NIST Special Publication 1000-5

June 2004
Progress Report on the Federal Building and Fire Safety Investigation of the World Trade Center Disaster

Volume 1

NIST National Institute of Standards and Technology • Technology Administration • U.S. Department of Commerce

National Institute of Standards and Technology Special Publication 1000-5
Natl. Inst. Stand. Technol. Spec. Publ. 1000-5, 1,054 pages (June 2004)
CODEN: NSPUE2

U.S. GOVERNMENT PRINTING OFFICE
WASHINGTON: 2004

For sale by the Superintendent of Documents, U.S. Government Printing Office
Internet: bookstore.gpo.gov — Phone: (202) 512-1800 — Fax: (202) 512-2250
Mail: Stop SSOP, Washington, DC 20402-0001

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List of Acronyms and Abbreviations

AAPOR	American Association of Public Opinion Research
ABC	American Broadcasting Company
ACI	American Concrete Institute
AISC	American Institute of Steel Construction
AISI	American Iron and Steel Institute
ALE	Arbitrary-Lagrangian-Evlerian
AMCBO	Association of Major City/County Building Officials
ANSI	American National Standards Institute
ANSYS	finite element model
ARA	Applied Research Associates, Inc.
ASCE	American Society of Civil Engineers
ASHRAE	American Society of Heating, Refrigerating and Air-Conditioning Engineers, Inc.
ASME	American Society of Mechanical Engineers
ASTM	ASTM International
AWS	American Welding Society
BOCA	Building Officials and Code Administrators
BOCA/BBC	BOCA Basic Building Code
BPAT	Building Performance Assessment Team
BPS	Building Performance Study
BSI	British Standards Institution
C/F	cancer free
CATI	computer-assisted telephone interviews
CBR	chemical, biological, and radiological
CBS	Columbia Broadcasting System
CERF	Civil Engineering Research Foundation
CFD	computational fluid dynamics
CIB	International Council for Research and Innovation in Building and Construction
CII	Construction Industry Institute
CNN	Cable News Network

CPP	Cermak Peterka Peterson, Inc.
CPU	central processing unit
CRT	cathode-ray tube
CTB&UH	Council on Tall Buildings and Urban Habitat
CTE	coefficients of thermal expansion
DC/F	BlazeShield DC/F fire protective insulation
DL	dead load
DTAP	dissemination and technical assistance program
EMS	Emergency Medical Service
EMT	Emergency Medical Team
ER&S	Emory Roth & Sons
FBI	Federal Bureau of Investigation
FCA	Flux cored arc
FDNY	New York City Fire Department
FDS	Fire Dynamics Simulator
FE	finite element
FEA	finite element analysis
FEM	finite element model
FEMA	Federal Emergency Management Agency
FMRC	Factory Mutual Research Corp.
FSI	Fire-Structure Interface
FVM	Finite Volume Method
GFI	Government Furnished Information
GG	glass over glass
GHz	gigahertz
GMS, LLP	Gilsanz Murray Steficek, LLP
HAZ	heat affected zone
HNSE	Hugo Nue Schnutzer East
HRR	heat release rate
HVAC	heating, ventilating, and air conditioning
IAQ	indoor air quality
IBC	International Building Code

ICBO	International Conference of Building Officials
ICC	International Code Council
IMTI	Integrated Manufacturing Technology
JFK	John F. Kennedy International Airport
JIS	Japan Industrial Standard
LERA	Leslie E. Robertson Associates
LES	Large Eddy Simulation
LL	live load
LSTC	Livermore Software Technology Corporation
MBC	BOCA National Building Code
MCC	Municipal Code of Chicago
MPI	Message Passing Interface
NBC	National Broadcasting Company
NBFU	National Board of Fire Underwriters
NCSBCS	National Conference of States on Building Codes & Standards, Inc.
NCST	National Construction Safety Team
NEMA	National Electrical Manufacturers Association
NFPA	National Fire Protection Association
NIBS	National Institute of Building Sciences
NIST	National Institute of Standards and Technology
NYC	New York City
NYCBC	New York City Building Code
NYCDOB	New York City Department of Buildings
NYPD	New York City Police Department
NYSBC	New York State Building Construction Code
P.L.	Public Law
PANYNJ	Port Authority of New York and New Jersey
PAPD	Port Authority Police Department
PC&F	Pacific Car and Foundry
PDM	Pittsburg-Des Moines
PONYA	Port of New York Authority
R&D	research and development

RWDI	Rowan Williams Davis and Irwin, Inc.
SBCCI	Southern Standard Building Code
SDL	superimposed dead load
SDO	standards development organization
SEAoNY	Structural Engineers Association of New York
SFPE	Society of Fire Protection Engineering
SFRM	spray-on fire resistant material or sprayed fire resistive materials
SHCR	Skilling, Helle, Christiansen, & Robertson
SI	metric
SLB	short legs back-to-back
SMA	Shielded Metal Arc
SOD	Special Operations Division
SOM	Skidmore, Ownings & Merrill
SPH	Smoothed Particle Hydrodynamics
SQL	Structured Query Language
SWMB	Skilling, Ward, Magnussen, and Barkshire
TL	Truss Lower Chord
TM	Truss Middle Chord
TU	Truss Upper Chord
UBC	Uniform Building Code
UL	Underwriters' Laboratories, Inc.
USC	United States Code
USM	United States Mineral Products Co.
VCBT	Virtual Cybernetic Building Testbed
WABC	WABC-TV New York
WCBS	WCBS-TV New York
WF	wide flange (a type of structural steel shape now usually called a W-shape). ASTM A 6 defines them as "doubly-symmetric, wide-flange shapes with inside flange surfaces that are substantially parallel."
WNBC	NBC4 New York
WNYW	FOX5 New York
WPIX	WPIX-TV New York
WTC	World Trade Center

WTC 1	World Trade Center Tower 1
WTC 2	World Trade Center Tower 2
WTC 7	World Trade Center Building 7

Abbreviations

×	by
±	plus or minus
°C	degrees Celsius
°F	degrees Fahrenheit
μm	micrometer
2D	two dimensional
3D	three dimensional
cm	centimeter
ft	foot
ft^2	square foot
F_y	yield strength (AISC usage)
g	acceleration (gravity)
g	gram
gal	gallon
h	hour
in.	inch
kg	kilogram
kip	a stress unit equal to 1,000 pounds
kJ	kilojoule
kN	kilonewton
kPa	kilopascal
klb	1,000 pounds
ksi	1,000 pounds per square inch
kW	kilowatt
kW/m^2	kilowatts per square meter
L	liter
lb	pound
m	meter
m^2	square meter
mm	millimeter
m/s	meters per second

min	minute
MJ	megajoule
MPa	megapascal
mph	miles per hour
ms	microsecond
Msi	millions pounds per square inch
MW	megawatt
N	newton
Pa	pascal
pcf	pounds per cubic foot
plf	pounds per linear foot
psf	pounds per square foot
psi	pounds per square inch
s	second

METRIC CONVERSION TABLE

To convert from	to	Multiply by
AREA AND SECOND MOMENT OF AREA		
square foot (ft^2)	square meter (m^2)	9.290 304 E-02
square inch (in^2)	square meter (m^2)	6.4516 E-04
square inch (in^2)	square centimeter (cm^2)	6.4516 E+00
square yard (yd^2)	square meter (m^2)	8.361 274 E-01
ENERGY (includes WORK)		
kilowatt hour (kW * h)	joule (J)	3.6 E+06
quad (1015 BtuIT)	joule (J)	1.055 056 E+18
therm (U.S.)	joule (J)	1.054 804 E+08
ton of TNT (energy equivalent)	joule (J)	4.184 E+09
watt hour (W * h)	joule (J)	3.6 E+03
watt second (W * s)	joule (J)	1.0 E+00
FORCE		
dyne (dyn)	newton (N)	1.0 E-05
kilogram-force (kgf)	newton (N)	9.806 65 E+00
kilopond (kilogram-force) (kp)	newton (N)	9.806 65 E+00
kip (1 kip=1000 lbf)	newton (N)	4.448 222 E+03
kip (1 kip=1000 lbf)	kilonewton (kN)	4.448 222 E+00
pound-force (lbf)	newton (N)	4.448 222 E+00
FORCE DIVIDED BY LENGTH		
pound-force per foot (lbf/ft)	newton per meter (N/m)	1.459 390 E+01
pound-force per inch (lbf/in)	newton per meter (N/m)	1.751 268 E+02
HEAT FLOW RATE		
calorieth per minute (calth/min)	watt (W)	6.973 333 E-02
calorieth per second (calth/s)	watt (W)	4.184 E+00
kilocalorieth per minute (kcalth/min)	watt (W)	6.973 333 E+01
kilocalorieth per second (kcalth/s)	watt (W)	4.184 E+03

To convert from	to	Multiply by
LENGTH		
foot (ft)	meter (m)	3.048 E-01
inch (in)	meter (m)	2.54 E-02
inch (in)	centimeter (cm)	2.54 E+00
micron (m)	meter (m)	1.0 E-06
yard (yd)	meter (m)	9.144 E-01
MASS and MOMENT OF INERTIA		
kilogram-force second squared per meter (kgf * s^2/m)	kilogram (kg)	9.806 65 E+00
pound foot squared (lb * ft^2)	kilogram meter squared (kg * m^2)	4.214 011 E-02
pound inch squared (lb * in^2)	kilogram meter squared (kg * m^2)	2.926 397 E-04
ton, metric (t)	kilogram (kg)	1.0 E+03
ton, short (2000 lb)	kilogram (kg)	9.071 847 E+02
MASS DIVIDED BY AREA		
pound per square foot (lb/ft^2)	kilogram per square meter (kg/m^2)	4.882 428 E+00
pound per square inch (*not* pound force) (lb/in^2)	kilogram per square meter (kg/m^2)	7.030 696 E+02
MASS DIVIDED BY LENGTH		
pound per foot (lb/ft)	kilogram per meter (kg/m)	1.488 164 E+00
pound per inch (lb/in)	kilogram per meter (kg/m)	1.785 797 E+01
pound per yard (lb/yd)	kilogram per meter (kg/m)	4.960 546 E-01
PRESSURE or STRESS (FORCE DIVIDED BY AREA)		
kilogram-force per square centimeter (kgf/cm^2)	pascal (Pa)	9.806 65 E+04
kilogram-force per square meter (kgf/m^2)	pascal (Pa)	9.806 65 E+00
kilogram-force per square millimeter (kgf/mm^2)	pascal (Pa)	9.806 65 E+06
kip per square inch (ksi) (kip/in^2)	pascal (Pa)	6.894 757 E+06
kip per square inch (ksi) (kip/in^2)	kilopascal (kPa)	6.894 757 E+03
pound-force per square foot (lbf/ft^2)	pascal (Pa)	4.788 026 E+01
pound-force per square inch (psi) (lbf/in^2)	pascal (Pa)	6.894 757 E+03
pound-force per square inch (psi) (lbf/in^2)	kilopascal (kPa)	6.894 757 E+00
psi (pound-force per square inch) (lbf/in^2)	pascal (Pa)	6.894 757 E+03
psi (pound-force per square inch) (lbf/in^2)	kilopascal (kPa)	6.894 757 E+00

To convert from	**to**	**Multiply by**
TEMPERATURE		
degree Celsius (°C)	kelvin (K)	T/K = t/ °C + 273.15
degree centigrade	degree Celsius (°C)	t/ °C ≈ t /deg. cent.
degree Fahrenheit (°F)	degree Celsius (°C)	t/ °C = (t/ °F 2 32)/1.8
degree Fahrenheit (°F)	kelvin (K)	T/K = (t/ °F + 459.67)/1.8
kelvin (K)	degree Celsius (°C)	t /°C = T/ K 2 273.15
TEMPERATURE INTERVAL		
degree Celsius (°C)	kelvin (K)	1.0 E+00
degree centigrade	degree Celsius (°C)	1.0 E+00
degree Fahrenheit (°F)	degree Celsius (°C)	5.555 556 E-01
degree Fahrenheit (°F)	kelvin (K)	5.555 556 E-01
degree Rankine (°R)	kelvin (K)	5.555 556 E-01
VELOCITY (includes SPEED)		
foot per second (ft/s)	meter per second (m/s)	3.048 E-01
inch per second (in/s)	meter per second (m/s)	2.54 E-02
kilometer per hour (km/h)	meter per second (m/s)	2.777 778 E-01
mile per hour (mi/h)	kilometer per hour (km/h)	1.609 344 E+00
mile per minute (mi/min)	meter per second (m/s)	2.682 24 E+01
VOLUME (includes CAPACITY)		
cubic foot (ft^3)	cubic meter (m^3)	2.831 685 E-02
cubic inch (in^3)	cubic meter (m^3)	1.638 706 E-05
cubic yard (yd^3)	cubic meter (m^3)	7.645 549 E-01
gallon (U.S.) (gal)	cubic meter (m^3)	3.785 412 E-03
gallon (U.S.) (gal)	liter (L)	3.785 412 E+00
liter (L)	cubic meter (m^3)	1.0 E-03
ounce (U.S. fluid) (fl oz)	cubic meter (m^3)	2.957 353 E-05
ounce (U.S. fluid) (fl oz)	milliliter (mL)	2.957 353 E+01

PREFACE

The National Institute of Standards and Technology (NIST) initiated the federal building and fire safety investigation of the World Trade Center (WTC) disaster on August 21, 2002. This WTC Investigation, led by NIST, is being conducted under the authority of the National Construction Safety Team Act (Public Law [P.L.] 107-231).

Goals of the WTC Investigation

- To investigate the building construction, the materials used, and the technical conditions that contributed to the outcome of the WTC disaster.
- To serve as the basis for:
  - Improvements in the way buildings are designed, constructed, maintained, and used
  - Improved tools and guidance for industry and safety officials
  - Recommended revisions to current codes, standards, and practices
  - Improved public safety

Objectives of the WTC Investigation

The objectives of the NIST-led Investigation of the WTC disaster are to:

1. Determine why and how WTC 1 and WTC 2 collapsed following the initial impacts of the aircraft and why and how WTC 7 collapsed
2. Determine why the numbers of injuries and fatalities were so high or low depending on location, including technical aspects of fire protection, occupant behavior, evacuation, and emergency response
3. Determine what procedures and practices were used in the design, construction, operation, and maintenance of WTC 1, 2, and 7
4. Identify, as specifically as possible, areas in current national building and fire model codes, standards, and practices that warrant revision

Authorities and Use of Information in Legal Proceedings

NIST is a nonregulatory agency of the U.S. Department of Commerce. NIST investigations are focused on fact finding, not fault finding. No part of any report resulting from a NIST investigation into a structural failure or from an investigation under the National Construction Safety Team Act may be used in any suit or action for damages arising out of any matter mentioned in such report (15 USC 281a, as amended by P.L. 107-231).

Organization of the WTC Investigation

The Investigation includes eight interdependent projects that, in combination, meet the objectives. A detailed description of each of these eight projects is available at http://wtc.nist.gov. The purpose of each project is summarized in Table P–1, and the key interdependencies among the projects are illustrated in Figure P–1.

Table P–1. Federal building and fire safety investigation of the WTC disaster.

Technical Area	Project No.	Project Purpose
Analysis of Building and Fire Codes and Practices	1	Document and analyze the code provisions, procedures, and practices used in the design, construction, operation, and maintenance of the structural, passive fire protection, and emergency access and evacuation systems of the WTC 1, 2, and 7.
Baseline Structural Performance and Aircraft Impact Damage Analysis	2	Analyze the baseline performance of WTC 1 and 2 under design, service, and abnormal loads, and aircraft impact damage on the structural, fire protection, and egress systems.
Mechanical and Metallurgical Analysis of Structural Steel	3	Determine and analyze the mechanical and metallurgical properties and quality of steel, weldments, and connections from steel recovered from WTC 1, 2, and 7.
Investigation of Active Fire-Protection Systems	4	Investigate the performance of the active fire protection systems in WTC 1, 2, and 7 and their role in fire control, emergency response, and fate of occupants and responders.
Reconstruction of Thermal and Tenability Environment	5	Reconstruct the time-evolving temperature, thermal environment, and smoke movement in WTC 1, 2, and 7 for use in evaluating the structural performance of the buildings and behavior and fate of occupants and responders.
Structural Fire Response and Collapse Analysis	6	Analyze the response of the WTC towers to fires with and without aircraft damage, the response of WTC 7 in fires, the performance of open-web steel joists, and determine the most probable structural collapse sequence for WTC 1, 2, and 7.
Occupant Behavior, Egress, and Emergency Communications	7	Analyze the behavior and fate of occupants and responders, both those who survived and those who did not, and the performance of the evacuation system.
Fire Service Technologies and Guidelines	8	Building on work done by the Fire Department of New York and McKinsey & Company, document what happened during the response by the fire services to the WTC attacks until the collapse of WTC 7; identify issues that need to be addressed in changes to practice, standards, and codes; identify alternative practices and/or technologies that may address these issues; and identify research and development needs that advance the safety of the fire service in responding to massive fires in tall buildings.

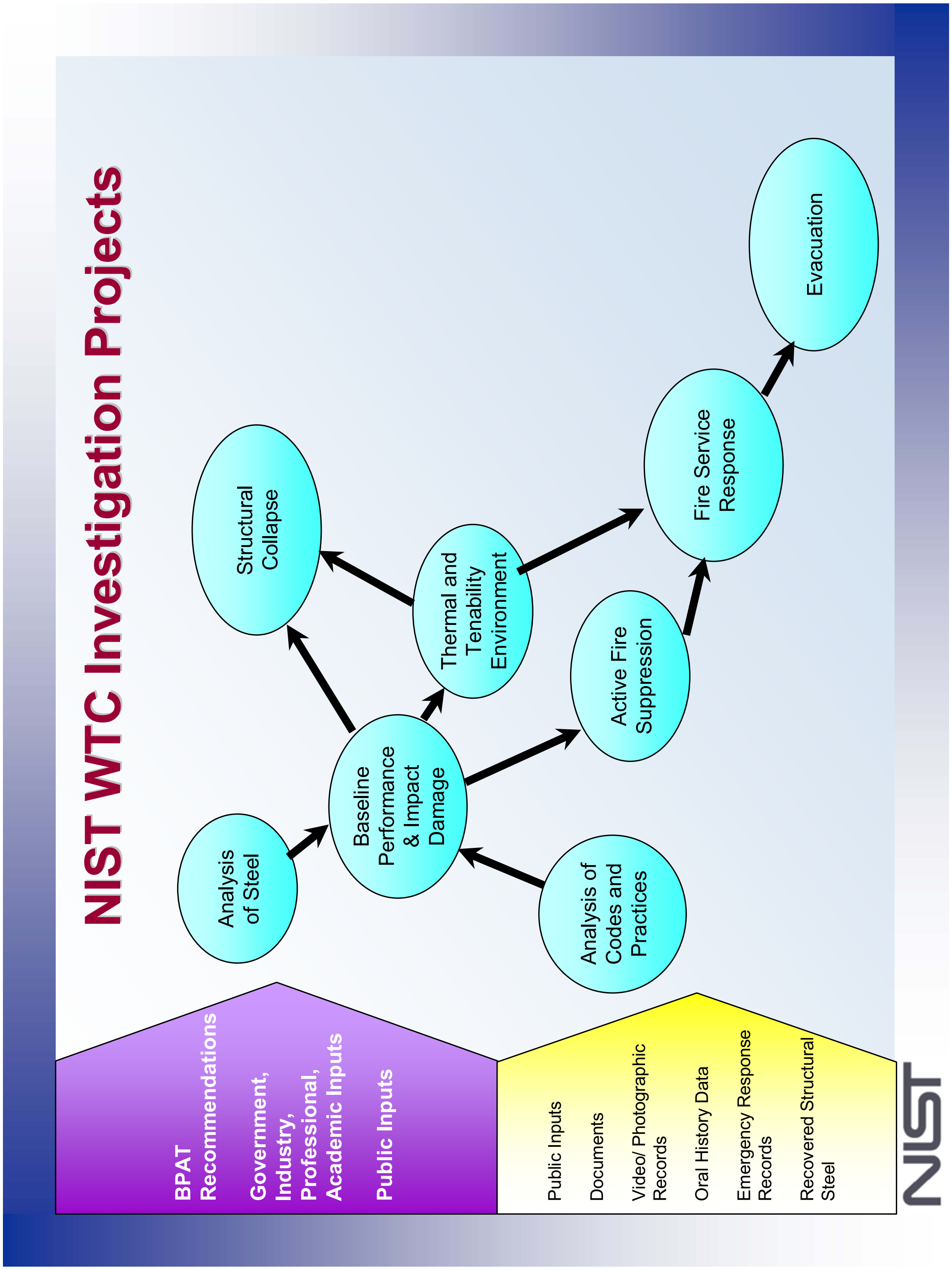

Figure P–1. The eight projects in the federal building and fire safety investigation of the WTC disaster.

NIST's WTC Public-Private Response Plan

The goal of the WTC Public-Private Response Plan is to develop the technical basis for standards, technology, and practices needed for cost-effective improvements to the safety and security of buildings and building occupants, including evacuation, emergency response procedures, and threat mitigation.

The strategy to meet this goal is a three-part NIST-led public-private response program that includes:

- A federal building and fire safety investigation to study the most probable factors that contributed to post-aircraft impact collapse of the WTC towers and the 47-story WTC 7, and the associated evacuation and emergency response experience.
- A research and development (R&D) program to provide a technical foundation that supports improvements to building and fire codes, standards, and practices that reduce the impact of extreme threats to the safety of buildings, their occupants and emergency responders.
- A dissemination and technical assistance program (DTAP) to engage leaders of the construction and building community in implementing proposed changes to practices, standards, and codes. This effort also will provide practical guidance and tools to better prepare facility owners, contractors, architects, engineers, emergency responders, and regulatory authorities to respond to future disasters.

The desired outcomes are to make all buildings safer for occupants and first responders and to ensure better evacuation systems and emergency response capabilities for future disasters.

EXECUTIVE SUMMARY

Background

In response to the terrorist attacks of September 11, 2001, the National Institute of Standards and Technology (NIST) initiated a formal federal building and fire safety investigation of the World Trade Center disaster on August 21, 2002. NIST issued two written updates on its WTC investigation activities (December 2002 and December 2003) and a detailed technical progress report in May 2003.

In addition, NIST held a public meeting in New York City on February 12, 2004 to solicit comments on (1) specific technical aspects of the investigation, (2) additional information that NIST might consider in the time remaining; and (3) areas that NIST should consider, within the scope of its investigation, in making recommendations for specific improvements to building and fire practice, standards, and codes, and their timely adoption.

The present report provides details of the technical progress made since the May 2003 report was published. NIST expects to release the draft of the final investigation report for public comment in December 2004. NIST's investigation is still ongoing. Current findings may be revised and additional findings will be presented in the December 2004 report. **NIST is not making any recommendations at this time. All recommendations will be made in the final report.**

Status of Progress

This report includes:

- A comprehensive summary of interim findings and accomplishments for each of the independent investigation objectives.
- A working hypothesis for the collapse of the WTC towers that identifies the chronological sequence of major collapse events and allows for different possible load redistribution paths and damage scenarios currently under analysis. The hypothesis will be refined on the basis of these analyses to determine the most probable collapse sequence for each building.
- A working hypothesis for the collapse of the 47-story WTC 7 based on an initiating event, a vertical progression at the east side of the building, a subsequent horizontal progression from the east to the west side of the building, and global collapse.
- Key visual observations on the building, fire, and smoke conditions in all three WTC buildings (the WTC towers and WTC 7) from analysis of a large collection of photographic and videographic images.
- A summary of major progress in building comprehensive models for analyzing the most probable collapse sequence, from aircraft impact to collapse initiation, and simplified analytical models with results to supplement those from detailed models.

- Results from experimental work to (1) analyze the recovered WTC structural steel, (2) support the fire dynamics and thermal modeling, and (3) conduct fire endurance testing of typical floor systems of the WTC towers based on ASTM E 119.

- Reports on the inventory and identification of the steels recovered from the WTC buildings and on the contemporaneous (1960s era) structural steel and welding specifications used to construct the WTC towers.

- First-person interviews of nearly 1,200 WTC occupants, first responders, and families of victims to collect data on occupant behavior, evacuation, and emergency response with some early results from analysis of that data.

- Review of the New York City 911 tapes and logs and the transcripts of about 500 interviews with Fire Department of New York (FDNY) employees involved in WTC emergency response activities with analysis still in progress.

- Preliminary analysis of emergency responder communication tapes recorded by the Port Authority, including the high-rise radio repeater, and by the New York Police Department (NYPD), including internal department operations.

- Analysis of building and fire codes and practices, including: a review of available documents related to the design, construction, operation, maintenance, and modifications to the three WTC buildings; and a comparison of selected building regulatory and code requirements.

- Analysis of the design, capabilities, and performance of the installed active fire protection systems for all three WTC buildings (i.e., fire alarm, sprinkler, and smoke management systems) with documentation of the fire history of the WTC towers.

- Progress on both the research and development and the dissemination and technical assistance programs related to the WTC Investigation.

- Seventeen appendices with detailed interim reports on specific technical tasks within the eight investigation projects where significant progress has been made.

NIST has received large amounts of data and information related to the design, construction, operation, inspection, maintenance, repair, alterations, emergency response, and evacuation of the WTC complex. NIST has received considerable cooperation from the Port Authority of New York and New Jersey (PANYNJ or Port Authority), the City of New York, the National Commission on Terrorist Attacks Upon the United States (9-11 Commission), designers, leaseholders, contractors, suppliers, insurers, news media, tenants, first responders, survivors, and families of victims.

NIST has received all of the essential information it needs for the WTC Investigation. NIST has made a few requests for materials that are lost, currently pending, or not yet located; NIST is making efforts to assemble this information from various sources since much of it was lost when the buildings collapsed. NIST continues to pursue other materials that can further clarify some aspects of the Investigation.

The Web site http://wtc.nist.gov provides comprehensive information on the WTC investigation and related work to improve the safety of buildings, their occupants, and first responders.

Investigation Objectives and Key Questions

The key interim findings are summarized below in subsections that relate to the investigation objectives contained in the NIST investigation plan (see Chapter 1 for a comprehensive discussion of all interim findings). The investigation objectives are:

1. To determine (a) why and how the WTC 1 and WTC 2 collapsed following the initial impact of the aircraft, and (b) why and how the 47-story WTC 7 collapsed.

2. To determine why the loss of life and injuries were so low or so high depending on location, including technical aspects of fire protection, occupant behavior, evacuation, and emergency response.

3. To determine the procedures and practices which were used in the design, construction, operation, and maintenance of the WTC buildings.

4. To identify, as specifically as possible, areas in national building and fire codes, standards, and practices that warrant revision.

Among the specific questions that NIST is investigating within the above four objectives are the following:

- How and why did WTC 1 stand nearly twice as long as WTC 2 before collapsing (103 min versus 56 min), though they were hit by virtually identical aircraft?

- What factors related to normal building and fire safety considerations not unique to the terrorist attacks of September 11, 2001, if any, could have delayed or prevented the collapse of the WTC towers?

- Would the undamaged WTC towers have remained standing in a normal major building fire?

- What factors related to normal building and fire safety considerations, if any, could have saved additional WTC occupant lives or could have minimized the loss of life among the ranks of first responders on September 11, 2001?

- How well did the procedures and practices used in the design, construction, operation, and maintenance of the WTC buildings conform to accepted national practices, standards, and codes?

Context for Findings

When reviewing these interim findings, the following should be considered:

- Buildings are not specifically designed to withstand the impact of fuel-laden commercial airliners. While documents from the PANYNJ indicate that the impact of a Boeing 707 flying at 600 mph, possibly crashing into the 80th floor, was analyzed during the design of the WTC towers in February/March 1964, the effect of the subsequent fires was not considered. Building codes do not require building designs to consider aircraft impact.

- Buildings are not designed for fire protection and evacuation under the magnitude and scale of conditions similar to those caused by the terrorist attacks of September 11, 2001.

- The load conditions induced by aircraft impact and the extensive fires on September 11, 2001, which triggered the collapse of the WTC towers, fall outside the norm of design loads considered in building codes.

- Prior evacuation and emergency response experience in major events did not include the total collapse of tall buildings such as the WTC towers and WTC 7 that were occupied and in everyday use; instead, that experience suggested that major tall building fires result in burnout conditions, not global building collapse.

- The PANYNJ was created as an interstate entity, under a clause of the U.S. Constitution permitting compacts between states, and is not bound by the authority of any local, state, or federal jurisdiction, including local building and fire codes. The PANYNJ's longstanding policy is to meet and, where appropriate, exceed the requirements of local building and fire codes.

Collapse of the WTC Towers

Working Hypothesis. The following chronological sequence of major events led to the eventual collapse of the towers; specific load redistribution paths and damage scenarios are currently under analysis to determine the most probable collapse sequence for each building:

- Aircraft impact damage to perimeter columns, resulting in redistribution of column loads to adjacent perimeter columns and to the core columns via the hat truss;

- After breaching the building's exterior, the aircraft continued to penetrate into the buildings, damaging core columns with redistribution of column loads to other intact core and perimeter columns via the hat truss and floor systems;

- The subsequent fires, influenced by the post-impact condition of the fireproofing, weakened columns and floor systems (including those that had been damaged by aircraft impact), triggering additional local failures that ultimately led to column instability; and

- Initiation and horizontal progression of column instability resulted when redistributing loads could not be accommodated any further. The collapses then ensued.

The working hypothesis (see Chapter 1 and Appendix Q for a detailed description) is consistent with all evidence currently held by NIST, including photographs and videos, eyewitness accounts, and emergency communication records. In addition to evidence of hanging floor slabs on the east and north faces of WTC 2 that were reported previously, new evidence has been found showing inward bowing of perimeter columns several minutes (less than 10 min) prior to collapse in both WTC towers. Inward bowing of about a quarter to a third of the perimeter columns was observed in photographs on the south face of WTC 1 and the east face of WTC 2 in regions that contained active fires. Further, initiation of global collapse was first observed by the tilting of building sections above the impact regions of both WTC towers. WTC 1 tilted to the south (observed via antenna tilting in a video recording) and WTC 2 tilted to the east and south and twisted in a counterclockwise motion.

Discussed below are interim findings for several factors relevant to the condition and collapse of the WTC towers, including the innovative structural system, aircraft impact and the ensuing fires, post-impact condition of the fireproofing, and the quality and properties of the structural steel used in the WTC towers. In addition to the role played by these factors in the collapse of the WTC towers, NIST continues to investigate the relative roles of the perimeter and core columns and the composite floor system, including connections.

Innovative Structural System. The WTC tower structures represented an innovative structural system when they were built, incorporating many new and unusual features. Among them, two features require additional consideration: the composite floor system, using open-web bar joist elements, to provide lateral stability and diaphragm action, and the use of wind tunnel testing to estimate lateral wind loads—which were a major governing factor in the design of the WTC tower structures.

The performance of the former under fire conditions, which is relevant to evaluating the collapse of the WTC towers, was of some concern to the building owner and designers throughout the life of the buildings. The concern stemming from the latter, identified by the leaseholder and insurers in litigation after September 11, 2001, is representative of the still evolving state-of-knowledge in the field of wind engineering and is relevant to establishing the baseline performance of the WTC towers and to assessing the practices and procedures used in design.

The fire protection of a truss-supported floor system by directly applying spray-on fireproofing to the steel trusses was innovative and not consistent with prevailing practice at the time the WTC towers were designed and constructed. The fireproofing thickness required to meet the 2 h fire rating evolved from the specified 1/2 in. when the WTC towers were built to 1-1/2 in. for use in upgrading the fireproofing some years prior to September 11, 2001. Unrelated to the WTC buildings, a model code evaluation service recommended in June 2001 a minimum thickness of 2 in. for a similar floor system. This three to four fold difference in specifying the fireproofing thickness to meet the required fire rating for a structural assembly is extraordinarily large and confirms the lack of technical basis in the selection of thickness.

While the benefits of conducting a full-scale fire endurance test to determine the required fireproofing thickness were recognized by the building designers, no tests were conducted on the floor system used in the WTC towers to establish a fire endurance rating. NIST has awarded a contract to Underwriters Laboratories (UL) to determine the fire resistance rating of typical WTC floor systems under both as-specified and as-built conditions. The tests, expected to be conducted in August 2004, are also designed to evaluate the effects of test scale, fireproofing thickness, and thermal restraint.

Further, use of the “structural frame” approach, in conjunction with the prescriptive fire rating, would have required the floor system trusses, the core floor framing, and perimeter spandrels in the WTC towers—essential to the stability of the building as a whole—to be 3 h fire-rated as the columns were required to be rated by the 1968 New York City (NYC) Building Code. This approach, which appeared in the Uniform Building Code (a model building code) as early as 1953, was carried into the 2000 International Building Code (one of two current national model codes). Neither the 1968 edition of the NYC Building Code which was used in the design of the WTC towers, nor the 2001 edition of the code, adopted the “structural frame” requirement.

Results of two sets of wind tunnel tests conducted for the WTC towers in 2002 by independent laboratories, and voluntarily provided to NIST by the parties to an insurance litigation, show large differences, of as much as about 40 percent, in resultant forces on the structures, i.e., overturning moments and base shears. In addition, the wind loads estimated from these tests are about 20 percent to 60 percent higher than those apparently used in the original design of the WTC towers, also obtained from wind tunnel testing. NIST is conducting an independent analysis to establish the baseline performance of the WTC towers under the original design wind loads and will compare those wind load estimates with then-prevailing code requirements. Wind loads were a major governing factor in the design of structural components that made up the frame-tube steel framing system.

Relative Roles of Aircraft Impact and Fires. The two WTC towers withstood the initial impact of virtually identical aircraft (Boeing 767-200ER) during the terrorist attacks of September 11, 2001. The robustness of the perimeter frame-tube system and large dimensional size of the WTC towers helped the buildings withstand the aircraft impact. The WTC towers displayed significant reserve capacity, vibrating immediately following impact with amplitudes that were about half the amplitudes for design wind conditions expected by the building designers and an oscillation period nearly equal to that measured for the undamaged building.

Preliminary aircraft impact damage analysis indicates that the impact of a fuel-filled wing section results in extensive damage to the exterior wall panel, including complete failure of the perimeter columns. A normal impact of the exterior wall by an empty wing segment produces significant damage to the perimeter columns, not necessarily complete failure. Also, engine impact against an exterior wall panel results in a penetration of the exterior wall and failure of impacted perimeter columns. The residual velocity and mass of the engine after penetration of the exterior wall is sufficient to fail a core column in the event of a direct impact.

Fires played a major role in further reducing the structural capacity of the buildings, initiating collapse. While aircraft impact damage did not, by itself, initiate building collapse, it contributed greatly to the subsequent fires by:

- Compromising the sprinkler and water supply systems;
- Dispersing jet fuel and igniting building contents over large areas;
- Creating large accumulations of combustible matter containing aircraft and building contents;

- Increasing the air supply into the damaged buildings that permitted significantly higher energy release rates than would normally be seen in ventilation limited building fires, allowing the fires to spread rapidly within and between floors; and

- Damaging ceilings that enabled "unabated" heat transport over the floor-to-ceiling partition walls and to structural components.

The jet fuel, which ignited the fires, was mostly consumed within the first few minutes after impact. The fires that burned for almost the entire time that the buildings remained standing were due mainly to burning building contents and, to a lesser extent, aircraft contents, not jet fuel.

By contrast, typical office furnishings were able to sustain intense fires for at least an hour on a given WTC floor. The typical WTC office floor using modern workstation furnishings had on average about 4 pounds per square foot (psf) of combustible materials on floors without unusual file rooms, film storage, etc. Further, the mass of aircraft solid combustibles was significant relative to the building combustibles in the immediate impact region of both WTC towers.

Consistent with available photographic and videographic evidence, computer simulations conducted by NIST have been able to capture the broad patterns of fire movement around the floors, with the flames in a given location lasting for about 20 min before spreading to adjacent, yet unburned combustibles. This spread is generally continuous due to the relatively even distribution of combustibles and the paucity of interior partitions. There are some observed instances where fires persisted over longer durations in regions with accumulated combustible debris and other instances of sudden or interrupted fire spread.

Applying the 1968 NYC Building Code, the WTC towers were required to have 1 h fire-rated tenant separations, but the code did not impose any minimum compartmentation requirements (e.g., 7,500 ft^2) to mitigate the horizontal spread of fire in buildings with large open floor plans. The sprinkler option was chosen for the WTC towers in preference to the compartmentation option in meeting the subsequent requirements of Local Law 5, adopted by New York City in 1973. The affected floors in the WTC towers were mostly open—with a modest number of perimeter offices and conference rooms and an occasional special purpose area. Some floors had two tenants, and those spaces, like the core areas, were partitioned (slab to slab). Photographic and videographic evidence confirms that even non-tenant space partitions (such as those that divided spaces to provide corner conference rooms) provided substantial resistance to fire spread in the affected floors. For the duration of about 50 min to 100 min prior to building collapse that the fires were active, the presence of undamaged 1 h fire-rated compartments may have assisted in mitigating fire spread and consequent thermal weakening of structural components.

Role of Fireproofing Conditions. NIST has developed a rigorous technical approach (see Appendix I for details) to evaluate the role fireproofing conditions may have played in the collapse of the WTC towers. The approach considers both the thickness and variability of fireproofing and the extent to which fireproofing may have been damaged due to aircraft impact.

In general, the floor systems in WTC 1 subject to aircraft impact and subsequent fires on September 11, 2001 had upgraded or thicker fireproofing (1.5 in. specified), while the affected floors in WTC 2 had the original fireproofing (0.5 in. specified).

The response of a structural component to fires is sensitive to variability in fireproofing thickness along its length. For the original fireproofing in the WTC towers, the as-applied fireproofing thickness on the floor trusses (0.75 in. average and 0.4 coefficient of variation) was found to be thermally equivalent to a uniform thickness of 0.6 in., which is greater than the specified minimum thickness of 0.5 in. For the upgraded fireproofing in some floors of the WTC towers, the as-applied upgraded fireproofing thickness (2.5 in. average and 0.24 coefficient of variation) was found to be thermally equivalent to a uniform thickness of 2.2 in., which is greater than the specified minimum thickness of 1.5 in. An alternative criterion for determining the equivalent thickness is currently being examined to confirm these findings.

Based on simplified analytical models, it was found that acceleration of a structural element, on the order of 100 to 150 times the acceleration due to gravity (or 100 g to 150 g), would be required to dislodge fireproofing similar to that used in the WTC towers with a typical thickness of about 1 in. from the structural component. Experiments are underway to verify the results of these simplified analyses. Similarly, analytical studies are underway to estimate the magnitude of accelerations of the structural members due to aircraft impact, from which the regions where fireproofing may have been dislodged will be identified. Those results are being used to analyze the role of the post-impact condition of the fireproofing, including its thickness, on the collapse of the WTC towers.

Analysis of Recovered WTC Steel. NIST has 236 pieces of steel in its possession; this collection of steel is adequate for purposes of determining the quality and properties of steel for the investigation. The regions of impact and fire damage were emphasized in the selection of steel pieces for the investigation. As a result, pieces of all 14 specified steel grades for exterior panels in the WTC towers are available, as well as the two specified grades that represent 99 percent of the core columns and both specified grades for the steel trusses that comprised the composite floor truss system.

Analysis of steel recovered from the WTC towers, based on stampings on the steel and mechanical tests, indicates that the correct specified materials were provided for the specified elements. When these data were combined with pre-collapse photographic images of damaged steel, it was found that aircraft impacted pieces of steel recovered from WTC 1 were in the precise locations as specified in the design drawings. Metallography and mechanical property tests indicate that the strength and quality of the steel used in the towers was as specified, typical of the era, and likely met all qualifying test requirements.

The room-temperature strength of the steel used in the towers met the relevant standards and, in many instances, exceeded the requirements by 5 percent to 10 percent. Work is ongoing to analyze the performance of the steel building components under impact and fire conditions up to initiation of global building collapse.

Collapse of the 47-Story WTC 7 Building

Working Hypothesis. The working hypothesis for the collapse of the 47-story WTC 7 building, if it remains viable upon further analysis, suggests that it was a classic progressive collapse including: an initiating event, a vertical progression at the east side of the building, a subsequent horizontal progression from the east to the west side of the building, and global collapse. The chronological sequence of major events under analysis is:

- An initial local failure at the lower floors (below Floor 13) of the building due to fire and/or debris induced structural damage of a critical column (the initiating event), which supported a large span floor bay with an area of about 2,000 ft^2;
- Vertical progression of the initial local failure up to the east penthouse, as large floor bays were unable to redistribute the loads, bringing down the interior structure below the east penthouse; and
- Horizontal progression of the failure across the lower floors (in the region of Floors 5 and 7, that were much thicker than the rest of the floors), triggered by damage due to the vertical failure, resulting in disproportionate collapse of the entire structure.

Visual Observations. The working hypothesis (see Chapter 1 and Appendix L for a detailed description) is consistent with all evidence currently held by NIST, including photographs and videos, eyewitness accounts, and emergency communication records. Specifically, the evidence indicates:

- The sequence of failures associated with the sinking of the east penthouse roof structure into the building, the near simultaneous window breakage along the east side of the north face, the sinking of the other roof structures, the near simultaneous breakage of a second set of windows along the west side of the north face, and the entire north façade above the 13th floor appearing to drop as an intact unit.
- Structural damage on the south face and southwest corner from WTC 1 debris that included (1) a multi-story gash across approximately a quarter to a third of the lower portion of the south face and extending inwards to the core, (2) approximately two columns in the southwest corner and related floor areas missing from Floors 8 to 18, and (3) severed spandrels between exterior columns near the southwest corner from the roof level for at least 5 to 10 floors.
- The sequence of fires in WTC 7—which began soon after WTC 1 collapsed—was observed (1) on the south face and near the southwest corner on Floors 22, 29, and 30, (2) across Floors 11 and 12 on the east face, from the south to the north, (3) on Floors 7 and 12 along the north face, (4) on Floors 8 and 13, with the fire on Floor 8 moving from west to east and the fire on Floor 13 moving from east to west, and (5) finally, on Floors 7, 8, 9, and 11 near the middle about half an hour before collapse; Floor 12 was burned out by this time. Interview responses indicate that there was no water in the standpipe system supplying the sprinklers in WTC 7.

Fuel System for Emergency Power. Based on a review of the fuel system for emergency power in WTC 7, Floor 5—which did not have any exterior windows and contained the only pressurized fuel distribution system on the south, west and north floor areas—is considered a possible fire initiation location, subject to further data and/or analysis that improve knowledge of fire conditions in this area.

Evacuation and Emergency Response

Building Population Characteristics. Based on information and data gathered during the first-person interviews of WTC surviving occupants:

- It is estimated that 17,400 occupants (± 1,200) were present in the WTC towers on the morning of September 11, 2001. The initial population of each tower was similar: 8,900 (± 750) in WTC 1 and 8,500 (± 900) in WTC 2. Of those present on September 11, 2001, 16 percent were also present during the 1993 bombing.

- About 6 percent of the surviving occupants reported a pre-existing limitation to their mobility. These limitations included obesity, heart condition, needing assistance to walk, pregnancy, asthma, being elderly, chronic condition, recent surgery or injury, and other.

- About 7 percent of the surviving occupants reported having special knowledge about the building. These included fire safety staff, floor wardens, searchers, building maintenance, and security staff. Searchers assist the floor wardens in facilitating evacuation.

Evacuation. Two-thirds of surviving occupants reported having participated in a fire drill in the 12 months prior to September 11, 2001, while 17 percent reported that they received no training during that same period. Of those participating in fire drills, 93 percent were instructed about the location of the nearest stairwell. Overall, slightly over half of the survivors, however, had never used a stairwell at the WTC prior to September 11, 2001.

Approximately 87 percent of the WTC tower occupants, including more than 99 percent of those below the floors of impact, were able to evacuate successfully. Two-thousand one-hundred fifty-nine building occupants (1,560 in WTC 1 and 599 in WTC 2) and an additional 433 first responders, including security guards but not aircraft passengers and crew or bystanders, were reported to have lost their lives on September 11, 2001.

Rough initial estimates indicate that about 20 percent or more of those who were in the WTC towers and lost their lives may have been alive in the buildings just prior to their collapse. This estimate, which will be refined as data analysis is completed, assumes that nearly all of the first responders and 76 building occupants below the floors of impact, but none of the people at or above the floors of impact, may have been alive. It is estimated that there were a total of 2,592 building occupants and first responders who were in the WTC towers and lost their lives.

Overall, about 7,900 survivors evacuated WTC 2 in 73 min (i.e., from the instant the WTC 1 was struck by aircraft until WTC 2 collapsed) while about 7,500 survivors evacuated WTC 1 in 103 min. Thus, the overall evacuation rate in WTC 2 (108 survivors per min) was about 50 percent faster than that in WTC 1 (73 survivors per min). Functioning elevators allowed many survivors to evacuate WTC 2 prior to aircraft impact. Most of the elevators in WTC 1 were not functioning, and survivors could only use the stairways. The stairwells, with partition wall enclosures that provided a 2 h fire-rating but little structural integrity, were damaged in the region of the aircraft impacted floors.

- After the first airplane struck WTC 1 and before the second airplane struck WTC 2, the survivors in WTC 2 were twice as likely as those in WTC 1 to have already exited the building (41 percent versus 21 percent). The rate of evacuation completion in WTC 2 was twice the rate in WTC 1 during that same period.

- Soon after WTC 2 was struck by the airplane until about 20 min before each building collapsed, the survivors in WTC 2 and WTC 1 had exited at about the same rate (the prior evacuation rate of WTC 1).

- During the last 20 min before each building collapsed, the evacuation rate in both buildings had slowed to about one-fifth the immediately prior evacuation rate. This suggests that for those seeking and able to reach and use undamaged exits and stairways, the egress capacity (number and width of exits and stairways) was adequate to accommodate survivors.

Preliminary results from application of existing computer egress models for a full capacity evacuation of a single WTC tower with 25,000 occupants and visitors indicate a movement time of 2 h and 15 min. This is a minimum time estimate; the simulation assumed that there was no survivor delay, continual movement on the stairs, and no damage to the egress system. It was also assumed that elevators were not available. The egress model estimate for a September 11, 2001 capacity evacuation under the same assumptions is about 50 min, which is 2.5 times less than the time estimate for evacuating 25,000 people.

Given that the actual evacuation time on September 11, 2001 was about 100 min without elevator use, a full capacity evacuation of each WTC tower with 25,000 people would have required about 4 h (2.5 times 100 min). To achieve a significantly faster total evacuation at full capacity would have required increases in egress capacity (number and width of exits and stairways).

In addition to the full evacuation of the WTC towers on September 11, 2001, a full evacuation was ordered during the 1993 bombing at the WTC site and during a 1977 terrorist threat associated with bombings in two remote midtown Manhattan buildings. Sufficient data do not exist on the frequency with which full evacuations are conducted in buildings not at risk for terrorist attacks and whether this frequency has increased since September 11, 2001 among the general population that did not directly experience the events on that day.

Roof Evacuation. A preliminary evaluation indicates that the PANYNJ's standard occupant evacuation procedures and drills required the use of stairways to exit at the bottom of the WTC towers. The standard procedures were to keep the doors to the roof locked with a key being required to gain roof access. The PANYNJ reports that it never advised tenants to evacuate upward.

There were at least two decedents who had tried to get to the roof and found the roof access locked to both the WTC towers. In addition, a PANYNJ employee trapped on Floor 105 of WTC 2 was unable to walk down the stairs, or go to the roof as instructed on radio by another PANYNJ employee.

The NYPD aviation unit arrived at the WTC site soon after WTC 1 was attacked. Despite repeated attempts to examine the possibility of roof rescue, smoke and heat conditions at the top of the WTC towers prevented the conduct of safe roof evacuation operations.

Considering the capacity of typical helicopters and travel times, it is not clear what fraction of the large number of occupants could have been evacuated from the WTC towers prior to their collapse had roof rescue been possible on September 11, 2001.

Emergency Communication Systems. A partial analysis of emergency responder communications (see Appendix P for details) has been completed, including:

- Audio communications tapes recorded by the PANYNJ, including a recording of the FDNY's city-wide high-rise Channel 7 (Port Authority Police Department's [PAPD] Channel 30) radio repeater that was located at the WTC.

- Audio tapes copied from original NYPD communications tapes, including NYPD internal department operations.

FDNY communications recordings from the WTC location on September 11, 2001, are not available because the primary field communication truck was in the shop for repairs. A back-up field-communications van used in its place—which did not have a recording capability—was destroyed when the WTC towers collapsed.

The best record of radio communications reflecting fire department operations came from the FDNY Channel 7/PAPD Channel 30 and first person accounts provided by FDNY personnel during their interviews. The PANYNJ installed the radio repeater system for use by FDNY after the 1993 bombing.

The analysis of the emergency responder communication tapes indicates that:

- After the first aircraft struck WTC 1, there was an approximate factor of 5 peak increase in traffic level over the normal level of emergency responder radio communications, followed by an approximate factor of 3 steady increase in the level of subsequent traffic.

- A surge in communications traffic volume made it more difficult to handle the flow of communications and delivery of information.

- Roughly a third to a half of the radio messages transmitted during these radio traffic surge conditions were not complete messages or understandable.

- FDNY's city-wide high-rise Channel 7 (PAPD Channel 30) radio repeater at the WTC site was operating.

- NYPD aviation unit personnel reported critical information about the impending collapse of the WTC towers several minutes prior to their collapse. No evidence has been found to suggest that the information was further communicated to all emergency responders at the scene.

Several FDNY personnel at the incident site did not think that the high-rise radio repeater was working. This is based on radio communications tests that were conducted by two chief officers working inside WTC 1 when the first command post was being set up in that lobby. Following this radio test, a chief officer involved in the test chose to use different channels for command and tactical communications during the incident. However, as FDNY operations increased in WTC 2, it was determined by FDNY members that the high-rise repeater was functioning, and use of the channel developed.

While the preliminary analysis indicates that the repeater was operating, there also appears to have been some type of malfunction with the communications equipment that was detected, but not identified, by

FDNY officers during the initial test. NIST continues to evaluate the repeater system and its operations, as well as the handheld radios, which were used on September 11, 2001. These findings will be updated and additional findings will be documented when the investigation is complete.

Command and Control. Based on face-to-face interviews, NIST has determined that first responders—including key incident commanders—did not have adequate information (voice, video, and data) on, nor an overall perspective of, the conditions in the WTC buildings and what was happening elsewhere at the WTC site. Interagency information sharing was inadequate.

The three FDNY suitcase-based, magnetic Command Boards that were set up at the incident site—on which a record is kept of the identification of the units on site, their assignment, location, and activities—were damaged and lost with the collapse WTC 2. Since there was no back-up capability for the Command Boards, all information related to command, control, and accountability was lost.

Active Fire Protection Systems. Investigation of the design, capabilities, and performance of the active fire protection systems in the WTC towers and WTC 7 indicates that:

- The smoke management systems in the WTC towers were not activated during the fires on September 11, 2001. It was determined that the likelihood of these systems being functional was very low due to the damage inflicted by the aircraft impacts.

- The analysis of smoke flow in WTC 1 and WTC 2 on September 11, 2001 shows that HVAC (heating, ventilation, and air-conditioning) ductwork was a major path for vertical smoke spread in the buildings. Fire dampers were installed in the systems, but not smoke dampers that could have provided a barrier to hot gas and smoke penetration into the vertical HVAC shafts. However, smoke dampers were not available when the towers were built.

- Modeling results show that stair pressurization systems would have provided minimal resistance to the passage of smoke in WTC 1 and WTC 2 had they been installed on September 11, 2001. While the existence of such systems was known when the WTC towers were built, the alternative smoke purge system used in the WTC towers was considered to be equivalent.

- The fire alarm system in WTC 7 sent only one signal (at 10:00:52 a.m. shortly after the collapse of WTC 2) to the monitoring company indicating a fire condition. The signal did not contain any specific information about the location of the fire within the building. Since the system was placed on TEST for a period of 8 h beginning at 6:47:03 a.m. on September 11, 2001, alarm signals would not have been shown on the operator's display; instead, they would have to be recorded into the history file.

- The resistance to failure of the fire alarm system communications paths between the fire command station and occupied WTC tower floors could have been enhanced if fiber optic communications cable had been used instead of copper lines. Fiber optic cable is not susceptible to electric short-circuits and would have provided full communications with fire alarm system components, including voice communications systems, to the point where the cable was severed. Electric shorts in the voice communications disable that communication system over the entire cable length affected by the electric short-circuit. During initial design of the system, the PANYNJ requested, but did not receive, approval of the City of New York for use of fiber optic communication cable in the system. The NYC code required copper wiring.

- There was adequate multiple point redundancy in the water supply to the sprinkler system, and the water flow rate exceeded the minimum requirement by a considerable margin. However, the potential for single point failure of the water supply to the fire sprinklers existed at each floor due to lack of redundancy in the sprinkler riser system that provided only one water supply connection on each floor. While this lack of redundancy may not have had an impact on September 11, 2001 because the sprinkler system was damaged by aircraft impact, it could have made a difference in other building emergencies.

Procedures and Practices

The 110-story WTC towers were among the world's tallest buildings, while the 47-story WTC 7 represented a more typical tall building. These buildings provide case studies to document, review, and, if needed, improve the procedures and practices used in the design, construction, operation, and maintenance of tall buildings. This investigation objective is independent of other objectives which are focused specifically on the consequences of the attack on September 11, 2001, viz., the building collapses, evacuation, and emergency response. While some findings under this objective are directly relevant to the events of September 11, 2001, others are concerned with general building and fire safety procedures and practices.

Applicable Building Codes. Although not required to conform to NYC codes, the PANYNJ adopted the provisions of the proposed 1968 edition of the NYC Building Code, more than three years before it went into effect. The 1968 edition allowed the PANYNJ to take economic advantage of less restrictive provisions compared with the 1938 edition that was in effect when design began for the WTC towers in 1962. The 1968 code:

- Eliminated a fire tower[1] as a required means of egress;

- Reduced the number of required stairwells from 6 to 3 and the size of doors leading to the stairs from 44 in. to 36 in.;

- Reduced the fire rating of the shaft walls in the building core from 3 h to 2 h;

- Changed partition loads from 20 psf to one based on weight of partitions per unit length (that reduced such loads for many buildings including the WTC buildings); and

- Permitted a 1 h reduction in fire rating for all structural components (columns from 4 h to 3 h and floor framing members from 3 h to 2 h).

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[1] A fire tower (also called a smoke-proof stair) is a stairway that is accessed through an enclosed vestibule that is open to the outside or to an open ventilation shaft providing natural ventilation that prevents any accumulation of smoke without the need for mechanical pressurization.

The NYC Department of Buildings reviewed the WTC tower drawings in 1968 and provided comments to the PANYNJ concerning the plans in relation to the 1938 NYC Building Code. The architect-of-record submitted to the PANYNJ responses to those comments, noting how the drawings conformed to the 1968 NYC Building Code.

In 1993, the PANYNJ and the NYC Department of Buildings entered into a memorandum of understanding that restated the PANYNJ's longstanding policy to assure that its facilities in the City of New York meet and, where appropriate, exceed the requirements of the NYC Building Code. The agreement also provided specific commitments to the NYC Department of Buildings regarding procedures to be undertaken by the PANYNJ to assure that buildings owned or operated by the PANYNJ are in conformance with the Building Standards contained in the NYC Building Code.

In 1993, the PANYNJ adopted a policy providing for implementation of fire safety recommendations made by local government fire departments after a fire safety inspection of a PANYNJ facility and for the prior review by local fire safety agencies of fire safety systems to be introduced or added to a facility. Later that year, the PANYNJ entered into an agreement with FDNY which reiterated the policy adopted by the PANYNJ and set forth procedures to assure that new or modified fire safety systems are in compliance with local codes and regulations.

Standards, Codes, and Regulations. NIST has reviewed the then-prevailing and current standards, codes, and regulations relevant to assessing the procedures and practices used in the design, construction, operation, and maintenance of the WTC buildings. That review raises the following issues that merit further consideration (see Chapter 1 for a discussion and WTC-related rationale):

- Code provisions that would detail procedures to analyze and evaluate data from fire resistance tests of other building components and assemblies to qualify an untested building element.
- Code provisions that would require the conduct of a fire resistance test if adequate data do not exist from other building components and assemblies to qualify an untested building element.
- Regulations that would adopt code provisions using the "structural frame" approach to fire resistance ratings which requires structural members, other than columns, that are essential to the stability of the building as a whole to be fire protected to the same rating as columns.
- Code provisions that would ensure that structural connections are provided the same degree of fire protection as the more restrictive protection of the connected elements.
- Code provisions and standards that would establish whether the minimum mechanical and durability related properties of spray-applied fire resistive materials (SFRM) are sufficient to ensure acceptable in-service performance in buildings. While minimum bond strength requirements exist, there are no serviceability requirements for such materials to withstand typical shock, impact, vibration, or abrasion effects over the life of a building.
- Rigorous field application and inspection provisions and regulatory requirements that would assure that the as-built condition of the passive fire protection, such as SFRM, conforms to conditions found in fire resistance tests of building components and assemblies.

- Rigorous provisions and regulatory requirements for in-service inspections of passive fire protection during the life of the building.
- Early installation of sprinklers in existing buildings, not as an option in lieu of compartmentation.
- Standards and code provisions that would provide minimum structural integrity for the means of egress (stairwells and elevator shafts) in the building core which are critical to life safety.
- Standards and code provisions that would permit the installation of fire-protected elevators and their use for routine emergency access by first responders or as a secondary method (after stairwells) for emergency evacuation of building occupants.
- Explicit standards and code provisions for structural integrity that would mitigate progressive collapse.
- Standards and code provisions for conducting wind tunnel tests and for the methods used in practice to estimate design wind loads from test results.
- Regulatory requirements for retention of documents related to the design, construction, operation, maintenance, and modifications of buildings, including retention off-site. For example, there are few, if any, requirements for retention of documents throughout the service life of a building.

Fire Safety and Egress Design Methods. Historical fire loss data over more than half a century, for different high-rise building occupancies, suggests that prescriptive requirements in standards and codes have considerable built-in conservatism to adequately protect building occupants. As a result, there has been a trend in recent decades to reduce fire rating and egress requirements, sometimes in conjunction with addition of other new and complementary fire protection requirements (e.g., detectors and sprinklers). The lower fire rating requirements when combined with the considerable increases in building design efficiency that have been achieved, have also led to reductions in the thermal mass of buildings—an indicator of how much heat energy a building can absorb passively without damage.

The empirical rules and test methods used in prescriptive design, which have evolved with experience over the years, do not lend themselves readily to evaluate whether the performance of building fire safety and egress systems is risk-consistent, considering both the hazards and the consequences of the hazards. Performance-based methods that explicitly define the design objectives and specific design-basis fire hazards or evacuation events are better suited to risk analysis, enabling appropriate protection to be provided where it is needed.

The increasing use of performance-based methods, as an alternative to prescriptive design, in fire safety and egress design, raises the following issues that merit further consideration (see Chapter 1 for a detailed discussion and rationale related to the WTC investigation):

- Considering fire as a design condition in structural design, including evaluation of the fire performance of the structure as a whole system. This design approach is already being used in building design practice for earthquake and wind hazards (e.g., a two-level design that includes an operational event with a 10 percent probability of occurrence in 50 years and a life safety event with a 2 percent probability of occurrence in 50 years).

- Detailed procedures to select appropriate design-basis fire scenarios for performance-based design of the sprinkler system (e.g., a frequent but low severity fire), compartmentation (e.g., a moderate severity but less frequent fire), and passive protection of the structure (e.g., a maximum credible fire).

- Validated and verified tools for use in performance-based design practice to analyze the dynamics of building fires and their effects on the structural system that would allow engineers to evaluate structural performance under alternative fire scenarios and fire protection strategies. While considerable progress has been made in recent years in advancing the tools that will help to improve the fire-safe design of new structures and analyze conditions of existing structures, significant work remains to be done before adequate tools are available for use in routine practice.

- The technical basis to establish whether the construction classification and fire rating requirements are risk-consistent. Specifically, it is not apparent how the current height and area tables in building codes consider the technical basis for the progressively increasing risk to an occupant on the upper floors of tall buildings that are much greater than 200 ft in height. The maximum fire rating in current codes applies to any building more than about 12 stories in height.

- Sprinklers improve safety in most common building fires and prevent them from becoming large fires. The technical basis to establish the “sprinkler trade-off” in current codes, considering fire safety risk factors such as: (1) the complementary functions of sprinklers and fire-protected structural elements, (2) the different design-basis fire scenarios for which each system is designed to provide protection, and (3) the need for redundancy should one system fail to function as intended is not available. The sprinkler trade-off provides an economic incentive to encourage installation of sprinklers by allowing a lower fire rating for sprinklered buildings.

- The design of egress systems to achieve a target performance (e.g., evacuation rate or time) for a given occupant population by adequately considering travel distance, remoteness requirements, and human factors such as occupant size, stairwell environmental conditions, visibility, and congestion.

Building Practices. While the PANYNJ entered into agreements with the NYC Department of Buildings in the 1990s with regard to conformance of PANYNJ buildings constructed in New York City to the NYC Building Code, the PANYNJ did not yield jurisdictional authority for regulatory and enforcement oversight to the NYC Department of Buildings. The PANYNJ was created as an interstate entity, under a clause of the U.S. Constitution permitting compacts between states, and is not bound by the authority of any local, state, or federal jurisdiction.

The architect is responsible for specifying the fire protection in current building practice. The structural engineer is not required to evaluate and certify that the passive fire protection is adequate to protect the structural system. In accordance with established practice, the structural engineer was not responsible for the passive fire protection in the design of the WTC tower structures. In addition, there is no requirement to involve a fire protection engineer in the design and evaluation of a building's fire protection system. In some cases, architects retain fire protection engineers to assist with the fire protection design for a building. There are only a few academic degree programs or continuing education programs that qualify engineers (or architects) to evaluate the fire performance of structures. The current state-of-practice is not sufficiently advanced for engineers to routinely analyze the performance of a whole structural system under a prescribed design-basis fire scenario.

Approach to Recommendations

In the United States, state and local governments are responsible for promulgating and enforcing building and fire regulations. With some exceptions, the state and local regulations are based on national model building and fire codes developed by private sector organizations. The model codes, in turn, reference voluntary consensus standards developed by a large number of private sector standards development organizations (SDOs) accredited by the American National Standards Institute (ANSI).

NIST is a non-regulatory agency of the U.S. Department of Commerce. NIST does not set building codes and standards, but provides technical support to the private sector and other government agencies in the development of U.S. building and fire practices, standards, and codes. NIST recommendations are given serious consideration by private sector organizations that develop national standards and model codes – which provide minimum requirements for public welfare and safety.

The NIST building and fire safety investigation of the WTC disaster has not yet formulated recommendations. However, in formulating its recommendations, NIST will consider the following:

- Findings from the first three independent investigation objectives related to building performance, evacuation and emergency response, and procedures and practices.
- Whether findings relate to the unique circumstances surrounding the terrorist attacks of September 11, 2001, or to normal building and fire safety considerations, including evacuation and emergency response.
- What technical solutions are needed, if any, to address potential risks to buildings, occupants, and first responders, considering both identifiable hazards and the consequences of those hazards?
- Whether the risk is in all buildings or limited to certain building types (e.g., height and area, structural system), buildings that contain specific design features, iconic/signature buildings, or buildings that house critical functions.

NIST urges organizations responsible for building and fire safety at all levels to carefully consider the interim findings contained in this report. NIST welcomes comments from technical experts and the public on the interim findings presented herein. Comments can be sent by e-mail to wtc@nist.gov, facsimile to (301) 975-6122, or regular mail to WTC Technical Information Repository, Stop 8610, 100 Bureau Drive, Gaithersburg, MD 20899-8610.

In its final report, a draft which is expected to be released in December 2004, NIST will recommend appropriate improvements in the way buildings are designed, constructed, maintained and used. It will be important for those recommendations to be thoroughly and promptly considered by the many organizations responsible for building and fire safety. As part of NIST's overall WTC response plan, the Institute has begun to reach out to these organizations to pave the way for timely, expedited consideration of recommendations stemming from this investigation. NIST also has expanded its research in areas of high priority need.

Chapter 1
INTERIM FINDINGS AND ACCOMPLISHMENTS

1.1 INTRODUCTION

In response to the terrorist attacks of September 11, 2001, the National Institute of Standards and Technology (NIST) initiated a formal federal building and fire safety investigation of the World Trade Center (WTC) disaster on August 21, 2002. At the same time, NIST also released the final plan for its investigation.

NIST has received large amounts of data and information related to the design, construction, operation, inspection, maintenance, repair, alterations, emergency response, and evacuation of the WTC complex. A summary is included in Section 2.1. NIST has received considerable cooperation from the Port Authority of New York and New Jersey (PANYNJ or Port Authority), the City of New York, the National Commission on Terrorist Attacks Upon the United States (9-11 Commission), designers, leaseholders, contractors, suppliers, insurers, news media, tenants, first responders, survivors, and families of victims.

NIST has received all of the essential information it needs for the WTC Investigation. NIST has a few requests for materials that are lost, currently pending, or not yet located; NIST is making efforts to re-create this information from various sources since much of it was lost when the buildings collapsed. NIST continues to pursue other materials that can clarify some aspects of the Investigation.

The interim findings summarized in this progress report are based on information contained in public updates issued by NIST in December 2002 and December 2003, a previous progress report issued in May 2003, and work completed since the last progress report. (Previous updates and progress reports may be found at http://wtc.nist.gov.)

NIST expects to release the draft of the final investigation report for public comment in December 2004. **NIST is not making any recommendations at this time. All recommendations will be made in the final report.** These interim findings and the working hypothesis for the collapse of the WTC towers and WTC 7 are subject to refinement or change as further information becomes available prior to release of the final investigation report. The final report will include any recommendations that NIST considers appropriate based on these and other findings yet to be made.

NIST welcomes comments from technical experts and the public on the interim findings presented herein. Comments may be sent by e-mail to wtc@nist.gov, facsimile to (301) 975-6122, or regular mail to WTC Technical Information Repository, Stop 8610, 100 Bureau Drive, Gaithersburg, MD 20899-8610.

The interim findings are presented in four subsections that relate to the investigation objectives listed below. They are then subdivided according to the technical areas to which they relate. NIST findings are presented in *italics* to distinguish them from narrative text throughout this section.

The stated objectives contained in the NIST investigation plan are:

1. To determine (a) why and how the WTC 1 and WTC 2 collapsed following the initial impact of the aircraft, and (b) why and how the 47-story WTC 7 collapsed.

2. To determine why the loss of life and injuries were so low or so high depending on location, including technical aspects of fire protection, occupant behavior, evacuation, and emergency response.

3. To determine the procedures and practices which were used in the design, construction, operation, and maintenance of the WTC buildings.

4. To identify, as specifically as possible, areas in national building and fire codes, standards, and practices that warrant revision.

1.2 COLLAPSE OF THE WTC TOWERS

Working Hypothesis

NIST is investigating possible collapse scenarios to establish the sequence of events that led to the collapse of the WTC towers following the initial impact of the aircraft. The objectives of the NIST analysis are to determine the most probable sequence of events from the moment of aircraft impact until the initiation of global building collapse and to identify the factors that have the strongest influence on the most probable sequence.

NIST has developed a working hypothesis to explain the collapse initiation of the WTC towers. The working hypothesis (summarized below and in Appendix Q) identifies the chronological sequence of major events as the WTC tower structures redistributed loads from structural element to structural element to accommodate the aircraft impact and subsequent fire damage until no further load redistribution was possible to maintain overall stability, thus, leading to collapse. The hypothesis:

- Is based on analysis of the available evidence and data, consideration of a range of hypotheses (including those postulated publicly by experts), and a newly enhanced understanding of structural and fire behavior.

- Is consistent with all evidence currently held by NIST, including photos and videos, eyewitness accounts, and emergency communication records.

- Allows for multiple load redistribution paths and damage scenarios for each building, currently under analysis.

- Will be further refined based on the results of NIST's continuing analyses to identify specific load redistribution paths and damage scenarios that are possible for each building, from which the most probable collapse sequence will be identified.

NIST welcomes comments from technical experts and the public on the working hypothesis. Among the key questions that NIST continues to investigate within the framework of the working hypothesis are the following:

- How and why did WTC 1 stand nearly twice as long as WTC 2 before collapsing (103 min versus 56 min), though they were hit by virtually identical aircraft?
- What were the relative roles of the perimeter and core columns[1] and the composite floor system,[2] including connections, in initiating the collapses?
- What was the post-impact condition of the fireproofing, especially the extent to which fireproofing may have been damaged due to aircraft impact?
- What factors related to normal building and fire safety considerations not unique to the terrorist attacks of September 11, 2001, if any, could have delayed or prevented the collapse of the WTC towers?

In evaluating the working hypothesis for the collapse of the WTC towers, NIST is also considering the following factors:

- The relative contributions of aircraft impact damage and subsequent fires;
- How safe each building was immediately after aircraft impact but before fire weakened the structures, i.e., to what extent the capacity of the buildings to carry design loads[3] was reduced;
- Whether the undamaged towers would have remained standing in a "maximum credible fire";[4] and
- The role compartmentation (i.e. areas divided by fire-rated walls) may have played, i.e., what would have happened if the floors had been separated into 7,500 or 10,000 ft^2 compartments with 1 h fire-rated partition walls or separations.

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[1] The perimeter columns were designed to carry both gravity and wind forces and acted together as a framed-tube system. The core columns were designed to carry only gravity loads and were not required to provide frame action.

[2] The composite floor truss system, which included long-span open-web bar joist elements, was designed to carry floor loads to the supporting core and perimeter columns. It also acted as a diaphragm that distributed wind forces to the perimeter columns of the framed-tube system and provided lateral stability to the perimeter columns.

[3] The design of the WTC towers was governed by gravity and lateral wind loads.

[4] A maximum credible fire for the WTC towers is assumed to have the following characteristics: the sprinkler system is compromised, overwhelmed, or not present; there is no active firefighting; combustible building contents averaging 10 psf (in the range of about 5 psf to 20 psf for conventional office buildings); floor-to-floor fire spread to next upper floor at 30 min or 60 min; and ventilation from windows broken by fire and a total of 50 ft^2 of air leakage between floors.

***Finding 1a.1:** The following chronological sequence of major events led to the eventual collapse of the towers; specific load redistribution paths and damage scenarios for each building continue to be refined:*

- Aircraft impact damage to perimeter columns, resulting in redistribution of column loads to adjacent perimeter columns and to the core columns via the hat truss;
- After breaching the building's exterior, the aircraft continued to penetrate into the buildings, damaging core columns with redistribution of column loads to other intact core and perimeter columns via the hat truss and floor systems;
- The subsequent fires, influenced by the post-impact condition of the fireproofing, weakened columns and floor systems (including those that had been damaged by aircraft impact), triggering additional local failures that ultimately led to column instability; and
- Initiation and horizontal progression of column instability resulted when redistributing loads could not be accommodated any further. The collapses then ensued.

Aircraft Impact. Buildings are not specifically designed to withstand the impact of fuel-laden commercial airliners. However, PANYNJ documents indicate that the impact of a Boeing 707 flying at 600 mph, possibly crashing into the 80th floor, was analyzed during the design of the WTC towers in February/March 1964. While NIST has not found detailed evidence of the analysis, the documents in NIST's possession state that the postulated aircraft collision would have resulted in only local damage that would not cause collapse or substantial damage to the WTC towers. The effect of the fires due to jet fuel dispersion and ignition of building contents was not considered. The loss of life in the immediate area of aircraft impact was recognized, but the loss of life due to the growth and spread of fires and smoke was not considered. Building codes do not require building designs to consider aircraft impact.

Role of the Hat Truss System

The purpose of the hat truss was to support gravity and wind loads on the antenna. It was not designed to resist lateral forces on the towers, and, in an undamaged state, it did not have a significant role in carrying gravity loads. Lateral loads due to wind were distributed to the framed-tube system via diaphragm action of the floor system. The hat truss was connected to each perimeter face at only four points, all at the same level (at the 108th floor just below the concrete floor slab). The 47 core columns were connected to diagonal elements, heavier transfer beams, or smaller beam elements of the hat truss. Most of the core columns extended to the roof level, but four core columns, which were only minimally connected to the hat truss, terminated at Floor 110. The hat truss provided minimal redistribution of loads (less than 10 percent) from perimeter columns to core columns. Most of the load redistributed due to aircraft impact damage occurred on the external face through vierendeel action.

***Finding 1a.2:** The two WTC towers withstood the initial impact of virtually identical aircraft (Boeing 767-200ER) during the terrorist attacks of September 11, 2001. The robustness of the perimeter frame-tube system and large dimensional size of the WTC towers helped the buildings withstand the aircraft impact. The WTC towers displayed significant reserve capacity, vibrating immediately following impact with amplitudes that were about half the amplitudes for design wind conditions expected by the building designers. WTC 2, which collapsed first and in about half the time as WTC 1, vibrated for over 4 min at an oscillation period nearly equal to that measured for the undamaged building. The lightly damped (about 1.2 percent of critical damping) oscillation had an initial amplitude of approximately 20 in at the roof level, where expected sway was about 3 ft to 4 ft under design wind conditions.*

Aircraft Impact Damage to Perimeter Columns. Based on the above information, structural damage to perimeter columns as a result of aircraft impact of the framed-tube system appears to have played a minimal role in initiating the collapse. Perimeter column bowing prior to collapse occurred on other faces (i.e., fire floors on the south face of WTC 1 and east face of WTC 2) that were not severed by the aircraft.

Aircraft Impact Damage to Core Columns. The core columns were designed to carry only gravity loads and not required to provide frame action. The aircraft trajectory at impact suggests damage to the core columns occurred as follows:

WTC 1—The aircraft was traveling about 450 mph and hit the tower near the center of the north face damaging Floors 93 to 99. The aircraft fully entered the core area and severed or damaged central core columns in the north-south direction. Aircraft and building debris accumulated in the remaining core area and south-side floor areas as contents were displaced from the point of impact.

WTC 2—The aircraft was traveling about 550 mph and hit the tower near the southeast corner of the building damaging Floors 77 to 85. Core columns to the south and east were severed or damaged. Aircraft and building debris accumulated in the core area and floor areas to the east and north.

Severed core columns redistributed their loads in three ways, depending on how many and which core columns were severed.

1. Isolated core columns were severed. Severed column and tributary floor loads, at and above the point of impact, were redistributed locally at each floor to adjacent intact core columns via core floor framing. This was limited by shear/bending capacity of floor-framing connections to adjacent columns.

2. Critical (e.g., corner) core columns and/or several other core columns were severed. The severed column and tributary floor loads, at and above impact floors, redistributed to intact core columns via the hat truss. Significant hat truss deflections may have occurred if there was adequate connection capacity since the severed core columns and the associated floors were hanging from the hat truss which was not designed to carry such loads. This was limited by the tensile capacity of bolted splices in the severed core columns, tensile/compression capacity of hat truss members, and tensile capacity of column connections to the hat truss.

3. Extent of core column failures precluded redistribution through the hat truss and/or exceeded redistribution capacity of the hat truss. Severed column and associated floor loads, at and above floors of impact, redistributed to intact core and perimeter columns via the core and composite truss floor system. Floors were subjected to combined bending and diaphragm action (e.g., consider the scenario of no core columns in the floor span direction to visualize this action). The overall capacity of the floors was limited by shear capacity of floor-to-column connections (including perimeter columns) and tensile/bending capacity of composite truss floor connections to core or perimeter columns. Significant sagging of the hat truss system may have occurred if its capacity was exceeded.

Relative Roles of Fires and Aircraft Impact

Finding 1a.3: *Fires played a major role in further reducing the structural capacity of the buildings, initiating collapse. The tower structures withstood the initial aircraft impacts and remained stable. While aircraft impact damage did not, by itself, initiate building collapse, it contributed greatly to the subsequent fires by:*

- Compromising the sprinkler and water supply systems;
- Dispersing jet fuel and igniting building contents over large areas;
- Creating large accumulations of combustible matter containing aircraft and building contents;
- Increasing air supply into the buildings (through broken windows and holes in the sides of the buildings, and between floors due to damaged floors, vertical shafts, and columns) that permitted significantly higher energy release rates than would normally be seen in ventilation-limited building fires, allowing the fires to spread rapidly within and between floors; and
- Damaging ceilings that enabled "unabated" heat transport over the floor-to-ceiling partition walls and to the floor trusses, spandrels, and tops of columns.

Finding 1a.4: *The jet fuel, which ignited the fires, was mostly consumed within the first few minutes after impact. The fires that burned for almost the entire time that the buildings remained standing were due mainly to burning building contents and, to a lesser extent, aircraft contents, not jet fuel.*

Thermal Effects on Columns and Floors. Some floors in WTC 2 experienced partial collapse due to aircraft impact. For example, partially collapsed floor slabs were visible on the east and north faces. This included failures at the edges with perimeter columns causing floor edge sagging. Based on photographs or videographs there is no visible evidence of partially collapsed floors in WTC 1.

Fires may have had the following thermal effects:

- Core columns and core floors may have been further weakened, with reduced ability to carry and/or redistribute load, causing such loads to be redistributed to other core and perimeter columns consistent with the residual reserve capacities of these columns and the transfer mechanisms (i.e., hat truss and floor system).
- The floor system may have been further weakened, either along the span of the floor system or localized at connections with columns. The weakening floor system may have pulled the perimeter columns inward (observed on the south face of WTC 1 and the east face of WTC 2 minutes prior to building collapse) and then initiated connection failures at perimeter or core columns.

Role of Fireproofing

The post-impact condition of the fireproofing played a key role in the structural response to fires. The post-impact condition of the fireproofing depends on the condition of the fireproofing prior to aircraft impact and the extent to which fireproofing was damaged due to aircraft impact. The fire-affected floors in WTC 1 had, in general, upgraded or thicker fireproofing (1.5 in. specified) while, in general, those in WTC 2 did not have upgraded fireproofing (0.5 in. specified).

- Perimeter columns may have been further weakened, with reduced ability to carry loads. Thermal effects could also cause inward bowing of perimeter columns due to differential temperatures between the inner and outer faces of the columns. The loads that could no longer be carried by the weakened columns would have been redistributed to adjacent perimeter columns.

Column Instability and Collapse Initiation. The perimeter columns were designed as part of a framed-tube system to carry both gravity and wind forces. Instability of perimeter columns resulted from a combination of (1) redistributed loads from the core columns via the floor system and possibly the hat truss, (2) inward bowing due to thermally-weakened and sagging floors, (3) increased unsupported length due to failed floors, and (4) thermal effects directly on the perimeter columns.

The instability of a few perimeter columns was observed to propagate across the entire face and around the corners just before or during collapse initiation. The initiation or spread of perimeter column instability also may have been facilitated by the hoop stress demand on the framed-tube system exceeding the capacity of the spandrels (horizontal steel plates) that tied the perimeter columns together (e.g., at the northeast corner of WTC 2).

The initiation of global collapse for both towers was first observed by the tilting of the sections above the impact regions of both WTC towers. The tilting and the propagation of column instability are synchronous processes that initiated global collapse. The tilting may have caused forces such as shear and torsion to spread the column instability laterally.

Issues Being Investigated. Over the next few months, NIST will continue to investigate the following technical issues and modify its working hypothesis as needed. Findings on these issues will be included in the final report.

- Aircraft impact damage to structural components, fireproofing, and hat truss connections.
- Distribution of aircraft/building contents.
- Thermal effects on core columns and core floors, especially extent of fires and growth history.
- Thermal effects on welded perimeter columns, especially temperature gradients on columns.
- Extent of load redistribution to intact core columns and their reserve capacity to accommodate thermal loads.
- Capacity of hat truss connections to perimeter columns, especially to meet the demands of aircraft impact and any torsional effects.
- Capacity of hat truss to accommodate the load redistribution from severed columns.
- Capacity of bolted splices in the severed core columns to carry loads to the hat truss.
- Relative magnitude of the load redistribution provided by the local core floor, hat truss, and the core-truss floor system for each tower.

- Axial/shear/bending capacity of floor connections to core and perimeter columns.
- Effect of localized fires on floor truss connections.
- Mechanisms to propagate instability laterally in the perimeter columns (e.g., shear and torsion forces induced by a rigid body movement)
- Capacity of spandrels, including splices, to carry shear transfer in the framed-tube system, especially at the corners.
- Role of bolted splices on instability of perimeter columns.
- Outward bowing of perimeter columns due to thermal expansion of floors.
- Effect of uneven floor thermal expansion on perimeter column instability due to potential biaxial bending.
- Comparison and reconciliation of working hypothesis with observed facts (photographs and videos, eyewitness accounts, emergency communication records).
- Examination of other possible or probable hypotheses.

Visual Observations

Photographic and video images of damage and fires in the WTC buildings are providing critical guidance for the investigation. NIST has collected and assembled the visual materials into a searchable computerized database. The database now contains well in excess of 6,000 photographs representing the work of more than 185 photographers, and 150 hours of video from most of the major media outlets and more than 20 individuals.

Based on an analysis of this visual evidence, NIST has identified significant events for WTC 1 and WTC 2 related to aircraft impact, fire development, and building damage, and work is progressing on WTC 7. As part of this analysis, NIST has developed detailed mappings and time-dependent visualizations for fire, smoke, and window conditions of the WTC towers, and similar efforts are nearing completion for WTC 7.

Finding 1a.5: *On the east face of WTC 2, what appears to be the 83rd floor slab was seen hanging across window openings over a large fraction of the 82nd floor. The object was observed in a number of photographs and videos very shortly after the plane strike and found to sag further prior to collapse. On the north face of WTC 2, shorter lengths of what appear to be the 81st, 82nd, and 83rd floor slabs were seen hanging through the windows in the floors below. There was no visible evidence of floors sagging in WTC 1. NIST continues to investigate what role aircraft impact and subsequent fires had on causing such floor failures.*

Finding 1a.6: *Several minutes (less than 10 min) prior to building collapse, inward bowing of about a quarter to a third of the perimeter columns was observed in photographs on the south face of WTC 1 and the east face of WTC 2 in regions that contained active fires.*

Finding 1a.7: *At 9:36 a.m. an occupant of WTC 2 called the New York City 911 telephone operator and reported that a floor in the 90's had collapsed underneath them and that they were now on the 105th floor. The NYPD aviation unit observed the following events with respect to WTC 2. At 9:49 a.m., they reported "large pieces" falling from WTC 2. At 9:58 a.m., it was reported that the south tower was coming down. With respect to WTC 1, at 10:06 a.m., an advisory was transmitted by the aviation unit that it wasn't going to take much longer before the north tower comes down and to pull emergency vehicles back from the building. At 10:20 a.m., the unit reported that the top of the (north) tower might be leaning. At 10:21 a.m., they reported that the north tower was buckling on the southwest corner and leaning to the south. At 10:27 a.m., a report was made that the roof was going to come down very shortly.*

Finding 1a.8: *The initiation of global collapse was first observed by the tilting of building sections above the impact regions of both WTC towers. WTC 1 tilted to the south (observed via antenna tilting in a video recording), and WTC 2 tilted to the east and south and twisted in a counterclockwise motion. The primary direction of tilt was around the weak axis of the core (north-south for WTC 1 and east-west for WTC 2). An earlier building performance study, performed by a private-public sector team with funding support from the Federal Emergency Management Agency (FEMA), concluded that the core failed first in WTC 1 based on vertical movement of the antenna observed in a video recording from due north that did not capture the antenna tilt due to the angle from which the video was shot. NIST is reevaluating this conclusion based on new visual information available from a different angle.*

Analysis of Recovered WTC Steel

NIST believes the collection of steel in its possession is adequate for purposes of analyzing the quality and properties of steel for the Investigation. NIST has 236 pieces of steel in its possession. The regions of impact and fire damage were emphasized in the selection of steel pieces for the investigation. As a result, NIST has all 14 specified steel grades for the exterior panels in the WTC towers, 2 specified grades that represent 99 percent of the core columns, and both specified grades for the steel trusses that comprised the composite floor truss system.

Finding 1a.9: *Analysis of steel recovered from the WTC towers, based on stampings on the steel and mechanical tests, indicates that the correct specified materials were provided for the specified elements. When these data were combined with pre-collapse photographic images of damaged steel, it was found that aircraft impacted pieces of steel recovered from WTC 1 were in the precise locations as specified in the design drawings.*

Finding 1a.10: *Metallography and mechanical property tests indicate that the strength and quality of the steel used in the towers was as specified, typical of the era, and likely met all qualifying test requirements. Further metallographic and fractographic analyses on the pieces from the impact zone are being considered.*

Finding 1a.11: *The room-temperature strength of the steel used in the towers met the relevant standards and, in many instances, exceeded the requirements by 5 percent to 10 percent. Ten different steel companies fabricated structural elements for the WTC towers, using steel supplied by at least eight different suppliers; four fabricators supplied the major structural elements from the 9th to the 107th floors. Work is ongoing to analyze the performance of the steel building components under impact and fire conditions up to initiation of global building collapse.*

Reconstruction of the Fires

NIST has completed three series of large-fire tests to enhance and validate the fire dynamics and thermal modeling tools being used in the WTC investigation, including:

- Single office workstation cubicle fire tests, based on descriptions of furnishings used in the WTC 1 office space, to generate a database on the thermo-physical properties of the materials for input to the fire dynamics simulation tool. The effects of debris and jet fuel on these fires were also investigated.

- Fire tests of multiple WTC workstation cubicles to validate the model predictions of the sensitivity of fire intensity, duration, and spread to the distribution and nature of the combustibles, effect of ventilation on the fire, and the effects of debris and jet fuel.

- Fire tests to measure the thermal environment (heat release and transfer rate to compartment gases) in a burning compartment and to establish a data set to validate the prediction of the temperature rise of structural steel components (similar to WTC steel trusses and columns, with and without fireproofing).

In addition, NIST has completed a series of experiments on ceiling tile systems similar to those present in the WTC towers. Shake table experiments were conducted to determine the magnitude of impulses that could result in damage to the ceiling tile systems, increasing the accessibility of the fire energy to the ceiling/floor membranes.

NIST has significantly enhanced the capabilities of computational tools for fire dynamics and thermal modeling and validated their predictions during the investigation. The tools can now be applied with greater confidence to recreate possible fires from the complex arrays of combustibles that existed in the WTC buildings, given initial damage conditions and descriptions of combustibles that have been collected by NIST. Key enhancements include: underventilated fire scenarios, charring materials such as those comprising much of the office furniture, fire spread, dimensional resolution in the vicinity of structural components, time-efficient computations for large simulations, and visualization of large data sets. These tools will help to improve the fire-safe design of new structures and analyze conditions of existing structures.

Finding 1a.12: *Unlike the jet fuel that was mostly consumed in only a few minutes, typical office furnishings were able to sustain intense fires for at least an hour on a given WTC floor. NIST has obtained generic and, in some cases, specific information about the furnishings in the WTC offices. In addition, NIST has obtained descriptions of the combustible contents of the aircraft. This includes cabin materials, aircraft components, and cargo bay contents. Based on a review of this information, NIST has found that:*

- *The typical WTC office floor, using modern workstation furnishings, had on average about 4 pounds per square foot (psf) of combustible materials on floors without unusual file rooms, film storage, etc. For conventional office buildings, the weight of combustibles varies from a combustible load commensurate with that in the WTC to about 10 psf to 20 psf when there are extensive bookcases, file storage, etc.*

- *The mass of the aircraft solid combustibles was significant relative to the mass of the building combustibles in the immediate impact region of each of the WTC towers.*

Following the experimental validation of the NIST Fire Dynamics Simulator for the WTC application, an extensive series of simulations of the fire floors in all three buildings is well underway. Input to the simulations includes the information gathered on the layouts of the floors, ventilation through the broken windows (as observed in the visual collection), the nature and loading of the combustible contents of the buildings, and preliminary (visual) estimates of damage to the towers from the aircraft impact. Alternative estimates of possible damage conditions in the interiors of the buildings are examined parametrically.

Consistent with available photographic and videographic evidence, these simulations have been able to capture the broad patterns of fire movement around the floors, with the flames in a given location lasting for about 20 min before spreading to adjacent, yet unburned combustibles. This spread is generally continuous due to the relatively even distribution of combustibles and the paucity of interior partitions and tends to be controlled by the air supply available through broken windows. There are some observed instances where fires persisted over longer durations in regions with accumulated combustible debris and other instances of sudden or interrupted fire spread. To the extent that these fires are locally of high intensity and duration and occur in a vicinity where they could contribute to structural weakening, they are being examined further via additional simulations.

Applying the 1968 NYC Building Code, the WTC towers were required to have 1 h fire-rated tenant separations, but the code did not impose any minimum compartmentation requirements (e.g., 7,500 ft^2) to mitigate the horizontal spread of fire in buildings with large open floor plans. The affected floors in the WTC towers were mostly open—with a modest number of perimeter offices and conference rooms and an occasional special purpose area. Some floors had two tenants, and those spaces, like the core areas, were partitioned (slab to slab). Photographic and videographic evidence confirms that even non-tenant space partitions (such as those that divided spaces to provide corner conference rooms) provided substantial resistance to fire spread in the affected floors. Exit access corridors had a 2 h fire-rating with 1 1/2 h fire-rated doors. Enclosures for vertical exits, exit passageways, hoistways, and shafts were required to be 2 h.

Finding 1a.13: *Laboratory experiments show that impulses like those estimated for the aircraft impact caused serious damage to the ceilings in the WTC towers. This is consistent with the accounts of survivors from floors below the impact region. This damage enables "unabated" heat transport over the floor-to-ceiling partition walls and to the steel trusses, spandrels, and tops of columns.*

Structural Response and Collapse Analysis

NIST has developed and adopted a comprehensive approach to identify the most probable of technically possible collapse sequences, from aircraft impact to collapse initiation. The approach:

- Combines mathematical modeling, well-established statistical and probability-based analysis methods, laboratory experiments, and analysis of photographic and videographic evidence.

- Allows for evaluation and comparison of possible collapse sequences based on different damage states, fire paths, and structural responses.

- Accounts for variations in models, input parameters, analyses, and observed events.

NIST has defined detailed requirements for this complex series of analyses, formulated detailed modeling approaches to capture important structural behavior, made significant progress in developing and evaluating the adequacy of the models, and obtained results from preliminary analyses using the models. The objective of the analyses is to simulate highly-complex failure modes at the component level, subsystem level, and over the entire structure due to aircraft impact and the subsequent fires. In many instances, NIST is testing the limits of current engineering software. Most such systems are used in general practice for design purposes, not for high-fidelity modeling and failure analysis of complex systems.

The computational models developed by NIST include:

- A detailed model of a typical truss-framed floor of the WTC towers with over 40,000 elements and 166,000 degrees of freedom.

- A detailed model of a typical beam-framed floor of WTC towers with over 12,000 elements and 35,000 degrees of freedom.

- A detailed global model of WTC 1 with over 80,000 elements and 218,000 degrees of freedom (with 17 flexible and other rigid diaphragm floors).

- A similar detailed global model of WTC 2 with over 78,000 elements and 200,000 degrees of freedom.

- A model of a typical turbofan engine of the Boeing 767-200ER aircraft with over 60,000 elements and 100,000 nodes.

- A comprehensive model of the Boeing 767-200ER aircraft, including engines, airframe, landing gear, fuel tanks, passenger cabin, and cargo bay, with over 530,000 elements and 740,000 nodes.

The first four models described above are being used to evaluate the baseline performance of the WTC towers under design gravity and wind loads. They also serve as *reference* models for other phases of the investigation involving analyses of aircraft impact damage and response of the thermally-insulated WTC structures to the subsequent fires.

Finding 1a.14: *The WTC tower structures represented an innovative structural system when they were built. The structural system incorporated many new and unusual features, including:*

- *First frame-tube framing system for a high-rise steel building.*

- *Composite floor system, using open-web bar joist elements, to provide lateral stability and diaphragm action.*

- *First extensive use of prefabricated perimeter panels (3 columns wide by 3 stories high) in steel construction with bolted butt-plate column splices.*

- *Uniform perimeter column geometry (14 in x 14 in cross-section) over most of the height of the 110-story buildings.*

- *First use of more than 14 different grades of specified steel in a tall building, with 14 grades specified for the uniform perimeter column geometry.*

- *Use of deep spandrel plates as beam elements connecting perimeter columns, enabling frame-tube action by providing lateral bracing around the structure.*

- *First use of wind tunnel testing to estimate the lateral wind loads in the design of a super tall building.*

- *First use of structural dampers to control dynamic motion in tall buildings, especially those due to winds (10,000 viscoelastic dampers were installed in each building connecting the floor trusses to the perimeter frame-tube system).*

- *Use of specially designed prefabricated panels to transfer forces at the chamfered corners of the frame-tube system.*

Finding 1a.15: *NIST has completed a preliminary stability analysis of the WTC towers. The findings from the preliminary analysis, if they remain viable upon further more detailed analysis, suggest that:*

- *For global instability of the WTC tower to occur under service loading conditions, five floors must have separated completely from all columns if the columns are at room temperature or four full floors must separate if the columns are uniformly heated to 600 ºC. Linear stability analysis indicates that some individual core columns begin buckling with fewer "failed" floors at both temperatures without significantly affecting global stability.*

- *For typical truss-framed floors under service loading conditions, if fifteen core columns are assumed severed due to aircraft impact, tension is induced in those columns by the floor immediately above the failure location of the columns. The tension force increases as more floor loads are picked up by the columns as they approach the hat truss at the roof level. The increase in tension load is limited by the tensile capacity of the column splice. When the tensile load exceeds the column splice capacity at a certain floor level, the splice fails, and all floors below the failed splices must redistribute their own loads directly to neighboring undamaged core columns. When fewer (only eight) core columns are assumed severed, the tension forces in the core columns are smaller due to the larger stiffness of the damaged floor area for eight severed columns, relative to that for 15 severed columns. The stiffer floor area redistributed more of the floor loads directly to neighboring undamaged core columns. The extent to which the severed core columns assist in transferring loads via the hat truss without failure of the column splices is sensitive to the relative magnitudes of the floor loads, column tension force, and column splice capacity.*

- *WTC 1 maintained stability after aircraft impact, with the highest stressed elements being the perimeter columns next to the region where columns and spandrels were severed on the north face of the tower. The analysis assumed eight columns in the core were severed due to aircraft impact. A "pushdown" analysis was used for evaluating structural stability accounts*

> *for geometric and material nonlinearities with plastic hinges. WTC 1 also maintained stability with remaining residual reserve capacity when additional perimeter columns were removed on the south face to represent the inward bowing observed in videos a few minutes prior to collapse. However, loss or weakening of additional core columns, weakening of additional perimeter columns, or loss of additional floors would be needed for global collapse of the tower to occur.*

NIST has completed a series of preliminary aircraft impact analyses using component-level models of tower perimeter and core columns with wing section and engine component models as impactors. These models were used to develop the simulation techniques required for the global analysis of the aircraft impacts into the WTC towers.

Finding 1a.16: *A 500 mph engine impact against an exterior wall panel results in a penetration of the exterior wall and failure of impacted perimeter columns. If the engine does not impact a floor slab, the majority of the engine core remains intact through the exterior wall penetration with a reduction in velocity between 10 percent and 20 percent. The residual velocity and mass of the engine after penetration of the exterior wall is sufficient to fail a core column in the event of a direct impact. Interaction with additional interior building contents prior to impact, or an indirect impact against the core column, could change this result.*

Finding 1a.17: *A normal impact of the exterior wall by an empty wing segment (toward the wing tip region) will produce significant damage to the perimeter columns, but not necessarily complete failure. This is consistent with photographs showing the exterior damage to the towers immediately after impact. Specific details of the damage depend on details of the impact orientation and locations of internal wing components such as control surface actuators and arms.*

Finding 1a.18: *Impact by a fuel-filled wing section (away from the wing tip toward the fuselage) results in extensive damage to the exterior wall panel, including complete failure of the perimeter columns. This is also consistent with photographs of the exterior damage. The resulting debris propagating into the building maintains the majority of the initial momentum of the wing prior to impact.*

NIST has completed detailed preliminary analyses of the response of a single floor truss assembly if it were subjected to a severe fire. These include the response of the truss and its seat connections (to columns) and knuckles (to provide composite action with the concrete floor slab) to service load conditions, uniformly increasing elevated temperatures in the steel, and increasing the temperature gradient in the concrete slab. The truss model includes all potential failure modes that may occur under loading and thermal conditions, though the actual sequence of failure may differ under other fire conditions.

Finding 1a.19: *NIST's preliminary analyses of a single floor truss assembly if it were subjected to a severe fire suggest the following sequence of events:*

- *The floor truss first experiences increasing vertical deflections at mid-span as it pushes (expands) outward and exerts a compressive lateral load on the exterior column. The exterior column begins to displace outward at the floor connection.*

- *Web diagonals begin to buckle at 340 ºC, the mid-span deflection continues to increase, but the horizontal displacement of the exterior column begins to decrease. The maximum horizontal displacement of the exterior column is approximately 0.7 in. when the diagonals begin to buckle. (The interior column is assumed to have no lateral displacements at the floor level, as it is braced by the core framing.)*

- *The shear connectors (steel-knuckle-to-concrete slab connections referred to as knuckles) at each end of the truss begin to fail as the steel and bottom surface of the slab reach 400 ºC, with such failures moving progressively inward from the truss ends. The failure of web diagonals and knuckles at the ends of the truss reduce the bending rigidity of the floor truss at the ends, further increasing the floor sag and decreasing the lateral outward force exerted on the columns.*

- *The truss bearing angle slips until the bolt is bearing against the edge of the slotted hole. The bolt shears off at the interior seat connection at approximately 500 ºC. The floor truss sag increases to 20 in. when the bolt fails.*

- *The interior end of the reinforced slab continues to carry vertical loads as the truss bearing angle continues to slip. At 560 ºC, the exterior column begins to be displaced inward as the floor truss continues to sag and exert vertical and horizontal tensile loads.*

- *At 650 ºC, the truss slides off the interior seat, followed by the gusset plate fracture at the exterior connection at 660 ºC.*

NIST has developed a rigorous technical approach to evaluate the role fireproofing conditions may have played in the collapse of the WTC towers, considering:

- Specified spray-applied fire resistive material (SFRM) and thicknesses for the various structural components.

- The as-built condition of the fireproofing prior to September 11, 2001, including the average SFRM thicknesses applied to different structural components, and variability in the thickness along the length of components.

- The mechanical and thermal properties of the fireproofing materials, including adhesive and cohesive bond strengths, and temperature-dependent heat capacity and thermal conductivity.

- The extent to which fireproofing may have been damaged due to aircraft impact via debris impact and local deformation/acceleration of structural components.

Finding 1a.20: *Available records suggest that the fireproofing of the columns, beams, and spandrels of the WTC towers was not a subject of concern to the building owner and designers, while fireproofing of the floor trusses was the focus of continuous reassessment and revision.*

- *The WTC towers were identified as Occupancy Group E – Business, and classified as Construction Class IB in accordance with the 1968 New York City Building Code. This* *classification required that the columns and floor systems of the towers have a 3 h and 2 h fire endurance, respectively.*

- *The steel trusses that supported the floors of WTC 1 and WTC 2 were specified to be fireproofed with 1/2 in. of SFRM, although the technical basis for the selection of fireproofing material and its thickness are not known.*
- *In 1999, a decision was made to begin upgrading the fireproofing to a specified 1.5 in. thickness as tenant spaces became unoccupied. In general, the floor systems in WTC 1 subject to aircraft impact and subsequent fires had been upgraded by September 11, 2001; the affected floors in WTC 2 had not.*
- *The fire protection of a truss-supported floor system by directly applying spray-on fireproofing to the steel trusses was innovative and not consistent with prevailing practice at the time the WTC towers were designed and constructed. While the benefits of conducting a full-scale fire endurance test were recognized by the building designers, no tests were conducted on the floor system used in the WTC towers to establish a fire endurance rating.*

Finding 1a.21: *The response of a structural component to fires is sensitive to variability in fireproofing thickness along its length. Such variations can be random in nature or in some instances as stark as bare spots. In the case of random variations, given an average fireproofing thickness and a coefficient of variation, it is possible to identify an equivalent uniform thickness without variation that gives nearly the same time history of temperature rise and component structural response under thermal loads.*

- *For the original fireproofing in the WTC towers, the as-applied fireproofing thickness (0.75 in. average and 0.4 coefficient of variation) on the floor trusses is thermally equivalent to a uniform thickness of 0.6 in. with no variation. This uniform thickness is greater than the specified minimum thickness of 0.5 in.*
- *For the upgraded fireproofing in some floors of the WTC towers, the as-applied upgraded fireproofing thickness (2.5 in. average and 0.24 coefficient of variation) is thermally equivalent to a uniform thickness of 2.2 in. with no variation. This uniform thickness is greater than the specified minimum thickness of 1.5 in.*
- *Thus, it is possible to evaluate if the as-applied average fireproofing thickness and variation on a component is thermally equivalent to a uniform thickness that is at least equal to the minimum fireproofing thickness specified on that component.*
- *For a specified fireproofing thickness, it is also possible to recommend an as-applied average thickness, given the expected variability (coefficient of variation) in quality of fireproofing application.*

NIST is currently examining an alternative performance criterion for determining the equivalent thickness based on restrained conditions to confirm the above finding.

Finding 1a.22: *Based on simplified analytical models, it was found that acceleration of a structural element, on the order of 100 times the acceleration due to gravity (or 100 g), would be required to* *dislodge 1 in thick SFRM from a planar surface. Acceleration on the order of 150 g would be required to dislodge a similar thickness of SFRM from a 1 in. diameter bar. In both cases, SFRM cohesive and adhesive strength properties and densities were typical of those used in the WTC towers. Experiments are underway to verify the results of these simplified analyses. In addition, analytical studies are underway to estimate the magnitude of accelerations of the structural members due to aircraft impact, from which the regions where fireproofing may have been dislodged will be identified.*

1.3 COLLAPSE OF THE 47-STORY WTC 7 BUILDING

Working Hypothesis

A working hypothesis has been developed around four phases of the collapse of WTC 7 that were observed in photographic and videographic records: an initiating event, a vertical progression at the east side of the building, a subsequent horizontal progression from the east to the west side of the building, and global collapse. The working hypothesis will be revised and updated as results of ongoing, more comprehensive analyses become available. NIST welcomes comments from technical experts and the public on this working hypothesis.

Finding 1b.1: *The working hypothesis for the collapse of the 47-story WTC 7, if it remains viable upon further analysis, suggests that it was a classic progressive collapse that included:*

- *An initial local failure at the lower floors (below floor 13) of the building due to fire and/or debris induced structural damage of a critical column (the initiating event), which supported a large span floor bay with an area of about 2,000 ft^2;*
- *Vertical progression of the initial local failure up to the east penthouse, as large floor bays were unable to redistribute the loads, bringing down the interior structure below the east penthouse; and*
- *Horizontal progression of the failure across the lower floors (in the region of floors 5 and 7, that were much thicker than the rest of the floors), triggered by damage due to the vertical failure, resulting in disproportionate collapse of the entire structure.*

WTC 7 Steel

No steel from WTC 7 has been identified from the pieces of recovered WTC steel in NIST's possession. WTC 7 had two specified grades of steel for columns and beams and four grades of steel for cover plates used in built-up columns. The specified grades (ASTM A36, A572 Grades 42 and 50, and A588 Grades 42 and 50) of steel are readily available. Properties were estimated from available test data in the literature.

Visual Observations

Finding 1b.2: *The first exterior sign of structural failure in WTC 7 was the sinking of the east penthouse roof structure into the building. Photographic and videographic records taken from the north have provided information about the sequence of failure events and their relative times. Other key observations* *include window breakage along the east side of the north face, occurring almost simultaneously with the sinking of the east penthouse structure, an approximate 5 s delay before the other roof structures also sank into the building core, a second set of window breakage along the west side of the north face occurring simultaneously with the other roof structure movements, and the appearance of the entire north façade above the 13th floor dropping as an intact unit 8 s after the east penthouse movement was first detected.*

Finding 1b.3: *Witnesses reported structural damage to WTC 7 on its south face and southwest corner from WTC 1 debris. A multi-story gash that extended across approximately a quarter to a third of the south face, in the lower portion of the face, was reported by a number of individuals, though details vary. This damage extended to the core area as two elevator cars were reportedly ejected from the elevator shaft at floor 8 or 9. Reported damage to the southwest corner was confirmed visually in photographic records, which show approximately two columns and related floor areas missing from floors 8 to 18. Multiple photographic and videographic records also appear to show damage on the south face that started at the roof level and severed spandrels between exterior columns near the southwest corner for at least 5 to 10 floors. However, the extent and details of this damage have not yet been discerned, as smoke is present in the photographs.*

Finding 1b.4: *Fires were first observed in WTC 7 after WTC 1 collapsed. Fires, or evidence of fires, were observed initially on the south face and near the southwest corner on Floors 22, 29, and 30. Many of these fires appeared to burn out before 2:00 p.m. Around 2:00 p.m., fires were observed in photographic and videographic records to be burning across Floors 11 and 12 on the east face, from the south to the north. Around 3:00 p.m., fires were observed on Floors 7 and 12 along the north face. The fire on Floor 12 appeared to bypass the northeast corner and was first observed at a point approximately one third of the width of the building from the northeast corner, and then spread both east and west across the north face. Sometime later, fires were observed on Floors 8 and 13 with the fire on floor 8 moving from west to east and the fire on Floor 13 moving from east to west. At this time, the fire on Floor 7 appeared to have stopped progressing near the middle of the north face. The fire on Floor 8 continued to move east on the north face, eventually reaching the northeast corner and moving to the east face. Around 4:45 p.m., a photograph showed fires on Floors 7, 8, 9, and 11 near the middle; Floor 12 was burned out by this time. Interview responses indicate that there was no water in the standpipe system supplying the sprinklers in WTC 7.*

Fuel System for Emergency Power

NIST has reviewed the fuel system for emergency power in WTC 7. There were two 12,000 gal fuel tanks below the first floor loading dock and one 6,000 gal above ground tank on the first floor. These tanks supplied fuel to 275 gal day tanks on floors 5, 7, and 8, and a 50 gal day tank on floor 9. In addition, there were two 6,000–gal tanks located below the first floor loading dock with pressurized pipes leading to floor 5.

Floor 5 did not have any exterior windows but it did have exhaust vents for generators near the south and north corners of the building. Any fires that may have burned on this floor would not have been visible in photographic or videographic records, except for smoke at the exhaust vents, which was not observed. The large opening created by the reported gash in the south face may have so altered the air flow on Floor 5 as to vent any smoke generated on this floor out the south face of the building, where overall smoke conditions prevented photographs or other observations. However, there was a pressurized fuel distribution system on the south, west and north floor areas. Given the variability of damage descriptions for the south face from WTC 1 debris impact, Floor 5 is considered a possible fire initiation location, subject to further data and/or analysis on building conditions that improve knowledge of fire conditions in this area.

Finding 1b.5: *The owner of the two 6,000 gal tanks supplying 5th floor generators through a pressurized piping system contracted with an environmental mitigation firm to recover any remaining fuel and to determine the extent of any contamination from fuel leakage from these tanks several months after the collapse. They reported that the tanks had been damaged by debris and were empty. No residual petroleum product or sludge was found in the tanks or piping. Examination of the gravel below the tanks and the sand below the slab on which the tanks were mounted showed some fuel contamination, but none was found in the organic marine silt/clay layer below. Witnesses also reported that the two 6,000 gal fuel tanks were always kept full for emergencies and were full that day. This finding allows for the possibility, though not conclusively, that the fuel may have contributed to a fire on Floor 5.*

1.4 EVACUATION AND EMERGENCY RESPONSE

Buildings are not designed for fire protection and evacuation under magnitude and scale of conditions similar to those caused by the terrorist attacks of September 11, 2001. Prior evacuation and emergency response experience in major events did not include the total collapse of tall buildings such as the WTC towers and WTC 7 that were occupied and in everyday use. Recent experience with major tall building fires suggests that they typically result in burnout conditions, not global building collapse. The intent of building codes is for buildings to withstand design loads without *local* structural collapse until the occupants can escape and the fire service can complete search and rescue operations. The load conditions induced by aircraft impact and the extensive fires on September 11, 2001, which triggered the collapse of the WTC towers, fall outside the norm of design loads considered in building codes.

NIST is interested in determining what factors related to normal building and fire safety considerations, if any, could have saved additional WTC occupant lives on September 11, 2001, or could have minimized the loss of life among the ranks of first responders. This is being accomplished by addressing the following key questions related to occupant behavior, evacuation, and emergency response:

- How did the evacuation technologies and practices affect the resulting fatalities and injuries?
- How did the first responder technologies and practices affect the resulting fatalities and injuries?
- How did the command, control, and communication systems support the activities of the first responders?
- What were the design, capabilities, and performance of the installed active fire protection systems (i.e., fire alarm, sprinkler, and smoke management systems)?
- What were the physical conditions within the buildings associated with occupant safety, tenability, and emergency responder operations?

- How did building design features affect egress and emergency access?

NIST is using multiple sources of data to investigate occupant behavior, evacuation, and emergency response. These sources include:

- Existing published first-person accounts of WTC evacuation; over 725 accounts collected and analyzed.
- Communication tapes from the PANYNJ and NYPD; 1,000-plus hours of taped recordings.
- Filings with the Occupational Safety and Health Administration by survivors and families of victims; about 60 written statements.
- Documents from the PANYNJ, FDNY, NYPD, and others on design of egress and emergency communication systems; WTC evacuation history; WTC evacuation planning and drills; emergency response preparedness and operational data.
- Photographic and videographic data on occupant behavior, evacuation, and emergency response.
- First-person data collection from WTC survivors, current and retired first responders, and families of victims.
- New York City 911 tapes and logs, and transcripts of about 500 interviews with FDNY employees involved in WTC emergency response activities.

Occupant Behavior and Evacuation

NIST is documenting occupant behavior and evacuation efforts by gathering and analyzing information about:

- Evacuation systems, emergency communications, and human factors;
- Occupant location, evacuation experience, and observed building conditions; and
- Interaction between occupants, first responders, and the buildings.

NIST has completed first-person interviews of nearly 1,200 WTC occupants and first responders to collect data on occupant behavior, evacuation, and emergency response, including:

- 803 telephone interviews with occupants of WTC 1 and WTC 2;
- 228 face-to-face interviews of WTC occupants and families of victims, including 28 near the floors of impact in both WTC towers, 33 persons with responsibility within the buildings, 15 who were in elevators, 13 who had a disability, 8 family members of victims, and 7 occupants from WTC 7; further, 8 interviews were conducted with key personnel present inside WTC 7;

- 108 face-to-face interviews of first responders, including 68 from FDNY, 24 from NYPD, 13 from the Port Authority Police Department (PAPD) and other PANYNJ safety and communications personnel, and 3 other building security and fire safety personnel;

- Six focus groups, including a group each from WTC 1 and WTC 2, maintenance and security personnel, floor wardens, people near/above the floor of impact, and mobility-challenged survivors.

Based on information and data gathered during these interviews with *surviving occupants:*:

Finding 2.1: *It is estimated that 17,400 occupants (± 1,200) were present in the WTC towers on the morning of September 11, 2001. The initial population of each tower was similar: 8,900 (± 750) in WTC 1 and 8,500 (± 900) in WTC 2. Of those present on September 11, 2001, 16 percent were also present during the 1993 bombing.*

Finding 2.2: *The average age of surviving occupants of the WTC towers was mid-forties, with a range of ages from their early twenties to mid-seventies. Occupants were twice as likely to be male (65 percent for WTC 1 and 69 percent for WTC 2) as female.*

Finding 2.3: *Two-thirds of WTC 1 surviving occupants had started working in the building during the previous four years (1998-2001), while half of WTC 2 occupants had begun working there during the same time period. The median residence time in WTC 1 was 2 years, while the median in WTC 2 was 3 years. In WTC 1, 4 percent of the occupants had worked in the building since 1975, while there was only one such respondent in WTC 2.*

Finding 2.4: *About 6 percent of the surviving occupants reported a pre-existing limitation to their mobility. These limitations included obesity, heart condition, needing assistance to walk, pregnancy, asthma, being elderly, chronic condition, recent surgery or injury, and other.*

Finding 2.5: *Overall, 7 percent of the surviving occupants reported having special knowledge about the building. These included fire safety staff, floor wardens, searchers, building maintenance, and security staff. Searchers assist the floor wardens in facilitating evacuation.*

Finding 2.6: *Approximately 87 percent of the WTC tower occupants, including more than 99 percent of those below the floors of impact, were able to evacuate successfully. Two-thousand one- hundred fifty-nine building occupants (1,560 in WTC 1 and 599 in WTC 2) and an additional 433 first responders, including security guards, were reported to have lost their lives that day. This does not include aircraft passengers and crew or bystanders.*

Rough initial estimates suggest that about 20 percent or more of those who were in the WTC towers and lost their lives may have been alive in the buildings just prior to their collapse. This estimate—which will be refined as analysis of the data is completed—assumes that nearly all of the first responders and 76 building occupants below the floors of impact but none of the people at or above the floors of impact who may have been alive. Below the floors of impact, there were 72 fatalities reported in WTC 1 and four fatalities reported in WTC 2, not including first responders. It is estimated that were a total of 2,592 building occupants and first responders who were in the WTC towers and lost their lives.

Finding 2.7: *Two-thirds of the surviving occupants reported having participated in a fire drill in the 12 months prior to September 11, 2001, while 17 percent reported that they received no training during that same period. Of those participating in fire drills, 93 percent were instructed about the location of the nearest stairwell. Overall, slightly over half of the survivors, however, had never used a stairwell at the WTC prior to September 11, 2001.*

Finding 2.8: *Overall, about 7,900 surviving occupants evacuated WTC 2 in 73 min (i.e., from the instant the WTC 1 was struck by aircraft until WTC 2 collapsed) while about 7,500 survivors evacuated WTC 1 in 103 min. Thus, the overall evacuation rate in WTC 2 (108 survivors per min) was about 50 percent faster than that in WTC 1 (73 survivors per min). Functioning elevators allowed many survivors to evacuate WTC 2 prior to aircraft impact.*

- *After the first airplane struck WTC 1 and before the second airplane struck WTC 2, the survivors in WTC 2 were twice as likely as those in WTC 1 to have already exited the building (41 percent versus 21 percent). The rate of evacuation completion in WTC 2 was twice the rate in WTC 1 during that same period. The elevators in WTC 2 were functioning at this time, while most of those in WTC 1 were not and survivors could only use the stairways. The stairwells, with partition wall enclosures that provided a 2 h fire-rating but little structural integrity, were damaged in the region of the aircraft impacted floors.*

- *Soon after WTC 2 was struck by the airplane until about 20 min before each building collapsed, the survivors in WTC 2 and WTC 1 had exited at about the same rate (the prior evacuation rate of WTC 1). Most of the elevators in both towers were not functioning at this time, and survivors could only use the undamaged stairways.*

- *During the last 20 min before each building collapsed, the evacuation rate in both buildings had slowed considerably to about one-fifth the immediately prior evacuation rate. This suggests that* for those seeking and able to reach and use undamaged exits and stairways, *the egress* capacity *(number and width of exits and stairways) was adequate to accommodate survivors.*

NIST has utilized existing computer egress models to better understand the evacuation experience on September 11, 2001. While these models were developed using data not indicative of the WTC buildings or events, they can provide some perspective into the relative magnitudes of evacuation times for phased evacuation (as the buildings are designed) and full evacuation of occupants. Three full evacuation scenarios are considered: a typical full capacity building evacuation assuming the WTC tower is fully occupied—with one case considering only tenants and another case considering both tenants and visitors; a full capacity building evacuation of each WTC tower with aircraft impact damage; and a September 11, 2001, capacity evacuation from a WTC tower.

NIST is using two classes of egress models in order to frame the evacuation questions: (1) partial behavior: simulates occupant movement and limited behavioral rules by including delay times, smoke effects, and occupant characteristics; and (2) behavioral: simulates movement and more comprehensive evacuation decisions and activities.

Finding 2.9: *Preliminary results from application of existing computer egress models for a full capacity evacuation of a single WTC tower with 25,000 occupants and visitors indicate a movement time of 2 h* *and 15 min. This is a minimum time estimate since the simulation assumed that there was no survivor delay, continual movement on the stairs, and no damage to the egress system. It was also assumed that elevators were not available. The egress model estimate for a September 11, 2001 capacity evacuation under the same assumptions is about 50 min, which is 2.5 times less than the time estimate for evacuating 25,000 people.*

Finding 2.10: *Given that the actual evacuation time on September 11, 2001 was about 100 min without elevator use, a full capacity evacuation of each WTC tower with 25,000 people would have required about 4 h (or 2.5 times 100 min). To achieve a significantly faster total evacuation at full capacity would have required increases in egress capacity (number and width of exits and stairways).*

Finding 2.11: *Ingress/egress was a tremendous physical challenge for first responders and many occupants; inadequate footwear presented a mobility challenge, particularly for many women. Many people were left shoeless, and their discarded shoes often littered the stairwells.*

NIST also has studied the possibility of roof evacuations from the WTC towers.

Finding 2.12: *A preliminary evaluation indicates that the PANYNJ's evacuation procedures did not include a plan to provide roof rescue for occupants trapped in a building incident at the WTC site. The standard policy was to keep the doors to the roof locked with a key being required to gain roof access. No fire safety procedures explicitly called for opening these doors, including anyone on a "key run." Instead, the standard occupant evacuation procedures and drills required the use of stairwells to exit at the bottom of the WTC towers. The PANYNJ reports that it never advised tenants to evacuate upward.*

Finding 2.13: *There were at least two instances reported on September 11, 2001, where roof access was found to be locked in both WTC towers. In the case of WTC 1, a decedent had called and informed a parent that they had tried to get to the roof and found the door locked. In the case of WTC 2, a decedent had called and informed a spouse that he had tried to get to the roof and found the door locked.*

Finding 2.14: *At least one case was reported on September 11, 2001, where a PANYNJ employee, trapped on Floor 105 of WTC 2, was instructed on the radio (PANYNJ Channel Y) by another PANYNJ employee to go to the roof of the building. The trapped occupant was unable to walk down the stairs, or go to the roof as instructed.*

Finding 2.15: *The NYPD aviation unit arrived at the WTC site soon after WTC 1 was attacked. Despite repeated attempts to examine the possibility of roof rescue, smoke and heat conditions at the top of the WTC towers prevented the conduct of safe roof evacuation operations. A helicopter, attempting to inspect the roof condition and determine if occupants were on the roof, experienced engine temperature increase as it approached WTC 1.*

Finding 2.16: *NYPD has an aviation-training manual to guide roof rescue in high-rise emergencies. The manual is not specific to the WTC. Considering the capacity of typical helicopters and travel times, it is not clear what fraction of the large number of occupants could have been evacuated from the WTC towers prior to their collapse had roof rescue been possible on September 11, 2001.*

The analysis of the first-person accounts collected from occupants by NIST and the evacuation modeling work is ongoing, and NIST will report its additional findings on occupant behavior and evacuation at a later date. That report will incorporate results from the analysis of previously published first-person accounts provided to the media by survivors and information released to the public by the 9-11 Commission.

Emergency Response and Communications

NIST's investigation of fire service technologies and guidelines, and more broadly the emergency response related to firefighting and evacuation on September 11, 2001, seeks to:

1. Document what happened during the emergency response to the attacks on the WTC up until the collapse of WTC 7;

2. Identify issues that need to be addressed in practices, standards, and codes;

3. Identify alternative practices and/or technologies that may address these issues; and

4. Identify technologies and guidelines to advance the safety of first responders in tall building emergencies.

Three FDNY suitcase-based, magnetic Command Boards were set up at the incident site. The unit identification and assignment for each unit that arrives at the scene is written on a magnetic chip and placed on the board. Information related to the location and activities of the units once they are on site are also recorded. One Command Board was set up at the original Incident Command Post in the lobby of WTC 1. The WTC 1 lobby Command Post became an Operations Post when the Incident Command Post moved outside. However, the original Command Board remained in place inside WTC 1 when the Incident Command Post was moved outside. The second Command Board was set up at the fire department's Operations Post in the lobby of WTC 2, and the third Command Board was set up at the new Incident Command Post established by the Chief of Department on West Street in front of the World Financial Center 2 building.

Finding 2.17: *The FDNY suitcase-based, magnetic Command Board system that was generally adequate for normal fire and rescue operations was not adequate for handling the massive operations that were necessary as a result of the terrorist attacks on the WTC. Interviews with FDNY personnel indicated that some FDNY personnel and others entered the towers and were not recorded on the Command Boards. This resulted in the lack of important command information. In addition, each of these Command Boards was damaged, and all were lost with the collapse WTC 2. There was no back-up capability for the Command Boards, and all information related to command, control, and accountability was lost.*

NIST has completed a partial analysis of emergency responder communications including:

- Digital copies of the audio communications tapes recorded by the PANYNJ, including communications from emergency response personnel, maintenance personnel, PAPD personnel, and a recording of the FDNY's city-wide high-rise Channel 7 (PAPD's Channel 30) radio repeater that was located at the WTC; and

- Audio tapes copied from original NYPD communications tapes, including NYPD internal department operations.

FDNY communications recordings from the WTC location on September 11, 2001, are not available because the primary field communication truck was in the shop for repairs, and a backup field communications van was used in its place. The backup van did not have the capability to record the on-scene incident command or tactical communications and was destroyed when the WTC towers collapsed.

The best record of radio communications reflecting fire department operations available to NIST came from the FDNY Channel 7/PAPD Channel 30 tape and first person accounts provided by FDNY personnel during their interviews. The tape provides limited information on FDNY communications and operations at the incident, but it does provide insight into FDNY operations inside WTC 2. FDNY Channel 7/PAPD Channel 30 was a city-wide channel designated by FDNY for use in high-rise building operations. The PANYNJ installed the radio repeater system at the WTC for use by FDNY after the 1993 bombing.

Finding 2.18: *The following findings have been drawn from the first-person interviews with emergency responders regarding telephone communications:*

- *Before the attack occurred, both the landline and cellular systems appeared to be working normally.*
- *Only moments after the first aircraft impacted WTC 1, the landline and cellular telephone systems were stressed by increased caller volume that made it difficult to get messages through. This condition continued for many hours following the attack.*
- *Telephone calls from the WTC to the 911 emergency operators, and statements from various individuals interviewed, show that even though WTC 1 and WTC 2 were severely damaged by aircraft impact and fires, many of the landline telephones in the buildings continued to work up until the collapse of WTC 2.*
- *After the collapse of WTC 2, a number of cellular phone systems were not functional in the area of lower Manhattan.*
- *After the collapse of WTC 2, there were still some landline telephones working within the city block areas adjacent to the WTC site.*

NIST has developed a preliminary chronology, based on analysis of selected communications messages, to provide information concerning (1) dispatch and arrival of emergency response units, (2) evacuation, (3) emergency response operations, (4) emergency response communications, and (5) observations of building conditions.

Finding 2.19: *The following findings have been drawn from the analysis of the emergency responder communication tapes:*

- *After the first aircraft struck WTC 1, there was an approximate factor of 5 peak increase in traffic level over the normal level of emergency responder radio communications, followed by an approximate factor of 3 steady increase in the level of subsequent traffic.*

- *A surge in communications traffic volume made it more difficult to handle the flow of communications and delivery of information.*

- *Roughly a third to a half of the radio messages transmitted during these radio traffic surge conditions were not complete messages or understandable.*

- *Preliminary analysis of the FDNY city-wide high-rise Channel 7 (PAPD Channel 30) recording indicates that the WTC site repeater was operating.*

- *Communications records and interviews with aviation unit personnel indicate that smoke and heat conditions on the top of the WTC towers prevented NYPD helicopters from conducting safe roof evacuation operations.*

- *NYPD aviation unit personnel reported critical information about the impending collapse of the WTC towers several minutes prior to their collapse. No evidence has been found to suggest that the information was communicated to all emergency responders at the scene.*

Finding 2.20: *Several FDNY personnel at the incident site did not think that the high-rise radio repeater was working. This is based on radio communications tests that were conducted by two chief officers working inside WTC 1 when the first command post was being set up in that lobby. This radio communications test was recorded on the FDNY Channel 7/PAPD Channel 30 tape. Following this radio test, a chief officer involved in the test chose to use different channels for command and tactical communications during the disaster. However, as FDNY operations increased in WTC 2, it was determined by FDNY members that the high-rise repeater was functioning, and use of the channel developed.*

Finding 2.21: *While the preliminary NIST analysis indicates that the repeater was operating, there also appears to have been some type of malfunction with the communications equipment that was detected, but not identified, by FDNY officers during the initial test. Three hypotheses are being investigated related to this malfunction: (1) damage to the repeater antenna system located in WTC 5, e.g., changing its direction, (2) failure of the radio handset located at the fire command desk in the lobby of WTC 1, and (3) the volume of the radio hand set not being turned up.*

NIST continues to evaluate the repeater system and its operations, as well as the handheld radios, which were used on September 11, 2001. The findings listed above will be updated and additional findings will be documented when the investigation is complete.

NIST has completed its review of the NYC 9-1-1 tapes and logs and the transcripts of about 500 interviews with FDNY employees involved in WTC emergency response activities. Analysis of this and other information is ongoing.

Finding 2.22: *Based on face-to-face interviews, NIST has determined that first responders— including key incident commanders—did not have adequate information (voice, video, and data) on and an overall perspective of the conditions in the WTC buildings and what was happening elsewhere at the WTC site; interagency information sharing was inadequate.*

NIST continues to analyze all of the data sources (documents, visual images, first-person accounts, and communications records) related to the emergency response activities at the WTC site, including deployment, evacuation, operations, and communication systems and protocols. A future report will document the findings from that analysis.

Fire History of the WTC Towers

NIST has completed a review of the history of post-occupancy fire incidents and identified significant fire incidents—those that exercised the fire suppression systems, specifically multiple sprinklers or one or more standpipes (with or without activation of at least one sprinkler).

The FDNY fire reports and fire investigation records indicate that in areas protected by automatic sprinklers, no fire activated more than three sprinklers. The design area for three sprinklers is a floor area of 63 m^2 (675 ft^2) in light hazard occupancy, such as a high-rise office building as specified in the National Fire Protection Association Standard for the Installation of Sprinkler Systems (NFPA 13).

Many of the fires that occurred were recorded as suspicious or unknown in cause, occurred during off-peak work hours, and involved materials such as trash or paper-based supplies. In cases where sprinklers were activated, the fire department records indicated that the sprinklers either extinguished the fire completely or aided in controlling the spread. In summary:

- 16 significant fires occurred in WTC 1, 2, and 7, with 12 in WTC 1, three in WTC 2, and one in WTC 7. Twelve of the 16 fires occurred between 6 p.m. and 4 a.m. when the number of occupants in the buildings was likely to be small.

- Of the 16 fires and their causes, five were labeled as unlisted or unclassified, six as suspicious or incendiary, two as discarded material, and three as an electrical failure or mechanical failure.

- Of the 16 fires, four were concentrated above the 100th floors and six fires were located in the basements. The other six were distributed throughout the rest of the buildings.

- 31 additional fires occurred in WTC 1 and WTC 2, which involved the use of one standpipe, with 23 in WTC 1 and eight in WTC 2.

- There is no known loss of life as a result of any of these fires (not including the 1993 bombing incident and September 11, 2001, terrorist attacks).

The following significant fires (not including the 1993 bombing incident and September 11, 2001, terrorist attacks) are noteworthy:

- February 14, 1975: Fire started on the 11th floor of WTC 1. Fire damage occurred on the 10th through the 19th floors. Approximately 9,000 ft^2 of 11th floor contents on the southeast corner was destroyed or damaged.

- April 19, 1980: Fire started on the 106th floor of WTC 1. Approximately 300 occupants from the Windows on the World restaurant on the 107th floor were evacuated.

- April 17, 1981: Fire started on the 7th floor of WTC 1. Approximately 1,500 occupants were evacuated from Floors 9 through 23.

Active Fire Protection Systems – Sprinkler Systems

Finding 2.23: *In WTC 1, 2, and 7, primary and secondary water supplies, fire pump size and locations, water storage tanks, and fire department connections provided multiple points of water supply redundancy. The potential for single point failure of the water supply to the fire sprinklers existed at each floor due to lack of redundancy in the sprinkler riser system that provided only one supply connection on each floor. As a result, the water supply to the sprinkler systems or a standpipe serving pre-connected hoses could be interrupted by routine maintenance needs (i.e., shutdown of the riser or standpipe) or by impairment due to deliberate acts to damage the sprinkler riser or standpipe systems. While this lack of redundancy may not have had an impact on September 11, 2001 because the sprinkler system was damaged by aircraft impact, it could have made a difference in other building emergencies.*

Finding 2.24: *Aided by the results of hydraulic modeling of a sprinkler system in WTC 1 and WTC 2—undamaged by aircraft impact and fully operational—the delivered water flow rate available from the automatic sprinkler systems was found to generally exceed the minimum requirements (by a considerable margin) for a high-rise office hazard classification in accordance with NFPA 13. In a number of cases, the amount of available water flow from sprinklers on specific floors was capable of protecting higher fire hazard classes than those associated with light hazard office buildings.*

Active Fire Protection Systems—Fire Alarm Systems

Finding 2.25: *The fire alarm system that was monitoring WTC 7 sent to the monitoring company only one signal (at 10:00:52 a.m. shortly after the collapse of WTC 2) indicating a fire condition in the building on September 11, 2001. This signal did not contain any specific information about the location of the fire within the building. From the alarm system monitor service view, the building had only one zone, "AREA 1." The building fire alarm system was placed on TEST for a period of 8 h beginning at 6:47:03 a.m. on September 11, 2001. Ordinarily, this is requested when maintenance or other testing is being performed on the system, so that any alarms that are received from the system are considered the result of the maintenance or testing and are ignored. NIST was told by the monitoring company that for systems placed in the TEST condition, alarm signals are not shown on the operator's display, but records of the alarm are recorded into the history file.*

Finding 2.26: *The resistance to failure of the fire alarm system communications paths between the fire command station and occupied WTC tower floors could have been enhanced if fiber optic communications cable had been used instead of copper lines. Extensive damage to the towers upon aircraft impact is likely to have cut and short-circuited the wiring of the alarm system network cables. If that occurred, communications between the distributed fire alarm panels, which are components of the integrated fire alarm system, would have been degraded and lost to certain panels depending on the location of those panels. Fiber optic cable is not susceptible to electric short-circuits and would have provided full communications with fire alarm system components, including voice communications systems, to the point where the cable was severed. Electric shorts in the voice communications disable that communication system over the entire cable length affected by the electric short-circuit. During initial engineering design for the fire alarm system in WTC 1 and WTC 2, the PANYNJ requested, but did* *not receive, approval of the City of New York for use of fiber optic communications cable in the system. The NYC code required copper wiring. As a result, ordinary copper wire communication cable was specified.*

A dedicated communications system for emergency responders, known as the “standpipe telephone” system, was installed in the stairwells of WTC 1 and WTC 2. To use the system, a compatible telephone handset was needed. Some firefighters that received handsets at the command post in the lobby of WTC 1 were interviewed as part of the Investigation. Every one of the firefighters interviewed indicated that they did not use the standpipe telephone communication system on September 11, 2001. Due to the loss of firefighters in WTC 2, there is no information about the use of the system in WTC 2.

Active Fire Protection Systems—Smoke Management System

The smoke management system in the WTC towers as designed and documented in the operation manuals consisted of a smoke purge mode using the components of the main HVAC (heating, ventilation, and air-conditioning) system. This system was intended to remove smoke and other gaseous combustion products from the fire area after a fire had been extinguished. This system was to be activated “manually” at the direction of FDNY.

Finding 2.27: *Based on the information reviewed, the smoke management systems were not activated during the fires on September 11, 2001. It was determined that the likelihood of these systems being functional in WTC 1 and WTC 2 was very low due to the damage inflicted by the aircraft impacts. In addition to the significant openings created in the building envelopes, the aircraft impacts are likely to have severed major vertical shafts through which ran electrical power supply and duct risers of the HVAC system, thereby causing the loss of power to the smoke management system air handlers and damage to the vertical HVAC duct risers used to provide smoke management (smoke purge).*

Finding 2.28: *The analysis of smoke flow in WTC 1 and WTC 2 on September 11, 2001, shows that HVAC ductwork was a major path for vertical smoke spread in the buildings. Fire dampers were installed in the systems, but not smoke dampers. Operational combined fire/smoke dampers in the HVAC ductwork on each floor would have provided a barrier to hot gas and smoke penetration into the vertical HVAC shafts in WTC 1 and WTC 2. However, smoke dampers were not available when the towers were built.*

Finding 2.29: *Modeling results show that in WTC 1 and WTC 2 stair pressurization systems would have provided minimal resistance to the passage of smoke had they been installed on September 11, 2001. While the existence of such systems was known when the WTC towers were built, the alternative smoke purge system used in the WTC towers was considered to be equivalent. Multiple stair doors being open for substantial periods of time due to occupant egress and stairway walls damaged by aircraft impact would have resulted in an inability to prevent smoke from entering stairwells.*

1.5 PROCEDURES AND PRACTICES

The 110-story WTC towers were among the world’s tallest buildings, while the 47-story WTC 7 represented a more typical tall building. These buildings provide case studies to document, review, and, if needed, improve the procedures and practices used in the design, construction, operation, and maintenance of tall buildings. This investigation objective is independent of other objectives which are focused specifically on the consequences of the attack on September 11, 2001, viz., the building collapses, evacuation, and emergency response. While some findings under this objective are directly relevant to the events of September 11, 2001, others are concerned with general building and fire safety procedures and practices.

NIST seeks to determine the building and fire safety procedures and practices that were used over the life of the WTC buildings and how well those procedures and practices conformed to accepted national building and fire safety practices, standards, and codes. The procedures and practices of interest to the investigation include those related to:

- Design and construction,
- New and innovative design features,
- New and innovative technologies and materials,
- Passive and active fire safety systems,
- Emergency access and egress systems, and
- Structural modifications, inspection, and maintenance.

To provide context for studying the specifications and criteria used for the WTC buildings, NIST has completed a preliminary comparison of building regulatory and code requirements. For the structural system, this comparison included the following building codes:

- New York City Building Code - 1968 edition
- New York State Building Code - 1964 edition
- Chicago Building Code - 1967 edition
- BOCA Basic Building Code - 1965 edition (a national model building code)
- New York City Building Code - 2001 edition

For the fire protection and egress system, the comparison included the five codes listed above plus the 1966 edition of the National Fire Protection Association Life Safety Code (NFPA 101).

Applicable Building Codes

Finding 3.1: *Although not required to conform to NYC codes, the PANYNJ adopted the provisions of the proposed 1968 edition of the NYC Building Code, more than three years before it went into effect. The 1968 edition allowed the PANYNJ to take economic advantage of less restrictive provisions compared* *with the 1938 edition that was in effect when design began for the WTC towers in 1962. The 1968 code:*

- Eliminated a fire tower[5] as a required means of egress;
- Reduced the number of required stairwells from 6 to 3 and the size of doors leading to the stairs from 44 in. to 36 in.;
- Reduced the fire rating of the shaft walls in the building core from 3 h to 2 h;
- Changed partition loads from 20 psf to one based on weight of partitions per unit length (that reduced such loads for many buildings including the WTC buildings); and
- Permitted a 1 h reduction in fire rating for all structural components (columns from 4 h to 3 h and floor framing members from 3 h to 2 h) by allowing the owner/architect to select Class 1B construction for business occupancy and unlimited building height.

Many of these newer requirements, instituted in the 1968 NYC Building Code, are contained in current codes.

Finding 3.2: *The NYC Department of Buildings reviewed the WTC tower drawings in 1968 and provided comments to the PANYNJ concerning the plans in relation to the 1938 NYC Building Code. The architect-of-record submitted to the PANYNJ responses to those comments, noting how the drawings conformed to the 1968 NYC Building Code. NIST continues reviewing documents to determine the level of review conducted by the NYC Department of Buildings and the six specific items identified in that review.*

Finding 3.3: *In 1993, the PANYNJ and the NYC Department of Buildings entered into a memorandum of understanding that restated the PANYNJ's longstanding policy to assure that its facilities in the City of New York meet and, where appropriate, exceed the requirements of the New York City Building Code. The agreement also provided specific commitments to the NYC Department of Buildings regarding procedures to be undertaken by the PANYNJ to assure that buildings owned or operated by the PANYNJ are in conformance with the Building Standards contained in the NYC Building Code. Some salient points included in this agreement and the 1995 enhancement to the agreement are:*

- *Each project would be reviewed and examined for compliance with the Code;*
- *All plans would be prepared, sealed, and reviewed by New York State licensed professional engineers or architects;*
- *The PANYNJ engineer or architect approving the plans would be licensed in the State of New York and would not have assisted in the preparation of the plans; and*

[5] A fire tower (also called a smoke-proof stair) is a stairway that is accessed through an enclosed vestibule that is open to the outside or to an open ventilation shaft providing natural ventilation that prevents any accumulation of smoke without the need for mechanical pressurization.

- *The person or firm performing the review and certification of plans for WTC tenants should not be the same person or firm providing certification that the project had been constructed in accordance with the plans and specifications.*

- *Variances from the Code, acceptable to the PANYNJ, would be submitted to the NYC Department of Buildings for review and concurrence.*

Finding 3.4: *In 1993, the PANYNJ adopted a policy providing for implementation of fire safety recommendations made by local government fire departments after a fire safety inspection of a PANYNJ facility and for the prior review by local fire safety agencies of fire safety systems to be introduced or added to a facility. Later that year, the PANYNJ entered into an agreement with FDNY which reiterated the policy adopted by the PANYNJ, recognized the right of FDNY to conduct fire safety inspections of PANYNJ properties in the City of New York, provided guidelines for FDNY to communicate needed corrective actions to the PANYNJ, assured that new or modified fire safety systems are in compliance with local codes and regulations, and required third-party review of such systems by a New York State licensed architect or engineer.*

Standard Fire-Resistance Tests

Finding 3.5: *Availability of code provisions with detailed procedures to analyze and evaluate data from fire resistance tests of other building components and assemblies to qualify an untested building element. Based on available data and records, no technical basis has been found for selecting the spray-applied fire resistive material (SFRM) used (two competing materials were under evaluation) or its thickness for the large-span open-web floor trusses of the WTC towers. The assessment of the fireproofing thickness needed to meet the 2 h fire rating requirement for the untested WTC floor system evolved over time:*

- *In October 1969, the PANYNJ directed the fireproofing contractor to apply 1/2 in. of fireproofing to the floor trusses.*

- *In 1999, the PANYNJ issued guidelines requiring that fireproofing be upgraded to 1-1/2 in. for full floors undergoing alterations.*

- *Unrelated to the WTC buildings, an International Conference of Building Officials (ICBO) Evaluation Service report (ER-1244), re-issued June 1, 2001, using the same SFRM recommends a minimum thickness of 2 in. for "unrestrained steel joists" with "lightweight concrete" slab.*

Finding 3.6: *Availability of code provisions that require the conduct of a fire resistance test if adequate data do not exist from other building components and assemblies to qualify an untested building element. Instead, several alternate methods based on other fire-resistance designs or calculations or alternative protection methods are permitted with limited guidance on detailed procedures to be followed. Both the architect-of-record (in 1966) and the structural-engineer-of-record (in 1975) stated that the fire rating of the floor system of the WTC towers could not be determined without testing. NIST has not found evidence indicating that such a test was conducted to determine the fire rating of the WTC floor system. The PANYNJ has informed NIST that there are no such test records in its files.*

NIST has awarded a contract to Underwriters Laboratories (UL) to determine the fire resistance rating of the WTC floor system under both as-specified and as-built conditions. The tests, which are expected to be conducted in August 2004, will provide the fire endurance ratings of typical WTC floor construction to evaluate three primary factors: (1) test scale, (2) fireproofing thickness, and (3) thermal restraint. The four tests will be performed as follows:

- 17 ft span assembly, thermally restrained, SFRM thickness of 1/2 in.
- 17 ft span assembly, thermally restrained, SFRM thickness of 3/4 in.
- 35 ft span assembly, thermally restrained, SFRM thickness of 3/4 in.
- 35 ft span assembly, thermally unrestrained, SFRM thickness of 3/4 in.

The first test represents current U.S. practice for establishing a fire endurance rating of a building assembly. The test assembly, fabricated to meet the design of the WTC steel joist-supported floor system, has a span of 17 ft. This span is typical of the floor assembly test furnaces used by the U.S. testing laboratories that routinely conduct the ASTM E119 test for the construction industry. As is common practice, the floor assembly will be tested in the thermally restrained condition. This test will be conducted at UL's Northbrook, Illinois, fire test facility. A second test will be identical except for the thickness of SFRM. The third and forth tests will be at twice the scale of the first two tests, with a span of 35 ft. This span represents a full-scale assembly of the 35 ft floor panel of the WTC floor system. The floor assembly for the third test will be thermally restrained as in the first two tests thereby allowing direct comparison for the determination of the effect of test scale on fire endurance rating. The fourth test will be conducted in the thermally unrestrained support condition which will allow direct comparison of the effect of thermal restraint on the fire endurance rating. The third and fourth tests will be conducted at the UL Canada fire test facility near Toronto.

Building Classification and Fire Rating

Finding 3.7: *Use of the "structural frame" approach, in conjunction with the prescriptive fire rating, would have required the floor trusses, the core floor framing, and perimeter spandrels in the WTC towers to be 3 h fire-rated, like the columns for Class 1B construction in the 1968 NYC Building Code. Neither the 1968 edition of the NYC Building Code which was used in the design of the WTC towers, nor the 2001 edition of the code, adopted the "structural frame" requirement. The "structural frame" approach to fire resistance ratings requires structural members, other than columns, that are essential to the stability of the building as a whole to be fire protected to the same rating as columns. This approach, which appeared in the Uniform Building Code (a model building code) as early as 1953, was carried into the 2000 International Building Code (one of two current model codes) which states: "The structural frame shall be considered to be the columns and the girders, beams, trusses and spandrels having direct connections to the columns and bracing members designed to carry gravity loads." The WTC floor system was essential to the stability of the building as a whole since it provided lateral stability to the perimeter columns and diaphragm action to distribute wind loads to the perimeter columns.*

Finding 3.8: *Availability of technical basis to establish whether the construction classification and fire rating requirements in modern building codes are risk-consistent with respect to the design-basis hazard and the consequences of that hazard. The fire rating requirements, which were originally developed* *based on experience with buildings less than about 20 stories in height, have generally decreased over the past 80 years since historical fire data for buildings suggested considerable conservatism in those requirements. However, for tall buildings, the likely consequences of a given threat to an occupant on the upper floors are more severe than the consequences to an occupant, say, on the first floor. It is not apparent how the current height and area tables in building codes consider the technical basis for the progressively increasing risk to an occupant on the upper floors of tall buildings that are much greater than 200 ft in height. The maximum required fire rating in current codes applies to any building more than about 12 stories in height. There are no additional categories for buildings above, for example, 40 stories and 80 stories, where different building classification and fire ratings requirements may be appropriate, recognizing factors such as the time required for stairwell evacuation without functioning elevators (e.g., due to power failure or major water leakage), the time required for first responder access without functioning elevators, the presence of sky lobbies and/or refuge floors, and limitations on the height of elevator shafts. The 110-story WTC towers, initially classified as Class IA based on the 1938 NYC Building Code, were classified as Class 1B before being built to take advantage of the provisions in the 1968 edition of the code. This re-classification permitted a reduction of 1 h in the fire rating of the components (columns from 4 h to 3 h and floor framing members from 3 h to 2 h).*

Fire Performance of Structures

Finding 3.9: *Structural design does not consider fire as a design condition, as it does the effects of dead loads, live loads, wind loads, and earthquake loads. Current prescriptive code provisions for determining fire resistance of structures—used in the design of the WTC towers and WTC 7— are based on tests using a standard fire that may be adequate for many simple structures and for comparing the relative performance of structural components in more complex structures. A building system with 3 h rated columns and 2 h rated girders and floors could last longer than 3 h or shorter than 2 h depending upon the performance of the structure as a 3-dimensional system in a real fire. The standard tests cannot be used to evaluate the actual performance (i.e., load carrying capacity) in a real fire of the structural component, or the structure as a whole system, including the connections between components. Performance-based code provisions and standards are not available for use by engineers, as an alternative to the current prescriptive fire rating approach, to (1) evaluate the system performance of tall-building structures under real fire scenarios, and (2) enable risk consistent design with appropriate thickness of passive protection being provided where it is needed on the structure. Standards development organizations, including the American Institute of Steel Construction, have initiated development of performance-based provisions to consider fire effects in structural design.*

Finding 3.10: *Availability of detailed procedures to select appropriate design-basis fire scenarios to be considered in the performance-based design of the sprinkler system, compartmentation, and passive protection of the structure. The standard fire in current prescriptive fire resistance tests is not adequate for use in performance-based design. While the NFPA 5000 model building code contains general guidance on design fire scenarios (the IBC Performance Code contains no such guidance), the details of the scenarios are left to the fire engineer and regulatory official. The three major scenarios that are not considered adequately are: frequent but low severity events (for design of sprinkler system), moderate but less frequent events (for design of compartmentation), and a maximum credible fire (for design of passive fire protection on the structure). The maximum credible fire scenario for passive protection of structures would assume that the sprinkler system is compromised or overwhelmed and that there is no active firefighting. These building-specific representative fire scenarios are similar in concept, though not* *identical, to the approach used in building design where the performance objectives and design-basis of the hazard are better defined (e.g., a two-level design that includes an operational event with a 10 percent probability of occurrence in 50 years and a life safety event with a 2 percent probability of occurrence in 50 years). The design-basis fire hazards for the WTC towers and WTC 7 are unknown, and it is difficult to evaluate the performance of the fire protection systems in these buildings under specific fire scenarios.*

Finding 3.11: *Availability of code provisions to ensure that structural connections are provided the same degree of fire protection as the more restrictive protection of the connected elements. The provisions that were used for the WTC towers and WTC 7 did not require specification of a fire-rating requirement for connections separate from those for the connected elements. It is not clear what the fire rating of the connections were when the connecting elements had different fire ratings.*

Finding 3.12: *Availability of technical basis to establish whether the minimum mechanical and durability related properties of SFRM are sufficient to ensure acceptable in-service performance in buildings. While minimum bond strength requirements exist, there are no serviceability requirements for such materials to withstand typical shock, impact, vibration, or abrasion effects over the life of a building. There are existing testing standards for determining many of these properties, but the technical basis is insufficient to establish serviceability requirements. Knowledge of such serviceability requirements is relevant to determine the post-impact fireproofing condition of the WTC towers.*

Finding 3.13: *Availability of validated and verified tools for use in performance-based design practice to analyze the dynamics of building fires and their effects on the structural system that would allow engineers to evaluate structural performance under alternative fire scenarios and fire protection strategies. Existing tools are either too simplified to adequately capture the performance of interest or too complex and computationally demanding, and lack adequate validation. While considerable progress has been made in recent years, significant work remains to be done before adequate tools are available for use in routine practice. NIST has had to further develop and validate existing tools to investigate the fire performance of the WTC towers and WTC 7.*

Structural Integrity

Finding 3.14: *Availability of explicit structural integrity provisions to mitigate progressive collapse. Federal agencies have developed guidelines to mitigate progressive collapse and routinely incorporate such requirements in the construction of new federal buildings. The United Kingdom incorporates such code requirements for all buildings. New York City adopted by rule in 1973 a requirement for buildings to resist progressive collapse under extreme local loads. The rules, which were adopted after the WTC towers were built but before WTC 7 was built, applied specifically to buildings that used precast concrete wall panels and not to other types of buildings. As stated in Finding 1b.1, the current working hypothesis for the collapse of the 47-story WTC 7, if it remains viable upon further analysis, would suggest that it was a classic progressive collapse.*

Finding 3.15: *Availability of minimum structural integrity provisions for the means of egress (stairwells and elevator shafts) in the building core that are critical to life safety. In most tall buildings the core is designed to be part of the vertical gravity load carrying system of the structure. However, in many of those buildings, especially in regions where earthquakes are not dominant, the core may not be part of* *the lateral load carrying system of the structure. Thus, the core may be designed to carry only vertical gravity loads with no capacity to resist lateral loads, i.e., overturning moment and shear loads. In such situations, the structural designer may prefer the use of partition walls over structural walls in the core area to reduce building weight. The decision to have the core carry a specified fraction of the lateral design loads or be made part of a dual system to carry lateral loads, each of which would enhance the structural integrity of the core if structural walls were used, is left to the discretion of the structural engineer. Alternatively, stairway/elevator cores built with concrete or reinforced concrete block, which are not part of the lateral load carrying system, may be able to provide sufficient structural integrity if they meet, for example, the hose-stream impact test already required by ASTM E 119, or other more appropriate test. In the case of the WTC towers, the core had 2 h fire-rated partition walls with little structural integrity and the core framing was required to carry only gravity loads. Had there been a minimum structural integrity requirement to satisfy normal building and fire safety considerations, it is conceivable that the damage to stairways, especially above the floors of impact, may have been less extensive.*

Finding 3.16: *Availability of standards and code provisions for conducting wind tunnel tests and for the methods used in practice to estimate design wind loads from test results. Building codes allow the determination of wind pressures from wind tunnel tests for use in design. Such tests are frequently used in the design of tall buildings. Results of two sets of wind tunnel tests conducted for the WTC towers in 2002 by independent commercial laboratories as part of insurance litigation, and voluntarily provided to NIST by the parties to the litigation, show large differences, of as much as about 40 percent, in resultant forces on the structures, i.e., overturning moments and base shears. Independent reviews by a NIST expert on wind effects on structures and a leading engineering design firm contracted by NIST indicated that the documentation of the test results did not provide sufficient basis to reconcile the differences. In addition, the wind loads estimated from these tests are about 20 percent to 60 percent higher than those apparently used in the original design of the WTC towers, also obtained from wind tunnel testing. NIST is conducting an independent analysis to establish the baseline performance of the WTC towers under the original design wind loads and will compare those wind load estimates with then-prevailing code requirements. Wind loads were a major governing factor in the design of structural components that made up the frame-tube steel framing system.*

Compartmentation and Sprinklers

Building fire protection is based on a four-level hierarchical strategy comprising detection, suppression (sprinklers and firefighting), compartmentation, and passive protection of the structure.

- Detectors are typically used to activate fire alarms and notify building occupants and emergency services.

- Sprinklers are designed to control small and medium fires and to prevent fire spread beyond the typical water supply design area of about 1,500 ft^2.

- Compartmentation mitigates the horizontal spread of more severe but less frequent fires and typically requires fire-rated partitions for areas of about 7,500 ft^2. Active firefighting also covers up to about 5,000 ft^2 to 7,500 ft^2.

- Passive protection of the structure seeks to ensure that a maximum credible fire scenario, with sprinklers compromised or overwhelmed and no active firefighting, results in burnout, not overall building collapse. The intent of building codes is also for the building to withstand local structural collapse until occupants can escape and the fire service can complete search and rescue operations.

Compartmentation of spaces has long been a cornerstone to building fire safety to limit fire spread. The WTC towers initially had 1 h fire-rated partitions separating tenants (demising walls) that extended from the floor to the suspended ceiling, not the floor above (the ceiling tiles were not fire rated). Over the years, these partitions were replaced with partitions that were continuous from floor to floor (separation wall) as required by the 1968 NYC Building Code. Some partitions had not been upgraded by 1997, and a consultant recommended to the PANYNJ that it develop and implement a survey program to assure that the remediation process occurred as quickly as possible. It appears that with few exceptions, nearly all of the floors not upgraded were occupied by a single tenant, and it is not clear whether separation walls would have mattered in terms of meeting the 1968 code. The PANYNJ adopted guidelines in 1998 that required such partitions to provide a continuous fire barrier from top of floor to underside of slab.

Finding 3.17: *Building codes typically require 1 h fire-rated tenant separations but do not impose minimum compartmentation requirements (e.g., 7,500 ft^2) for buildings with large open floor plans to mitigate the horizontal spread of fire. This is the case with both the 1968 NYC Building Code, which did not require above-ground sprinklers, and the 2001 NYC Building Code, which requires sprinklers. The sprinkler option was chosen for the WTC towers in preference to the compartmentation option in meeting the subsequent requirements of Local Law 5 adopted by New York City in 1973. Thus, if there was only one tenant on a WTC floor there would be no horizontal compartmentation requirement. Conversely, if there were a large number of tenants on a WTC floor, it would be highly compartmented with separation walls. The affected floors in the WTC towers were mostly open—with a modest number of perimeter offices and conference rooms and an occasional special purpose area. Some floors had two tenants and those spaces, like the core areas, were partitioned (slab to slab). Photographic and videographic evidence confirms that even non-tenant space partitions (such as those that divided spaces to provide corner conference rooms) provided substantial resistance to fire spread in the affected floors. For the duration of about 50 to 100 min prior to collapse of the WTC towers that the fires were active, the presence of undamaged 1 h fire-rated compartments may have assisted in mitigating fire spread and consequent thermal weakening of structural components.*

Finding 3.18: *Availability of state and local building regulations for early installation of sprinklers in existing buildings, not as an option in lieu of compartmentation. Functioning sprinklers can provide significant improvement in safety for most common building fires and prevent them from becoming large fires. NYC promulgated local laws in 1973 and 1984 to encourage installation of sprinklers in new buildings, and is now considering a law to require sprinklers in existing buildings. The WTC towers were fully sprinklered by 2001, about 30 years after their construction. Sprinklering of the tenant floors in the WTC towers was completed by October 1999, while sprinklering of the sky lobbies was still underway at that time. The sprinkler system was installed in three phases. Phase 1 was completed during initial building construction and included the sub-grade areas. Phase 2 was done in 1976, in compliance with Local Law 5, and included sprinklering the corridors, storage rooms, lobbies, and certain tenant spaces. Phase 3 was begun in 1983 and completed in 2001 and resulted in fully sprinklering the complex.*

Finding 3.19: *Modern building codes allow a lower fire rating for structural elements when a building is sprinklered. This trade-off provides an economic incentive to encourage installation of sprinklers. Sprinklers provide better intervention against small and medium fires, fires which are more likely to occur than a WTC disaster, as long as the water supply is not compromised and there is redundant technology in place. The required technical basis is not available to establish whether the "sprinkler trade-off" in current codes adequately considers fire safety risk factors such as: (1) the complementary functions of sprinklers and fire-protected structural elements, (2) the different fire scenarios for which each system is designed to provide protection, and (3) the need for redundancy should one system fail. It is noteworthy that the British Standards Institution (BSI) has established a group to review all the sprinkler trade-offs contained in their standards. While the classification and fire rating of the WTC towers did not take advantage of the sprinkler-tradeoff since such provisions were not contained in the 1968 NYC Building Code, had such provisions existed, they would have required a lower fire rating for many WTC building elements.*

Occupant Behavior and Evacuation

The capacity of egress systems in very tall buildings is based on the phased evacuation concept, where occupants are evacuated first from the three floors closest to the emergency (e.g., fire), while others wait their turn. Such systems require a voice communication system to manage the process from a fire command center and, e.g., in New York City, fire wardens on each floor directing the flow. These systems are not designed to accommodate a total or full emergency evacuation of the building.

There were at least three instances where a full evacuation of the WTC towers was ordered; a 1977 terrorist threat associated with bombings in two midtown Manhattan buildings, during the 1993 bombing, and on September 11, 2001. During the 1977 event, a full evacuation of the WTC complex was ordered at 11:45 a.m., and fire safety teams searched and evacuated 35,000 occupants. The evacuation was orderly, and no one was injured. The WTC complex was reopened to the public at 3:00 p.m. In addition, about 1,500 people were evacuated from 15 floors of WTC 1 during a fire on the morning of April 17, 1981.

Sufficient data do not exist on the frequency with which full evacuations are conducted in buildings not at risk for terrorist attacks and whether this frequency has increased since September 11, 2001. In one of the three instances, the WTC towers had to be fully evacuated even though the terrorist threat was to other remote buildings in the city. Based on NIST interviews of WTC survivors and anecdotal data reported to the 9-11 Commission, it appears that since September 11, 2001, building occupants may be more likely to evacuate even when the safety risk at their location may not be sufficient to require them to evacuate. It is not clear how widespread this sentiment is among the general population that did not experience the events of September 11, 2001 directly.

Finding 3.20: *Availability of technical basis for the design of egress systems. Current prescriptive methods (e.g., unit width or inches per person) for minimum stair width or number of stairs do not provide information on the target performance to be achieved. For example, what would be the evacuation rate or time, for a given occupant population, considering travel distance, remoteness requirements, and human factors such as occupant size (reflecting current population data), stairwell environmental conditions, visibility, and congestion. Also, the technical basis for the "sprinkler trade-off") in egress specifications (generally a doubling of allowed travel distance is not available).*

Further, proposals for increases in stair width do not consider the effect of doors in limiting the flow. Also, proposals for increases in the number of stairs do not balance the need to meet remoteness requirements (physical separation of stairways that are located in separate enclosures) while possibly permitting scissor stairs (two separate stairways within the same enclosure and separated by a fire rated partition). Scissor stairs are credited as a single stair but provide additional capacity and additional doors that achieve real increases in evacuation rate with only minor impact on leasable space. The egress capacity in the WTC towers was based on the unit-width method contained in the 1968 NYC Building Code; it is not possible to assess the adequacy of the resulting egress capacity to achieve a target performance (e.g., evacuation rate or time) under a design-basis evacuation event. Further, although the NYC code permitted scissor stairs—which are prohibited in most other codes—none were used in the WTC towers or WTC 7.

Finding 3.21: *Consideration of counterflow, e.g., due to first responder emergency access, in the design of egress systems. For typical short height buildings, the occupants are evacuated from the affected floors within a matter of several minutes, before first responders arrive and climb up the stairs. While such evacuation still would be ongoing after arrival of first responders in taller buildings, NIST interviews with WTC occupants suggest that on September 11, 2001, with about one-third of building occupants present, there were only occasional encounters with first responders, and counterflow was not a significant issue. This finding is consistent with the Finding 2.8—based on occupant first-person interviews—related to adequacy of the egress capacity in the WTC towers on September 11, 2001.*

Use of Elevators in Emergencies

Finding 3.22: *With a few special exceptions, building codes in the United States do not permit the use of fire-protected elevators for routine emergency access by first responders or as a secondary method (after stairwells) for emergency evacuation of building occupants. The use of elevators by first responders would additionally mitigate counterflow problems in stairwells. While the United States conducted research on specially protected elevators in the late 1970s, the United Kingdom has required such "firefighter lifts," located in protected shafts, for a number of years. Without functioning elevators (e.g., due to a power failure or major water leakage), first responders carrying gear typically require about a minute per floor to reach an incident using the stairs. While it is difficult to maintain this pace for more than about the first 20 stories, it would take a first responder about an hour to reach, for example, the 60th floor of a tall building if that pace could be maintained. Such a delay, combined with the resulting fatigue and physical effects on first responders that were reported on September 11, 2001, would make firefighting and rescue efforts difficult even in tall building emergencies not involving a terrorist attack. Each of the WTC towers had 100 elevators, and WTC 7 had 38 elevators. By code, the elevators could not be used for fire service access or occupant egress during an emergency since they were not fire-protected, nor were they located in protected shafts. The elevators were equipped through normal modernization with fire service recall. Most were damaged by the aircraft impacts; though prior to the impact in WTC 2 the elevators were functioning and contributed greatly to the much faster initial evacuation rate in WTC 2 as stated in Finding 2.8.*

Building Practices

Finding 3.33: *While the PANYNJ entered into agreements with the NYC Department of Buildings in the 1990s (with regard to conformance of PANYNJ buildings constructed in New York City to the NYC*

Building Code), the PANYNJ did not yield jurisdictional authority for regulatory and enforcement oversight to the New York City Department of Buildings. The PANYNJ was created as an interstate entity, under a clause of the U.S. Constitution permitting compacts between states, and is not bound by the authority of any local, state, or federal jurisdiction.

Finding 3.34: *Availability of rigorous field application and inspection provisions and regulatory requirements to assure that the as-built condition of the passive fire protection, such as SFRM, conforms to conditions found in fire resistance tests of building components and assemblies. For example, provisions are not available to ensure that the as-applied average fireproofing thickness and variability (reflecting the quality of application) is thermally equivalent to the specified minimum fire proofing thickness. In addition, requirements are not available for in-service inspections of passive fire protection during the life of the building. The adequacy of the fireproofing of the WTC towers posed an issue of some concern to the PANYNJ over the life of the buildings, and the availability of accepted requirements and procedures for conducting in-service inspections would have provided useful guidance.*

Finding 3.35: *State and local jurisdictions do not require retention of documents related to the design, construction, operation, maintenance, and modifications of buildings, with few exceptions. These documents are in the possession of building owners, contractors, architects, engineers, and consultants. Such documents are not archived for more than about 6 to 7 years and there are no requirements that they be kept in safe custody physically remote from the building throughout its service life. In the case of the WTC towers, the PANYNJ and its contractors and consultants maintained an unusually comprehensive set of documents, a significant portion of which had not been destroyed in the collapse of the buildings but could be assembled and provided to the investigation. In the case of WTC 7, the situation was more typical of current practice, and a significant portion of the documents could not be assembled since they were lost in the collapse of the building. However, NIST has adequate information for its investigation. Neither the original general contractor nor the architects was able to supply more than a few documents.*

Finding 3.36: *The architect is responsible for specifying the fire protection and designing the evacuation system in current building practice. The structural engineer is not required to evaluate and certify that the passive fire protection is adequate to protect the structural system. In accordance with established practice, the structural engineer was not responsible for the passive fire protection in the design of the WTC tower structures. In addition, there is no requirement for a fire protection engineer to be part of the team designing the overall fire protection (including detection, suppression, compartmentation, and passive fire protection) and evacuation systems for the building. A change in this respect is already underway for signature/iconic buildings, where it is becoming more common for a fire protection engineer to be included in the design team. In the case of the WTC towers, the building owner played a significant role in specifying the fire protection and evacuation systems; a fire protection engineer was not part of the original design team. There are only a few academic degree programs or continuing education programs that qualify engineers (or architects) to evaluate the fire performance of structures. The current state-of-practice is not sufficiently advanced for engineers to routinely analyze the performance of a whole structural system under a prescribed design-basis fire scenario.*

1.6 APPROACH TO RECOMMENDATIONS

In the United States, state and local governments are responsible for promulgating and enforcing building and fire regulations. While states are increasingly adopting a single, uniform set of statewide regulations, in many instances major cities and counties adopt their own regulations. The national organization representing state building officials is the National Conference of States on Building Codes and Standards (NCSBCS)—a body of the National Governors Association—and that representing building officials of major local jurisdictions is the Association of Major City/County Building Officials (AMCBO).

With some exceptions, the state and local regulations are based on national model building and fire codes developed by private sector organizations, the International Code Council (ICC) and the NFPA. The model codes, in turn, reference voluntary consensus standards developed by a large number of private sector standards development organizations (SDOs). They include organizations such as NFPA, ASTM International, the American Society of Civil Engineers (ASCE), the American Institute of Steel Construction (AISC), and the American Concrete Institute (ACI). The SDOs are accredited by the American National Standards Institute (ANSI).

Other key stakeholder groups involved in the design, construction, operation, and maintenance of buildings include organizations representing building owners and managers, real estate developers, contractors, architects, engineers, suppliers, and insurers.

NIST is a non-regulatory agency of the U.S. Department of Commerce. NIST does not set building codes and standards, but provides technical support to the private sector and other government agencies in the development of U.S. building and fire practices, standards, and codes. NIST recommendations are given serious consideration by private sector organizations that develop national standards and model codes – which provide minimum requirements for public welfare and safety. The model codes become regulation when adopted by state and local governments.

The NIST building and fire safety investigation of the WTC disaster has not yet formulated recommendations under this objective. However, in formulating its recommendations, NIST will consider the following:

- Findings from the first three independent investigation objectives related to building performance, evacuation and emergency response, and procedures and practices.
- Whether findings relate to the unique circumstances surrounding the terrorist attacks of September 11, 2001, or to normal building and fire safety considerations, including evacuation and emergency response?
- What technical solutions are needed, if any, to address potential risks to buildings, occupants, and first responders, considering both identifiable hazards and the consequences of those hazards?
- Whether the risk is in all buildings or limited to certain building types (e.g., height and area, structural system), buildings that contain specific design features, iconic/signature buildings, or buildings that house critical functions?

NIST urges organizations responsible for building and fire safety at all levels to carefully consider the interim findings contained in this report. NIST welcomes comments from technical experts and the public on the interim findings presented herein. Comments can be sent by e-mail to wtc@nist.gov, facsimile to 301-975-6122, or regular mail to WTC Technical Information Repository, Stop 8610, 100 Bureau Drive, Gaithersburg, MD 20899-8610.

In its final report, a draft which is expected to be released in December 2004, NIST will recommend appropriate improvements in the way buildings are designed, constructed, maintained and used. It will be important for those recommendations to be thoroughly and promptly considered by the many organizations responsible for building and fire safety. As a part of NIST's overall WTC response plan, the Institute has begun to reach out to those organizations to pave the way for a timely, expedited consideration of recommendations stemming from this investigation. NIST also already has expanded its research in areas of high priority need.

Chapter 2
PROGRESS ON THE WORLD TRADE CENTER INVESTIGATION

2.1 STATUS OF DATA COLLECTION EFFORTS

The National Institute of Standards and Technology (NIST) is basing its review, analysis, modeling, and testing work for the World Trade Center (WTC) Investigation on a solid foundation of technical evidence. This requires access to critical data such as building documents, videographic and photographic records, emergency response records, and oral histories, in addition to the samples of steel that have been recovered.

NIST has received considerable cooperation and large volumes of information from a variety of organizations and agencies representing the building designers, owners, leaseholders, suppliers, tenants, first responders, contractors, insurers, news media, survivors, and families of victims. In addition, NIST has received and is grateful for cooperation from The National Commission on Terrorist Attacks Upon the United States (9-11 Commission). The documents and other information relate to the design, construction, operation, inspection, maintenance, repair, alterations, emergency response, and evacuation of the WTC complex.

Local authorities providing information include the Port Authority of New York and New Jersey (PANYNJ or Port Authority) and its consultants and contractors; the New York City Fire Department (FDNY); the New York City Police Department (NYPD); the New York City (NYC) Law Department; the NYC Department of Design and Construction; the NYC Department of Buildings; and the NYC Office of Emergency Management. In addition, the Occupational Safety and Health Administration provided correspondence sent to it regarding the evacuation experience of WTC occupants on September 11, 2001.

NIST also has received information from Silverstein Properties and its consultants and contractors; the group of companies that insured the WTC towers and its technical experts; Nippon Steel; Laclede Steel; U.S. Mineral Products Co. and Isolatek International; Morse Zehntner Associates; W.R. Grace & Co.; Citigroup, formerly Salomon Smith Barney; United Airlines; American Airlines; and Boeing. NIST also received information on floor plans, furnishings, and contents from tenants of all three buildings.

The information from Silverstein and the insurance companies includes the large body of technical work completed by both parties as part of the insurance litigation involving the WTC towers, such as reports on the structural collapse, fire spread and severity, and wind tunnel test results for the WTC towers. In addition, technical experts for both parties independently provided extensive briefings to the WTC investigation team and discussed the tenability environment and the evacuation procedures in the buildings.

NIST has received all of the essential information it needs for the WTC investigation. That information includes NYC 9-1-1 tapes, the transcripts of approximately 500 interviews of employees of the FDNY who were involved in WTC emergency response activities, and supporting documents for McKinsey & Company's FDNY study.

The following is the list of documentary information received or inspected by NIST.

December 2002

- The original design drawings (structural, architectural, mechanical, electrical, plumbing) and the original fabrication and construction drawings for the WTC towers
- Tenant alteration application reports, including drawings and specifications, for the WTC towers and WTC 7, and associated construction audit reports
- Tenant design standards manuals for structural; architectural; heating, ventilating, and air-conditioning (HVAC); fire protection; plumbing; electrical; fire alarm; and construction review
- Emergency evacuation procedures manuals, including fire safety guide
- Operations manuals for the fire protection system, including sprinklers, standpipes, alarm system and communication protocols, and water and power supply
- Operations manuals for the HVAC systems
- Reports on facility condition surveys and structural integrity inspections for the WTC towers and WTC 7
- Recent inspection and maintenance reports for the elevators and escalators in the WTC towers; elevator numbering system
- Reports on pre-design tests of structural components, including dampers for the WTC towers
- Reports on wind tunnel tests of the WTC towers and wind speed measurements near the WTC site
- Reports on the 1993 bombing damage assessment and repairs, and documentation of changes made to the evacuation system after 1993
- Documents related to the location, approval, and inspection of fuel tanks in WTC 7
- Documents related to fire rating and fireproofing of structural steel members in the WTC towers
- Documents related to PANYNJ building and fire code requirements and practices
- Correspondence sent to the Occupational Safety and Health Administration regarding the evacuation experience of WTC occupants on September 11, 2001
- Documents related to the lease of the WTC towers by Silverstein Properties
- Reports prepared by McKinsey & Company for FDNY and NYPD

- Basic FDNY dispatch data, including time of dispatch and unit identification
- Firefighter fatality and injury data from FDNY

<u>May 2003</u>

- More than 1,000 hours of recordings made by PANYNJ on September 11, 2001 (from 0705 through 1900 hours) of telephone calls, as well as police, fire, operations, maintenance, security, and other radio transmissions from four distinct locations
- Personal injury data from FDNY and Port Authority of New York and New Jersey Police Department (PAPD)
- Handwritten notes on the events of September 11, 2001, by PAPD staff
- Emergency responder fatality data for FDNY, NYPD, and PAPD
- WTC list of tenants with contact information from PANYNJ and Silverstein
- WTC list of occupants issued security badges by PANYNJ
- Report on WTC smoke management system by Hughes Associates, Inc.
- Phase I and final reports on fire engineering of WTC steelwork by Buro Happold
- Transcripts of depositions by two PANYNJ staff in the WTC insurance litigation
- Documents, videos, and photographs related to the fireproofing of the WTC tower structures
- WTC floor plan for the fire alarm system and drawings of WTC subgrade plumbing and city water main
- Information regarding building contents such as partitions and furnishings from a key WTC tower tenant, to characterize the types of combustibles and estimates of the mass loading in the region of the fires
- FDNY WTC incident summary, September 20, 2001
- FDNY reports on the fire history of WTC 1, 2, and 7 from 1970 to 2001
- FDNY reports related to inspections of WTC 1, 2, and 7 from 1999 to 2001
- FDNY policies and practices on operations specific to the WTC buildings and on accountability of firefighters at incidents
- FDNY information on dispatched units, apparatus, command posts, and staging areas
- FDNY information on number of command and company officers and firefighters operating in and around WTC 1, 2, and 7 with number of surviving personnel

- Detailed briefing on the NYPD communications system, including 9-1-1 system and radio networks

August 2003

- Design and structural calculations from Leslie E. Robertson Associates (LERA) for the WTC towers, including TV antenna, beams, and beam girders, as well as wind analysis and calculations
- Correspondence from LERA during the time of construction
- Laclede floor truss shop drawings (1,364 sheets) and other documents on steel and joints
- Information on steel from Nippon
- List of WTC drawings in possession of Yamasaki and Associates
- Information on the flammable contents of the American Airlines B-767 aircraft
- Information regarding building contents and floor layouts from some WTC tower and WTC 7 tenants
- Mechanical and electrical specifications for WTC 7
- Asbestos litigation documents from PANYNJ
- Underwriters' Laboratories, Inc. (UL) test reports regarding spray-on fireproofing from supplier (Isolatek)
- Correspondence on the selection of WR Grace fireproofing products, test data, and UL design listings (WR Grace)
- Data on the WTC internal radio system and FDNY radio repeater from PANYNJ
- Some FDNY training practices for operations in high-rise buildings
- Global positioning system coordinates and map where human remains and equipment were located from FDNY
- Information on FDNY personnel killed on September 11, 2001, and map of fire and Emergency Management Services Command Post Locations
- NYPD internal communications concerning the terrorist attacks on WTC (43 cassette tapes)
- Disaster Response Plan, Patrol Guide Procedures, and other guides and manuals from NYPD, including the Unusual Occurrence Report on the 1993 WTC bombing
- A large portion of NYPD and FDNY extensive photographic and videographic collection

- Updated badge list of WTC occupants maintained by PANYNJ

- WTC fire safety and PA/FDNY WTC training videos and pre-September 11, 2001 WTC photographs

<u>September 2003</u>

- Information on the flammable contents of the United Airlines B-767 aircraft

- Documents from PANYNJ on accessibility for disabled persons, active fire protection systems, and adoption of revisions to NYC Building Code

- Elevator and escalator contract information from PANYNJ

- Status of changes to WTC towers (March 1973) from PANYNJ

- Transcripts from September 11 PAPD audiotapes, police reports, and PAPD special awards ceremony documents for September 11, 2001

- Additional documents from PANYNJ on asbestos litigation

<u>October 2003</u>

- Supporting documents for McKinsey & Company's FDNY and NYPD studies

- Review of UL test reports regarding spray-on fireproofing from supplier (W.R. Grace)

- Information from Boeing on flammable contents of aircraft that contributed to fires

<u>May 2004</u>

- Review of NYC 9-1-1 tapes and logs, transcripts of about 500 first responder interviews with employees of the FDNY who were involved in WTC emergency response activities

- General description of WTC building systems and capital program

- WTC documents presented as exhibits in asbestos litigation

- Additional documents on WTC maintenance services, accessibility, elevators, code compliance, fire rating, fire detection system, fire alarm system, etc.

- Photographs of WTC 7 construction project

- Architectural and HVAC drawings for WTC 7, including modifications

- Well in excess of 6,000 photographs representing more than 185 professional and amateur photographers. Organizations that have provided materials include FDNY, NYPD, Associated Press, Corbis, Reuters, *The New York Times*, *The New York Daily News*, and the

Star Ledger. Many organizations have provided both published and unpublished photographs.

- In excess of 150 hours of videotapes from news media (NBC, CBS, ABC, CNN, and local New York stations WABC, WCBS, WNBC, WPIX, WNYW, and New York One), FDNY, NYPD, and more than 20 individuals. In many cases, the videos provide not only broadcast material (known as air checks), but also material that was recorded but not broadcast (known as outtakes).

The few NIST requests for materials that are lost, currently pending, or not yet located include:

- Original contract specifications for WTC towers (lost in the collapse of the buildings)
- Construction and maintenance logs for WTC 1, 2, and 7 (lost in the collapse of the buildings)
- Calculations and analyses that supported the original aircraft impact studies (lost in the collapse of the buildings)
- Descriptions of partitions and furnishings in most of the tenant spaces of WTC 2 and WTC 7 in the fire and impact zones
- Shop drawings showing connection details of WTC 7

NIST is making efforts to assemble this information from various sources because much of it was lost when the buildings collapsed. NIST continues to pursue other materials that can further clarify some aspects of the Investigation.

2.2 ANALYSIS OF BUILDING AND FIRE CODES AND PRACTICES (PROJECT 1)

2.2.1 Project Objective

One of the four primary objectives of the NIST Investigation of the WTC disaster is to determine the procedures and practices that were used in the design, construction, operation, and maintenance of the WTC towers and WTC 7. A key focus is on acceptance procedures and practices for innovative systems, technologies, and materials, and for variances from requirements of building and fire code provisions. This documentation of historical information is expected to be of value to the professional community in identifying and adopting changes to procedures and practices that may be warranted.

For most buildings constructed in the United States, building codes adopted by local jurisdictions establish minimum requirements for design and construction. However, because PANYNJ is an interstate agency, its construction projects are not required to comply with any local or national model building code. Thus, to determine the criteria, procedures, and practices that were used in the design, construction, operation, and maintenance of WTC 1, 2, and 7, Project 1, Analysis of Building and Fire Codes and Practices, has the following objectives:

- Document the requirements that governed the design and construction of WTC 1, 2, and 7

- Document any differences between the Port Authority requirements used for design and the then current building code requirements of other jurisdictions and the appropriate model building code

- Document the procedures used by the Port Authority to accept new and innovative design features that deviated from the Port Authority building design requirements

- Document the procedures used to accept new technologies and materials that were not recognized by then-current standards

- Document passive and active fire safety, emergency access and egress provisions that were incorporated in the original design and subsequent modifications during occupancy

- Document major modifications made to structural, fire protection, and egress systems of WTC 1, 2, and 7

- Document the inspection and maintenance procedures used for WTC 1, 2, and 7.

2.2.2 Project Approach

The design and construction documents of the WTC buildings that were kept centrally at the Port Authority office in WTC 1 were destroyed when the tower collapsed. Thus, existing copies of design and construction documents of WTC 1, 2, and 7 had to be assembled from various sources that were associated with these projects. Documents were obtained principally from:

- The Port Authority, and

- Architectural and engineering firms who designed and inspected the WTC buildings.

In addition, information was obtained from others who were associated with WTC 1, 2, and 7 construction projects.

The information collected will enable the NIST investigators to accomplish the five tasks of Project 1:

- **Task 1**. Document the design and construction of structural systems to determine:

  - Provisions used to design and construct the buildings. This will include the Port Authority building design and construction requirements, the building code used, standards referenced, and Port Authority policies and agreements with the NYC Department of Buildings regarding building code requirements.

  - Tests performed to support the design, such as wind tunnel tests and tests of structural assemblies.

  - Criteria used to proportion structural members and other components of the buildings including structural connections.

  - Innovative systems, technologies, and materials that were used, and the acceptance procedures used by the Port Authority.
  - Variances granted by the Port Authority, including the justification for those variances.
  - Special fabrication and inspection requirements.
  - Inspection protocols used during construction.
  - Technical problems that occurred during construction of the buildings and their resolution.

- **Task 2.** Document the design and construction of the fire protection and egress systems to determine:
  - Provisions used to design and construct the fire protection (passive and active) and egress systems of the buildings. This will include the Port Authority building and fire regulatory requirements, the building and fire code used, and standards referenced.
  - Building regulations adopted after the issuance of the certificates of occupancy, or equivalent, that were applied to the buildings through retroactivity (including any provisions of NYC Local Laws), and any permits issued or special inspections required resulting from the installation of special hazards or equipment in the buildings.

- **Task 3.** Document the fuel system for emergency power in WTC 7 to determine:
  - Locations of emergency power generating systems;
  - Size and locations of the fuel storage tanks and distribution systems;
  - Specific fire protection systems used for the fuel storage and distribution systems;
  - Normal and emergency operating procedures; and
  - Maintenance history.

- **Task 4.** Compare building regulatory and code requirements to document:
  - Port Authority building regulatory requirements.
  - Differences among the Port Authority building regulatory requirements and the then-current NYC, New York State, Chicago, and Building Officials Conference of America (now known as the Building Officials and Code Administrators [BOCA]) building code provisions.
  - Differences between the Port Authority building regulatory requirements and the current (2001) NYC Building Code provisions.

  - Evolution of the life safety provisions in the NYC Building Code since the design of WTC 1 and WTC 2.

- **Task 5.** Document maintenance of and modifications to the structural, fire protection, and egress systems to determine:

  - Guidelines used by the Port Authority for inspection, repair, and modifications to structural, fire protection, and egress systems.

  - Structural integrity inspection programs during the occupancy of the buildings.

  - Any significant modifications and/or repairs of the original structural framing system by the owner or tenants during original construction and occupancy.

  - Any repairs and modifications made to the passive and active fire protection systems from initial occupancy to September 11, 2001.

2.2.3 Status of Tasks

Except for the task of comparing building regulatory and code requirements (Task 4), the tasks depend upon the availability of design, construction, and maintenance documentation related to WTC 1, 2, and 7. Efforts by NIST to obtain needed documents are described briefly, and the current status of each of the five tasks is stated below.

Sections 2.2.4 and 2.2.5 present salient points related to Tasks 1 and 2, particularly how changes in NYC Building Code provisions from the 1938 edition to the 1968 edition, and subsequent promulgation of NYC Local Laws, affected the design, construction and maintenance of WTC 1, 2, and 7. Section 2.2.6 describes the fuel system that powered the emergency generators in WTC 7 (Task 3). Comparison of the requirements of several building codes (Task 4) pertaining to structural and fire safety is presented in Appendix A, Interim Report on the Analysis of Building and Fire Codes and Practices, of this report. Task 5, which deals with maintenance and modifications to the structural, fire protection, and egress systems of WTC 1, 2 and 7, is near completion.

Collection of Design and Construction Data

NIST requested that the Port Authority and the design and construction firms for WTC 1, 2, and 7 provide design, construction, and maintenance documents. NIST obtained a considerable amount of information (design drawings, shop drawings, specifications, project correspondence, and inspection reports) related to WTC 1 and WTC 2 from the structural engineers who were involved in the original design and subsequent modifications to the towers. The Port Authority provided construction related files for WTC 1, 2, and 7, mostly pertaining to tenant alteration projects, wherein tenants modified parts of the buildings to meet their needs. No document was obtained from the general contractor of WTC 1, 2, and 7. The general contractor retains construction documents for about 7 years. As a result, few records are available related to changes to the structural and fire safety systems that were made during construction of these buildings. However, documents obtained from the structural engineer included revisions to structural modifications for tenant renovations.

It should be pointed out that there is no mandated requirement by NYC for design and construction firms to retain their documents for a specific duration. Currently, no municipalities in the United States have document retention policies that require design and construction firms to retain their documents.

Collected documents have been examined by NIST, and pertinent documents have been organized into a searchable database. Using keywords, the user of the database can retrieve relevant documents. NIST has engaged a contractor to assist in reviewing the vast amount of documentation that has been collected.

Task Status and Report Preparation

Under NIST guidance and direction, a team of NIST contractors, led by Rolf Jensen and Associates, Inc., has made an in-depth review of the relevant documents and is in the process of preparing draft reports that address the Project 1 tasks. Independent examination of documents by NIST engineers together with the contractor's reports will be the basis for the final report of this project. Table 2–1 indicates the status of each of the tasks.

Table 2–1. Status of Project 1 tasks.

Task	Status
1	Documented code provisions used to design and construct WTC 1, 2 and 7. Documented criteria used to design WTC 1 and WTC 2. Contractor submitted final draft report to NIST.
2	Documented code provisions used to design and construct the passive and active fire protection systems for WTC 1, 2, and 7. Documented adoption of Local Laws that modified and/or amended the fire safety provisions of the 1968 NYC Building Code. Contractor submitted second draft report to NIST.
3	Documented all emergency power generating systems and the size and locations of the fuel storage tanks and distribution system in WTC 7. Contractor submitted final draft report to NIST.
4	Documented line-by-line comparison of structural and life safety provisions of the 1968 NYC Building Code vs. three other contemporaneous building codes and the 2001 NYC Building Code. Contractor submitted final draft report to NIST.
5	Documented repairs, modifications to the structural and fire protection systems, and emergency access and egress systems. Contractor submitted final draft to NIST.

2.2.4 Building Codes

As discussed in Appendix A, the Port Authority adopted the 1968 NYC Building Code (NYCBC 1968) for the final design of the WTC buildings. Therefore, this code served as the basis for the code comparison. NIST examined the structural and fire safety provisions in several contemporaneous codes, including the 1964 New York State Building Construction Code (NYSBC 1964), the 1965 BOCA model building code (Basic Building Code), the 1967 Municipal Code of Chicago (MCC 1967), and the 1966 National Fire Protection Association (NFPA) 101 Life Safety Code (NFPA 1966) egress requirements. A comparison was also made between the 1968 NYC Building Code and the current (2001) NYC Building

Code. The current NYC Building Code (NYCBC 2001) is basically the document adopted in 1968 with modifications made over the years by adoption of Local Laws and rules.

Because fire protection and fire safety provisions are of major importance to this Investigation, this section provides background information on building codes, focusing on matters related to fire protection. Appendix A provides additional background information and provides a summary of the code comparison.

Code Provisions on Fire Safety

The fire safety provisions in building codes can be confusing to those who are not familiar with the code provisions that have evolved over the past century. The following provides basic concepts related to fire protection and fire safety.

Fire Rating

This is a time expressed in hours or minutes. It represents fire resistance assigned to a building element on the basis of a test. Fire rating is the time that a test assembly is able to withstand the furnace temperature exposure specified in American Society for Testing and Materials (ASTM) E 119 (ASTM 2003) without exceeding one of the "failure conditions." Thus, a fire rating of 2 h for a structural member indicates that when the member was tested in accordance with ASTM E 119, it performed successfully for at least 2 h. It is not necessary to conduct a test for all materials to be used in a building if data are available from past standard tests showing acceptable performance, and the same materials and application methods will be used. The fire rating of a member or assembly does not indicate for how long a similar member or assembly in the building would perform under a real fire because the actual fire exposure and structural configuration are never the same as during the ASTM E 119 test.

Occupancy Group

Buildings and spaces are classified according to how the buildings and spaces will be used. The concept of "occupancy group" is used to define different types of occupancy or use such as storage, industrial buildings, general assembly, business, and so forth. A given occupancy group is associated with a different level of fire risk. Factors such as amount of combustible material, ignition sources, and characteristics of the occupants are considered in developing these groupings. In some codes, such as the 1968 NYC Building Code, occupancy groups are listed in a hierarchal sequence (highest to lowest hazard) and assigned an overall "fire index" rating in hours. For example, "high hazard" occupancy is assigned a fire index of 4 h, while "business" occupancy is assigned a fire index of 2 h.

Construction Classification

The nature of the materials used in constructing exterior walls and interior building elements define the construction type. In general, building codes characterize construction materials as "combustible" and "noncombustible" materials. For example, the 1968 NYC Building Code uses the designations "Group I" and "Group II" for noncombustible and combustible construction materials, respectively. Other codes, however, also consider whether these types of materials are used in exterior walls, or interior elements, or both. For example, International Building Code (IBC) 2000 (ICC 2000) lists Type I through Type V. These types, or groups, are further divided into different subclasses that are designated with letters, such as Class IA, Class IB, Class IIA, Class IIB, or in some codes as Type IA, Type IB, Type IIA, Type IIIA, and so forth. Building codes typically include tables that specify fire ratings for different structural elements (columns, walls, beams) and for different construction classifications (Type IA, Type IB, and so forth).

Height and Area Limits

Tall buildings and buildings with large floor areas pose greater risks in the event of fire. Building codes, therefore, place limitations on the heights and floor areas of buildings based on the construction classification, the occupancy group, and whether or not the building has sprinklers.

Partitions

Partitions, in general, are walls that provide separations between spaces within the story of a building. Partitions may or may not require minimum fire ratings, depending on the spaces being separated. In general, three types of partitions require fire ratings:

- Walls that provide separations between different occupancy groups
- Walls that provide separations between tenants (often called demising partitions)
- Walls that separate large floor areas into smaller compartments

Buildings codes specify different minimum fire ratings based on the type of partition and the types of occupancies in the spaces being separated.

Variances

All building codes make allowances for obtaining approval of materials and methods not strictly in compliance with code provisions, but which are judged to be equivalent. Normally, the regulatory authority makes this equivalence determination during the plan approval process, and a variance is issued. Compliance with building code provisions is verified by controlled inspections by building (and fire) inspectors at specified points in the construction process using sets of approved plans showing all such variances.

Evolution of Fire Safety Provisions in Model Building Codes

Model building codes are documents prepared by qualified nongovernmental organizations. When adopted by local jurisdictions, the code provisions become law. The following provides a short review of the evolution of provisions in model codes related to types of construction and fire resistance requirements of structural elements.[6]

The 1927 edition of the Uniform Building Code (ICBO 1928) placed office occupancies in Group F Division 1 and allowed only Type 1 construction for such buildings. This required a 4 h fire rating for

[6] This historical summary was provided by Joseph Messersmith of the Portland Cement Association with a letter of July 31, 2002. See also Messersmith (2002).

columns, beams, and girders and a 3 h rating for floors for buildings (steel and reinforced concrete) over eight stories or 85 ft in height.

The 1934 (5th) edition of the National Building Code (NBFU 1934) required business buildings over 75 ft in height to be "fireproof" construction, defined as having a 4 h fire rating for bearing walls, firewalls, party walls, piers and columns, and a 3 h rating for other walls, girders, beams, and floors.

The 1946-1947 edition of the Southern Standard Building Code (SBCC 1946) required Type 1 construction for business occupancies over 80 ft in height. Type 1 was defined as a 4 h fire rating for columns, bearing walls, trusses or girders supporting masonry or bearing walls, columns or girders, and beams, and a 2 1/2 h rating for floors. There is, however, a note under Type 2 construction that allows residential and business occupancies of unlimited height with a rating of 3 h for columns and a rating of at least 2 h for other structural members including floors.

The 1950 Basic Building Code (BOCA 1950) permits Group E business occupancies of unlimited height to be Type 1A or 1B. Type 1A requires a 4 h fire rating for bearing walls and for columns supporting more than one floor, and 3 h rating for floors including beams. Type 1B reduces those to 3 h and 2 h, respectively. Because this is the model code used in the Northeastern United States, this may have provided the basis for the changes from the 1938 to the 1968 NYC Building Code (see next section). However, because records of technical substantiations for code changes were not kept in this era, conclusive evidence does not exist.

Evolution of the NYC Building Code

Historically, NYC has developed and promulgated its own building code, in contrast to most jurisdictions that adopt (locally modified) versions of one of the model building codes. At the time the WTC project was begun (early 1960s), the 1938 NYC Building Code, which was first adopted January 1, 1938, was in effect and enforced throughout the five boroughs.

In the late 1950s, it was noted that "great changes have occurred in all facets of the building industry" and that "As a result of these developments, and the failure in many instances, of the Code to keep pace, there had been a growing dissatisfaction with it" (Schaffner 1964). Thus in 1960, the Building Commissioner requested the New York Building Congress to form a working committee to study the problem. The committee recommended that the Code should not be rewritten by a group of volunteers and that a local educational institution should conduct a study to develop an approach to solve the problem. The Polytechnic Institute of Brooklyn conducted the study, and in July 1961, the Institute made the following recommendations (Schaffner 1964):

1. The NYC Building Code be completely rewritten. The new Code should provide for frequent periodic revision through a committee or board appointed solely for this purpose.

2. The new Code be a combination of performance and specification types with heavy emphasis on performance, wherever possible, and with liberal reference to accepted national standards.

3. The BOCA Basic Building Code be used as a guide for the development of the NYC Building Code.

4. The Code be rewritten by a private professional group such as an engineering company, architectural firm, educational institution, or any combination of the three. Those rewriting the Code should work closely with the NYC Building Department. They should be supported, for review purposes, by volunteer committees composed of representatives of professional, trade, and industry associations."

In April 1962, NYC signed an agreement with the Polytechnic Institute of Brooklyn for the writing of a new Code to be completed in 3 years. The first draft was completed in 1964. A public relations document highlighted the "major advantages to be gained from recommendations in the proposed new Building Code" (Bell and Stanton 1964). One of these related to the "area and height limitations," and it was stated that:

> Area and height limitations will be liberalized and present unrealistically high construction requirements for fire protection in structures of low combustible content such as auditorium, halls, schools, institutions and residences will be significantly reduced and considerable economy will result.

On December 6, 1968, Local Law 76 repealed the 1938 code and replaced it with the 1968 code, which itself was subsequently amended by Local Laws. As is the general custom with changes to building codes, the new provisions generally are not applied to existing buildings (those approved under the prior code) provided they do not represent a danger to public safety and welfare.

There were 79 Local Laws adopted between 1969 and 2002 that modified the 1968 code. Of particular importance with regard to fire protection and life safety are Local Law 5, adopted in 1973, and Local Law 16, adopted in 1984. Local Law 5, among other things, added requirements on compartmentation of large floor areas, and Local Law 16 added requirements for sprinklers in high-rise buildings (greater than 100 ft). Local Law 5 is particularly significant because its provisions, which are reviewed in a subsequent section, applied retroactively to existing office buildings. Local Law 84, which was passed in 1979, revised the compliance dates of Local Law 5 so that full compliance was required by February 7, 1988.

As discussed in Appendix A, the 1968 NYC Building Code contained a number of provisions not addressed in the other codes of the time, but which were added to these other codes at later times.

Selection of Construction Type for WTC Towers

The 1938 NYC Building Code recognized one construction type for buildings of unlimited height and area, namely Class 1—Fireproof Structures, which required a 4 h fire rating for columns and a 3 h rating for floors. In the 1968 code, Group I (Noncombustible) construction was subdivided into "Class 1A—4-hr protected" and "Class 1B—3-hr protected" construction. Class 1A specifies similar protection as the previous Class 1, and Class 1B specifies a 3 h rating for columns and girders supporting more than one floor and a 2 h rating for floors including beams. Both Class 1A and Class 1B construction permit unlimited height and area for unsprinklered business occupancy.

If a building qualifies for more than one construction classification, such as Class 1A or Class 1B, codes are silent on which classification should be used. In such situations, the classification selected for construction is at the discretion of the owner/architect. To date, no contemporaneous documentation has been found that provides the rationale for the decision to select Class 1B for the WTC towers. This decision, however, appears to have been made by the architect-of-record on the basis of economics. In a 1987 memorandum on the subject of fire rating of the WTC buildings, the following statement was included (Feld 1987):

> For office buildings there is no economic advantage in using Class 1A Construction, and ER&S [architect-of-record] used Class 1B Construction for the WTC Towers and Plaza Buildings which are Occupancy Group "E" (Business) with a fire index of 2 hours.

An interoffice memorandum between staff of the general contractor written in 1969 is the only contemporaneous document found to date that refers to the classification of the WTC towers (Bracco 1969). The following statement is included in that memorandum:

> The WTC towers would be classified, by our interpretation of the code, as occupancy Group E, Business; Construction Group 1, Non-combustible; and Construction Classification 1-B, (since there are no area or height limitations applicable).

2.2.5 The 1968 New York City Building Code

Applicability to Port Authority Properties

Established in 1921, the Port Authority is a self-supporting, public interstate agency and is not subject to the local laws of jurisdictions where its properties are constructed. This means that for the construction of the WTC buildings, the Port Authority was not bound by the NYC Building Code or any regulations requiring inspection or approval of the building construction or operation. The Port Authority could establish its own requirements, conduct its own inspections, and enforce its own rules without independent oversight.

According to a joint report written by the Fire Commissioner and Commissioner of Buildings on March 15, 1993 (after the 1993 bombing), in 1975 the NYC Council submitted a resolution to the New York State Legislature to require the Port Authority to comply with the NYC Building Code when building within the City (Rivera and Rinaldi 1993). The 1993 report includes the following statements with respect to jurisdiction over the WTC complex:

> After several major fires in the 1970s, the Fire Department in 1975 testified at the City Council for the need to have jurisdiction over this complex as well as other buildings owned by public benefit corporations, again particularly for Local Law 5 compliance. As a result, the City Council forwarded a Resolution dated August 29, 1975, to the State legislature. … Proposed legislation which would have granted City agencies jurisdiction was introduced in the State legislature over the years; the State has not enacted such legislation.

It appears that there was friction between the FDNY and the Port Authority as evidenced by the following statements in the same joint report (Rivera and Rinaldi 1993):

> Prior to the February 26, 1993, explosion, the Fire Department acted pursuant to the joint protocol for inspectional activity at the WTC which was signed in 1986. The Port Authority's policy was to voluntarily cooperate with the Fire Department 'to the fullest extent practicable.' Fire Department representatives met continually with Port Authority officials to discuss problems with the WTC's emergency procedures and fire safety equipment. Generally, the Port Authority was cooperative and verbally informed the Fire Department that it was their intent to fully comply with Local Law 5. However, since its compliance with fire code requirements was dependent upon economic and design feasibility, the PA agreed to comply with selected provisions of the code, but has not fully done so. Moreover, it was difficult for the Fire Department to monitor code compliance by the WTC because the WTC consistently asserted its legal exemption from local law. Fire officials relied on persuasion and negotiation to gain compliance. The extent of these negotiations is reflected in the voluminous WTC files maintained at the Fire Department. Code compliance at the WTC has been dealt with by every Fire Commissioner and Chief of the Department over the last twenty-five years.

It was not until 1993 that a formal agreement was reached between the Port Authority and the NYC Department of Buildings with regard to code conformance for Port Authority buildings constructed in NYC (PANYNJ and NYCDOB 1993). The introduction of the memorandum of understanding contained the following statements:

> While the facilities of the Port Authority, an agency of the States of New York and New Jersey, are not technically subject to the requirements of local building codes, the long-standing policy of the Port Authority has been to assure that its facilities meet and, where appropriate, exceed Code requirements.
>
> The purpose of this Memorandum is not only to restate that longstanding policy as part of an understanding with the City but to provide specific commitments to the Department, as the agency of the City responsible for assuring compliance with the Code, regarding procedures to be undertaken by the Port Authority for any Project at its facilities in the City to assure that the buildings owned or operated by the Port Authority within the City are in conformance with the Building Standards contained in the Code.

Some salient points included in this agreement are:

- Each project would be reviewed and examined for compliance with the Code;
- All plans would be prepared, sealed, and reviewed by New York State licensed professional engineers or architects; and

- The Port Authority engineer or architect approving the plans would be licensed in the State of New York and would not have assisted in the preparation of the plans.

This agreement was enhanced in 1995 by the approval of a supplement to the 1993 memorandum of understanding (PANYNJ and NYCDOB 1995). The supplement added that:

- The person or firm performing the review and certification of plans for WTC tenants should not be the same person or firm providing certification that the project had been constructed in accordance with the plans and specifications.

In 1993, the Port Authority entered also into an agreement with the FDNY related to fire safety inspections (PANYNJ and FDNY 1993). The introduction to the memorandum of understanding contains the following statements:

> On April 15, 1993, the Port Authority, in order to maintain and enhance the safety of Port Authority facilities, adopted a policy providing for the implementation of fire safety recommendations made by local government fire departments after a fire safety inspection of a Port Authority facility and for the prior review by local fire safety agencies of fire safety systems to be introduced or added to a facility.
>
> The purpose of this Memorandum of Understanding is to reiterate the Port Authority's commitment to this policy and to set forth certain procedures to facilitate the implementation of this policy for buildings at Port Authority facilities located in New York City.

The agreement recognized the right of the FDNY to conduct fire safety inspections of Port Authority properties in NYC and provided guidelines related to corrective actions.

Port Authority's Transition from the 1938 to the 1968 Code

As discussed in Appendix A, in 1963 the Port Authority instructed the designers of the WTC to follow the then current 1938 NYC Building Code. During this time, the code was in the process of being revised (as noted above), and in 1965, the Port Authority directed its designers to adopt the draft version of the new code for their final designs. Some of the advantages of the new draft code were noted to be the following (Levy 1965):

- Fire towers[7] could be eliminated;
- Provisions for exit stairs were more "lenient;" and
- Criteria for partition weights were more "realistic."

It was not certain whether all the changes being proposed to the 1938 code would be incorporated into the final version of the new code. Thus in 1966, the Chief Engineer of the Port Authority suggested that the

[7] A "fire tower" is a stair tower enclosed within a 4 h fire rated shaft that is entered through a naturally ventilated vestibule. The 1938 Code stipulated that one of the required exits in most buildings over 75 ft in height be a fire tower.

"architect/engineers prepare a listing of the elements of the design which do not conform to old code requirements, but are acceptable under the new. With this list in hand, we could initiate discussions, at top level in the Building Department, to see if we can secure agreement to go along with our design (Kyle 1966)."

A one-page document,[8] dated "2/15/67", with the initials "CKP" listed the following items:

1. Fire tower corridors [sic] eliminated.

2. Number of stairs reduced from 6 to 3. (Old plans had 5 stairs at 3'-8" and 1 stair at 4'-8" for a total population of 390. New plans have 2 stairs at 3'-8" and 1 stair at 4'-8" allowing a population of 390.)

3. The size of doors leading to the stairs are [sic] changed from 3'-8" to 3'-0".

4. All stairs exit through a lobby. Old plans had fire tower stair exiting through a fire enclosed corridor.

5. Shaft walls are changed from a 3-hour rating to a 2-hour rating.

6. Corridors are limited to a 100' dead end and with a 2-hour rating.

7. Additional [word(s) missing] changed from 20 pounds per square foot to 6 pounds per square foot (based on partition weight of 50 pounds to 100 pounds per linear foot).

Apparently, the above list represents elements of the WTC design that would not have satisfied the 1938 code, but did satisfy the then-current draft version of the new code.

A letter dated February 18, 1975, from the architect-of-record to the Port Authority discusses compliance with the 1968 NYC Building Code (Solomon 1975). This letter begins with the following paragraph:

> In accordance with the instructions issued by the Port Authority at the start of the project, construction drawings for the World Trade Center were to conform with requirements of the Building Code of New York City, and any variations therefrom were to be called to the attention of the Port Authority for final decision and authorization. This procedure has been followed in the production of the contract drawings and, with the exceptions authorized by the Port Authority noted below, the drawings are in accordance with the new Building Code adopted in December, 1968. The Building Department reviewed the tower drawings in 1968 and made six comments concerning the plans in relation to the old code. Specific answers noting how the drawings conformed to the new code with regard to these points were submitted to the Port Authority on March 21, 1968.

[8] "Changes to Building to Conform to New New York City Building Code," dated 2/15/67.

The same letter continues with a list of four items that the architect was "instructed by the Port Authority to deviate from code (Solomon 1975)." The four items are:

- Omission of vents from closed shafts.
- Demising partitions to stop at suspended ceiling or bottom of truss instead of running from slab to slab.
- Omission of fire protected openings on exterior walls with separation of less than 30 ft.
- Treatment of the concourse level as Underground Street.

Section C26-504.3(a) of the 1968 NYC Building Code required that tenant spaces be separated "by fire separations having at least the fire resistance rating prescribed in table 5-1, but in no case less than 1 hr, and shall continue through any concealed spaces of the floor or roof construction above." The Port Authority chose to stop tenant (demising) partitions at the bottom of the suspended ceiling and use 10 ft strips of 1 h rated ceiling on either side of the partition (Solomon 1969). The general contractor stated in a letter to the Port Authority "…we have been unable to find any precedent for the fire rated ceiling 10' on either side of the demising partitions beyond the one you described from your construction experience on Port Authority hangers [sic] (Endler 1969)."

In a code compliance evaluation report written in 1997, it was stated "Tenant demising partitions, including separations from the public corridor, do not in all cases meet the requirement of being built to the slab above (Coty 1997)." The author of the report recommended that: "Generally, this condition has been and will continue to be remediated as a requirement of new tenant alterations. However, it is recommended that the Port Authority develop and implement a survey program to assure that this remediation process occurs as quickly as possible."

The tenant alteration guidelines issued in 1998 require that tenant partitions have a 1 h fire rating, and the standard details for fire rated partitions indicate a continuous fire barrier from top of floor to bottom of slab (PANYNJ 1998).

Compartmentation and Sprinklers

Neither the 1968 NYC Building Code nor any of the other contemporaneous codes that were examined required sprinklers in tall buildings except for underground spaces. Thus, only the parking garage under WTC 1 and WTC 2 was originally sprinklered. Although Local Law 16, adopted in 1984, required sprinklers in new office occupancies, it was not retroactive. The incentive to retrofit for sprinklers (as explained below) was the passage of Local Law 5 in 1973, which was retroactive.

In the 1968 NYC Building Code, Class 1B construction for business occupancies had no limit on floor area. Local Law 5 required compartmentation of large floor areas in existing business occupancies over 100 ft in height by the installation of fire rated partitions in accordance with the following:

- Compartmentation to 7,500 ft^2 with 1 h partitions; or
- Compartmentation to 10,000 ft^2 with 2 h partitions; or

- Compartmentation to 15,000 ft^2 with 2 h partitions and smoke detectors.

Compartmentation was not required, however, if "complete sprinkler protection" were provided. Compliance dates for these provisions were revised in 1979 by Local Law 84 so that one-third of the total area of buildings had to be in compliance by December 13, 1981, two-third of the total area had to comply by August 7, 1984, and full compliance was required by February 7, 1988.

Following the February 13, 1975, fire in the lower stories of WTC 1 (Powers 1975), an independent consultant was retained to review WTC life-safety provisions, including response to Local Law 5. It is reported that the "consultant concluded that the existing structural fire retardants of the building are sufficient to make the probability of serious structural damage extremely remote and the degree of vertical compartmentation provided sufficiently limits the spread of fire in the structures but that the spread of smoke requires attention from a life safety standpoint (PONYA 1976)." The consultant reported that "…either of the two fire protection options provided for under Local Law 5 would provide a good level of occupant life safety within the World Trade Center complex, provided that whichever is selected is supplemented by certain additional measures." The consultant provided a series of recommendations to supplement either the compartmentation option or the sprinklering option.

The Port Authority initially decided to adopt the compartmentation option in response to Local Law 5. The summary of the January 1976 report on the *Fire Safety of the World Trade Center* lists the following actions to be implemented to enhance the fire safety of the WTC towers (PONYA 1976):

1. The openings between floors of telephone closets, which was a source of fire spread during the February 13, 1975, fire should be closed. This work has been accomplished to prevent any reoccurrences of a similar condition.

2. In addition, the Port Authority will proceed with the compartmentation option of Local Law 5, including all of its requirements for fire alarm, communications, and stairway pressurization.

3. Sprinklering of all storage rooms, janitor closets, mail rooms and file rooms in the central core of each floor.

4. Building additional sprinkler capacity and provisions for extension of a sprinkler system to any area of such usage requiring it in the event of an occupancy change.

5. Equipping those doors which are normally kept open to the corridor system, such as doors at consumer service areas, with electromagnetic 'hold open' devices which would be activated by smoke detectors to close the doors.

6. Providing fail-safe automatic door closers, arranged to close upon activation by smoke detectors, for the overhead rolling fire doors separating the below-grade truck dock from the elevator lobby.

7. Developing an optimum mode of operation of the building air-conditioning system to remove smoke from the central core compartments without contaminating adjacent areas.

Thus, while the Port Authority initially chose to implement the compartmentation option, it also chose to provide "for extension of sprinkler system to any area of such usage requiring it." According to the 1993 joint report written by the NYC Fire Commissioner and Commissioner of Buildings, in the 1980s the Port Authority began "a program to fully sprinkler the Tower buildings (Rivera and Rinaldi 1993)." The report goes on to state that by March 1993 sprinklering was " nearly complete in Tower 2 and 85 percent complete in Tower 1." The report also included a table that summarized "the major system requirements of Local Laws 5/73 and 16/84 with conditions in place when the1993 explosion occurred." The content of that table is reproduced here as Table 2–2.

The tenant alteration guidelines issued in 1998, contained the following requirement and information (PANYNJ 1998):

> All tenant spaces shall be sprinklered. Except for a few areas, most tenant floors in The World Trade Center are provided with wet-pipe sprinkler systems. New tenants normally require a new sprinkler system. For renovations of existing spaces, modifications to the existing system are normally needed to comply with any new partition configuration.

Because Local Law 16 required that business occupancies taller than 100 ft be sprinklered, WTC 7 was sprinklered during the original construction.

Emergency Egress

The 1968 NYC Building Code has requirements for the number and capacity of stairs and for the assumed occupant load that are similar to requirements in the other contemporaneous codes (see Appendix A). Codes of the time required that multiple stairs be located "as remote from each other as practicable." NYC permits scissor stairs,[9] and the code requires the exit doors to be at least 15 ft apart. Local Law 16 (1984) first imposed a remoteness requirement of 30 ft or one-third the maximum travel distance of the floor (whichever is greater), which was not retroactive, so it did not apply to WTC 1 and WTC 2 but did apply to WTC 7.

The 1968 NYC Building Code also has a requirement that, " …vertical exits should extend in a continuous enclosure to discharge directly to an exterior space or at a yard, court, exit passageway or street floor lobby …" (C26-602.4). Similar requirements are found in the 1965 BOCA Basic Building Code and in 1966 NFPA 101, but not in the 1964 New York State Building Construction Code or the 1966 Municipal Code of Chicago. Current code language (2003 IBC, section 1003.6) defines continuous as: not " … interrupted by any building element other than a means of egress component."

This requirement was the subject of ongoing discussion with respect to the stairs in WTC 1 and WTC 2 discharging onto the mezzanine level, which was not at street level but rather at the Plaza level. It was the

[9] Scissor stairs refers to two separate interior stairways contained within the same enclosure and separated by a fire rated partition.

position of the Port Authority that the Plaza was like a street, and the arrangement met the intent of the Code.

Table 2–2. Summary of compliance with Local Laws 5/73 and 16/84 provided in March 1993 report by Fire Commissioner and Commissioner of Buildings (Rivera and Rinaldi 1993).

	Type of Work {Code Section}	Compliance
1	Compartmentation {504.1(c)}	Not required in sprinklered buildings
2	Smoke shaft of stair pressurization {504.15(c)}	Not required in sprinklered buildings. However, smoke purge and pressurization of corridors with 100% fresh air is provided.
3	Emergency power exit lights {605.2(b)}	Exceeds requirement. Required — On separate circuit ahead of main switch Provided — Separate feeders and emergency generators (NOTE A)
4	Emergency power exit signs {606.2(b)}	Exceeds requirement. Required — On separate circuit ahead of main switch Provided — Separate feeders and emergency generators (NOTE A)
5	Stair and elevator signs {608.0}	Yes
6	Emergency power {610.0}	Exceeds requirement. Required — None Provided — See NOTE A above
7	Sprinklers {1703.1}	Yes 95% completed for one tower [WTC 2] 85% completed for other tower [WTC 1]
8	Class "E" fire alarm signal system {1704.5(f)}	Yes — But air supply and exhaust air to fire floor are not closed off when sprinklers are activated. Note: equivalent system provided by item #2 above and smoke detectors at fans, which stop fans.
9	Fire command and communication {1704.8}	Yes — except that each building does not have its own fire command station
10	Elevator in readiness {1800.8(b)}	Yes — See NOTE A above
11	Removal of locks on elevators and hoistway doors {1801.4}	Yes
12	Firemen's service operation {1801.5}	Yes — See NOTE A above

Fire Alarm Systems

Consistent with practice at the time, the original fire alarm system in WTC 1 and WTC 2 was a manual system with four smoke detectors on each tenant floor, positioned to monitor for smoke entering the HVAC returns and arranged to stop the fans to prevent smoke circulation to non-fire areas. Local Law 5 (1973) included retroactive requirements for fire alarm systems and emergency voice communication systems in business occupancies over 100 ft in height. Subsequently, such systems were installed in WTC 1 and WTC 2 with the required fire command center located in the underground parking garage. Following the 1993 bombing, the fire command stations were relocated to the tower building lobbies with a third monitoring location in the Port Authority offices. There are no code requirements for off-site monitoring of fire alarm systems in this occupancy.

Elevators

Local Law 5 requires that elevators be provided with an emergency recall system. This requirement was incorporated subsequently into the American Society of Mechanical Engineers (ASME) A17.1, Safety Code for Elevators and Escalators, that governs elevator design and operation in all the building codes. The ASME Code requires that:

- All passenger elevators be marked with signs stating that they cannot be used during a fire;
- Fire detectors installed in every elevator lobby and machine room be arranged to initiate a recall of the elevators to the ground floor where the doors open and the elevator is taken out of service; and
- Fire service personnel can use a special key to operate any individual car in a manual mode as long as they feel it is safe to do so.

The elevator and building codes require that at least one elevator serving every floor be connected to emergency power. Refer to Table 2–2 for elevator status in WTC 1 and WTC 2 in 1993.

Structural Stability

As discussed in Appendix A, provisions related to structural stability in the 1968 NYC Building Code were in general agreement with those of the contemporaneous codes that were compared. There were, however, a number of provisions in the NYC code that were not included in the other codes such as uniform partition dead loads based on the weight of the partitions, consideration of loads due to thermal expansion/contraction and shrinkage of concrete, minimum strength requirements for bracing of compression members, and allowance for design wind loads based on wind tunnel tests. The NYC code, however, does not provide a standard protocol for wind tunnel testing to establish design wind loads.

2.2.6 WTC 7 Fuel System

WTC 7 was constructed and owned by Silverstein Properties on land owned by the Port Authority. It was built and operated by Silverstein as a Port Authority tenant alteration (see Appendix A.1). Many of the tenants conducted critical business operations in the building and required uninterruptible power to prevent the loss of information or operational continuity in the event of a power failure. This backup power was provided by diesel generators located in the mechanical spaces of the building. These generators were designed to start automatically in the event of an interruption of the utility supply. The total generator capacity and quantity of fuel stored in the building was sized to tenant needs.

Code Requirements

Design and installation of the WTC 7 emergency power and associated fuel systems were to follow the NYC Building Code. The base system was installed in 1987 with modifications occurring in 1990, 1994, and 1999. Over the period 1987 to 1999, the NYC Building Code provisions discussed below were not changed, so all systems were installed to the same requirements. Some of the key code provisions for the construction and location of fuel storage tanks, piping, and controls are discussed here, and additional details will be published in a separate report.

Tanks [27-828 and 27-829] [10]

All tanks must be fabricated of steel and coated to prevent corrosion. Minimum thicknesses are specified by tank diameter for storage tanks and for so-called "day tanks" (60 gal or 275 gal). Large storage tanks (up to 20,000 gal) may be buried inside or outside the building or on the lowers floor of the building with protection related to the tank capacity. For example, tanks from 550 gal to 1,100 gal must be enclosed in 2 h fire rated, noncombustible construction and tanks larger than 1,100 gal in 3 h construction.

Tanks on floors above the lowest floor are limited to 275 gal and one such tank per story. These "day tanks" must be surrounded by a concrete curb or steel pan with the capacity to hold twice the volume of the tank in the event of a leak. The curb or pan must be provided with a float switch to sound an alarm and shut off the transfer pump in case of tank failure. Appropriate controls (generally a float switch in the day tank) are provided to transfer fuel from the storage tanks to the day tank through a transfer pump and piping, with only one such transfer pump and piping network per day tank.

Piping [27-830]

Piping from transfer pumps to day tanks is required to be enclosed in a shaft of 4 in. thick concrete or masonry with a 4 in. clearance to the fuel pipe. Horizontal offsets may be enclosed in a steel sleeve two (pipe) sizes larger and enclosed in 2 h fire rated construction. The spaces between the fuel pipe and sleeve or shaft must lead to an open sight drain or an open sump so leaks can be detected.

Power Systems Designs

NIST located and reviewed specifications and drawings for each of the emergency power systems. It was noted that some of the fuel risers were installed in existing shafts containing other utilities. The NYC Building Code requires that pipe shafts containing piping from the transfer pump to storage tanks above the lower floors not be penetrated by or contain other piping or ducts [27-830(f)(5)]. Correspondence relating to the system for the Mayor's Office of Emergency Management shows that this system was

[10] Sections of the New York City Building Code in which these requirements are found. These provisions are found in the subchapter on "Heating and Combustion Equipment."

reviewed and inspected by the FDNY, a list of needed corrections was produced, and each item was initialed as the corrections were verified.

Base Building System

The initial base emergency power system was installed in 1987, and consisted of two 900 kW generators and a 275 gal day tank located on floor 5. Main fuel storage was in two 12,000 gal tanks buried under the loading dock on the south side of the building. The tanks were double wall fiberglass[11] with leak detectors between the walls.

Fuel was transferred by one of the two pumps through a 2 in. supply line in an existing shaft containing other utilities, near the west bank of passenger elevators. The transfer pump was controlled by a float switch in the day tank with a low (pump on) and high (pump off) position. An alarm would be sounded if the fuel level in the day tank fell below the low level or went above the high level. The day tank was located within a 550 gal pan fitted with an alarm and another pump cutoff. The vent for the day tank terminated outside the south wall.

The 2 in. fuel lines were encased in a second pipe covered with 2 in. of calcium silicate to provide the required 2 h fire rating. Pipe supports were located approximately 10 ft apart, and inspection plugs were provided approximately 50 ft apart. Mechanical equipment rooms were sprinklered (ordinary hazard group I), and the fuel pump room was sprinklered (ordinary hazard group III). The generator area on floor 5 was not sprinklered.

Modifications to System

From 1990 to 1999, four major modifications (additions) were made to the base emergency power system. These modifications are summarized in Table 2–3. Of significance are the 1990 modification (Salomon Brothers) that required a pressurized fuel supply system, because a day tank already existed on floor 5, and the 1999 modification (Mayors' Office of Emergency Management) that required a separate 6,000 gal tank on the first floor. Figure 2–1 is a schematic of the locations of the various components of the base system and the four major modifications.

For the Salomon Brothers system, the transfer pumps were powered from the output of the generators. In the event of a failure of utility power, all nine generators were started automatically to ensure that if any of the nine did not start there would be enough power. Once the generators were up to speed, the control system would shut down those that were not needed, but these could be restarted later if power demand increased. There was enough fuel and residual pressure in the lines to start the generators and to run them for a few minutes, but once running, the fuel pumps were powered to supply fuel. As long as any one generator was running, the pumps ran at full capacity.

[11] While the NYCBC requires steel tanks, effective in November of 1985 the U.S. Environmental Protection Agency required (40CFR280) that all new underground fuel storage tanks be double wall fiberglass and that any steel tanks older than 20 years be replaced by double wall fiberglass.

Table 2–3. Summary of modifications to base emergency power system in WTC 7.

Year	Day Tank/Generator	Storage Tank	Piping	Comments
1990	No day tank permitted since base design included one on floor 5/9 generators on floor 5, 1750 kW combined capacity	Two 6,000 gal next to base tanks.	Two 2 1/2 in. pipes in separate rated shaft	50 psi pressurized fuel system
1994	50 gal/125 kW on floor 9; generator room sprinklered	Used existing base tanks	1 1/4 in. in new 2 h rated dedicated shaft	New transfer pump connected to existing storage tanks
1994	275 gal/350 kW on floor 8; generator room sprinklered	Used existing base tanks	2 in. in same dedicated shaft as above	New transfer pump connected to existing storage tanks
1999	275 gal/three 500 kW on floor 7; smoke detectors in generator room	6,000 gal on floor 1, in 4 h rated enclosure; gaseous (clean) fire suppression system; space below tank sprinklered	10 gauge conduit in 2 h rated enclosure	Storage tank kept filled from base storage tanks.

2.2.7 Preliminary Findings

The following preliminary findings are based on (1) review of the design and construction documents of WTC 1, 2, and 7; (2) review of the 1968 NYC Building Code, the 1964 New York State Building Code, the 1967 Municipal Code of Chicago, the 1965 BOCA Basic Building Code, and the 2001 NYC Building Code; and (3) correspondence of the Port Authority, design consultants, and general contractor.

1. Building code used for design of WTC 1, 2, and 7

   When the design of WTC 1 and WTC 2 began in 1962, the governing building code in NYC was the 1938 edition. In September 1965, the Port Authority instructed its consultants to revise their design for WTC 1 and WTC 2 to comply with the second and third drafts of the new building code of NYC that was under development (Levy 1965). The new building code was adopted on December 6, 1968.

   The Port Authority took advantage of some of the less restrictive provisions of the 1968 Code compared with the outdated 1938 Code. Some of these new provisions included:

   a. Elimination of a fire tower as a required means of egress;

   b. Reduction in the number of required stairs;

   c. Reduction in fire rating of shaft walls from 3 h to 2 h;

   d. Use of uniform partition load that depends on weight of partition per unit length; and

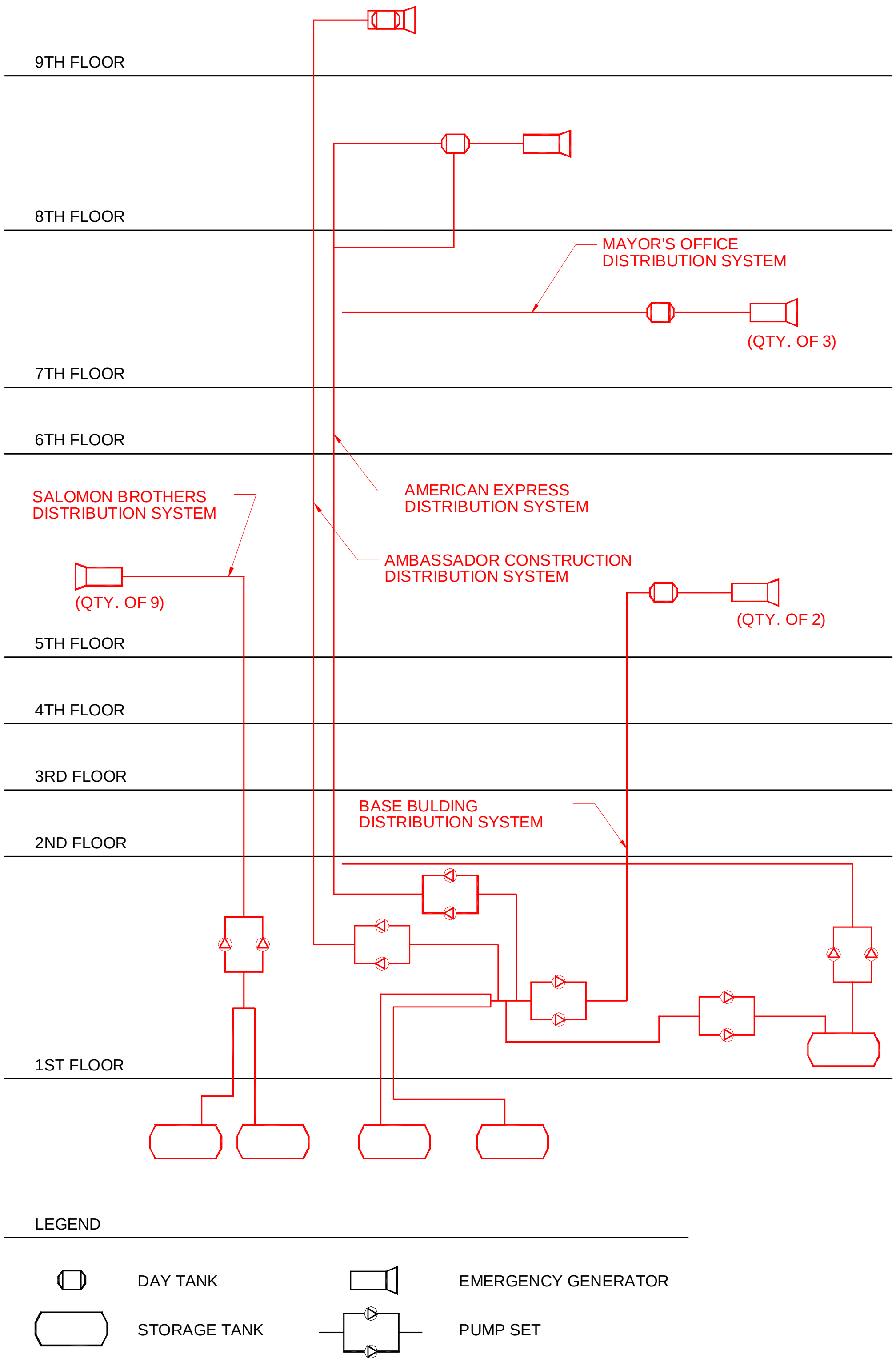


Figure 2–1. Locations of emergency generators and fuel tanks in WTC 7.

e. Allowance of Class 1B construction for business occupancy and unlimited building height.

One of the reviewed documents noted that "For office buildings there is <u>no</u> economic advantage in using Class 1A Construction, and ER&S [architect-of-record] used Class 1B Construction for the WTC Towers and Plaza Buildings…." (Feld 1987).

The design of WTC 7 followed the 1968 code as amended by Local Laws, which includes Local Law 16, requiring sprinklers in buildings taller than 100 ft.

2. Review of design documents by the Department of Buildings of NYC

In 1963, the Port Authority instructed its consultants that the design of the WTC should comply with NYC Building Code. The Port Authority also stated that: "When preliminary designs have been completed, the Chief Engineer will review all design concepts with the appropriate municipal agencies before the consultants proceed with the final design (Levy 1963)." This implies that the NYC Department of Buildings would be involved in reviews of the design. A letter in 1975 from the architect-of-record to the Port Authority indicates that in 1968 the NYC Department of Buildings reviewed the tower drawings and "made six comments concerning the plans in relation to the old code (Solomon 1975)." It was stated further that on March 21, 1968, the architect submitted to the Port Authority responses to these comments "noting how the drawings conformed to the new code." NIST is attempting to locate a copy of this correspondence to determine, if possible, the level of review conducted by the Department of Buildings and the specific six items identified in that review.

3. The 1968 NYC Building Code compared with contemporaneous codes

The 1968 NYC Building Code was more comprehensive in the coverage of provisions in certain areas compared with the other contemporaneous codes that were reviewed. For example, the 1968 NYC Building Code requires special design consideration for expansion and contraction due to temperature and shrinkage of concrete, and it permits determination of design wind loads from tests. In general, except for permitted construction classifications, provisions for structural stability, fire safety and egress were similar among the four contemporaneous codes that were compared (see Appendix A.5.1 and A.5.2 for details). There were, however, differences in permitted live load reduction, design wind pressures, and the treatment of partition loads. Table A–8 in Appendix A provides a summary of fire safety provisions for high-rise buildings used for business.

In the 1993 joint report, the Fire Commissioner and Commissioner of Buildings made this comment about the NYC Building Code (Rivera and Rinaldi 1993):

> We pride ourselves that our codes are among the most stringent in the nation, and we have been in the forefront in applying technological advances to assure fire and structural safety in buildings.

4. Permitted construction classification for high-rise buildings

   The 1938 NYC Building Code permitted only Class 1 (fireproof) construction for unsprinklered office buildings of unlimited height and area, which required a 4 h fire rating for columns and a 3 h rating for floor framing members (4-3 rated construction). The 1968 New York Building Code subdivided Group 1 (noncombustible) construction into Class 1A and 1B. Class 1B required a 3 h fire rating for columns and a 2 h rating for floor framing members (3-2 rated construction). There was precedence in earlier model building codes for permitting 3-2 fire rated construction for unsprinklered high-rise office buildings (BOCA 1950 and SBCC 1946). Of the contemporaneous codes that were reviewed, the 1964 New York State Building Code and the 1965 BOCA Basic Building Code permitted 3-2 rated construction for unsprinklered office buildings of unlimited height and area. The 1967 Chicago Municipal Building Code permitted only 4-3 rated construction for high-rise office occupancy. (See Appendix A.4.4 for more detail.)

5. Deviations from NYC Building Code

   In 1975, the architect-of-record wrote a letter to the Port Authority pointing out that the Port Authority had provided instructions to deviate from the NYC Building Code with respect to four items (Solomon 1975). One of these was a deviation from the requirement that tenant fire-rated partitions be continuous from floor to floor. Over the years, these partitions were replaced with partitions that were continuous as required by the 1968 Code. In the 1997 report on code compliance, it was noted that some partitions did not meet the requirement (Coty 1997). The 1998 tenant alteration guidelines require that core walls have a 2 h fire rating and walls separating tenants (demising walls) have a 1 h rating (PANYNJ 1998). The standard details for 2 h and 1 h rated partitions show that the partitions provide a continuous fire barrier from top of floor to underside of slab.

6. Compartmentation and sprinklering

   Following the passage of Local Law 5 in 1973, the Port Authority implemented a program to proceed with the compartmentation option of Local Law 5 and to provide for the extension of the sprinklering system beyond the below grade spaces installed during original construction. In addition, sprinklers were installed in storage rooms, janitor closets, mailrooms and other spaces in the core area of each floor, and outside the core for tenants not selecting the compartmentation option. In the 1980s, the Port Authority began a program to sprinkler the remaining tenant spaces, initially as tenants changed, and later on negotiated schedules. According to Local Law 86, passed in 1979, full compliance with Local Law 5 was required by February 7, 1988. A report in 1997 states that there were four floors and the sky lobbies (all in WTC 1) that remained to be sprinklered, and that installation of sprinklers on these floors was in progress (Coty 1997). In the October 1999 report on code compliance, it is stated that sprinklering of the tenant floors was completed and sprinklering of the sky lobbies was “currently under way” (PANYNJ 1999). The tenant design guidelines in 1998 require that all tenant spaces be sprinklered (PANYNJ 1998).

7. Fuel system for emergency generators in WTC 7

   The generators and associated fuel distribution system installed in WTC 7 followed the requirements of the NYC Building Code with two exceptions. First, the underground storage tanks were fiberglass and not steel, but this is consistent with federal requirements promulgated by the U.S. Environmental Protection Agency, which preempt the City requirements. The second deviation was the installation of the fuel risers for the base system in an existing shaft with other utilities. All of the subsequent sets of fuel risers were installed in a separate 4 h rated shaft.

   The modification in 1990 included a pressurized fuel system, and the generators powered the fuel pumps. As long as one generator was running, the pumps ran at full capacity.

2.3 BASELINE STRUCTURAL PERFORMANCE AND AIRCRAFT IMPACT DAMAGE ANALYSIS (PROJECT 2)

2.3.1 Project Objective

Project 2 of the NIST investigation into the collapse of the WTC towers focuses on (1) establishing the baseline structural performance of each of the two towers under design gravity and wind loads and (2) analyzing the aircraft impacts into each of the two towers to estimate the damage to the towers and establish the initial conditions for the fire dynamics modeling in Project 5 and thermal-structural response and collapse initiation analysis in Project 6. The objective of the project is to evaluate the role of the structural system and the abnormal loads from aircraft impact on the collapse of the WTC towers by (1) developing reference structural models of the WTC towers that serve as reference for more detailed models to be developed for Projects 2 and 6, (2) using these models to establish the baseline performance of each of the towers under design loading conditions, (3) estimating probable damage to the structural, mechanical, and architectural systems of the towers due to aircraft impacts, (4) evaluating the role of floor diaphragms and hat trusses on the structural integrity of the towers, and (5) estimating the structural reserve capacities of the towers under service loading conditions after losing a number of exterior and core columns and floor segments due to aircraft impact.

2.3.2 Project Approach

This project is divided into two primary focus areas. The first, related to establishing the baseline performance of the towers under design loading conditions, is divided into the following tasks:

- **Develop Structural Databases of the Primary Components of WTC 1 and WTC 2.** To develop electronic databases for the major structural components of the WTC towers from original computer printouts of the structural design documents and modifications made after construction. This task will also estimate all cross-sectional properties and link the databases into a format suitable for the development of the reference structural models of the towers.

- **Develop Reference Structural Analysis Models of WTC 1 and WTC 2.** To use the structural databases to develop finite element structural models of WTC 1 and WTC 2 that capture the intended behavior of the towers. The models include typical floor models and global models of the towers. The models are used to establish the baseline performance of each of the towers under gravity and wind loads and serve as reference for more detailed structural models to be used for other phases of the investigation.

- **Estimate Wind Loading Criteria on the WTC Towers Based on the State of the Art.** To develop wind loads on the towers based on currently available aerodynamic information (from two wind tunnel tests conducted recently by parties to an insurance litigation concerning the towers) and on extreme climatological information from available data and applicable standards.

- **Establish the Baseline Performance of WTC 1 and WTC 2 Under Design Loading Conditions.** To use the reference structural models to analyze the two towers to estimate stresses, deflections, and member utilization ratios under the following loads:

  – Gravity loads considering the following cases: (1) dead loads, (2) live loads used in the original design of the towers, and (3) live loads according to the current American Society of Civil Engineers (ASCE) 7 Standard.

  – Lateral wind loads considering the following cases: (1) wind loads used in the original design of the towers, and (2) wind loads based on the state of the art.

The second focus area, related to analyzing the aircraft impacts into each of the two towers, is divided into the following tasks:

- **Analyze the Aircraft Impacts into WTC 1 and WTC 2.** To analyze the aircraft impact into each of the two towers to provide the following: (1) estimates of the damage to structural systems due to aircraft impact – including exterior walls, floor systems, and interior core columns; (2) estimates of the aircraft fuel dispersal during the impact; (3) estimates of accelerations and deformations in each of the two towers due to aircraft impact to be used for estimating damage to fire proofing; and (4) a database of the major fragments of the aircraft and destroyed structural components of the towers to be used for estimating damage to the mechanical and architectural systems inside the towers. The impact analyses are conducted at various levels including: (1) the component level, (2) the subassembly level, and (3) the global level to estimate the probable damage to the towers due to aircraft impact. The analyses also include simplified and approximate methods. This task will include the development of detailed models of the aircraft and the towers at the impact zone.

- **Analyze the Post-Impact Stability of WTC 1 and WTC 2.** To examine the stability of each of the two towers and determine the reserve capacity after losing columns and floor segments due to aircraft impact and show that the towers did not collapse immediately after impact. The analyses will help understand the mechanism by which the towers remained standing after impact, including the load redistribution provided by the hat truss system, and determine how close to collapse were each of the towers immediately after impact.

- **Perform Sensitivity and Probabilistic Analysis of Aircraft Impact.** To (1) conduct a sensitivity analysis to assess the effects of variability associated with various input parameters and identify the most influential parameters that affect the damage estimates and

(2) perform probabilistic analysis to determine the probabilities associated with different damage estimates.

Work completed to date on the above tasks is summarized in the following sections.

2.3.3 Development of Structural Databases for WTC 1 and WTC 2

The development of structural databases of the primary components of the towers has been completed under a contract from NIST by the firm of LERA, the firm responsible for the structural engineering of the WTC towers. The work included digitization of the original drawing books with tabulated information, a quality control procedure to ensure consistency of the generated databases with original design documents, cross sectional property calculations, and developing relational databases to link the database files into a format suitable for models development. The developed databases include modifications made after construction.

NIST has implemented a rigorous review procedure to mitigate potential conflicts of interest and to ensure the integrity and objectivity of the deliverables. The review procedure includes an in-house NIST review and a third-party review by the firm of Skidmore, Owings, & Merrill (SOM) also under a contract from NIST. The third-party review by SOM included random checks of the digitized structural databases and cross section property calculations. The review indicated no discrepancies between the developed databases and the original drawing books. The in-house NIST review included: (1) line-by-line review of all database files, (2) random checks on the developed databases by project leader, and (3) calculation of all cross section properties and comparing with those in the developed databases. The review indicated minor discrepancies between the developed databases and the original drawing books. These discrepancies were reported to LERA, who implemented the changes and modified the databases accordingly. Consequently, the structural databases have been approved by NIST and are being made available for other phases of the NIST investigation.

Additional details on the development of the structural databases appear in Appendix B.

2.3.4 Development of Reference Structural Models for WTC 1 and WTC 2

The development of the reference structural models for the towers has been completed by LERA. These are three-dimensional, linear, finite element models (FEMs) of the towers developed using SAP2000 software. The models include:

- Typical truss-framed floor model (floor 96 of WTC 1): The model contains all primary structural members of the floor system, including primary and bridging trusses, beams in the core, strap anchors, viscoelastic dampers, exterior and core columns above and below floor level, spandrel beams, and concrete slabs. Initial verification of the model has also been performed.

- Typical beam-framed (mechanical) floor model (floor 75 of WTC 2): The model contains all primary structural members of the floor system, including composite beams, horizontal trusses, viscoelastic dampers, exterior and core columns above and below floor level, spandrel beams, and concrete slabs. Initial verification of the model has also been performed.

- Global models of each of the two towers: These are models of the 110-story above grade and 6-story below grade structure for each of the two towers and include the following six main parts: core columns, exterior wall (foundation to floor 7), exterior wall trees (floors 7 to 9), exterior wall (floors 9 to 106), exterior wall (floors 107 to 110), and hat trusses. These models were developed separately and then assembled into a unified model. Rigid and flexible diaphragms representing the floor systems, core bracing, and loads were then added to the unified model. Parametric studies were undertaken to establish the idealizations used in the global models. These studies included detailed shell element and simplified beam element models for typical exterior wall panels and exterior corner panels. The parametric studies also included development of a simplified flexible floor diaphragms calibrated against the detailed floor models. Initial verification of the global models has also been performed.

Similar to the structural databases, the developed reference models were thoroughly reviewed. As part of the review process, NIST conducted a workshop for NIST investigators, outside experts, and contractors to review the reference structural models developed by LERA. The purpose of the workshop was to discuss the methodology, assumptions, and details of the developed reference models. The feedback from individual workshop participants was included in the final review of the models.

The in-house NIST review and the third-party review by SOM included: (1) checks on the consistency of the developed reference models with the original structural drawings and drawing books, and (2) verification and validation of the models, including reviewing assumptions and level of detail and performing analyses using various loading conditions to assess the accuracy of the models. The reviews indicated minor discrepancies between the developed reference models and the original design documents. The reviews also indicated that, in general, the modeling assumptions and level of detail in the models were accurate and suitable for the purpose of the project. The reviews identified two areas where the models need to be modified. The first is the effect of additional vertical stiffness of the exterior wall panels due to the presence of the spandrel beams. The second area is the modeling of the connections of the floor slab to the exterior columns of the typical beam-framed floor model, where this connection appeared to be fixed while the connection should be modeled as pinned. The minor discrepancies and the areas identified for modification were reported to LERA, who implemented the changes and modified the models accordingly. Consequently, the reference structural models have been approved by NIST and are being made available for other phases of the NIST investigation.

More details on the development of the reference structural models appear in Appendix B.

2.3.5 Estimates of Wind Loads on the WTC Towers

The development of estimates of wind loads on the WTC towers has been completed by NIST on the basis of the current state of the art in wind engineering. The estimates make use of wind tunnel test results and extreme wind climatological estimates obtained by Rowan Williams Davis and Irwin, Inc. (RWDI) and by Cermak Peterka Peterson, Inc. (CPP) as part of insurance litigation concerning the WTC towers. In addition, the estimates of wind-induced forces and moments on the WTC towers make use of independent extreme wind climatological estimates performed by NIST, based on airport wind speed data obtained from the National Climatic Data Center, National Oceanic and Atmospheric Administration, and on the NIST hurricane wind speed database.

A comparison of estimates by CPP and RWDI of wind-induced maximum base moments on WTC 2 indicates a difference of about 40 percent between the two estimates. NIST studied the two wind tunnel reports and attempted to identify the sources of disagreement between them in order to develop the wind loading on the towers. The NIST study included: estimates of the wind speeds for the direction that corresponds in the CPP and RWDI reports to the peak wind-induced base moment, and a critique of wind profiles used in estimation of wind loads by RWDI and methods used to integrate aerodynamic and extreme wind climatological data (the sector-by-sector approach in the CPP report and the up-crossing method in the RWDI report).

The wind load estimates are currently being reviewed by SOM. Upon completion of the third-party review, the loads will be applied to the global models of the towers as part of the baseline analysis.

2.3.6 Baseline Performance Analysis of the Towers

Work is under way to complete this portion of the study. Significant progress has been made in using the reference models subject to gravity loads (dead loads and live loads used in the original design and according to ASCE 7-02 Standard) and wind loads used in the original design of the towers. Upon completion of loads application into the models, the models will be analyzed to establish the baseline performance of the towers. The results of the analysis will be reported at a later date.

2.3.7 Analysis of Aircraft Impacts into WTC 1 and WTC 2

The objective of the analysis of aircraft impacts into the WTC towers is to estimate the impact response of the towers, including damage to structural systems, acceleration environment, and fuel and debris dispersion. The analysis is being conducted at various levels including: (1) the component level, (2) the subassembly level, and (3) the global level to estimate the probable damage to the towers due to aircraft impact. The analyses also include simplified and approximate methods. NIST is working with experts from Applied Research Associates, Inc. (ARA) under a contract from NIST to conduct these analyses. The commercially available finite element analysis (FEA) software, LS-DYNA is being used for most impact analyses in this project.

The development of constitutive models describing the actual behavior of the structure under the dynamic impact conditions of the aircraft is an important step prior to conducting the impact analyses. Significant progress has been made to identify the proper constitutive relationships, including high strain-rate effects and failure criteria for the various materials included in the analysis of aircraft impacts into the WTC towers. These materials include the various grades of steels used in the exterior walls and core columns of the towers, weldment, bolts, reinforced concrete, and aircraft materials. Details on the development of the materials constitutive models appear in Appendix C.

Another important step prior to conducting the various impact analyses is the development of an aircraft model to be used in the component, subassembly, and global analyses. The model is developed based on information gathered from documentary aircraft structural information, and data from measurements on a Boeing 767 aircraft. The development of the Boeing 767 aircraft model for impact analysis is nearing completion. The engine and wing models have been completed and are being used in the component and subassembly analyses. Also completed is the empennage and landing gears. Work is under way to finalize the model of the fuselage, nose, and nonstructural components of the aircraft. Details on the development of the aircraft model appear in Appendix C.

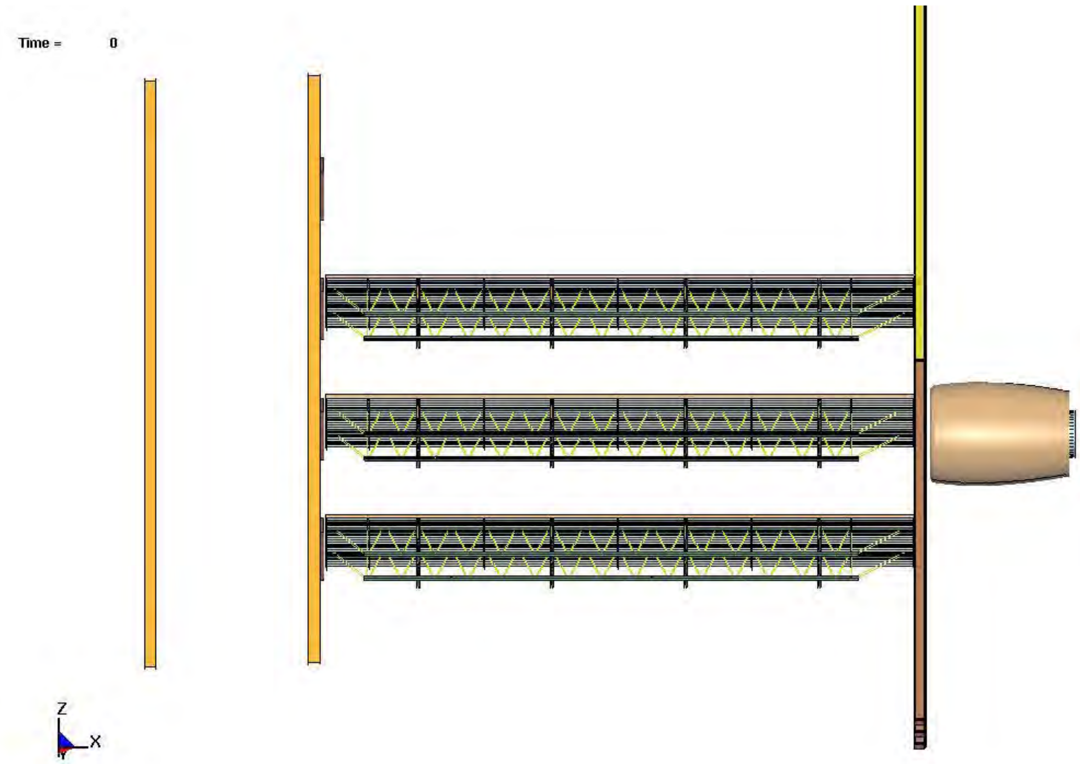
The WTC towers and Boeing 767 aircraft are complex structural systems, and a large database of detailed structural information has been collected on them. In the model development process, the objective was to include all of the primary structural components and details of both the aircraft and towers. This approach, however, results in very large models. The component and subassembly analyses were used to determine model simplifications that can reduce the overall model size while maintaining fidelity in the analysis. Therefore, a series of component impact analyses were performed. The primary objectives of component modeling are to (1) develop understanding of the interactive failure phenomenon of the aircraft and tower components and (2) develop the simulation techniques required for the global analysis of the aircraft impacts into the WTC towers, including variations in mesh density and numerical tools for modeling fluid-structure interaction for fuel impact and dispersion. The approach taken for component modeling is to start with finely meshed, brick element models of key components of the tower structure and progress to relatively coarsely meshed beam and shell element representations that will be used for the global models. Much progress has been made on the component level analyses using models of tower exterior and core columns with column end bolted connections and spandrel bolted connections, as well as floor segments impacted separately with an engine or a wing section with and without fuel. This analysis is nearing completion, and details on the analysis methodology and results appear in Appendix C.

Not reported in Appendix C is progress made on the subassembly analysis. This work is under way. Preliminary subassembly engine impact analyses into a strip from the exterior wall to the core of WTC 1 have been performed. An example analysis, shown in Fig. 2–2, is for a 500 mph engine impact centered on the spandrel for exterior panel 121A at floor 96 and includes core columns 503A and 603A between floors 94 and 98. This model includes a single width exterior panel and floor assembly of the same width. The concrete slab is modeled with brick elements, and the diagonal round bar members in the floor trusses are modeled with beam elements. The remainder of the structures, including the columns, metal decking, and truss upper and lower chord components, are modeled with shell elements. An alternate view of the impact damage at 0.25 s is shown in Fig. 2–3. Current work focuses on expanding the size of the model in width (larger number of exterior panels), height (larger number of floors), and depth (extension all the way through the core) to minimize the effect of boundary conditions on the model response. Details of further work on the subassembly analysis will be reported at a later date.

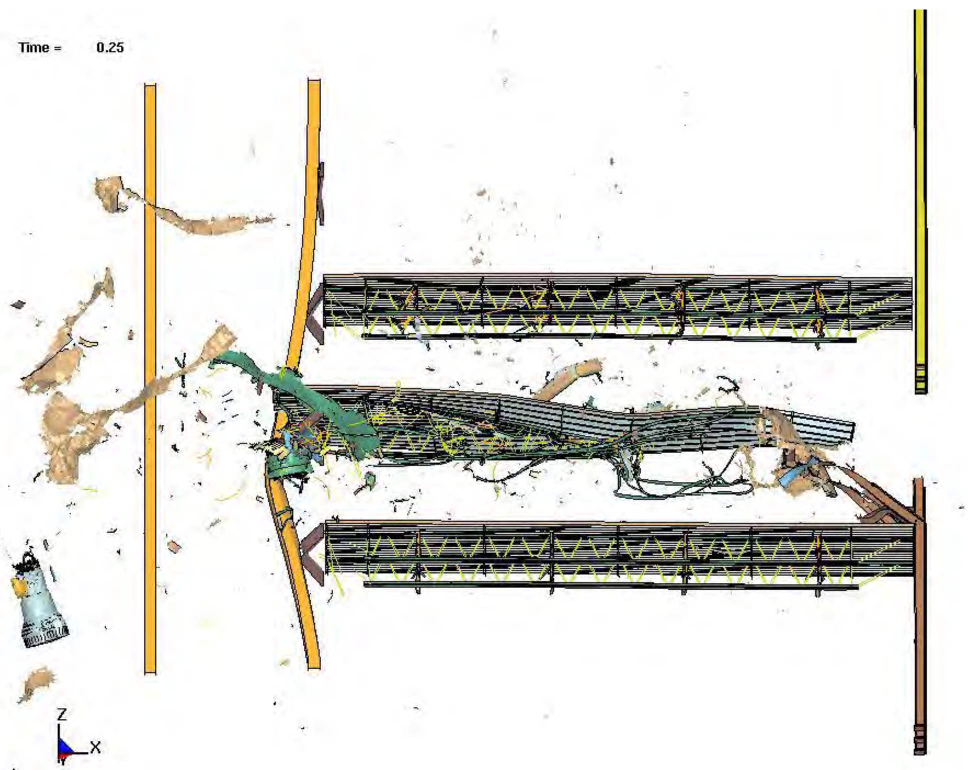
Also not reported in Appendix C is progress made on the development of the models of the towers in the impact zone to be used for the global impact analysis. This work is ongoing. Examples include single floor models in the core (Fig. 2–4), multiple floor models (Fig. 2–5), and exterior wall models (Fig. 2–6). Details of further work on the development of the global models will be reported at a later date.

2.3.8 Preliminary Stability Analysis of the WTC Towers

Preliminary system stability analyses of the WTC towers have been performed to: (1) examine the overall stability of the undamaged tower upon removal of floors, (2) study possible load redistribution mechanisms upon losing columns in the core due to aircraft impact, and (3) study the response of WTC 1 when columns in the exterior walls and the core are assumed destroyed due to aircraft impact and columns in the exterior are damaged due to subsequent fire effects.



(a) Initial configuration



(b) Impact response at 0.25 s

Figure 2–2. Example engine impact subassembly analysis.

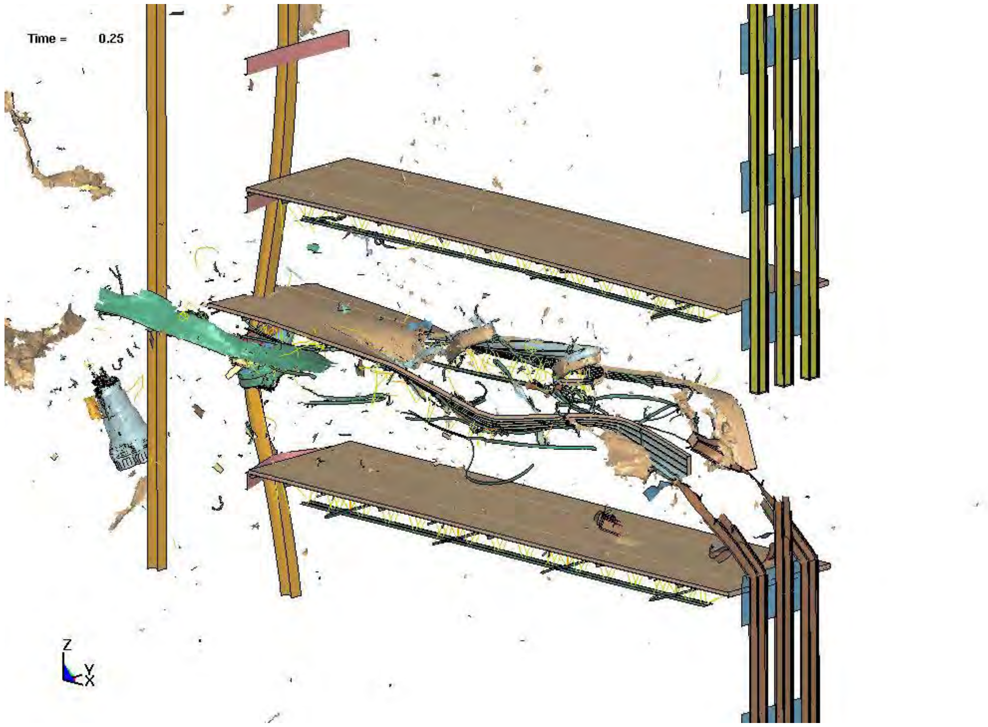


Figure 2–3. Oblique view of the subassembly engine impact damage.

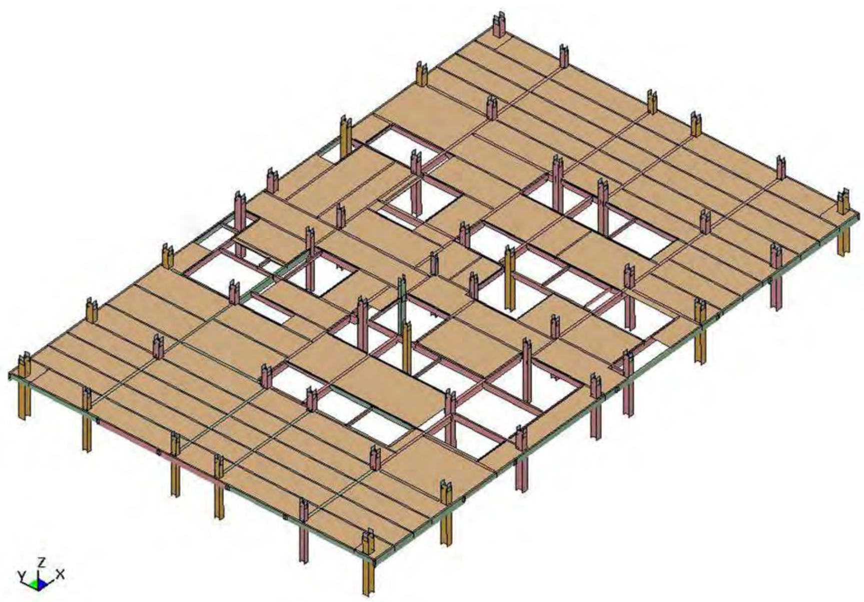


Figure 2–4. Model of the 96th floor and columns of WTC 1.

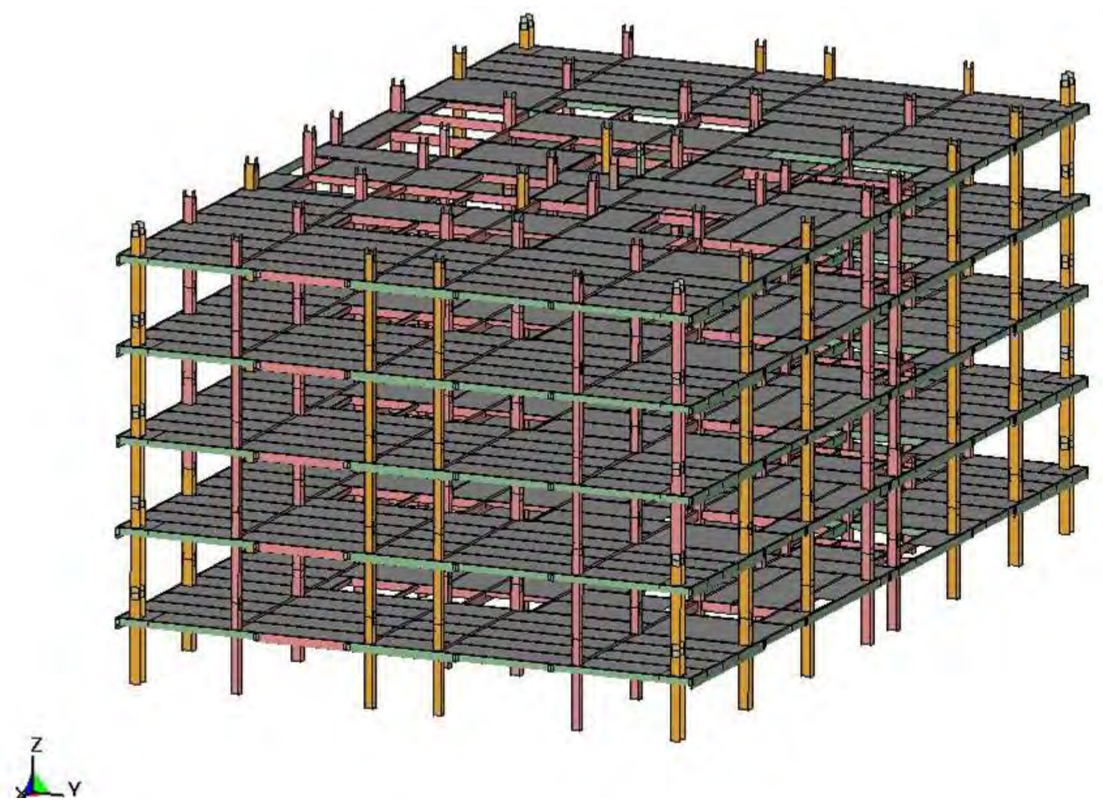


Figure 2–5. Model of the core of WTC 1, floors 94–98.

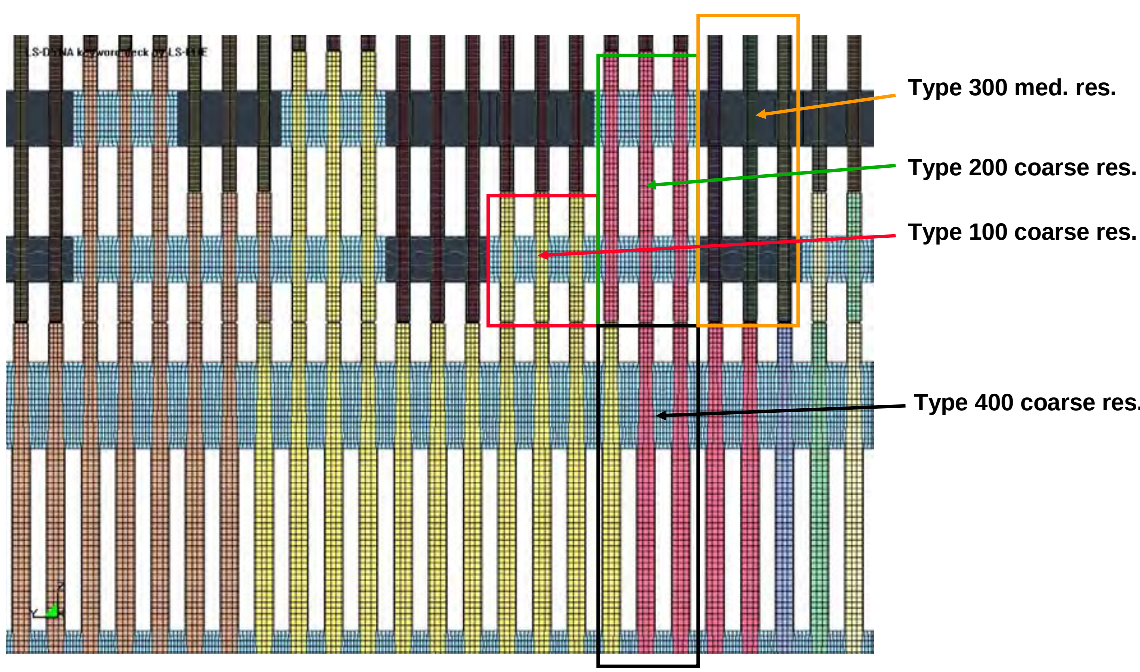


Figure 2–6. Detail of the WTC 2 impact zone exterior column panels.

The analyses used the typical truss-framed floor model and a reduced version of the global reference model of WTC 1 (see Section 2.3.4) with proper modifications. Modifications included adding vertical springs at the bottom of the reduced models to account for the removed lower portion of the towers, and using actual steel properties and actual loads on the towers. The analyses used staged construction technique to account for the sequential construction of the towers, especially in the zone of the hat trusses. Linear buckling analysis and nonlinear analysis with plastic hinges were used to study the effects of removal of floors and loss of exterior and core columns, respectively. In addition, analysis of the floor system, where severed core columns were replaced by equivalent springs representing the stiffness of the hat trusses and columns between the floors and hat trusses, was conducted to study the mechanism by which the floor loads were redistributed when the core columns were severed by aircraft impact.

Details on the preliminary stability analyses appear in Appendix D.

2.3.9 Summary and Preliminary Findings

Significant progress has been made on the first focus area of this project dealing with the baseline performance of the WTC towers. This includes the completion, review, and, final approval by NIST of the structural databases and reference structural models of the towers. Also completed are the NIST estimates of the wind loading on the towers based on the state-of-the-art, which is currently under review. Progress has been made on performing the baseline analysis.

For the second focus area, dealing with aircraft impact into the towers, work is nearing completion on the development of materials constitutive modeling, aircraft model, and component level analyses. Progress has been made on the subassembly and global models development. In addition, preliminary stability analyses of the towers under damage from aircraft impact have been performed.

The following presents some preliminary findings obtained from the component impact analyses (see Appendix C):

- A 500 mph engine impact against an exterior wall panel results in a penetration of the exterior wall and failure of impacted exterior columns. If the engine does not impact a floor slab, the majority of the engine core will remain intact through the exterior wall penetration with a reduction in velocity of about 10 percent and 20 percent. The residual velocity and mass of the engine after penetration of the exterior wall is sufficient to fail a core column in a direct impact condition. Interaction with additional interior building contents prior to impact or a misaligned impact against the core column could change this result.

- A normal impact of the exterior wall by an empty wing segment from the wing tip region will produce significant damage to the exterior columns, but not necessarily complete failure. A fuel-filled wing section impact results in extensive damage to the exterior wall, including complete failure of the exterior columns. This is consistent with photographs showing the exterior damage to the towers due to impact.

- Three different numerical techniques were investigated for modeling impact effects and dispersion of fuel: (1) standard Lagrangian FEA with erosion, (2) Smoothed Particle Hydrodynamics (SPH) analysis, and (3) Arbitrary-Lagrangian-Eulerian (ALE) analysis. Of these approaches, SPH analyses appear to offer the greatest potential for modeling fuel in the global impact analysis due to the combination of both computational efficiency and modeling fidelity.

The following presents some preliminary findings obtained from the preliminary stability analyses under service live loads and subject to the assumptions and the limitations of these models (see Appendix D):

- Linear stability analysis was used to examine the stability of the undamaged WTC 1 under service loads through increased unbraced column lengths (floor removal). The tower was stable when two floors were removed. Two core columns buckled when three floors were removed, but the tower maintained its overall stability. The tower also maintained its stability when four columns buckled with four floors removed. The analysis suggested that global instability of the tower occurred when five floors were removed from the model. Assuming that all columns at the region of the removed floors reached a temperature of 600 °C (reduced modulus of elasticity), the analysis indicates that removal of four floors would induce global instability.

- Analysis of the typical truss-framed floor model with fifteen severed core columns indicated that, under service loads, the floors first attempted to redistribute their loads to the hat trusses through tension in the columns above the damage. The load followed this path due to the relatively large stiffness of the hat trusses-column system compared to the flexural stiffness of the floors. At a certain floor level, column splices fail due to the large tensile forces and the floors below the failed splices must redistribute their loads directly to neighboring undamaged core columns. When only eight core columns were assumed severed, the analysis indicated that the tensile forces in the columns were smaller, due to the relatively larger stiffness of the floor. These forces may still have failed the columns at the splices.

- Nonlinear analysis that included geometric nonlinearities and material nonlinearities using plastic hinges was conducted on the reduced global model of WTC 1. The model assumed the following damage to the tower: (1) due to aircraft impact, loss of columns and spandrels in the north face, and an exterior panel in the south face of the tower, as well as eight columns in the core; and (2) due to fire, loss of columns in the south face, which were shown in videos to be bowing inward a few minutes prior to collapse. The analysis indicated that after aircraft impact, the tower maintained its stability, where the highest stressed elements were the exterior columns next to the damaged area on the north face of the tower. The tower also maintained its stability after losing columns in the south wall due to fire effects with some reserve capacity left, indicating that additional loss or weakening of columns in the core, weakening of additional columns in the exterior, or additional loss of floors is needed to collapse the tower. More detailed models will account for local bucking of columns, and the failure and role of the floor system in redistributing the loads; factors that are not considered in this analysis.

2.4 METALLURGICAL AND MECHANICAL ANALYSIS OF STRUCTURAL STEEL (PROJECT 3)

2.4.1 Project Objective

Structural steel recovered from the WTC site provides information essential to understanding the events of September 11, 2001. Important data available from analysis of the steel include failure modes of the steel that provide clues to the interaction of the aircraft with the buildings and mechanical properties of the steel that assist in modeling of the buildings during impact and under the high temperatures concomitant with the fires. The steel may provide additional clues, such as information on the extent of high temperature exposure of the steel in the fires.

Thus, the objective of Project 3 is to analyze structural steel available from WTC 1, 2, and 7 for determining the metallurgical and mechanical properties and quality of the metal, weldments, and connections, and providing essential data to other investigation projects.

2.4.2 Project Approach

This project is divided into five substantive tasks as follows:

- **Task 1–Physical Evidence.** Collect and catalog the physical evidence (structural steel components and connections) and other available data, such as specifications for the steel, the location of the steel pieces within the buildings, and the specified steel properties.

- **Task 2–Visual Observations.** Document failure mechanisms and damage based on visual observations of recovered steel, especially for available columns, connections, and floor trusses. Photographs taken before collapse will be used to determine damage occurring to the recovered steel before collapse.

- **Task 3–Mechanical Properties.** Determine the metallurgical and mechanical properties of the steel, weldments, and connections, including temperature dependence of properties. The grades of steel will be identified in the columns, welds, spandrels, trusses, truss seats, and fasteners. The identification will include composition, microstructure, mechanical, and impact properties. This task will provide steel property data, including models of elevated temperature behavior for relevant steels, to estimate damage to the structural steel members from aircraft impact, evaluate structural fire response, and study the initiation of structural collapse in Project 6, Structural Fire Response and Collapse Analysis.

- **Task 4–Correlation with Engineering Drawings.** Correlate determined steel properties with the specified properties for construction of the buildings. The quality of the steel used in the buildings will be compared with that specified.

- **Task 5–High Temperature Excursions.** Analyze the steel metallographically to estimate maximum temperatures reached. It is recognized that high temperature exposure before the collapse may be difficult to distinguish from exposure during post-collapse fires.

2.4.3 Physical Evidence

NIST has studied steel elements from the WTC buildings and collected and analyzed documents on steel and welding specifications from the 1960s applicable to the WTC towers. This analysis has resulted in the documents described below.

Catalog of Structural Steel

NIST has catalogued the 236 structural steel elements from the WTC buildings recovered for the investigation. These pieces represent a small fraction of the enormous amount of steel examined at the various recovery yards where the debris was sent as the WTC site was cleared. Components include full exterior column panels, core columns, portions of the floor truss members, channels used to attach the floor trusses to the interior columns, and other smaller structural components (e.g., bolts, diagonal bracing straps, aluminum facade).

NIST catalogued and documented the steel pieces, and when possible, identified markings on the steel which pinpoint the intended as-built location within the buildings. Roughly 0.25 percent to 0.5 percent of the 200,000 tons of steel used in the construction of the two towers was recovered. The recovered steel includes portions of:

- 90 exterior column panels; the as-built location of 41 distinct sections has been unambiguously identified within WTC 1 and WTC 2:
  - 26 panels from WTC 1: 22 from near the impact floors, 4 hit directly by the airplane
  - 15 panels from WTC 2: 4 from near the impact floors.
- 55 wide flange sections and built-up box sections; 12 core columns have been positively identified from WTC 1 and WTC 2, including 1 column from the impact zone of WTC 1 and 2 columns from the impact zone of WTC 2.
- 23 pieces of floor truss material from WTC 1 and WTC 2; however, the as-built location of the trusses within the buildings could not be identified.
- 25 pieces of channel sections that connected the floor trusses to the core columns in WTC 1 and WTC 2; however, the as-built location of the channels could not be identified.

The design drawings for WTC 1 and WTC 2 designate 14 different grades (or strengths) of steel for the exterior panels, four grades for the core columns, and two grades for the floor trusses. Stampings on identified perimeter and core columns indicate that the steel supplied was the appropriate strength as indicated on the design drawings, with the exception that 100 ksi plate was used for the 85 ksi and 90 ksi material called for in the design, leading to a total of 12 grades of steel in the buildings. The recovered structural elements have yielded representative samples of the following:

- All 12 grades of exterior panel material;

- Two grades of core column steel (representing 99 percent, by total number, of the columns); and
- Both grades for the floor truss material.

The collection of steel from the WTC towers is adequate for purposes of NIST's investigation (i.e., chemical, metallurgical, and mechanical property analyses as well as a substantial damage assessment and failure mode examination) to examine why and how WTC 1 and WTC 2 collapsed following the impact of the aircraft and ensuing fires.

More detail on the recovered steel appears in Appendix F, Inventory and Identification of Steels Recovered from WTC Buildings.

Contemporaneous Specifications and Other Documents

As part of an analysis of contemporaneous (1960s era) documents, NIST has studied the building drawings to ascertain the major structural elements and grades of steel in the towers relevant to the investigation. Also, 1960s era steel and welding specifications used to construct the WTC towers have been located and analyzed. The many steels (combinations of strengths and manufacturers) that were used have been characterized based on structural engineering specifications for the buildings and manufacturer documents of the era. Appendix E, Contemporaneous Structural Steel Specifications, also describes the major structural elements in the towers relevant to the investigation.

Ten steel companies fabricated structural elements for the two towers. The floors involved in the aircraft impact and major fires contained steel from four of these companies. Laclede Steel (St. Louis, Missouri) fabricated the trusses for the floor panels that spanned the opening between the core and the perimeter columns. They used steels conforming to ASTM A36 and A242, which they made and rolled in their own mill. NIST chemical analyses and strength tests, as well as contemporaneous mill reports indicate that many of the floor truss components specified as ASTM A36 were actually fabricated with a micro-alloyed steel of considerably higher yield strength.

Pacific Car and Foundry (Seattle, Washington) fabricated the perimeter box column panels (generally 3 columns wide by 3 stories tall) above Floor 9. Although 14 grades of steel (36 ksi to 100 ksi yield strength) were specified in the structural steel drawings, only 12 grades were supplied due to an upgrading of two of the specified steels. Most of the steel came from Yawata Iron and Steel (now Nippon Steel) and Kawasaki Steel, although about 10 percent of the plate was produced domestically, primarily by Bethlehem Steel. Many of these steels were relatively new proprietary steels and were not covered by ASTM standards of the time. In the impact zones of the towers, the perimeter columns damaged by the aircraft were largely of three specified grades: 55 ksi, 60 ksi, and 65 ksi steels.

Stanray Pacific (Los Angeles, California) fabricated the welded core box columns (rectangular columns assembled from four steel plates) above Floor 7, primarily using steels conforming to ASTM A36. The thicker plates came from Colvilles, Ltd. (Motherwell, Scotland, now Corus Steel), while the thinner plates came from Fuji Steel (now Nippon Steel).

Montague-Betts (Lynchburg, Virginia) fabricated the rolled wide-flange core columns and beams above the Floor 9. Much of the steel for the wide-flange columns came from Yawata Iron and Steel. The rest

Montague-Betts purchased from numerous domestic suppliers. For WTC 1, the core columns damaged by impact and fire were mostly wide-flange shapes, since the highest floors of the buildings contained few box columns. In WTC 2, the damaged columns were a roughly equal mix of welded box columns and wide-flange shapes.

Details on the 1960s era documents appear in Appendix E.

2.4.4 Visual Observations

Visual analysis includes analysis of both photographic evidence just before the collapse and analysis of the recovered steel for clues to the performance of the steel structure throughout the event. Airplane impact damage to the towers has been characterized from enhanced precollapse photographs (Fig. 2–7). Such analyses provide input for validation of airplane impact models. In addition, these images allow investigators to determine whether damage observed in the recovered steel occurred before or after the collapse, greatly aiding the failure analyses of these pieces.

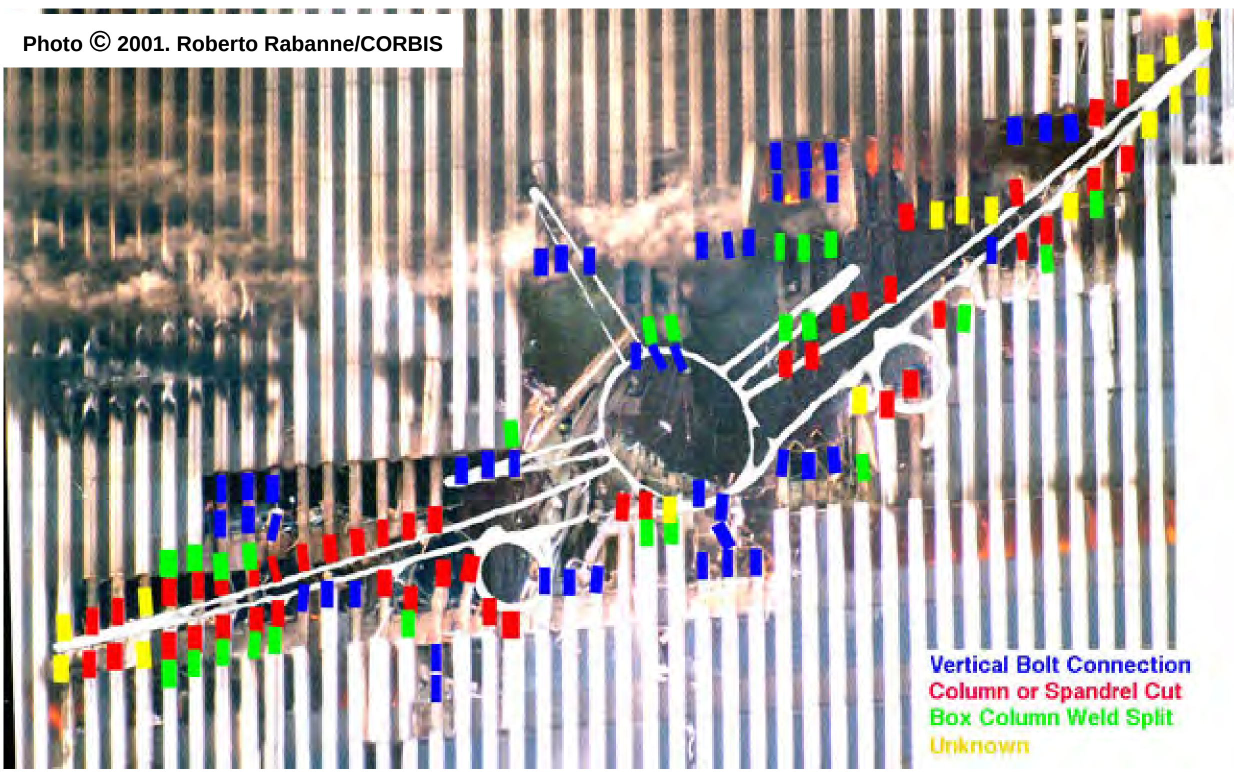


Figure 2–7. Enhanced precollapse image of the north face of WTC 1 with superimposed outline of the Boeing 767. Likely failure modes of damaged columns are indicated.

The recovered steel has been studied to determine failure modes of the various components and connections. In addition, a NIST contractor, Wiss, Janney, Elstner, has surveyed the recovered steel to characterize failure behavior.

2.4.5 Mechanical Properties

Mechanical tests of steel at room temperature (for baseline performance), high temperature (for strength and deformation behavior in fire conditions) and high rates of deformation (to calibrate strength enhancements occurring during airplane impact) are completed and are being analyzed. Preliminary analysis of the perimeter column and truss steel indicates that the quality and strength (relative to required minimum values) of the steel is as expected for steel of the period. Results for the core columns and connections are still being analyzed. Data are being provided to the impact damage and fire modeling teams.

Mathematical models of the stress-strain behavior of 21 steels (various grades and manufacturers) in the WTC towers have been developed and provided to contractors for use in computer models of the behavior of the buildings. The models of the steel behavior are based on the conventional room temperature tests, high strain rate tests, and high temperature characterization.

2.4.6 Correlation with Engineering Drawings

In order to determine if the great variety of steels were in the proper positions in the buildings, NIST has correlated stamped yield strength values (generally stamped on each plate in the perimeter panels) and the measured mechanical properties with the yield strengths on the design drawings. This correlation is largely complete, and there are no indications that any inappropriate steel was in place in the buildings.

2.4.7 High-Temperature Excursions

The structural steel from the impact area of the towers is being characterized to determine maximum exposure temperatures for input to Project 5. After surveying a number of possibilities, NIST developed a technique to map thermal exposure of the relevant pieces by characterization of paint condition. By this means, sections with no damage (i.e., no “mud-crack” patterns) to the paint are known to have remained below approximately 250 °C, and paint with mud-cracks, but remaining relatively intact, remained below approximately 750 °C. Above 750 °C the paint becomes powdery and flakes off. Mapping of the steel is nearly complete, and data will be supplied to Project 5.

2.4.8 Significant Interim Results

1. NIST has cataloged the 236 pieces of recovered steel (Appendix F).

2. Material and construction specifications of the construction period, as well as steel fabrication documents, have been located, analyzed, and documented (Appendix E).

3. Mechanical tests at room temperature (for baseline performance), high temperature (for strength and deformation behavior in fire conditions) and high rates of deformation (to calibrate strength enhancements occurring during airplane impact) are completed and are being analyzed.

4. Mathematical models of the stress-strain behavior of 29 steels (various grades and manufacturers) in the WTC towers have been developed for use in computer models of the behavior of the building during the airplane impact and during the resulting fires. The models of the steel

behavior are based on the conventional room temperature tests, high strain rate tests, high temperature characterization and literature on properties of steel produced in the era in which the WTC towers were constructed.

5. Airplane impact damage has been characterized from enhanced precollapse photographs and correlated with recovered steel.

6. Ten different steel companies fabricated structural elements for the towers, using steel supplied from at least eight different suppliers; four fabricators supplied the major structural elements of floors 9 to 107.

7. Documents from the era of WTC tower construction, from the steel suppliers and others, were used to estimate average yield strengths for each of the supplied steels. These strengths typically exceed the specified minimum strengths given in the engineering drawings by 5 percent to 10 percent.

8. In contrast to the above, extensive studies of steel from the construction period show that due to statistical variation expected in steel products, a fraction of mechanical tests would be expected to fall below specified minimums.

9. Although ASTM structural steel standards have evolved since the construction of the towers, changes have been minor and do not represent changes to the basic mechanical properties of the steels.

2.4.9 Preliminary Findings

1. Analysis of recovered samples of the many grades of steel in the towers indicates that, based on stampings on the steel and mechanical tests, the correct specified grades of steel were provided for the specific fabricated elements. Furthermore, when this data is combined with pre-collapse photographic images of the five recovered WTC 1 panels in NIST's possession that were damaged by the aircraft impact, it has been shown that these particular elements contained proper steel in the precise locations as specified in the design drawings.

2. Metallography and mechanical property tests indicate that the strength and quality of steel in the towers was adequate, typical of the era, and likely met all qualifying test requirements.

2.5 INVESTIGATION OF ACTIVE FIRE PROTECTION SYSTEMS (PROJECT 4)

2.5.1 Project Objectives

The active fire protection systems studied in this investigation are the automatic fire sprinklers, fire detection and alarms, smoke purging, and preconnected hose lines. The automatic fire sprinkler system is the first line of defense against fires in these buildings. Water stored in the building, from public sources and even pumped from fire apparatus can be supplied through dedicated piping to the area of the fire. Also present in the buildings were preconnected hose lines connected to a water supply through standpipes located in the stairwells and other utility shafts. The standpipes provided hose connections at each floor for the FDNY. In addition, standpipe preconnected hoses were installed for trained occupants to manually suppress fires. The heart of the fire detection system is the automatic fire alarm and emergency notification system. Occupants in the building depend on this system to detect fires and provide information for emergency evacuation. Capabilities were also designed for the ventilation system to operate in a way to purge smoke produced by fires from the building. Smoke purge was intended to be used for post-fire clean-up, but could be used during a fire event at the discretion of the FDNY.

All of the active fire protection systems provide capabilities that are important for fire control, providing information for occupants and first responders, and limiting the effects of the fire on the building and its occupants. Therefore, this project has the objectives of documenting and evaluating the performance of the installed active fire protection systems in WTC 1, 2, and 7 and assessing their role in fire control, emergency response, and the fate of occupants and responders.

2.5.2 Project Approach

The tasks of this project are to (1) document fire protection systems design and installation, and (2) evaluate performance without the benefit of any physical evidence from the collapsed buildings. The need to document facts associated with the installed fire protection systems was made difficult because many of the relevant documents for WTC 1, 2, and 7 were lost in the collapse of those buildings.

With the cooperation of the PANYNJ and Silverstein Properties Inc., information was obtained from other locations and from contractors, consultants, and operators. As an example, some information was obtained from the engineering offices of PANYNJ in Newark. Other written materials describing the design and operation of active fire protection systems were obtained through files maintained by contractors. Lastly, information from engineers and system operators was helpful in clarifying details of the installation and operation.

NIST investigators led three groups of fire protection systems contract experts. Each group specialized in one of the fire protection systems being investigated – fire sprinkler, fire alarm, and smoke management. The group examining the sprinkler system was also tasked with investigation of the other water-based fire suppression systems—the standpipe and preconnected hoses.

Technical assistance to NIST in the investigation of the sprinklers, standpipes and preconnected hoses was provided by Hughes Associates Inc. of Baltimore, Maryland. This group was tasked with:

1. Documenting the design and installation of the systems;

2. Documenting the design and capacity of the water supply including provisions for redundancy;

3. Identifying differences in the designs used in WTC 1, 2, and 7;

4. Documenting the normal operation and effect of the fully functional systems for fire control;

5. Assessing the probable performance of the systems in WTC 1 and WTC 2 on September 11, 2001; and

6. Assessing the installed systems with respect to present best practices.

The amount of water that the water supply and sprinkler systems were capable of delivering for a series of fire scenarios was determined using a hydraulic model of the sprinkler system and the associated water supply.

Technical assistance to NIST in the investigation of the fire alarms was provided by Rolf Jensen and Associates, Inc., of Fairfax, Virginia. This group was tasked with:

- Documenting the design and installation of the system;
- Documenting the normal operation and effect of the fully functional systems, including provisions for redundancy;
- Documenting modifications made to the fire alarm systems in WTC 1 and WTC 2 after the 1993 bombing;
- Assessing the probable performance of the systems in WTC 1 and WTC 2 on September 11, 2001; and
- Assessing the installed systems with respect to present best practices

Technical assistance to NIST in the investigation of the smoke management systems was provided by Hughes Associates, Inc., of Baltimore, Maryland. This group was tasked with:

- Documenting the design and installation of the systems;
- Describing the normal operation in fire emergencies;
- Assessing the probable performance of the systems in WTC 1 and WTC 2 on September 11, 2001; and
- Assessing the installed systems with respect to present best practices.

The NIST building airflow and contamination dispersal computer model, CONTAM, was used to evaluate the performance of several smoke management system configurations in WTC 1 and WTC 2 under specific fire scenarios.

Significant fires in WTC 1, 2, and 7 prior to September 11, 2001, were of interest to understand, in particular, how the fires were suppressed. Information was sought on all fires that activated multiple sprinklers or where hose lines were used to suppress the fires. Because the records of fire events in the buildings maintained by the PANYNJ were destroyed in the fire and collapse of WTC 1, information was collected from FDNY fire reports.

2.5.3 Fire History of WTC 1, 2, and 7

Fires occurred in WTC 1, 2, and 7 prior to September 11, 2001. The facts related to the performance of automatic sprinkler, manual suppression, fire detection and smoke purge systems during significant fires in the buildings after first occupancy were documented.

Extensive records of fire incidents kept in the WTC 1 offices of the PANYNJ were lost in the collapse of the building; however, FDNY maintains records of the responses to all fires. These records consist of standardized forms on which fire events are described using codes from a predefined list of descriptive phrases and categories.

The FDNY provided 397 Bureau of Operations Fire Reports and 112 Bureau of Fire Investigation Records (Fire Marshals' Reports) that served as the basis for this summary of the fire history in the WTC 1, 2, and 7. NIST reviewed these reports of fires for the period of 1970 to 2001 and fire investigation records between 1977 and 2001 for WTC 1, 2, and 7. All of these records consist of standardized forms that may be supplemented with other materials. Many were for minor fire events, such as fires that were extinguished by occupants before FDNY arrival. These were not of interest for this investigation. The records of significant fires were identified. Significant fire incidents were those involving the discharge of multiple sprinklers, use of a standpipe connected hose, or the combination of a single sprinkler discharge and a hose. As an aside, the majority of fire records for significant fires documented the performance of the detectors and sprinkler systems, but almost all reports lacked information about the performance of the smoke purge system.

Table 2–4 contains the categorization of all structural fire incidents contained in the FDNY records for WTC 1, 2, and 7 available to this Investigation. This information was obtained from 345 of the 397 Bureau of Operations Fire Reports that reported structural fire incidents. The table contains information on the category of fire incident, the time period over which the fire occurred, the number of records in that category, and a descriptive statement about the category.

Forty-seven of these cases were considered significant fires based on information about the number of sprinklers activated, and/or hose lines used to suppress the fire. Sixteen fire incidents exercised multiple sprinklers or multiple standpipes (with or without the activation of at least one sprinkler). Thirty-one fires involved the use of one standpipe line or one standpipe line and discharge of one sprinkler. These incidents are documented further in Appendix G. In addition, the appendix contains information from publicly available investigation reports of the 1975 office fire in WTC 1 and the 1993 bombing incident.

The FDNY fire reports and fire investigation records indicate that in areas protected by automatic sprinklers, no fire activated more than three sprinklers. The design area for three sprinklers is a floor area of 63 m^2 (675 ft^2) in a light hazard occupancy, such as a high-rise office building as specified in the NFPA Standard for the Installation of Sprinkler Systems (NFPA 13).

Many of the fires that occurred were recorded as suspicious or unknown in cause, occurred during off-peak work hours, and involved materials such as trash or paper-based supplies. In cases where sprinklers were activated, the FDNY records indicated that the sprinklers either extinguished the fire completely or aided in controlling the spread.

Table 2–4. Summary of historical fires in WTC 1, 2, and 7.

WTC 1			
Category	**Dates**	**Number**	**Generalization of Incidents**
No detection, no sprinkler	1980–2001	66	Unattended food/appliances, overheated elevator equipment, discarded material, welding operations, electrical failure and suspicious fires
No detection information and no sprinklers	1970–1979	79	Trash can fires, discarded material, food on stove, electrical failure, overheated equipment
Detection, no sprinklers	1980–2000	57	Unattended food/appliances, overheated elevator equipment, discarded material, welding operations, electrical failure
Detection and sprinklers	1977–1999	18	Suspicious, electrical failure, discarded material
WTC 2			
Category	**Dates**	**Number**	**Generalization of Incidents**
No detection, no sprinkler	1980–1999	37	Discarded material, welding too close, overheated equipment, suspicious, elevator motor
No detection information and no sprinklers	1975–1979	40	Discarded material, fire in office furniture, trash can fires
Detection, no sprinklers	1981–1999	40	Food on stove, small elevator fire, electrical failure, suspicious, overheated equipment
Detection and sprinklers	1977–2000	5	Mechanical failure, suspicious
WTC 7			
Category	**Dates**	**Number**	**Generalization of Incidents**
No detection, no sprinkler	2000	1	Trash can fire/discarded material
Detection, no sprinklers	1990	1	Electrical switch on floor – explosion
Detection and sprinklers	1988	1	Suspicious

2.5.4 Fire Sprinkler, Standpipe, and Preconnected Hoses

Resources used in the investigation of the sprinkler, standpipe, and preconnected hoses in WTC 1, 2, and 7 are being documented by NIST investigators and subject experts at Hughes Associates, Inc. This information will be included in the final report

The design and installation of the fire sprinkler, standpipe, and preconnected hoses are described in a report being prepared for NIST by Hughes Associates, Inc. This report will provide an analysis of the performance capabilities of the suppression systems based on hydraulic modeling of the water supply and sprinkler distribution system.

Preliminary Findings

1. **Sprinkler Risers and Standpipes**. In WTC 1, 2, and 7, primary and secondary water supplies, fire pump size and locations, water storage tanks, and FDNY connections provided multiple

points of water supply redundancy. The potential for single point failure of the water supply to the fire sprinklers existed at each floor due to lack of redundancy in the sprinkler riser system that provided only one supply connection on each floor. As a result, the water supply to the sprinkler systems or a standpipe serving preconnected hoses could be interrupted by routine maintenance needs (i.e., shutdown of the riser or standpipe) or by impairment due to deliberate acts to damage the sprinkler riser or standpipe systems. While this lack of redundancy may not have had an impact on September 11, 2001 because the sprinkler system was damaged by aircraft impact, it could have made a difference in other building emergencies.

2. **Water Flow Rate to Sprinklers**. Aided by the results of hydraulic modeling of the sprinkler system in WTC 1 and WTC 2—undamaged by aircraft impact and fully operational—the delivered water flow rate available from the automatic sprinkler systems was found to generally exceed the minimum requirements (by a considerable margin) for a high-rise office hazard classification in accordance with NFPA 13. In a number of cases, the amount of available water flow from sprinklers on specific floors was capable of protecting higher fire hazard classes than those associated with light hazard office buildings.

2.5.5 Fire Alarm Systems

The WTC 7 fire alarm system was monitored by AFA Protective Systems, Inc., at a location remote from the WTC site. AFA Protective Systems furnished the record from the fire alarm system history tape to NIST for use in the Investigation.

Other resources used in the investigation of the fire alarm systems in WTC 1, 2, and 7 are being documented by NIST investigators and subject experts at Rolf Jensen and Associates, Inc. This information will be included in the final report.

The design and installation of the fire alarm systems will be described in a report being prepared for NIST by Rolf Jensen and Associates, Inc. This report will provide the analysis of performance based on the design and programming of the systems.

WTC 7 Alarm System Monitoring Record

Although a great amount of information is normally collected and stored by any fire alarm system from fire detectors installed throughout a building, typically, and in the case of WTC 7, no specific fire information is sent to the monitoring site beyond the fact that a fire condition has been detected.

The information from the WTC 7 alarm system monitoring record for September 11, 2001, is shown in Fig. 2–8.

09/11/01	14:48:22	DYJ	4612	**** FULL CLEAR ****		
09/11/01	14:47:22	LATE	3923	SYSTEM TEST OVER		
09/11/01	14:47:22	COMMENT:		TEST: ALL		
09/11/01	14:47:21	COMMENT:		LAST SET: 091101 64742		
09/11/01	10:00:52	1	1510	CO TO CLASS E	AREA:1	*T
09/11/01	06;47:43	COMMENT:		RIC: WILLIAMS		
09/11/01	06:47:03	RIC	4210	PLACE ON TEST	CAT:11	
09/11/01	06:47:03	COMMENT:		091101 647 091101 1447		
09/11/01	06:47:02	COMMENT:		TEST: ALL		
09/11/01	06:05:01	RP		20 TIMER TEST		

Figure 2–8. Monitoring station history tape record for the WTC 7 fire alarm system on September 11, 2001.

The fire alarm history tape record is read from the bottom to the top. Some entries occur as the result of normal operations, and others are the result of actions taken by operators. The bottom line of the record shows that at 6:05:01 a.m. on September 11, 2001, the fire alarm system completed a normal communications check with the central monitoring station. This check is made every day.

At 6:47:02 a.m., AFA placed WTC 7 in a "TEST: ALL" condition. This is normally done in response to a request from the building manager. Ordinarily, it is requested when maintenance or other testing is being performed on the system, so that any alarms that are received from the system are considered the result of the maintenance or testing and are ignored. NIST was told by AFA that for systems placed in the TEST condition, alarm signals are not shown on the operator's display, but records of the alarm are recorded into the history file.

At 6:47:03 a.m., the record includes an explanation of the request to put the system in the TEST condition. Continuing to read from bottom to top, the date and time the system was placed in TEST is recorded. In this case it is 091101 647 (6:47 a.m., September 11, 2001), and the system will automatically go back to normal monitoring after 8 hr, a system default value, at 091101 1447 (2:47 p.m., September 11, 2001). On the next line above, "RIC" identifies the AFA operator; 4210 is a code number for the "PLACE ON TEST" message. CAT:11 indicates the authority of the person requesting the action. On the next line above, the comment entered by RIC identifies that the person who requested that the system be placed on TEST was Williams. This action appears to be common for the building alarm system. Records show that the system was placed on test condition every morning for the 7 days preceding September 11, 2001.

At 10:00:52 a.m., a fire condition [1 1510 CO TO CLASS E] was indicated in WTC 7 by sensing performed by the fire alarm system. The *T at the right end of that record indicates that the system was in TEST at the time. The alarm record also shows that the fire condition is in AREA 1. NIST has been told by AFA that AREA 1 is not a specific area within the building, but a reference to a zone consisting of the entire building. That is to say, fires detected in any fire alarm zone in the building by the fire alarm system would result in the same AREA 1 identification at the monitoring station. The time 10:00:52 a.m.

is shortly after the collapse of WTC 2. It is unknown if this fire alarm was triggered by smoke from a fire or dust entering smoke detectors.

At 2:47:21 p.m. and 2:47:22 p.m. (14:47:21 hr and 14:47:22 hr), at the time the 8 hour "TEST: ALL" condition was set to expire, additional actions are recorded that end in an operator (DYJ) entry to "FULL CLEAR."

Alarm System Network Communications Paths

During initial engineering design for the fire alarm system in WTC 1 and WTC 2, PANYNJ requested approval of the City of New York for use of fiber optic communications cable in the system. NYC Building Authorities denied the use of fiber optic cable. As a result, ordinary copper wire communication cable was specified. The copper communications cable is susceptible to electrical shorts that can prevent any communication between all of the distributed control units of the system. Fiber optic cable is not susceptible to electrical shorts. If fiber optic communications cable had been used, communications between panels where the fiber optic cable had not been severed would continue to be able to communicate with each other.

Severing either type of data communications cabling without electrical shorts would have produced the same effect on the system. The system was designed with redundant communication paths to provide Class A signaling circuits. If one communication path is served without electrical shorts, a trouble condition would be annunciated, but communications would not be impaired.

"Standpipe Telephone" System

A dedicated communications system for emergency responders was installed in the stairwells of WTC 1 and WTC 2. To use the system, a compatible telephone handset was needed. This system was known as the "standpipe telephone" system.

Preliminary Findings

1. **Fire Alarm Monitoring System.** The fire alarm system monitoring WTC 7 sent to the monitoring company only one signal indicating a fire condition in the building on September 11, 2001. This signal did not contain any specific information about the location of the fire within the building. From the alarm system monitor service view, the building had only one zone, "AREA 1."

2. **Alarm System Communications Paths.** The resistance to failure of the fire alarm system communications paths between the fire command station and occupied floors may have been enhanced if fiber optic communications cable had been used instead of copper lines. Extensive damage to the towers upon aircraft impact is likely to have cut and shorted the wiring of the alarm system network cables. If that occurred, communications between the distributed fire alarm panels, which are components of the integrated fire alarm system, would have been degraded and lost to certain panels depending on the location of those panels. Fiber optic cable is not susceptible to electric short-circuits and would have provided full communications with fire alarm system components, including voice communications systems, to the point where the cable was

severed. Electric shorts in the voice communications disable that communication system over the entire cable length affected by the electric short-circuit. During initial engineering design for the fire alarm system in WTC 1 and WTC 2, the PANYNJ requested, but did not receive, approval of the City of New York for use of fiber optic communications cable in the system. The NYC code required copper wiring. As a result, ordinary copper wire communication cable was specified.

3. **"Standpipe Telephone" System.** Some firefighters that received handsets at the command post in the lobby at WTC 1 were interviewed as part of the investigation. Every one of the firefighters interviewed indicated that they did not use the standpipe telephone communication system on September 11, 2001. Due to the loss of firefighters in WTC 2, there is no information about the use of the system in WTC 2.

2.5.6 Smoke Management

Resources used in the investigation of the fire alarm systems in WTC 1, 2, and 7 are being documented by NIST investigators and subject experts at NIST contractor, Hughes Associates, Inc. This information will be included in the final report.

The design and installation of the smoke management systems will be described in a report being prepared for NIST by Hughes Associates, Inc. This report will provide the analysis of performance based on the design and programming of the systems.

The smoke management systems as designed and documented in the operation manuals consisted of a smoke purge mode using the components of the main HVAC systems. The systems were intended to remove smoke and other gaseous combustion products from the fire area after a fire was extinguished. This system was to be activated "manually" at the direction of FDNY.

Preliminary Findings

1. **Smoke Management Systems Performance on September 11, 2001.** Based on the information reviewed, the smoke management systems were not activated during the fires on September 11, 2001. It was determined that the likelihood of these systems being functional in WTC 1 and WTC 2 was very low due to the damage inflicted by the aircraft impacts. In addition to the significant openings created in the building envelopes, the aircraft impacts are likely to have severed major vertical shafts through which ran electrical power supply and HVAC system duct risers, thereby causing the loss of power to the smoke management system air handlers and damage to the vertical HVAC duct risers used to provide smoke management (smoke purge).

2. **Fire/Smoke Dampers.** The analysis of smoke flow in WTC 1 and WTC 2 on September 11, 2001, shows that HVAC ductwork was a major path for vertical smoke spread in the buildings. Fire dampers were installed in the systems, but not smoke dampers. Operational combined fire/smoke dampers in the HVAC ductwork on each floor would have provided a barrier to hot gas and smoke penetration into the vertical HVAC shafts in WTC 1 and WTC 2. However, smoke dampers were not available when the towers were built.

3. **Stair Pressurization Systems.** Modeling results showed that in WTC 1 and WTC 2 stair pressurization systems would have provided minimal resistance to the passage of smoke had they been installed on September 11, 2001. While the existence of such systems was known when the WTC towers were built, the alternative smoke purge system used in the WTC towers was considered to be equivalent. Multiple stair doors being open for substantial periods of time due to occupant egress, and stairway walls damaged by aircraft impact, would result in an inability to prevent smoke from entering stairwells.

2.6 RECONSTRUCTION OF THERMAL AND TENABILITY ENVIRONMENTS (PROJECT 5)

2.6.1 Project Objective

The collapse of the WTC towers resulted from a combination of aircraft impact damage and the ensuing fires. However, both the relative importance of these two factors and their interaction leading to the observed total collapse is at present unclear. It is also unresolved:

- Which structural features of the buildings were affected, and thus what location, magnitude, and duration of fire brought about the collapse, and

- Whether the nature of the fires is typical of what might be expected in common occupancies, or whether there were special features that made these fires especially severe.

These facets are even more pivotal for WTC 7, where the fires that led to the unexpected collapse followed an unknown ignition in an unknown location.

In addition to the flames and heat, the smoke from the fires plays multiple roles, for example:

- It serves as a telltale for the locating of fires, although the determination of location also requires a knowledge of smoke movement within the buildings.

- Its visual obscuration and perhaps its toxicity may have affected choices made by people as they decided direction of movement, whether to wait for rescue, etc.

Thus, Project 5 has as its objective to reconstruct, with assessed uncertainty limits, the time-evolving temperature, thermal radiation, and smoke fields in WTC 1, 2, and 7 for use in understanding the behavior and fate of occupants and responders and the structural performance of the buildings.

2.6.2 Project Approach

Due to the near absence of physical evidence, the recreation of the fires depends on computer modeling. NIST is redefining the state-of-the-art in fire and thermostructural modeling, since this type of reconstruction has never been done before. Fire experiments are being used to guide adaptation of existing models and develop new algorithms for them; additional experiments form the basis for validating the models.

The models will then be exercised for a range of possible initial conditions. Those simulations that agree with the photographic evidence and the eyewitness information will be accorded a higher likelihood of being correct. To the extent that simulations contradict the evidence, they will be deemed less plausible.

There is essential input that will arise from other projects, for example:

- Specifications of the liquid fuel storage systems in WTC 7 – Project 1
- Degree and nature of aircraft impact damage – Project 2
- Maximum temperatures experienced by the structural steel – Project 3
- Performance specifications for the smoke handling system – Project 4
- Extent and location of structural weakening needed for collapse to be initiated – Project 6
- Eyewitness accounts of building damage and fire locations – Projects 7 and 8

The interdependence with Projects 2, 6, and 7 is particularly broad, with Project 5 providing, for example:

- Establishment of the nature and precision of the information needed from the aircraft input modeling – Project 2
- Descriptions of the duration and intensity of likely fires for use in assessing the possible locations of collapse initiation – Project 6
- Description of fire and damage information to be requested of survivors – Project 7

Project 5 is divided into the following eight tasks:

- **Visual Collection and Time Line Development for WTC 1, 2, and 7.** To acquire and use photographs, videos, and other relevant information to develop detailed time lines for the spread and growth of fires at the peripheries of WTC 1, 2, and 7 and to organize the information such that it can be utilized by other investigation team members. The cataloging and analysis will provide guidance on the initial conditions for modeling the fires, the rates of spread of the fires, the floors on which the structural collapses appear to have begun, etc.
- **Characterization of Combustibles.** To gather data on and characterize the types, mass and distribution of combustibles in the pertinent floors of WTC 1, 2, and 7 at the time of the September 11, 2001, disaster. The results are to serve as input to the overall Project 5 effort to reconstruct the thermal and tenability environment within the three buildings.
- **Characterization of Partitions.** To identify the location of and characterize the fire endurance properties of the internal partitions (floors, walls, and ceilings) in the pertinent floors of WTC 1, 2, and 7 at the time of the September 11, 2001, disaster. This entails obtaining existing data on the fire performance of floor, wall, ceiling systems, and complementing this with additional measurements as needed. The results will help in

determining the potential and rates of intercompartment fire spread and also the degree to which the interior of a building was visible in the photographs and videos.

- **Characterization of Structural Insulation.** To determine the effective thermal properties of the structural fireproofing systems, the effect of vibration, impact, and shock on their thermal insulation performance, and whether chemical interaction between the fireproofing materials and the steel at elevated temperatures could degrade the steel and fireproofing performance during thermal insult. This will enable simulation of the temperature rise within the structural elements as a result of the changing thermal environment.

- **Model Development.** To upgrade the NIST computational fluid dynamic (CFD) Fire Dynamics Simulator (FDS) for its application to the reconstruction of the fires in WTC 1, 2, and 7. This will affect a pragmatic fire growth routine and also improve the efficiency of the model, enabling more extensive simulations during the timeframe of the Investigation. In addition, this task will develop a computational method for relating the turbulent fire environment to the transport of heat to and through the insulating layer to the underlying structural steel. This will enable simulation of the temperature rise and resulting loss of structural capability of the steel.

- **Experiments for Model Development.** To provide input parameters and guidance for the FDS combustion submodel.

- **Fire Reconstruction.** To reconstruct the gaseous thermal environment (radiation and temperature fields) surrounding the structural elements and in the inhabitable spaces within WTC 1, 2, and 7. Using such input information as the estimated aircraft damage from Project 2, the contents and layout of the building from the above tasks, NIST will use FDS to simulate fully involved fires in the three buildings, with and without the initial damage from the aircraft or incident debris, enabling addressing the extent to which that damage affected the thermal environment felt by the structure. Parameters in the re-creation of the fires will enable estimating the roles of jet fuel and building contents, ventilation system, compartment damage, pressurized core, and fire protection system on the growth and spread of fire. The use of statistical design for the sets of simulations will lead to identification of those input conditions to which the results are the most sensitive and those combinations of input conditions that lead to the best agreement with the photographic evidence.

- **Reconstruction Validation.** To generate and use experimental data for assessing the accuracy of the fire model prediction of thermal insult on structural members such as columns, trusses, beams and other support structures like those in WTC 1, 2, and 7. Comparison of the data from large-compartment tests of fire growth and heat transfer to steel specimens will establish the accuracy of FDS in simulating heat transfer and complex burning at a realistic scale.

2.6.3 Collection of Photographic Evidence

NIST has compiled an extraordinary collection of still and video images of the three buildings. These have been digitized and organized into a searchable database. The user can organize a search to view each of the two towers from all four sides from the time of the airplane strike to the time of collapse, although there are still significant gaps in time and vantage point. The collection is less definitive for WTC 7, due mainly to the high hazard level in the vicinity following the collapses of the two towers and to obscuration of the building by other structures and the smoke cloud from the tower collapses.

To facilitate comparison of the predictions of the fire modeling (see below) with the photographic evidence, NIST has created animations of the building facades that depict the evolving breaking of windows, the emanation of smoke from the windows, the appearance of fire through the windows, and the emanation of flames out the windows.

More detail on the collection appears as Appendix H.

2.6.4 Data on the Building Interiors

The solicitation of information on the tenant spaces was focused on those floors of the three buildings in which physical damage was observed and those floors where fires were observed or might have existed unobserved (as shown in Table 2–5).

Table 2–5. Floors of visible damage.

Building	Aircraft Impact Damage	Observed Fires
1	93–99	92–99, 100, 104
2	77–85	78–83
7	–	7, 8, 9, 11, 12, 13, 22, 29, 30

Floor Plans

Examination of documents combined with discussions with architects, product manufacturers, occupants, and building managers indicate a general picture of the building interior. The floor slab was generally carpeted; there are some cases of raised floors and of wood- or stone-covered areas. There were glass walls at the entrances to some of the suites. For multi-tenant floors, the demising walls between the tenants were of gypsum board over steel studs and ran from the floor slab to the bottom of the slab above. Tenant space interior walls were of similar construction, but ran from the floor slab to just above the drop ceiling. Thus, the joist space was often open across a whole story or large fractions of a story. The drop ceiling systems, one for the tenant spaces and one for the core areas, were designed for the WTC.

NIST requested that tenant companies provide architectural drawings of the most recent renovation of their space. The following features are of particular importance:

- Location of walls.
  - These can act as a barrier to fire spread. In the towers, the fire resistance time of the demising and interior walls may have been comparable to the time between aircraft impact and building collapse. Even though the overall duration of the fires in WTC 7 was much longer, the walls could have limited the rate of fire spread.

- When there were walled offices at the perimeter of a floor, the interior walls block the view of the interior from the exterior. Thus, the nonobservation of fire through a particular window could indicate either the absence of a fire on that floor or the presence of a vision-obstructing wall.

- Air flow between floors. Fires need both fuel and air. To the extent that the flow of air to the fire is limited, so is the size of the fire. The direction of air access will also determine the direction(s) of fire spread. Thus, it is important to know where there were perforations, e.g., interior stairwells, air ducts, to have the most realistic sets of input conditions for the fire simulations.

NIST has obtained floor plans for a large fraction of the floors of interest in the three buildings. For the few that are missing, NIST is working with the design drawings and is estimating their similarity to the layouts on September 11, 2001, from eyewitness, tenant, and manager accounts.

Combustibles

While much of the public attention has been focused on the jet fuel, this was fully combusted in only a few minutes. By contrast, typical office furnishings can sustain intense fires of at least an hour's duration on a given floor. NIST has obtained generic information about the furnishings in many of the suites and specific details for a few. This information has already been of use in the design of workstation fire tests (see below). NIST is checking to identify the location and size of any unusual fuel loads, such as file rooms, film storage, etc.

In addition, NIST has obtained descriptions from the airlines of the combustible contents of the airplanes on that day. This includes the cabin materials (both installed and carried on by the passengers), aircraft components (e.g., wire insulation, flammable fluids other than the jet fuel), and the cargo bay contents. A preliminary estimate indicates that the combined mass of aircraft-borne combustibles is a considerable fraction of the building combustibles in the impact zone.

For WTC 7, NIST is aware of two special sources of combustible fluids. Rolf Jensen and Associates, Inc., a NIST contractor, is gathering data on the fuel tanks and distribution lines for the emergency generators in WTC 7. NIST has obtained information on the magnitude of the volume of the transformer fluids located in the power substation in WTC 7.

2.6.5 Insulation of Structural Members

The required fire resistance ratings of the structural members in the three buildings were obtained by gypsum framing of some columns or the use of spray-applied fireproofing on other columns and web joists. The architectural drawings provide definition of the former. Working from documents and discussions with engineers, NIST has identified the various spray-applied fireproofing materials and where each was used, as shown in Table 2–6. For the exterior columns in the towers, both Vermiculite plaster and BlazeShield DC/F were used. NIST is still investigating the specific locations for each.

Table 2–6. Types and locations of spray-applied fireproofing on fire floors.

Building	Fireproofing Material	Locations		
		Interior Columns	Floor Systems	Exterior Columns
WTC 1	BlazeShield II		Floors 92 to 100, 102	
	BlazeShield DC/F	Yes	Remaining floors	
	Vermiculite plaster and BlazeShield DC/F			Yes
WTC 2	Vermiculite plaster and BlazeShield DC/F	Yes	Yes	Yes
WTC 7	Monokote ML-5	Yes	Yes	Yes

The ability of the fireproofing to delay the rise of temperature in the protected structural steel depends on:

- **The thermophysical properties of the insulation material**. NIST has obtained samples of the three types of spray-applied insulation and four types of gypsum wallboard and has sent them to testing laboratories for determination of their thermal conductivity, density, and heat capacity, all as a function of temperature from ambient. The spray-applied material data will be from 25 °C to 1,200 °C; the wallboard data will be from 25 °C to 600 °C.

- **The thickness of the insulation**. The gypsum wallboard thickness is described in the architectural drawings. As documented in the May 2003 Progress Report, NIST has traced the evolution of the intended thickness of the spray-applied material. NIST has evaluated the variation in the actual thickness of the fireproofing in the WTC towers. Appendix I presents the results of that evaluation.

- **Any damage to the layer during construction or refitting of the building.Damage from the impact or shock of the incident airplane (WTC 1 and WTC 2)**. NIST will obtain estimates of the impact intensity from ARA, which is the NIST contractor modeling the aircraft impact under Project 2. Using both standard and custom measurement methods, NIST is determining the cohesive properties (shear strength and tensile strength) to steel of the two types of spray-applied insulation used in WTC 1 and WTC 2. NIST is also developing models to predict dislodgement of the insulation.

- **Damage from distortion of the WTC 1 and WTC 2 structures from the impact or the fires.**

- **Damage to the insulation in WTC 7 from incident debris from the collapse of the towers.**

Prior to this Investigation, there was no computational method for modeling in three dimensions the effect of a fire on structural assemblies, that is, modeling the absorbance of incident heat from a turbulent fire by an insulating surface, the transport of heat through the insulating layer, and the distribution of heat throughout the steel structure. This is in large part because the turbulent fire is characterized by short time steps and large computational cell size, while the structural member is characterized by longer time steps (slower changes in temperature) and smaller computational cell size.

NIST has developed a tool to do just this, the Fire-Structure Interface (FSI), for the first time linking FDS with ANSYS (a finite element, FE, thermostructural model). Of particular importance is the relationship between the thickness of the insulation and the time for the underlying steel to reach a temperature at which the steel strength is compromised. Variations in thickness could be random in nature or as stark as bare spots.

The first computational runs were for a column as heavy as the thickest core column on the floors of impact in WTC 1. For each of three cases, a sequence of portraits of the temperature distribution in the steel was generated throughout an exposure to a uniform external temperature of 1,100 °C. From these depictions, the following times were determined for when the steel temperature reached 600 °C, a temperature near which significant compromising of the steel's structural properties would ensue:

- Insulation with the (estimated) properties of BlazeShield applied to a thickness of 13 mm. The time to reach 600 °C was over 10 h.
- A 20 percent reduction in the total mass of insulation, with the loss of thickness being varied randomly. The time to reach 600 °C was about 6 h.
- All insulation removed from one face of the column. The time to reach 600 °C was about 12 min.

In a second set of calculations, the same material was applied to a bar 25 mm × 25 mm × 1,500 mm. A 25 mm notch (to bare metal) of insulation was removed from the midpoint of the bar. Upon exposure to the same thermal environment, the temperature of the steel reached 600 °C along its full length in a matter of minutes.

A further description of the interface and sample calculations appears in Appendix J. The preliminary indications are that, in the future predictions of the impact of the WTC fires on structural members, NIST should expect the results to be sensitive to small gaps in the thickness of the insulation. In other words, small areas of thin or missing insulation can lead to accelerated heating, and this effect could be felt well away from the susceptible sites.

2.6.6 Modeling the Fires

In simulating the fires, NIST will be examining the effects of uncertainty in knowing the initial conditions of the fire and the building. Thus, it is critical that the accuracy of the fire model itself be established so that the uncertainty in the model's predictions is small compared to the effects caused by the differences in the initial conditions.

As a first step, certain enhancements to FDS were implemented:

- Realistic state relation curves for underventilated fire scenarios. The prior computational code used ideal state relations for its combustion routine, i.e., user-prescribed values for the combustion efficiency that did not change as the ventilation within the fire compartments evolved.

- The combustion module was enhanced to enable the inclusion of charring materials, such as those that comprise much of the office furniture.

- Computational code enhancements to enable time-efficient computations. Multiblock gridding now enables needed dimensional resolution in the vicinity of the structural components. The FDS code was also re-written for parallel processing.

- Fire spread. Each combustible is characterized by a heat of gasification, with the surface irradiance calculated from the thermal radiation field generated by existing fire. When the mixture fractions in two adjacent computational cells straddle the stoichiometric value, the flame then extends to the interface between the cells.

- Enhanced visualization. Smokeview has been modified to handle the extremely large data sets that will be generated in these simulations.

A first set of experiments was conducted in the NIST Large-scale Fire Laboratory to assess the accuracy with which FDS predicts the thermal environment in a burning compartment and to establish a data set to validate the prediction of the temperature rise of structural steel elements using FSI. Within a large test compartment, assorted steel members were exposed to controlled fires of varying heat release rate and radiative intensity. The steel members were bare or coated with spray-applied fireproofing of two thicknesses. The thermal profile of the fire was measured at multiple locations within the compartment. Temperatures were also recorded at multiple locations on the surfaces of the steel, the insulation, and the compartment. Prior to each test, a prediction of the thermal environment in the compartment was determined using FDS. Following the tests, the prediction and experimental results were compared.

Much of the combustible material on the fire floors of the WTC buildings consisted of employee workstations. Each such space was a combination of desk space, generally made of fiberboard with a laminated finish; file cabinets; carpet; chair; computer; paper; etc. NIST conducted a set of fire tests of a generic workstation in our Large Fire Laboratory. A single unit was burned under a large hood with a soffitted ceiling. Ignition was by a 2 MW spray burner, simulating an already burning adjacent workstation. Test variables included the combustible mass, the presence of jet fuel, and the presence of inert material (simulating fallen ceiling tiles or wall fragments). Gasification data for the combustible was generated using a Cone Calorimeter. These data plus the geometry of the workstation were used as input to the fire model. The intent was to use the experiments to identify any needed changes in the combustion algorithms. In fact, little adjustment was needed. These tests and their analyses are detailed in Appendix J.

A third set of tests was conducted to determine the accuracy of the FDS under conditions simulating a portion of a representative floor of the WTC towers. Thus, the predictions of the outcome of the tests were performed prior to the experiments. Three workstations were situated in a large compartment. Two were contiguous, the third was across an aisle. The open end of the compartment had windows of aspect ratio similar to those in the towers. The test variables included the presence of jet fuel, the presence of inert material, the location of the ignition burner (at or away from the windows), and the extent to which the workstations had been reduced to rubble. The analysis of the results is under way, but preliminary indications are that the model predictions closely resembled the test results.

Additional information on the three sets of experiments appears in Appendix J. Full reports on the first two sets of experiments are expected in the coming months.

2.6.7 Preliminary Findings

To date, the calculations and analyses have led to several interim findings that will guide the reconstruction of the fires in the three WTC buildings:

Observed Fires

1. Despite the airplane striking the center of the north face of WTC 1, the resulting fires are not symmetric about the centerline of the building. After the initial fireballs, the flames damped considerably. The early fire growth was on the north face, the center of the east face and the west side of the south face. On some floors, there was continuous spread; in some instances sudden, noncontiguous fires appeared.

2. The damage and initial fires in WTC 2 were highly asymmetric, as the airplane struck off center to the east. Burning debris piles of long duration were observed at the northeast corner of some floors. In general, the fires spread less actively than in WTC 1, but there were sudden fires here as well. There was visual evidence of collapsed floors.

Building Interiors

1. The view through many windows was blocked by interior walls.

2. The mass of aircraft solid combustibles was significant relative to the mass of the building combustibles in the impact zone.

3. In laboratory experiments, impulses like those estimated from the aircraft caused serious damage to the ceilings. This is consistent with the accounts of survivors from floors below the impact zone. This damage enabled “unabated” heat transport over the walls and to the joists.Small areas of thin or missing insulation can lead to accelerated heating over much larger lengths of steel.

Combustion Modeling

1. FDS is a useful tool to recreate the burning of the complex arrays of combustibles that existed in the WTC buildings, provided that the initial damage conditions and combustible descriptions are accurate.

2. FSI is a tractable construct for linking the output of a computational fluid dynamic model of the fire-generated thermal environment in the building compartments to a FEM of the building structure.

2.7 STRUCTURAL FIRE RESPONSE AND COLLAPSE ANALYSIS (PROJECT 6)

2.7.1 Project Objective

Both the north and south towers, WTC 1 and WTC 2, were severely damaged by the impact of Boeing 767 aircraft, yet they remained standing for some time. The ensuing fires were observed to move through both buildings until their eventual collapse. The extent and relative importance of the structural damage caused by the aircraft impact, and subsequent weakening due to the fires, is still being investigated. WTC 7 was reported to be damaged by falling debris from the collapse of WTC 1. The fires in WTC 7 that burned for much of the day appeared to play a key role in the building collapse. Project 6 addresses the first primary objective of the NIST-led technical investigation of the WTC disaster: to determine why and how WTC 1 and WTC 2 collapsed following the initial impacts of the aircraft and why and how WTC 7 collapsed. Specifically, the objective of this Project is to determine the response of structural components and systems to the impact damage and fire environment in WTC 1, 2, and 7, and to identify probable structural collapse mechanisms.

2.7.2 Project Approach

Three steps are required to determine the response of structural components and systems to fire conditions. First, the thermal environment (radiation flux and temperature fields) for the floors involved in fire are determined using computational fluid dynamics calculations. The predicted upper layer gas temperatures vary both spatially and temporally. Next, transient thermal analysis is used to predict the time-temperature relationship for the structural components and systems for bare and fireproofed steel conditions. Finally, the time-dependent structural response of the components and systems to the estimated service loads and elevated temperatures is computed using thermal-mechanical FEA.

Project 6 relies heavily on information provided by other projects, specifically:

- Reference structural models of typical floor and exterior wall subsystems of each WTC 1 and WTC 2 tower (Project 2)
- Extent of aircraft damage to WTC 1 and WTC 2 (Project 2)
- Mechanical properties of the steels, welds, and bolts used in the construction of the towers, including elastic, plastic, and creep properties from 20 ºC to 700 ºC (Project 3)
- Thermal properties of spray-on fire resistant materials (SFRM) (Project 5)
- Temperature time-histories for various components, sub-systems and systems for both standard fires (e.g., ASTM E 119) and real fires based on fire dynamics simulations (Project 5).

This project is divided into several tasks as follows:

- Evaluate the structural response of floor and column subsystems under fire conditions.

- Evaluate the response of the WTC towers under fire conditions, both with and without aircraft impact damage.
- Identify and evaluate candidate hypotheses for initiation and propagation of collapse, and estimate the uncertainty for probable collapse initiation and propagation mechanisms.
- Conduct tests of structural components and systems under fire conditions.
- Report on the performance of open-web steel trussed joist systems in fire.
- Analyze the response of WTC Building 7 under fire conditions.

Work completed to date on the above tasks is summarized in the following sections.

2.7.3 Fireproofing of WTC Towers

In May 2003, NIST issued an interim report on the Procedures and Practices Used for Passive Fire Protection of the Floor System of the World Trade Center Tower Structures as Section 3.3 of the May 2003 Progress Report. The report summarized factual data contained in documents provided to NIST by the PANYNJ and its contractors and consultants; by Laclede Steel Company, the firm that supplied the floor trusses for the WTC towers; and by United States Mineral Products Co. (USM) doing business as Isolatek International, the manufacturer of the fireproofing material.

The report discusses the applicable building codes and building classification system, which dictates the fire rating required for structural members and assemblies. The structural system for the WTC towers was constructed predominantly with steel, which, in general, requires protection from fire to maintain its strength and stiffness. Available information on the spray-on fireproofing and the procedures and practices used in its selection and application is presented. Additionally, the report discusses the procedures and practices used to determine whether tests were needed to evaluate the fire endurance of the structural elements, and it presents the results from one such test.

In May 1963, the Port Authority instructed its consulting engineers and architects to comply with the NYC Building Code for the design and construction of the WTC towers. Because the NYC Building Code was being revised during this period, the plans for fire protection of the structural steel underwent concurrent modification. While available records suggest that the fireproofing of the columns, beams, and spandrels was not a subject of concern, fireproofing of the floor bar joists was the focus of continuous reassessment and revision.

A few of the more significant interim findings are:

- The WTC towers were identified as Occupancy Group E – Business, and classified as Construction Class IB in accordance with the 1968 NYC Building Code. This classification required that the columns and floor systems of the towers have a 3 h and 2 h fire endurance, respectively.

- The steel trusses that supported the floors of WTC 1 and WTC 2 were fireproofed with specified 1/2 in. of spray-on fire-protection although the technical basis for the selection of fireproofing material and its thickness are not known.

- In 1999, a decision was made to begin upgrading the fireproofing to a specified 1 1/2 in. thickness as tenant spaces became unoccupied. In general, the floor systems in WTC 1 subject to impact and fire conditions had been upgraded; the floors in WTC 2 subject to impact and fire conditions had not been upgraded.

- The fire protection of a truss-supported floor system by directly applying spray-on fireproofing to the steel trusses was innovative at the time the WTC towers were designed and constructed and, while the benefits of conducting a full-scale fire endurance test were recognized by the building designers, no tests were conducted on the floor system used in the WTC towers to establish a fire endurance rating.

The specified material and thickness of SFRM at the time of construction are as given in Table 2–7.

Table 2–7. Specified passive fire protection.

Structural Component	Member Size	Location	Material	Thickness (in.)
Floor trusses	All	All	Cafco DC/F	1/2
Interior columns	< 14WF228	All	Cafco DC/F	2 3/16
	≥ 14WF228	All	Cafco DC/F	1 3/16
Exterior columns	“Heavy”	Exterior faces	Cafco DC/F	1 3/16
	“Heavy”	Interior faces	Vermiculite aggregate	7/8
Spandrel beams	All	Exterior face	Cafco DC/F	1/2
	All	Interior face	Vermiculite aggregate	1/2

2.7.4 Response of WTC 1 and WTC 2 Floor and Column Systems under Fire Conditions

The detailed component and subsystem models will provide guidance for the analysis of the larger, global analysis of each WTC tower under damage and fire conditions. To determine the structural response of components and subsystems under fire conditions requires the development of nonlinear structural models that account for gravity (service) and thermal loads, temperature dependent material properties, and nonlinear structural behavior, such as plastification and large deflection effects, including instability. These nonlinear structural models are then subjected to thermal-mechanical analysis to determine the time-dependent structural response to the estimated service and fire loads. The commercially available FEA code, ANSYS (version 8.0), is being used for the thermal and mechanical analyses.

The analytical work is being conducted with the assistance of Simpson Gumpertz & Heger Inc. under a contract from NIST and includes the following tasks:

- **Task 1**. Component, Connection, and Subsystem Structural Analysis
- **Task 2**. Global Analysis of the WTC Towers Response to Fire Without Impact Damage
- **Task 3**. Global Analysis of the WTC Towers Response to Fire With Impact Damage

The scope of work under Task 1 includes: (1) the development and validation of ANSYS models of the full floor and exterior wall subsystems, (2) evaluation of structural responses under dead and live loads and elevated structural temperatures, (3) identification of failure modes and failure sequences, and the associated temperatures and times-to-failure, and (4) identification of simplifications for the global models and analyses.

Selected technical results and findings for the typical floor system and its components are presented in the following sections. More detailed coverage is given in Appendix K, Interim Report on Structural Fire Response and Collapse Analysis. A simplified approach to the analysis of the WTC truss-framed floor system response to fire is presented in Appendix M, Interim Report on 2-D Analysis of WTC Towers Under Gravity Load and Fire.

Structural Models and Analyses

Structural FEMs of components and subsystems have been developed for the following:

- Shear connector between the truss and concrete slab, referred to as a knuckle, shown in Figure 2–9
- Truss-to-column bearing seats
- Truss section, including composite floor slab, knuckles, and truss seat connections to columns
- Single-story exterior column for a 9-story height
- Exterior wall subsystem consisting of a 3-by-3 panel section of the exterior wall, where a panel is 3 columns wide and 3 stories long
- Full floor subsystem including the concrete slab, truss seat connections to columns, and core floor area

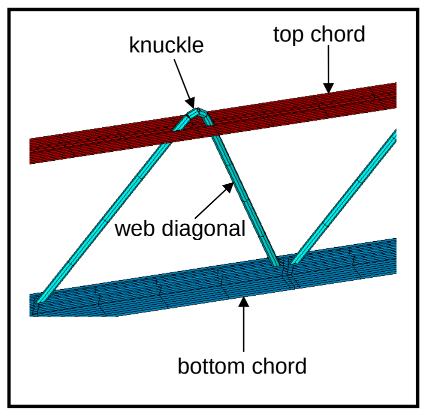


Figure 2–9. Features of truss model.

The truss section model (see Figs. 2–9 and 2–10) includes the following:

- Temperature-dependent elastic material properties for both steel and concrete
- Temperature-dependent steel plasticity

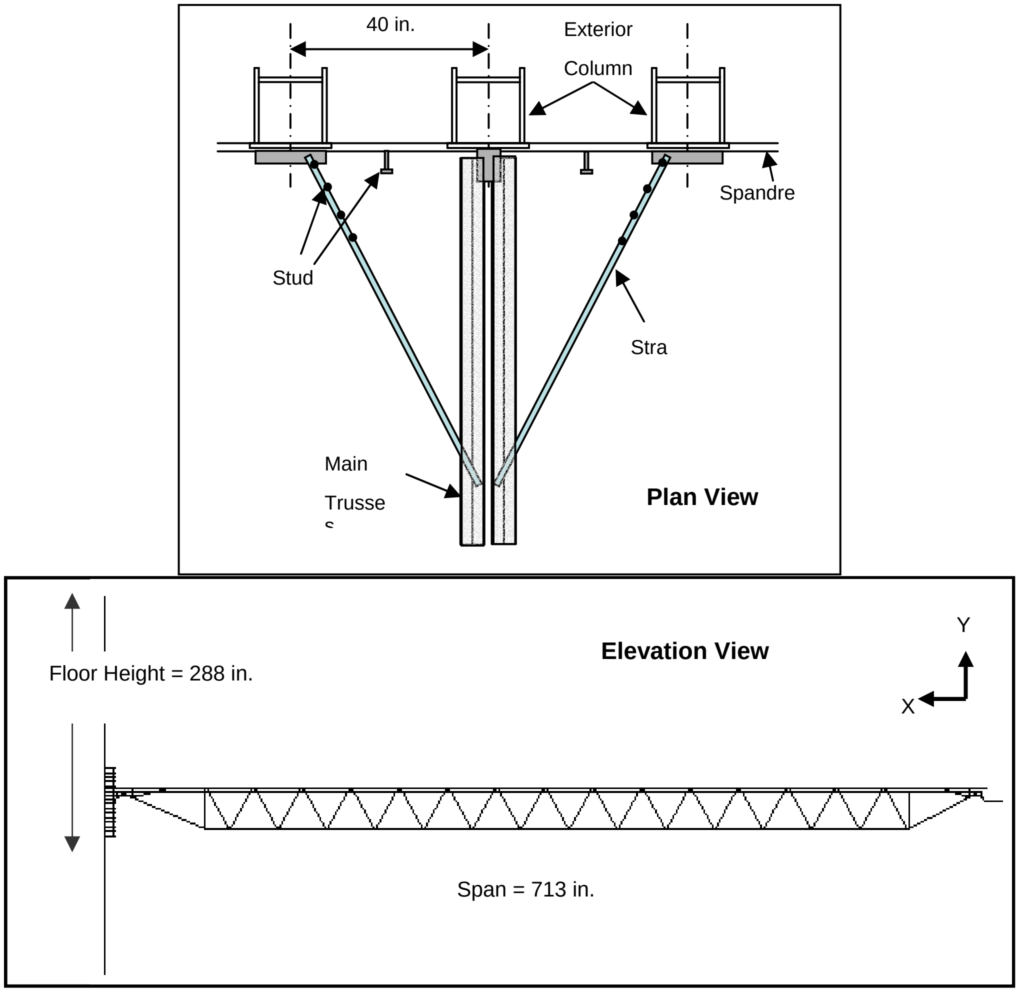


Figure 2–10. Truss model.

- Buckling of truss members
- Failure of knuckle leading to loss of composite action
- Failure of studs on the strap
- Failure of stud between the spandrel and the concrete slab
- Failure of truss resistance welds between the web diagonals and the chords
- Failure of the exterior and interior truss seats

The full floor subsystem model includes the following:

- The main trusses and bridging trusses
- Concrete slab with metal deck
- Strap and seated connections to columns

- Restraint provided by interior and exterior columns

The model was developed by translating a SAP2000 full floor model developed under Project 2 into ANSYS format and validating the analysis results against the SAP2000 model for design loads. A corner of the full floor model is shown in Fig. 2–11.

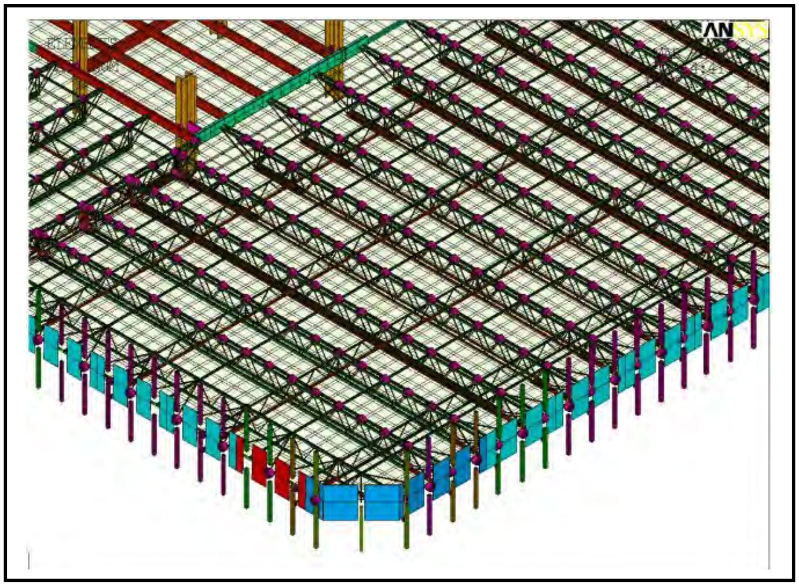


Figure 2–11. Converted ANSYS model of floor 96.

Summary of Technical Results

Capacity of Truss-to-Column Connections

The horizontal and vertical load capacities of the twelve truss-to-column connection configurations on floor 96 have been calculated. These calculated capacities were used to develop simplified models of the connection behavior for use in the floor and exterior wall subsystem analyses. As an illustration, Fig. 2–12 shows a seated connection of the floor truss to the exterior wall of the tower (spandrel plate). The connection is designed to carry vertical floor loads and horizontal loads that are at least 2 percent of the column design load. These connections may be subjected to large horizontal forces, and the capacity of the exterior truss connection under such circumstances must be ascertained.

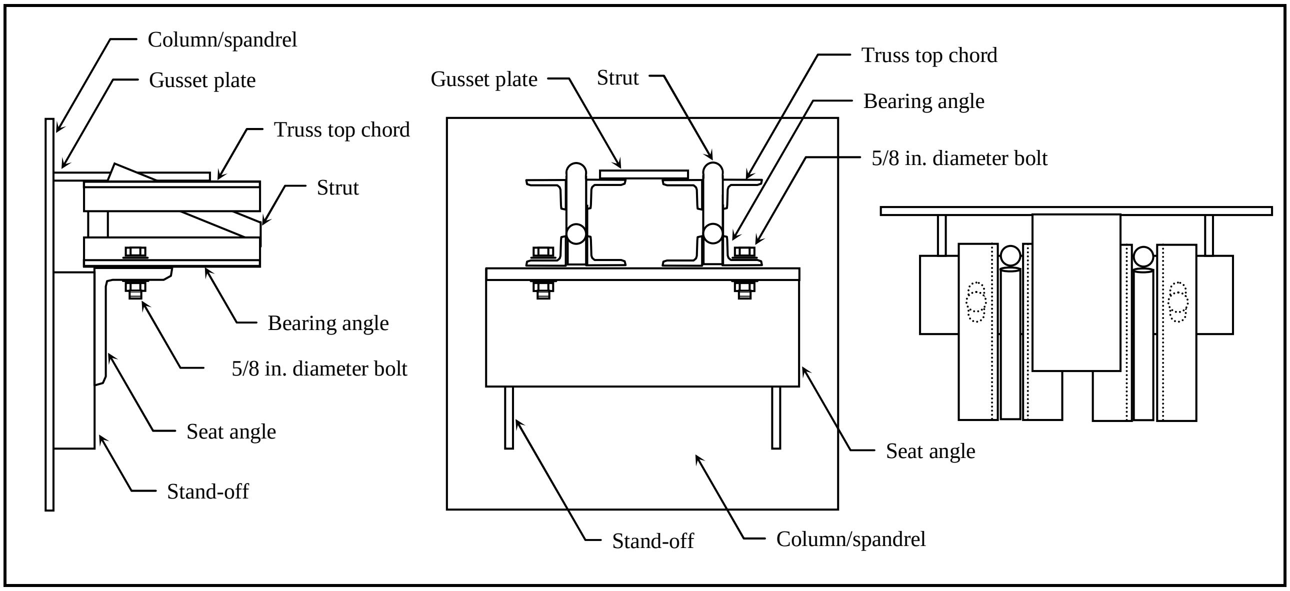


Figure 2–12. Exterior connection of the floor truss to the spandrel plate.

The failure modes considered for the truss-to-column connections are: (1) failure of the groove weld between gusset plate and spandrel, (2) failure of the fillet weld between the gusset plate and the truss top chord, (3) tensile failure of the gusset plate, (4) bolt shearing off, (5) bolt bearing, (6) bolt tear-out, and (7) block shear failure. Possible failure sequences are illustrated in Fig. 2–13. Of the seven exterior connections types that were analyzed, path A is the failure sequence most frequently followed, which is described as follows: first the gusset plate yields across its section and then fractures, followed by truss sagging and deformation and the bolts slipping until they bear against the edge of the slotted hole, then the bolt shears off, and finally the truss walks off the seat. The travel distance for the truss to walk-off of the seat is 4 5/8 in. Sequence (A) and typical tensile force resistance for an exterior seat is shown in Fig. 2–14.

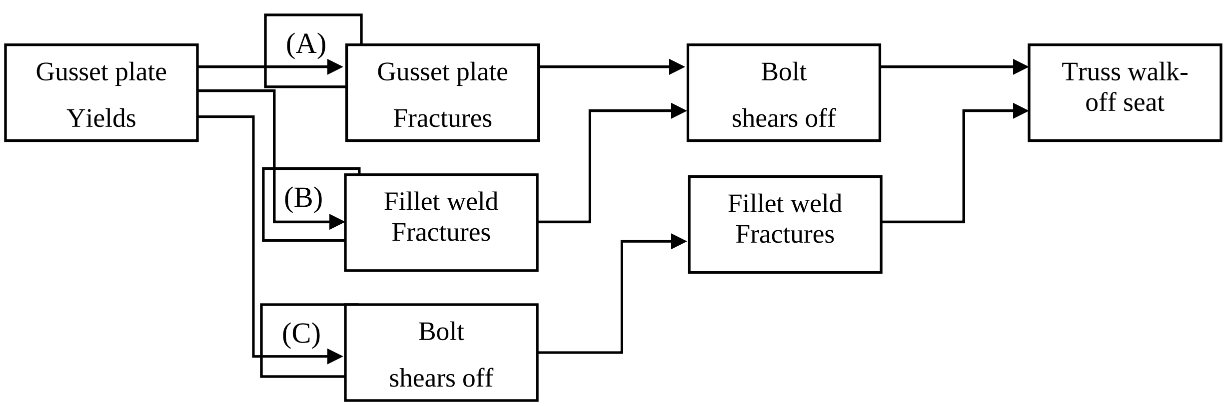


Figure 2–13. Failure sequence of exterior seats for tensile forces.

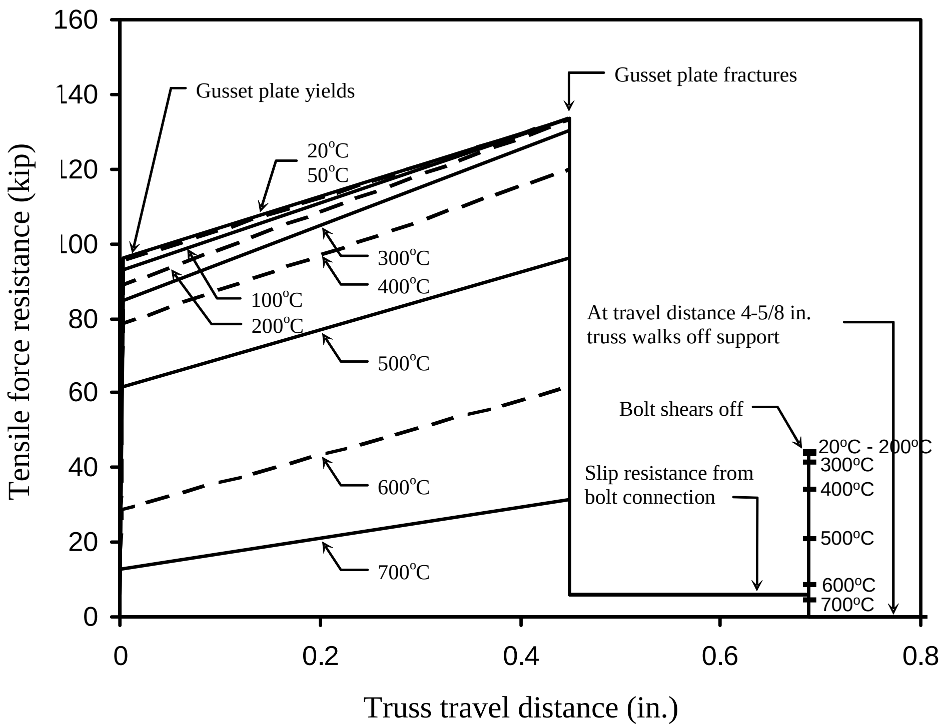


Figure 2–14. Typical tensile force resistance of exterior seat connection.

Knuckle Analysis

The knuckle is a shear connector formed by the extension of the truss diagonals into the concrete slab. Composite action is developed due to the shear transfer between the knuckle and the concrete slab in both the longitudinal and transverse truss directions. The objective of the knuckle analysis is to predict its shear capacity when the truss and concrete deck act compositely and to develop a simplified model of the knuckle behavior for the full floor subsystem model. FEAs have been conducted and calibrated against tests of both longitudinal and transverse loading conditions that were conducted by Laclede Steel in 1967.

Figure 2–15 shows the FEM of the longitudinally loaded knuckle, representing one quarter of the knuckle test specimen. The ANSYS LS-DYNA program, which is part of the ANSYS software package for explicit nonlinear structural analysis, was used for the analysis of the knuckle tests as it had a concrete material model for nonlinear behavior. Solid steel elements were used for the knuckle and channel members and the Psuedo Tensor material model in LS-DYNA was used for the concrete. The knuckle-to-concrete interface was modeled as a bonded or no-friction contact. The finite-element analysis results of the knuckle capacity depended on the steel-concrete interface assumption of bonded or no-friction contact.

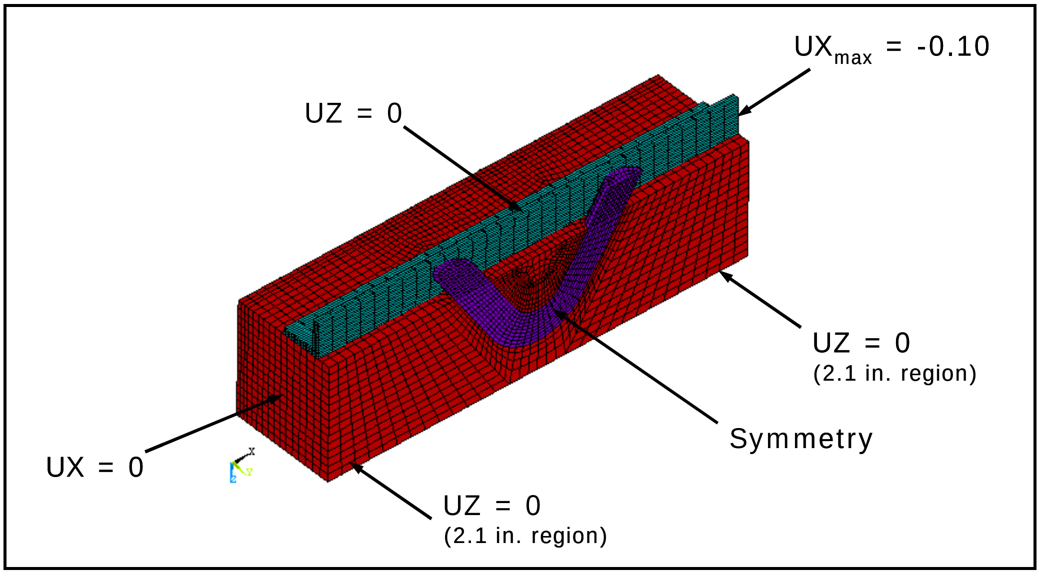


Figure 2–15. Finite element model of transversely loaded knuckle.

Truss Analysis

A typical long-span truss, designated as C32T1, is modeled to study its response to failure when subjected to dead and live loads and thermal loads. The model includes the following:

- One truss of the pair of trusses at column line 143 in floor 96 of WTC1,
- Two exterior columns (columns 143 and 144) with half the area and bending properties (see plan view of Fig. 2–10) , and a length of 24 ft (12 ft above and below the floor level),
- The portion of the spandrel between the two exterior columns,
- The portion of the slab (40 in. wide) between the two exterior columns,
- One strap anchor that is attached to the truss top chord, concrete slab and the adjacent exterior column (Column 144), and
- Exterior and interior seats, and the top plate at the exterior end.

A typical slab section consists of 4 in. thick lightweight concrete on 22 gauge metal deck and has two layers of welded wire fabric. An equivalent thickness of 4.35 in. is used as the slab thickness to account for the fluted metal deck profile. The metal deck and the welded wire fabric are not included in the truss model. Steel bar joist trusses support the concrete slab and act compositely with it. The chords of the trusses consist of double angles while the web members are round bars.

The truss and the columns are modeled with temperature-dependent elastic and plastic material properties. The concrete slab is modeled with shell elements. The nodes of the concrete slab are located at the neutral plane of the concrete slab with an offset relative to the nodes of the top chords. A low tensile yield stress is used to simulate concrete cracking. At knuckle locations, the top chord elements and the elements representing the concrete slab are connected by control elements with capacities determined from the knuckle analysis. Studs on the strap between the top chord and column are also modeled by control elements that connect the strap to the slab. The exterior and core truss seats are modeled by a combination of control elements and link elements, which can have temperature-dependent capacities determined from the truss seat analysis. The interior column is modeled as a fixed support for the interior truss connection, allowing no lateral displacements at the floor level, as the column was braced by the core framing.

Loading consists of gravity dead and live loads and temperature time-histories for all steel members, including the truss seats. The gravity loads include weight of the structure, superimposed dead load (including nonstructural dead loads due to architectural items and fixed service equipment), and a service live load equal to 25 percent of design live load. The steel and concrete temperatures were subjected to a uniform heating condition by ramping to a maximum temperature over 1,800 s and then holding the maximum temperatures for another 1,800 s. The steel temperature increased from 20 °C to 700 °C; the bottom surface of the slab increased from 20 °C to 700 °C, and the top surface of the slab increased from 20 °C to 300 °C. This thermal load creates a linear temperature gradient through the slab from 300 °C at the top surface to 700 °C at the bottom surface of the slab. Elevated temperatures are not applied to the columns.

The truss model can capture the following:

- Temperature-dependent elastic material properties for both steel and concrete
- Temperature-dependent steel plasticity
- Buckling of truss members
- Failure of knuckle – loss of composite action
- Failure of knuckle causing loss of composite action
- Failure of studs on the strap
- Failure of stud between the spandrel and the concrete slab
- Failure of the exterior and interior truss seats

Figure 2–16 shows that the top chords of the truss yield in compression beyond 300 ^{o}C (for clarity, the concrete floor slab is not shown). This is due to a significant difference of coefficients of thermal expansion (CTE) between concrete and steel. At 500 ^{o}C, the CTE of steel is twice that of light-weight concrete. Bottom chords remain in the elastic range throughout the thermal loading. Web diagonal buckling starts around 350 ^{o}C and, as seen in the figure, some diagonals are bent significantly in the plane of the truss by high axial force and end moments.

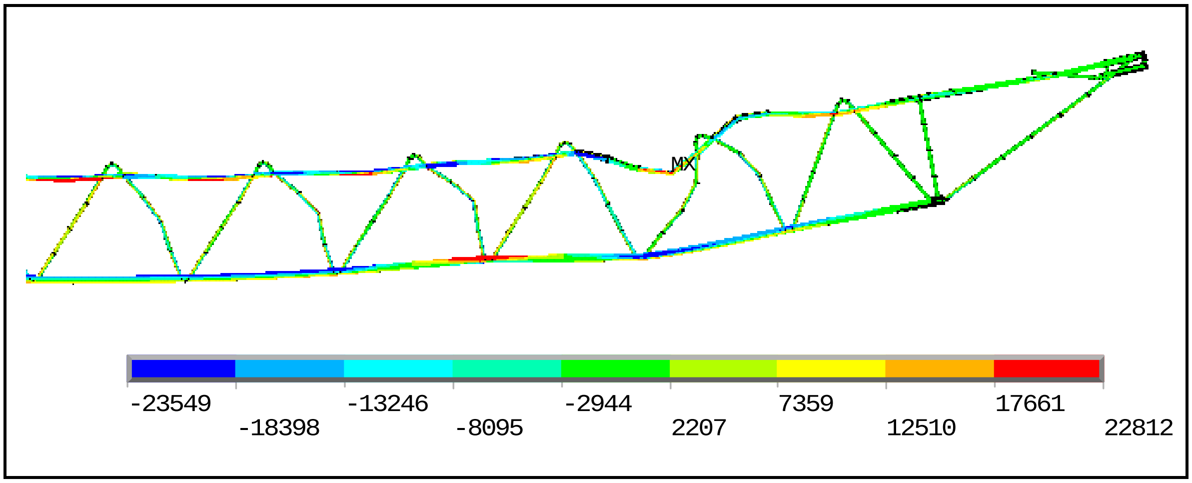


Figure 2–16. Finite element solution for floor truss under gravity and temperature loads.

Preliminary Findings

Preliminary findings are described here for floor truss connection capacities, knuckle capacities, and floor truss response to the uniform heating condition.

Floor Truss Connection Capacities

Capacities of the interior and exterior floor truss connections, for both vertical (gravity) and horizontal forces (tension and compression), have been computed for the variety of connection types found on floor 96 of WTC 1. In all cases, the sequences of failures of the connection components have been taken into account, as illustrated in Fig. 2–13, for the exterior seated connection under horizontal (tension) force. Capacities have been computed as a function of temperature for the applicable plate, weld, or bolt properties. It should be noted that while the computed vertical and horizontal capacities are primarily due to the loads they must support, construction-related decisions may have increased the capacity. For example, an available bolt or steel section size or a minimum allowable weld thickness may provide greater capacity than that required for design loads.

Failure mode of the interior truss seat for vertical force is the fracture of the fillet welds at the seat-to-channel beam connection. Failure mode of the exterior truss seat for vertical force is fracture of the fillet welds at the stand-off-to-spandrel connection. Preliminary findings of connection capacities for vertical bearing forces at room temperature (20 °C), 400 °C, 600 °C, and 700 °C are summarized in Fig. 2–17.

Similarly, connection capacities for horizontal tensile forces at the same temperatures are shown in Fig. 2–18. For interior truss seat connections, the shear strength of the two bolts controls the horizontal tensile capacity. The connection capacity of exterior truss seats that follow failure sequence (A), as shown in Fig. 2–13, equals the failure load for the tensile capacity of the gusset plate. Note that the strength of the truss seat #1013 increases by approximately 38 percent at a temperature of 100 °C. For temperatures less than 100 °C, the horizontal capacity is controlled by the gusset fillet weld strength, and for temperatures above 100 °C, the bolt bears against the edge of the slotted hole and increases the capacity of the connection.

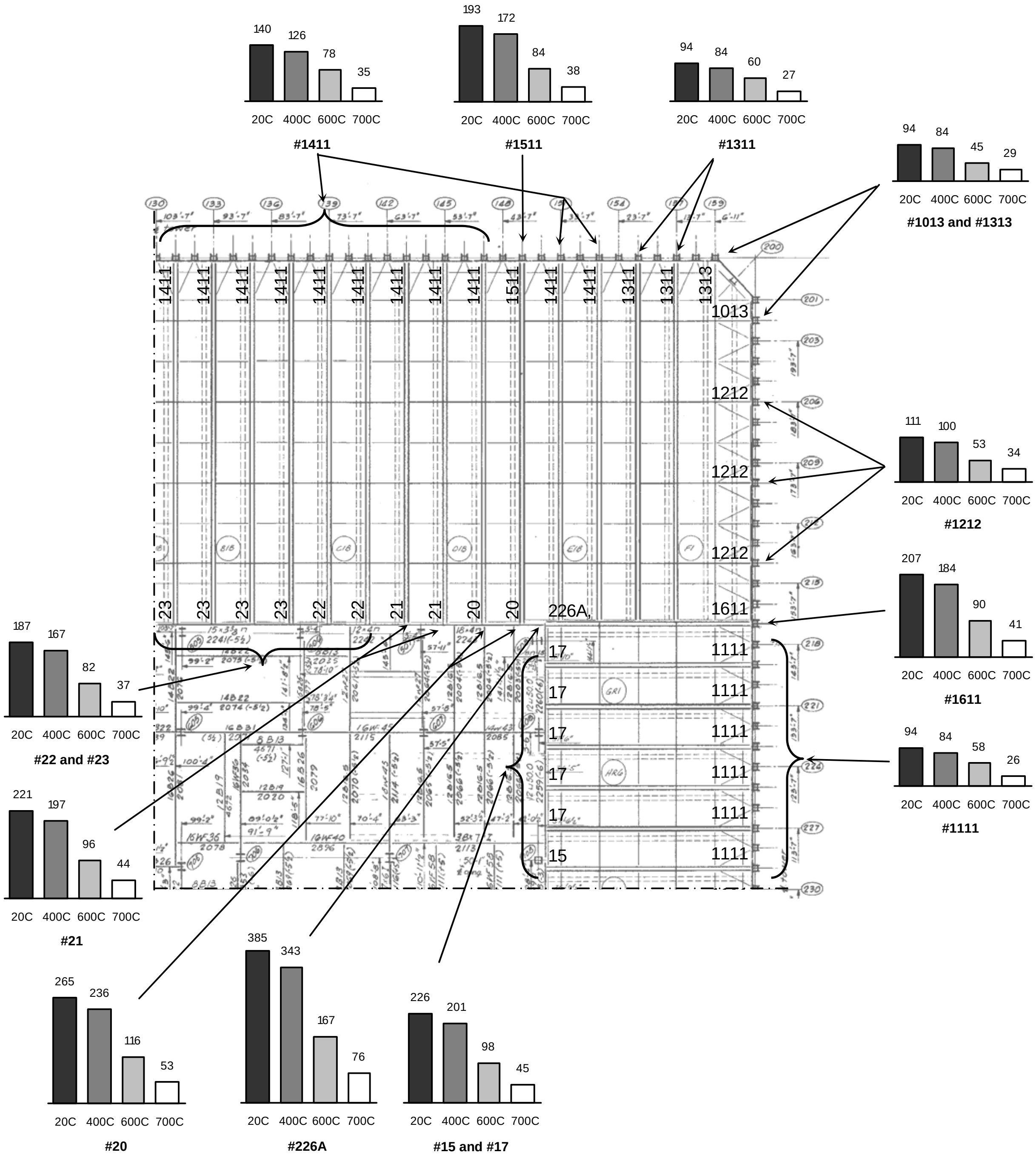


Figure 2–17. Truss seat capacity for vertical forces.

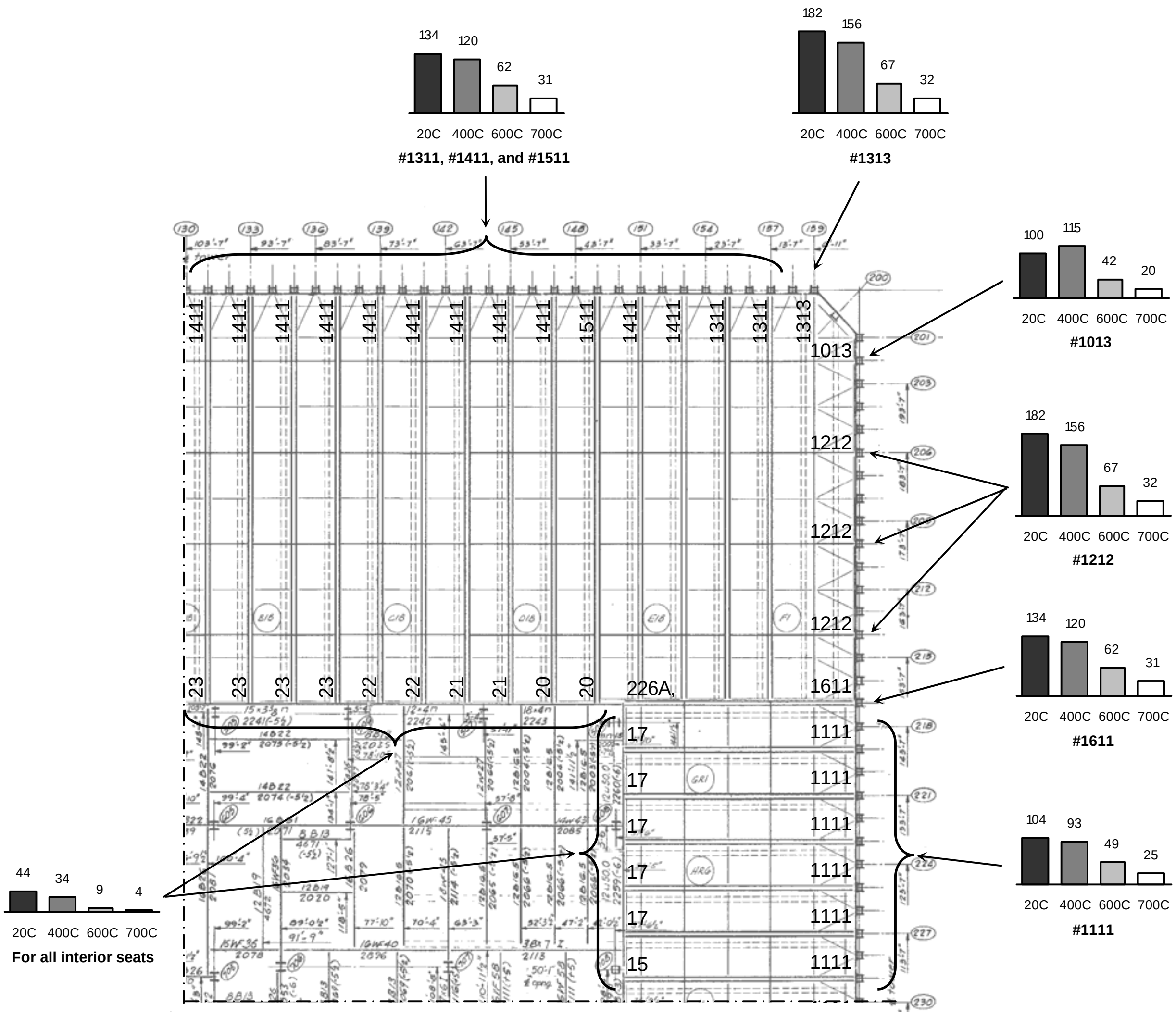


Figure 2–18. Truss seat capacity for horizontal forces.

Knuckle Analysis

FEA results of the knuckle under longitudinal loading are shown in Fig. 2–19. Displacement results shown in Fig. 2–20 indicate a significant dependence on the characteristic of the interface between the steel and concrete. Results show that each knuckle has a capacity in the range of 15 klb to 35 klb, depending on the steel-to-concrete interface assumption. Results of longitudinal shear tests conducted by Laclede Steel in 1967, using normal weight concrete with an average compressive strength of 3,707 psi, indicate an average shear capacity of approximately 28.3 klb per knuckle. After adjusting for the strength of in-place light-weight concrete of 4,100 psi (i.e., multiplying 28.3 klb by the ratio of 4,100 to 3,707 psi), the longitudinal shear capacity of the knuckle is approximately 31 klb per knuckle. This is consistent with the finite element solution for the fully bonded case shown in Figure 2–21 (note results are for two knuckles). A shear capacity of approximately 30 klb per knuckle is used for subsequent analyses of the floor truss.

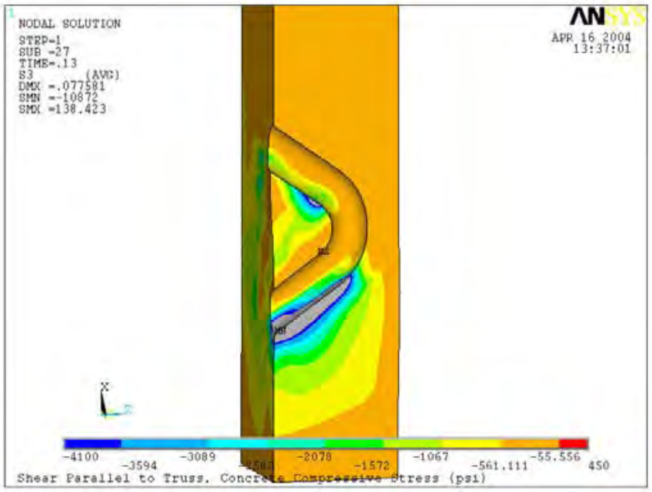


Figure 2–19. Compressive stress in longitudinal shear.

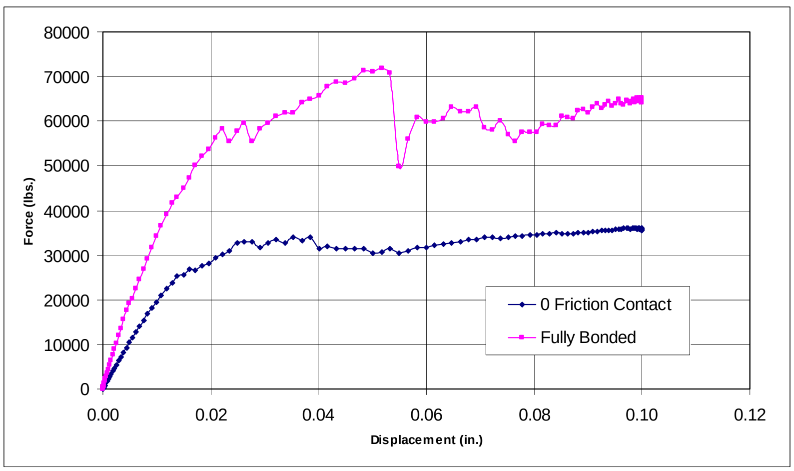
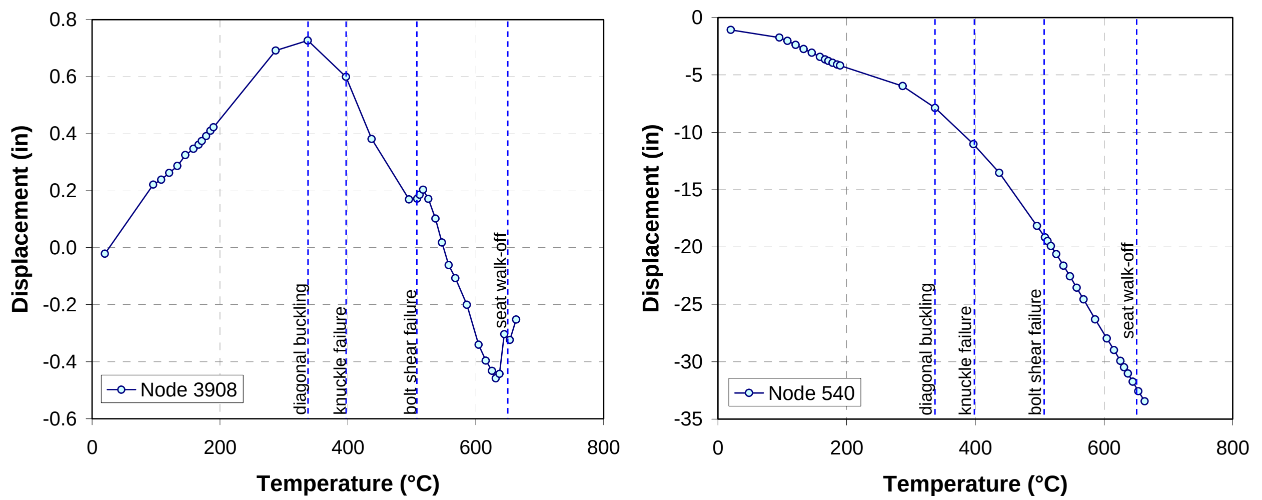


Figure 2–20. Shear force versus displacement for two knuckles under longitudinal shear.

Truss Analysis

The floor truss analysis was carried out dynamically with 5 percent Rayleigh damping and a temperature ramp set to 1.0 s. The floor slab had only gravity loads applied; no other loads related to floor diaphragm action were included. The analysis of truss behavior under the gravity plus thermal loading proceeded to a temperature of T=663 °C. Figure 2–20 shows the horizontal displacement of the column and the vertical midspan displacement of the truss. A positive horizontal displacement indicates that the exterior columns are pushed out, and a negative vertical displacement indicates that the truss is deflected downward.



(a) Horizontal displacement at Column 143 (b) Vertical displacement at midspan

Figure 2–21. Floor truss response due to gravity load and uniform heating.

As the truss and floor slab heat up, the column is pushed outward by thermal expansion. The trust top chord begins to yield in compression around 300 ^{o}C due to the difference in coefficient of thermal expansion between steel and lightweight concrete. At approximately 340 ^{o}C, web diagonals begin to buckle and the horizontal displacement at the exterior column reverses and begins to decrease. At 400 ^{o}C, knuckles start to fail sequentially from both interior and exterior supports toward the center. With the loss of composite action, the floor begins to sag at an increasing rate. Eventually, at about 500 ^{o}C, with the truss sagging almost 20 in., the bolts at the interior connection are found to shear. At 560 ^{o}C, the exterior columns begin to displace inward, and the truss begins to act as a catenary. At 650 ^{o}C, the truss walks off the interior seat while the interior end of slab remains intact and continues to carry vertical load. For the truss to walk off the interior seat, the truss must shorten by 4 in. This shortening is caused mainly by the significant plastic deformation of the top chord of the truss (Fig. 2–14), resulting from differences in the thermal expansion of the top chord relative to the slab and the failure of the first two knuckles near the interior seat. At roughly 660 ^{o}C, the gusset plate fracture at the exterior end which precipitates vertical failure of the exterior seated connection.

The results for the additional debris weight show that the knuckles start failing when 2.4 times dead load is applied. Most knuckles fail before 3.0 times dead load. After the knuckle failure, the truss loses composite action between the truss and the concrete slab, and the vertical displacement increases significantly. As a result, the horizontal reaction force increases.

Models of the truss, including knuckles with temperature-dependent capacities, diagonal weld failure, steel creep strains, and concrete cracking and crushing, are under study.

Summary of Preliminary Findings

Preliminary findings for a single truss and its seat connections and knuckles subject to service load conditions, uniformly increasing elevated temperatures in the steel, and an increasing temperature gradient in the concrete slab, can be summarized as follows:

- The floor truss first experiences increasing vertical deflections at midspan as it pushes outward and exerts a compressive lateral load on the exterior column. The exterior column begins to displace outward at the floor connection.

- As web diagonals begin to buckle at 340 ^{o}C, the midspan deflection continue to increase but the horizontal displacement of the exterior column begins to decrease. The maximum horizontal displacement of the exterior column is approximately 0.7 in. when the diagonals begin to buckle. The interior column is assumed to have no lateral displacements at the floor level, as it was braced by the core framing.

- Knuckles at each end of the truss begin to fail as the steel and bottom surface of the slab temperatures reach 400 ^{o}C, with knuckle failures moving progressively inward from the truss ends. The failure of web diagonals and knuckles at the ends of the truss reduce the flexural rigidity of the floor truss at the ends, further increasing the floor sag and decreasing the lateral outward force exerted on the columns.

- The truss bearing angle slips until the bolt is bearing against the edge of the slotted hole. The bolt shears off at the interior seat connection at approximately 500 ^{o}C. The floor truss sag increases to 20 in. when the bolt fails.

- The interior end of the reinforced slab continues to carry vertical loads as the truss bearing angle continues to slip. At 560 ^{o}C, the exterior column begins to be displaced inward as the floor truss continues to sag and exert vertical and horizontal tensile loads.

- At 650 ^{o}C, the truss begins to walk off the interior seat, followed by fracture of the gusset plate at the exterior connection at 660 ^{o}C. Fracture of the gusset plate precipitates weld failure of the exterior seat connection resulting in complete loss of vertical support of the truss.

- The truss model, with knuckle and seat connections, includes all potential failure modes that may occur under loading and thermal conditions, though the actual sequence of failure may differ under other loading and fire conditions.

2.7.5 Standard Fire Endurance Tests of Floor System

Standard Fire Tests of the steel truss-supported concrete slab floor system used in the WTC towers are being conducted by UL. The results of the testing will provide the fire endurance ratings of typical floor construction to evaluate three primary factors: (1) test scale, (2) fireproofing thickness, and (3) thermal restraint. Four ASTM E 119 Standard Fire Tests of the WTC floor construction will be performed as follows:

- 17 ft span assembly, thermally restrained, SFRM thickness of 1/2 in.

- 17 ft span assembly, thermally restrained, SFRM thickness of 3/4 in.

- 35 ft span assembly, thermally restrained, SFRM thickness of 3/4 in.

- 35 ft span assembly, thermally unrestrained, SFRM thickness of 3/4 in.

The first test represents current U.S. practice for establishing a fire endurance rating of a building construction. The test assembly, fabricated to meet the design of the World Trade Center steel joist-supported floor system, has a span of 17 ft. This span is typical of the floor assembly test furnaces used by the U.S. testing laboratories that routinely conduct the ASTM E 119 test for the construction industry. As is common practice, the floor assembly will be tested in the thermally restrained condition. This test will be conducted at UL's Northbrook, Illinois, fire test facility. A second test will be identical except for the thickness of SFRM.

The third and forth tests will be at twice the scale of the first two tests, with a span of 35 ft. This span represents a full-scale assembly of the 35 ft floor panel of the WTC floor system. The floor assembly for the third test will be thermally restrained as in the first two tests, thereby allowing direct comparison for the determination of the effect of test scale on fire endurance rating. The fourth test will be conducted in the thermally unrestrained support condition, which will allow direct comparison of the effect of thermal restraint on the fire endurance rating. The third and fourth tests will be conducted at the UL Canada fire test facility near Toronto.

In all tests, individual structural members of the steel trusses with varying thickness of SFRM will be exposed to the standard fire environment, and temperatures will be recorded. This will allow comparison of results for various amounts of fireproofing based on the end point criteria for steel temperatures.

The test specimens have been designed and fabricated to duplicate as closely as possible the actual floor system in the WTC towers. Laclede Steel shop drawings were used to ensure the specimens were dimensionally accurate. Properties of the constituent materials and components have been duplicated as closely as possible, including the steel angles and rods that make up the floor trusses, concrete (lightweight aggregate, air entrainment, etc.), metal deck, welded wire fabric, reinforcing steel, shop primer, and Cafco DC/F SFRM. NIST has overseen fabrication of the steel trusses, assembly of test specimens, and casting of the concrete slab and test cylinders. NIST will continue oversight of the installation of instrumentation and application of SFRM.

The test assemblies have been fabricated as shown in Fig. 2–22, and at this time, the concrete floor slabs are drying to the ASTM E 119 specified moisture condition under controlled conditions to obtain concrete design strength. The specimens are currently drying in a temperature/humidity controlled environment to achieve the ASTM E 119 prescribed moisture equilibrium.

Figure 2–22. Fabrication of 35 ft span ASTM E 119 test assembly.

2.7.6 WTC 7

The structural response of WTC 7 to damage from debris and fires is being evaluated to identify possible collapse sequences and critical components that are consistent with the videographic and photographic records, interview accounts by individuals that were in or around WTC 7, and other available data.

The analytical work is being conducted with the assistance of Gilsanz Murray Steficek LLP under a contract from NIST and includes the following tasks:

- **Task 1**. Structural response analysis to identify critical components
- **Task 2**. Structural analysis of possible collapse initiation hypotheses

The scope of work under Task 1 includes (a) develop a nonlinear global structural model of WTC 7 and evaluate its performance under design gravity loads, (b) identify credible failure sequences for the structural model with service loads and initial structural damage by analyzing the effect of component failures (that may have occurred directly or indirectly from fires) on the structural system stability, (c) identify dominant failure modes for critical components and subsystems determined in (b) for service loads and elevated structural temperatures, (d) conduct parametric studies of critical subsystems to identify influential parameters, and (e) develop approaches to simplify structural analyses for global modeling and analyses.

Selected technical results and finding for progress on Task 1 (a), (b), and (c), data collection of building conditions, working collapse hypotheses, and supporting analyses are presented in the following sections. Appendix L presents more detailed information about the WTC 7 structural design, observations about damage and fires, a timeline and description of the collapse sequence from videographic records, and working collapse hypotheses developed to date. Detailed thermal-structural analyses of selected collapse sequences are planned to refine the working hypotheses presented here and identify probable collapse hypotheses.

Summary of Data Collection for Building Conditions

Data that have been obtained about building conditions from photographic and videographic records and interview accounts include:

- Structural damage to the south face and to the southwest corner from WTC 1 debris was reported by witnesses. A multi-story gash that extended across approximately a quarter to a third of the south face, in the lower portion of the face, was reported by a number of individuals, though details vary. This damage extended to the core area as two elevator cars were reported to be ejected from the elevator shaft at floor 8 or 9. Reported damage to the southwest corner was also seen in photographic records, which show approximately 2 columns and related floor areas missing from floors 8 to 18. Multiple photographic and videographic records also appear to show damage on the south face that started at the roof level and severed the spandrels between exterior columns near the southwest corner for at least 5 to 10 floors. However, the extent and details of this damage have not yet been discerned, as smoke is present.

- The south face was covered by smoke the entire day, following the collapse of WTC 1. This smoke appeared to be emanating from fires in WTC 5, 6, and 7, though the contribution from each building cannot be discerned. The smoke was dense enough that no information about structural damage to the south face has been seen in photographic or videographic records.

- No fires were observed in WTC 7 after WTC 2 collapsed, but fires were observed after WTC 1 collapsed. Fires, or evidence of fires, were observed initially on the south face and near the southwest corner. Many of these fires appeared to burn out before noon to 2 p.m. Around 2 p.m., fires were observed in photographic and videographic records to be burning across floors 11 and 12 on the east face, from the south to the north. Around 3 p.m., fires were observed on floors 7 and 12 along the north face. The fire on floor 12 appeared to bypass the northeast corner and was first observed at a point approximately one third of the width from the northeast corner, and then spread both east and west across the north face. Some time later, fires were observed on floors 8 and 13, with the fire on floor 8 moving from west to east and the fire on floor 13 moving from east to west. At this time, the fire on floor 7 appeared to have stopped progressing near the middle of the north face. The fire on floor 8 continued to move east on the north face, eventually reaching the northeast corner and moving to the east face. Around 4:45 p.m., a photograph showed fires floors 7, 8, 9, and 11 near the middle; floor 12 was burned out by this time.

- Floor 5 did not have any exterior windows, and any fires that may have burned on this floor would not have been visible in photographic or videographic records. However, there was a fuel distribution system on the south, west and north floor areas. Given the variability of damage descriptions for the south face from WTC 1 debris impact, fires on floor 5 will be considered as a possible fire location, subject to further data and/or analysis on building conditions that improve knowledge of fire conditions in this area.

- The first exterior sign of structural failure in WTC 7 was the sinking of the east penthouse roof structure into the building. Photographic and videographic records taken from the north have provided information about the sequence of failure events and their relative times.

Other key observations include window breakage along the east side of the north face, occurring almost simultaneously with the sinking of the east penthouse structure, an approximate 5 s delay before the other roof structures also sink into the building core, a second set of window breakage along the west side of the north face occurring simultaneously with the other roof structure movements, and the appearance of the entire north façade above floor 13 appearing to drop as an intact unit 8 s after the east penthouse movement was first detected.

Structural Models and Analysis

Analyses have been conducted to assess proposed collapse hypotheses that are based upon available information about the building conditions and sequence of events prior to the global collapse. Models and analyses to date have included the following:

- Structural analysis of WTC 7, as built in 1985, for design gravity and wind loads on a global structural model. Development of a reference model for understanding global behavior of the structure, and providing a foundation for other models.

- The global structural model was modified for reported structural damage and estimated service loads for analysis of the structural system response to building condition after debris impact.

- A kinematic structural analysis assisted with identifying possible failure sequences following an initiating event, such as a column or group of columns becoming unstable as steel temperatures reach critical levels.

- Typical tenant floors were analyzed to identify the sequence of floor system load redistribution and component failure for initiating events, such as failure of a support column.

- The global structural model was used to develop a submodel of the lower 10 floors of WTC 7 to evaluate the effect of component failure and load redistribution within this portion of the structural system.

- Thermal-structural analysis of critical columns to fire scenarios for proposed collapse hypotheses have been conducted to evaluate the effect of component response to fires, including time to reach critical uniform elevated temperatures and temperature gradients across component cross-section and length.

- These models were used to consider a range of possible failure scenarios, based upon knowledge of the reported damage, observed or possible fire scenarios, and the exterior failure sequence recorded on videos.

Summary of Technical Results

Some of the more important technical results developed to date, based upon known building conditions and analyses, include the following:

- The perimeter moment frame was highly redundant and was able to redistribute the loads around the reported damage areas without over-stressing or failing surrounding members.

- The working hypothesis has been developed around four phases of the collapse that were observed in photographic and video records: the initiating event, a vertical progression at the northeast corner of the building, and horizontal progression from the east to west side of the building, and global collapse.

- The first exterior sign of failure in WTC 7 was the displacement at the center of the east penthouse roofline, which appeared to be a kink in the roof line. This kink aligns with columns 79, 80, and 81. This observation has led to postulating initiating event failure sequences that lead to the failure of one of these columns.

- Thermal-structural analyses have been used to evaluate components in postulated initiating event failure sequences. Analyses to date have included single and multi-story columns subject to severe fires (gas temperatures of 1,100 ºC), intact and damaged fireproofing, intact and missing lateral support conditions, and temperature-dependent material properties (yield strength, ultimate tensile strength, thermal expansion, and creep strains) to evaluate the structural response to thermal softening, axial expansion, and bowing from thermal gradients. These types of analyses continue to be developed and refined. Simpler initiating events that have been analyzed appear unlikely as initiating events, such as the direct failure of columns 79, 80, or 81 in the lower portion of the building for intact fireproofing and the fires observed in the photographic and videographic records. Other initiating event sequences continue to be postulated and analyzed. Possible initiating events include consideration of interior columns 69, 72, 75, 78, and 78A, the east transfer girder (which supports column 78A and frames into transfer truss #2), and adjacent framing and floor systems and their response to possible structural and fireproofing damage from debris impact and subsequent fire growth and progression. See Appendix L for component locations.

- Interior columns 79, 80, and 81, were located directly below the east penthouse on the roof and supported large tributary areas. The tenant floor areas on the east side of the building had spans of approximately 50 ft between columns. Their failure would likely result in failure of the tributary floor system, as analysis indicates that the floors would not be able to redistribute their loads. This failure mechanism would progress vertically upward within the failed bay to the roof level, and would not be visible from the exterior until the east penthouse lost support, as shown in Fig. 2–23. Available information on the floor-to-column connections indicate that the connections would fail under this scenario without significantly damaging the perimeter or interior columns.

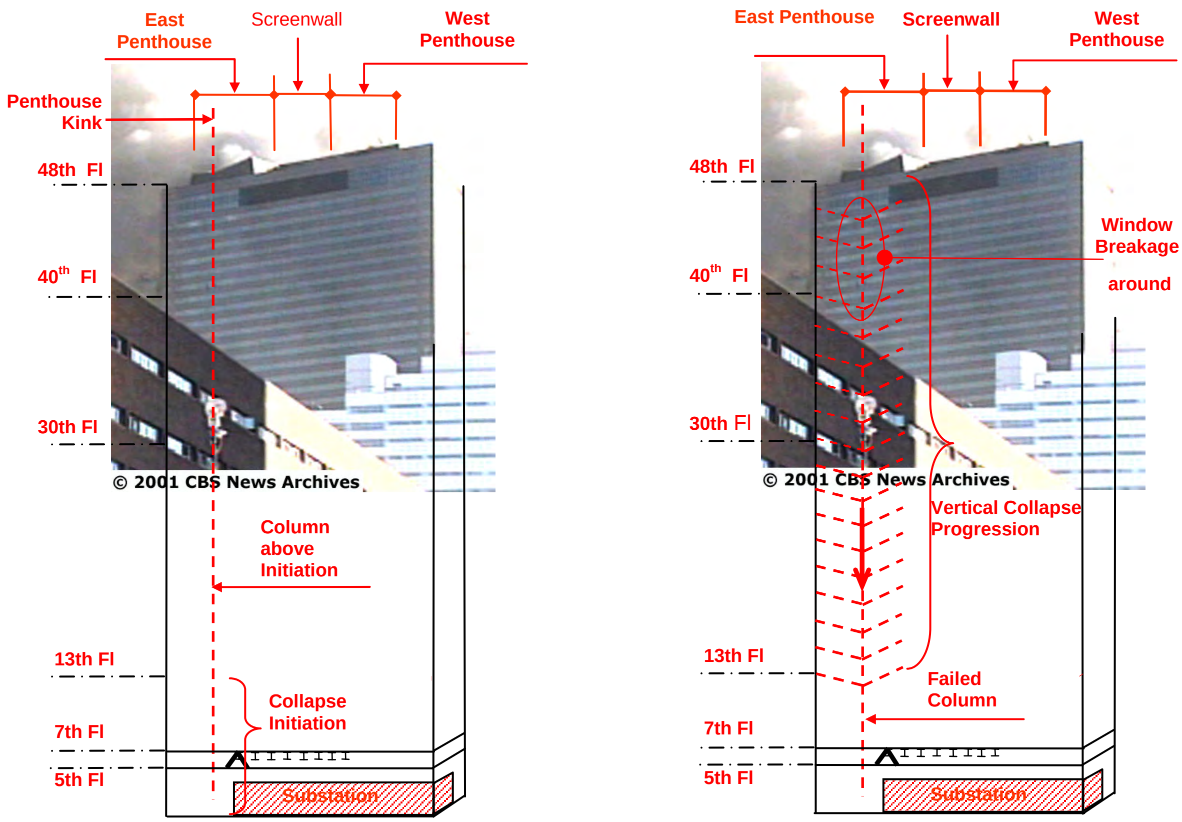


Figure 2–23. Collapse initiation and vertical progression on the east side of WTC 7.

- The debris pile from a vertical failure progression on the east side of the building would damage or sever transfer girders and trusses between the fifth and seventh floors, shown in Fig. 2–24. Two system responses to this secondary damage have been postulated and are being further investigated. (1) The columns supported by these transfer components would become unstable, and their loads would transfer to adjacent core columns. If the columns could not support the transferred loads, the column instability would progress sequentially to adjacent core columns. (2) The floor-to-column connections in the fifth and seventh floors are strong enough to impose lateral displacements upon the other core columns, particularly in the center of core where there were elevator areas without reinforced concrete slab. Such a horizontal pull would fail the columns at their connection near the seventh floor, as shown in Figs. 2–25 and 2–26.

- The core columns failed sequentially and redistributed loads until the building loads could no longer be supported, and the global collapse occurred with few external signs prior to the system failure, as illustrated in Fig. 2–26.

This working hypothesis has been developed to date for the data and analyses described. Continued analyses and evaluation continue toward determining probable collapse sequences.

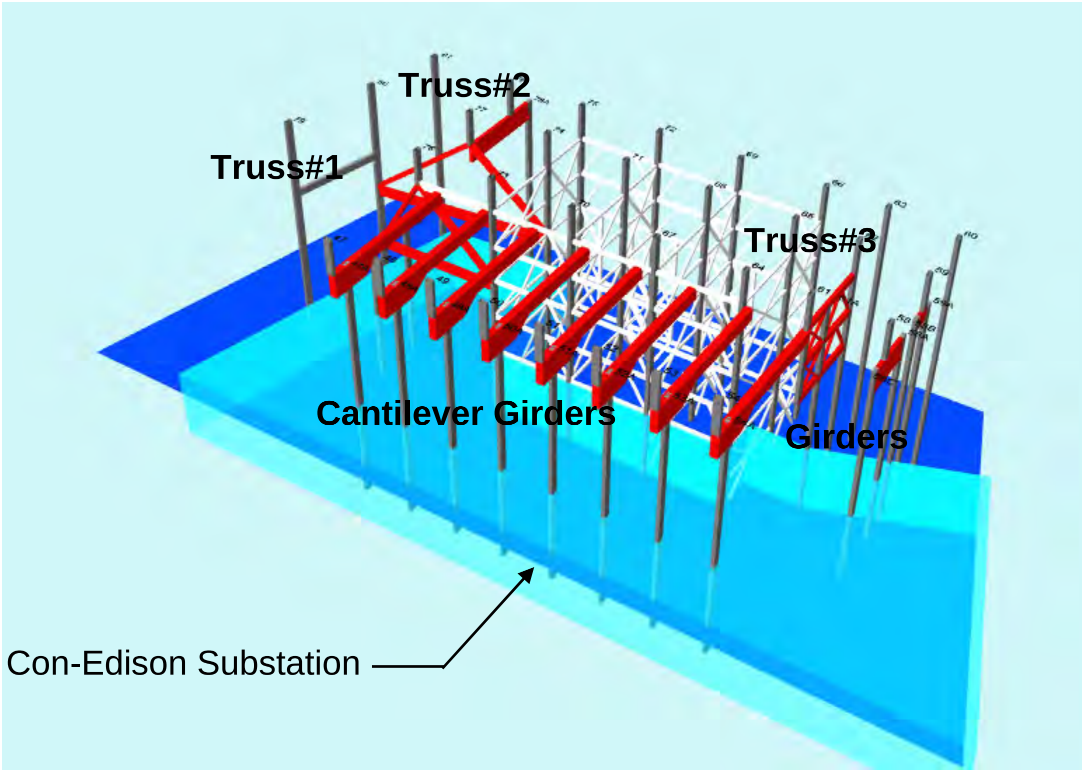


Figure 2–24. Transfer trusses and girders between the fifth and seventh floors.

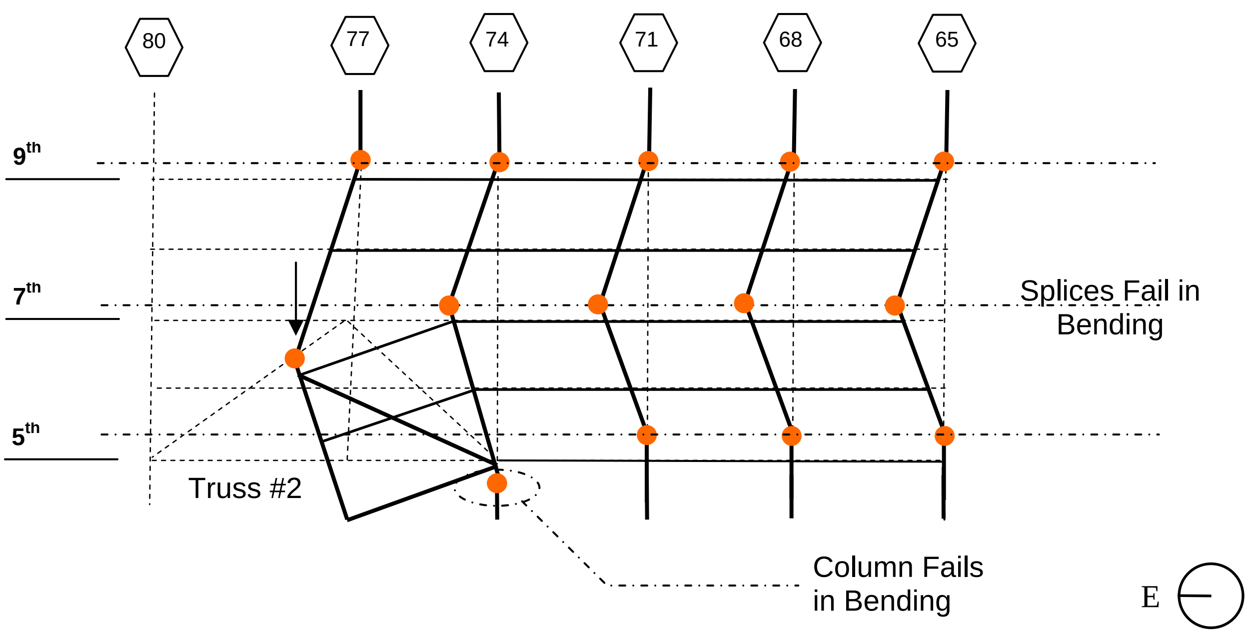


Figure 2–25. Example of horizontal progression of failure in core columns following damage to transfer components on the east side of the building.

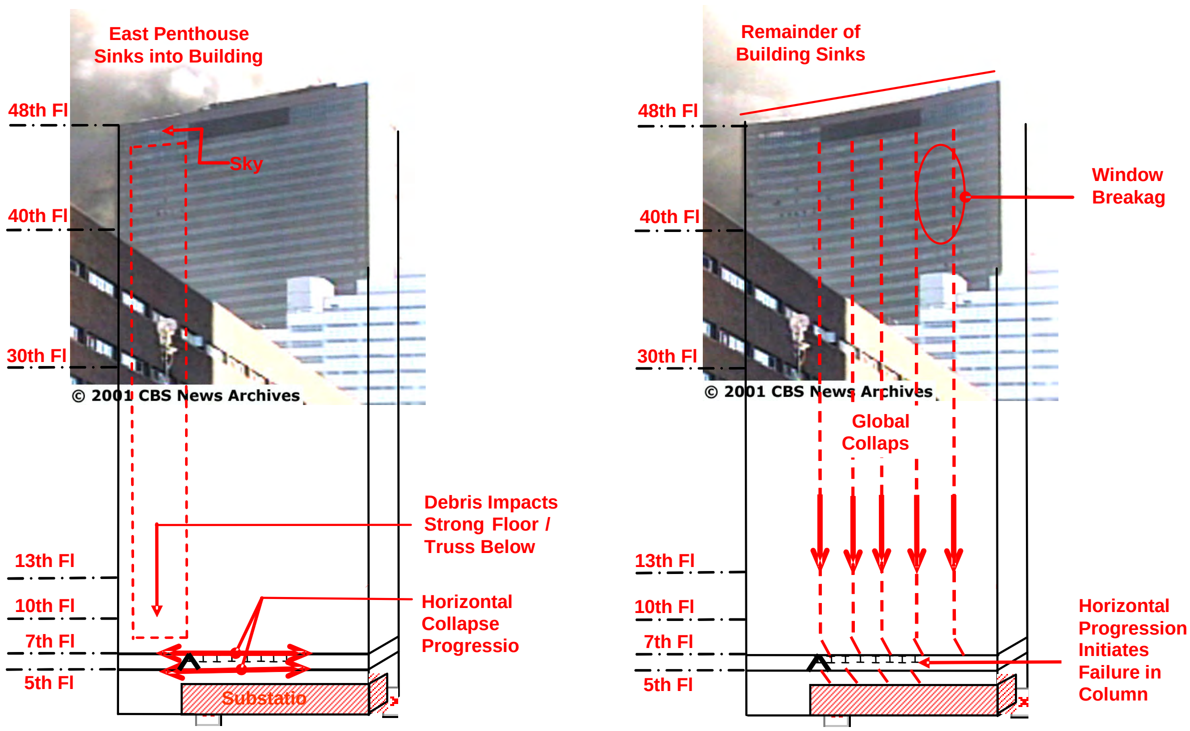


Figure 2–26. Horizontal progression to the west side of WTC 7.

2.8 OCCUPANT BEHAVIOR, EGRESS, AND EMERGENCY COMMUNICATIONS (PROJECT 7)

The purpose of this project is to determine the behavior and fate of occupants and responders - both those who survived and those who did not - by collecting and analyzing information on occupant behavior, human factors, egress, and emergency communications in WTC 1, 2, and 7, and evaluating the performance of the evacuation system on September 11, 2001.

2.8.1 Project Objectives

This project is divided into six tasks as follows:

- **Task 1.** Gather baseline information on the evacuation of the WTC buildings on September 11, 2001 through a comprehensive, systems-oriented, and interdisciplinary data collection effort focused on occupant behavior, human factors, egress, and emergency communications (including instructions given, interpretation of instructions, and response to instructions). This involves the collection of new data from people affected by the WTC attacks (e.g., building occupants, building operators, and first responders via direct accounts from survivors and families of victims), especially those who had to evacuate the buildings. Experts in human behavior and statistical sampling were used develop a data acquisition strategy that considers various data collection methods such as interviews and questionnaires. Inputs and suggestions were obtained from organizations with an interest in the content of the data collection effort. Additionally, written accounts, transcripts of (emergency) communications, published accounts, and other sources of egress related information were obtained, in coordination with other data collection efforts for the investigation.

- **Task 2.** Collect archival records from prior WTC evacuation incidents (e.g., 1975 fire, 1977 blackout, 1980 bomb scare, 1990 power outage, and 1993 bombing) and practice evacuations, including oral history data from floor wardens and fire safety directors. These records are compared and contrasted with the September 11, 2001, incident evacuation. Changes made to the evacuation procedures following the earlier incidents and in recent years will be evaluated in the context of the experience on September 11, 2001.

- **Task 3.** Document pre-event data for WTC Buildings 1, 2, and 7. This information includes, but is not limited to, physical aspects of building egress components, such as stairs (width, number, location, vertical continuity), evacuation lighting, back-up power, elevators (number, operational before and after impact, role in evacuation), and active fire protection systems (sprinklers, manual suppression, fire alarms, smoke control). Building plans, emergency plans, type and frequency of evacuation drills, occupancy level and distribution on the morning of September 11, 2001, and communications also constitute pre-event data. This information provides a baseline for evaluating the performance of the egress system.

- **Task 4.** Store the information collected in Task 1 in a database. Additionally, information from third-party sources, such as television interviews and newspaper articles, as well as other relevant published material, will be analyzed, examined, and assembled in the database.

- **Task 5.** Analyze the data to study the movement of people during the evacuations, decision-making and situation awareness, and issues concerning persons with disabilities. A timeline of the evacuation will be developed using the results of these analyses together with other data sources. This timeline will be compared with the timeline of the structural response, the development of the interior conditions (fire and smoke), as well as activation of the active fire protection systems. The characteristics of the WTC evacuation designs and protocols will be evaluated, including the performance of stairs and elevators, emergency communications, and the temperature and smoke conditions. The designs will also be compared with building code requirements and practices for tall buildings in other major cities worldwide. The observed evacuation data will be compared with results obtained using alternate egress models to better understand occupant behavior and identify needed improvements to existing egress models. In addition, the evacuation experience will be compared with previous evacuation incidents in these buildings. The results of the analyses will be reviewed in the context of occupant protection practices for tall buildings, including the consideration of full evacuation and phased evacuation strategies.

- **Task 6.** Report preparation. The results of this project will be synthesized into a chapter to describe the occupant behavior, egress, and emergency communications in WTC 1, 2, and 7, and the performance of the evacuation system. The project staff will contribute to drafting the final investigation report for review by the National Construction Safety Team (NCST) Advisory Committee.

2.8.2 Project Status

Project 7 has made significant progress since the last interim report. Data collection is substantially complete, including interviews and focus groups with survivors and family members. Significant portions of the data analysis are complete, and egress modeling is under way.

Face-to-Face Interviews, Telephone Interviews, and Focus Groups

Over 220 face-to-face interviews have been completed with occupants of WTC 1, 2, and 7, and family members of victims. These numbers do not reflect interviews conducted with first responders or other key building personnel. A preliminary analysis of the interview data indicates that the topic areas of interest and physical building locations of the occupants have resulted in an adequate number of interviews to support analysis. While additional face-to-face interviews may occur in the near future should conditions necessitate, no further interviews are scheduled. Face-to-face interviews typically required two hours, with some requiring significantly more or less depending upon the particular experiences of the respondent. Data were collected using a cooperative, electronic format, previously described as the cue-action-reason technique. The technique formalized a logical, chronological data collection, enabling detailed recall, and diminishing gaps in continuity of actions.

Egress Simulation

The purpose of the modeling project is to obtain evacuation times for two different evacuation procedures for the WTC towers, phased evacuation and total evacuation of the occupants from the buildings. The first objective is to simulate a phased evacuation of WTC 1 or WTC 2, which involves the evacuation of the occupants on the fire floor, the floor above, and the floor below to a specific floor of the building. For this simulation, a representative fire floor was chosen within one of the towers, and the simulation consisted of occupants from the fire floor, the floor below, and the floor above evacuated to two floors below the fire floor. The purpose of the phased evacuation simulation is to obtain evacuation results (time) on how the evacuation from a fire emergency was supposed to work.

The second objective is to simulate total evacuation of the building, and involves three different scenarios. The first total evacuation scenario is the simulation of a full capacity building evacuation (without damage) involving all occupied floors of a WTC tower. The second scenario involves a full capacity building evacuation of WTC 1 with plane damage blocking floors 91 to 110 and a full capacity building evacuation of WTC 2 with plane damage blocking floors 78 to 110. The second scenario will be used to show how long it would have taken occupants in a scenario resembling the September 11, 2001, emergency to evacuate if each tower was fully occupied. Finally, the third scenario is a simulation of a September 11, 2001, capacity building evacuation from a WTC tower. The results from the simulation of the third full evacuation scenario will be compared with evacuation time results obtained from the telephone interviews in an attempt to verify the accuracy of the model. None of the simulations run for this project involves the simulation of the fire environment.

Three evacuation models will be used to perform the modeling objectives outlined in the previous paragraphs. These models are Simulex (IES 2000 and IES 2001), EXIT89 (Fahy 1999), and buildingEXODUS (Galea et al. 2001). Simulex will be used in a limited capacity to simulate the phased evacuation and observe occupant movement on specific floors. EXIT89 and buildingEXODUS will be used in all of the objectives stated above.

Other September 11, 2001, Data Collection

A systematic review of all 9-1-1 emergency calls related to the WTC attacks between 8:46 a.m. and 10:28 a.m. was completed. Occupants from inside the WTC called 9-1-1, sometimes repeatedly, in order to report where they were trapped, the conditions on the floor, and how many other people they were with. The callers often requested advice, guidance, information about the attack, and the progress of the rescue efforts. The information relayed to the 9-1-1 system was unique and invaluable in specificity and timing. Information related to building damage, fire and smoke spread, and occupant mobility has been integrated with other aspects of the NIST investigation.

Seven-hundred forty-five media accounts from were compiled and analyzed. Finally, information about significant previous building fires and evacuations has been compiled as background. An overview of the preliminary results of both analyses is included in the next section, while more detailed analyses are contained in Appendix N and Appendix O. The database can be obtained electronically at http://wtc.nist.gov.

Collection of documents related to the design and maintenance of the egress and emergency communication systems is complete. This collection was coordinated with Project 1, Analysis of Codes, Standards, and Practices.

Data Analysis

Every data channel is being analyzed and synthesized to form a complete understanding of the evacuation of WTC 1, 2, and 7 on September 11, 2001. There are two primary analysis modes, proceeding in parallel: quantitative and qualitative. Quantitative analysis techniques are being implemented with the telephone interview data. An example of the quantitative data analysis can be found in Appendix N. The SPSS 12.0 data analysis package is being used to analyze the telephone interview data. The qualitative analysis is being consolidated with the ATLAS.ti 4.2 program. All face-to-face interviews, 9-1-1 emergency call records, focus group notes, emergency communications, and formal complaints will be simultaneously analyzed using over 100 coding variables. The coding variables encompass the entire scope of Project 7 objectives and are coordinated with the analyses conducted by other projects within the investigation, particularly Project 8, First Responder Technologies and Guidelines.

2.8.3 Interim Findings and Key Issues

Significant Historical Building Incidents

Although historically the WTC attack, subsequent fires, and building collapses are arguably the most significant fire event where building egress played a critical role, concern for fires in large buildings is hardly new. In many cases, provisions in current building codes evolved as a result of high-rise fires as early as 1911 where egress or collapse was an important issue. These provisions include remoteness and protection of egress stairways, the need for control of combustible contents of buildings, the need to provide sprinklers or other alternatives for high-rise buildings, and concern that sprayed-on fireproofing may not adhere properly to surfaces or may be dislodged.

Analysis of Published Accounts

NIST contracted with the NFPA to collect first-person accounts from newspapers, radio and television programs, e-mail exchanges, and a variety of websites. Over a period of 18 months, a total of 745 first-person accounts were collected. These accounts had been published up to 14 months after the event. Although media accounts do not provide the scientific rigor of a proper study, they do present important insights into the events of the day. The objectives of the analysis of the first-person accounts were to gain insight into the variability of human behavior and response time displayed during the evacuation, with the findings to be used as a guide for additional investigation. It should be acknowledged that content analysis of first-person accounts has important limitations: the questions asked by journalists are usually unknown, and, some details might be left unreported and the most dramatic stories over represented. Consequently, the results cannot be generalized to the overall population of the WTC towers.

To analyze the content of the first-person accounts, a questionnaire tool was developed and used to "interview" each account. The questionnaire had 33 questions such as: "On what floor was the person?," "What was the first cue of the event?," "Was the person injured?," or "What were the conditions in the stairs?" Not every account provided answers for all 33 questions, since some accounts lacked certain details, but this is similar to a respondent who did not answer some questions in a survey. Once the 745 first-person accounts were summarized, multiple accounts from the same person were merged into one, which provided accounts for 465 individuals. (Some survivors provided multiple accounts through different sources.) Before any analysis began, the database was further limited to the 435 civilian building occupants who were in either WTC 1 or WTC 2 on that day.

In summary, the accounts analyzed were from 435 individuals; 251 occupants of WTC 1 and 184 occupants of WTC 2. They represented the three different floor strata of the two towers. The accounts were mainly from men (314 versus 118) and from people varying in age from 20 to 89 years old. Among the interesting results found was the means of egress used that morning. Out of 158 people who mentioned their means of egress in WTC 2, 18 used the elevators, and 26 used a combination of stairs and elevators to leave the tower. It was found that the higher the person was located in the tower initially, the more likely it was that this person used an elevator to evacuate. In WTC 1, out of 202 people who mentioned their means of egress, 198 used the stairs, 1 used an elevator, and 3 used a combination of stairs and elevator. This does not include the 22 people who were stuck in elevators when WTC 1 was hit. The most common adverse floor condition mentioned by people in WTC 1 was the presence of smoke (mentioned by 74 people), debris or collapsed walls, ceilings, or floors (72 people), and fires (41 people). In WTC 2, 37 people reported debris or collapsed walls, ceilings or floors on their floor, and 25 people saw smoke.

The most prevalent condition reported for the stairwell was that it was crowded and hot (mentioned by 106 people). A particular condition mentioned for the stairs in both towers was the presence of smoke, mentioned by 78 people in WTC 1 and 29 in WTC 2. The presence of water, usually on the lower stairwell floors, was mentioned by 49 people in WTC 1 and four people in WTC 2. Jammed or locked doors were mentioned by 20 people in WTC 1 and two people in WTC 2.

In WTC 2, 96 people mentioned hearing a message over the communication system to "stay in or return to their office." The majority of them, 69 people, decided to disregard the instructions and continued their evacuation. The 16 people who decided to remain in their offices or decided to turn back didn't have time to travel very far before the second plane hit; at that point they all resumed their evacuation downward.

Among the accounts analyzed, 27 people reported having a disability, and 47 were injured that morning. All these people were supported in their evacuation by coworkers. Half of them stated that they started their evacuation immediately, and one-third mentioned some delay to get organized and seek first-aid. Several people who were disabled or injured evacuated the towers swiftly as occupants formed a single line to let them through rapidly down the stairwell. Many people (143 in WTC 1 and 26 in WTC 2) mentioned being reassured and feeling safe when meeting firefighters in the building. Although the emergency crews disrupted the evacuation in the stairwell by going against traffic, the people appreciatively cheered them on. Phone calls were made by 151 survivors to family and friends to give and obtain information; 20 people called their bosses or colleagues; and another 12 people made calls to authorities. Another 14 people used e-mail wireless technology and pagers to exchange information, which seems to be the only reliable devices used from inside the stairwells.

Telephone Interviews

The survey objectives of the telephone interviews called for collecting 800 computer-assisted telephone interviews (CATI) of persons occupying either of the two WTC towers at the time of the terrorist attacks on September 11, 2001. Attempts were made to equally divide the respondents among WTC 1 and WTC 2 occupants (i.e., n = 400 occupant interviews from each tower). Within each of the WTC buildings, independent, proportionate, stratified samples of survivors were drawn. Eight-hundred three telephone interviews were completed, with 440 from WTC 1 and 363 from WTC 2. Additional discussion of the sampling methodology and disposition can be found in Appendix O.

A response rate analysis indicated differential nonresponse, more noticeably near the impact floors in WTC 1. In other words, respondents were less likely to complete a telephone interview if they had been near the impact floor than respondents who had been lower in the building. Thus, percentages presented in this summary are weighted, unless otherwise indicated. Weighting preserves the ability to accurately generalize the results.

Population of WTC 1 and WTC 2 on September 11, 2001

The total building population is the sum of survivors and decedents. At the time of this report, the City of New York has officially determined that 2,749 people were killed at the WTC on September 11, 2001; no official breakdown of where people were killed presently exists. While an analysis of this issue by Dennis Cauchon, a reporter for *USA Today*, in the months immediately following September 11, 2001, was remarkably complete (Cauchon 2001), differences exist between his projections and the official numbers from the City of New York and other official sources. These differences are shown in Table 2–8. For example, the number of first responders depends on the definition of first responder. The City of New York published an occupational analysis of WTC decedents based on a Census of Fatal Occupational Injuries (U.S. Department of Labor, Bureau of Labor Statistics, in cooperation with the NYC Department of Health and Mental Hygiene and State and Federal agencies). Four hundred and thirty-three decedents' occupations were listed as firefighting, police, or security. This number exceeds by 30 the number of FDNY, NYPD, and PAPD reported killed. This may be attributable to private security forces present inside the towers on September 11 and/or first responders not employed by New York City or PANYNJ. NIST is attempting to resolve these differences in order to fully understand the initial building population.

Table 2–8. Reports of WTC decedents.

<table>
<tr><th>Decedent</th><th colspan="2">Official Numbers</th><th>USA Today[a]</th></tr>
<tr><td>WTC 1 occupants</td><td colspan="2"></td><td>1,434</td></tr>
<tr><td>At or above impact</td><td colspan="2"></td><td>1,360</td></tr>
<tr><td>Below impact</td><td colspan="2"></td><td>72</td></tr>
<tr><td>WTC 2 occupants</td><td colspan="2"></td><td>599</td></tr>
<tr><td>At or above impact</td><td colspan="2"></td><td>595</td></tr>
<tr><td>Below impact</td><td colspan="2"></td><td>4</td></tr>
<tr><td>First responders (total)</td><td colspan="2">433[b,c]</td><td>479</td></tr>
<tr><td>FDNY</td><td rowspan="3">403[d]</td><td>343[e]</td><td></td></tr>
<tr><td>NYPD</td><td>23[f]</td><td></td></tr>
<tr><td>PAPD</td><td>37[g]</td><td></td></tr>
<tr><td>UA 175 and AA 11</td><td colspan="2">157[d]</td><td>157</td></tr>
<tr><td>Uncertain location in towers</td><td colspan="2"></td><td>147</td></tr>
<tr><td>Bystanders</td><td colspan="2"></td><td>10</td></tr>
<tr><td>Total number of decedents</td><td colspan="2">2,749[b,h]</td><td>2,826</td></tr>
</table>

a. Cauchon, Dennis. ‘For many on September, 11, 2001, survival was no accident.’ USA Today, December 20, 2001.
b. Summary of Vital Statistics 2002: The City of New York. Bureau of Vital Statistics, NYC Department of Health and Mental Hygiene. December 2003.
c. Table WTC 8: Occupation of Decedents. All decedents classified as ‘protective service’ occupations, which includes firefighting, police, and guards.
d. World Trade Center Building Performance Study. FEMA 403. May 2002. Includes 10 hijackers as passengers.
e. Increasing FDNY’s Preparedness (McKinsey Report). Available at: http://www.ci.nyc.ny.us/html/fdny/html/mck_report/index.shtml
f. Available at http://www.ci.nyc.ny.us/html/nypd/html/memorial_01.html
g. Available at http://www.panynj.gov/AboutthePortAuthority/PortAuthorityPolice/InMemorium/
h. Does not include 10 airplane hijackers for whom the City has not issued death certificates.

Using the known eligibility rates allows for a projection of the survivors of WTC 1 and WTC 2 present in the building at 8:46 a.m. on September 11, 2001. The analysis indicates that WTC 1 had approximately 7,500 ± 750 surviving occupants, while WTC 2 had approximately 7,900 ± 900 surviving occupants. Thus, the total population of survivors from both towers was 15,400 ± 1,200. Table 2–9 summarizes the projection of population of WTC 1 and WTC 2 on September 11, 2001. Pending resolution of decedent locations, the total building population at the time of the first airplane impact was 17,400 ± 1,200, calculated using the building decedent locations reported by Cauchon.

Table 2–9. Occupancy estimates on September 11, 2001, by tower.

	WTC 1	WTC 2	Total
Estimated total population of survivors	7,500	7,900	15,400
Statistical Precision Calculations			
Sample n	427	376	803
Standard error (p)	1.90 %	1.92 %	1.36 %
Standard error (total)	750	900	1,200
Confidence limits at 5%	±1,470	±1,790	±2,320
Number of Occupant Decedents			
Decedents	1,434[a]	599[a]	2,033[a] – 2,236[b]
Total Building Population			
Total population	8,900	8,500	17,400

a. Calculated as 2,749 – 343 FDNY – 23 NYPD – 147 airplane passengers (not including hijackers).
b. Calculated as 2,749 – 403 First Responders – 157 airline passengers (not including hijackers).

Previous Evacuation Experience

Whether an occupant had a previous evacuation experience may have affected the decisions an individual made during the September 11, 2001, evacuation. NIST will conduct further analysis to develop this hypothesis. Of the WTC 1 occupants present on September 11, 2001, 16 percent (n = 64) were also present during the 1993 bombing. Sixty percent (n = 38) of WTC 1 evacuees in 1993 reported that they evacuated immediately, 30 percent (n = 20) reported that they waited to evacuate, and 9 percent (n = 6) did not recall. Most (95 percent [n = 53]) who were able to recall their evacuation decision felt that they made the right decision, while 5 percent (n = 3) did not believe they made the right decision.

Similarly, 16 percent (n = 59) of WTC 2 evacuees on September 11, 2001, also evacuated in 1993. In WTC 2, however, only 75 percent (n = 42) felt that they made the right decision in 1993, possibly due to the fact that many more waited to evacuate in 1993 in WTC 2 (69 percent [n = 39]) than did so in WTC 1. Only 31 percent (n = 17) who reported their decision evacuated immediately from WTC 2 in 1993, keeping in mind that the bomb had a more significant impact on WTC 1 in 1993.

Preparedness and Training

Long a cornerstone of public policy on the emergency preparedness of office workers around the country, the Port Authority required tenants to conduct fire drills every 6 months and appoint employee floor wardens and searchers. Sixty-six percent (n = 529) of WTC 1 and WTC 2 occupants reported participation in at least one fire drill in the 12 months immediately prior to September 11, 2001. Seventeen percent (n = 139) reported that they did not participate in any fire drills in the 12 months prior to September 11, and 17 percent (n = 135) did not know. Fire drill participation rates were similar between the two towers (as shown in Table 2–10).

Table 2–10. WTC fire drills in 12 months prior to September 11, 2001.[a]

Number of Drills	WTC 1	WTC 2
None	18 % (n = 78)	17 % (n = 61)
1	13 % (n = 57)	8 % (n = 29)
2	21 % (n = 90)	24 % (n = 88)
3	11 % (n = 47)	15 % (n = 53)
4	10 % (n = 44)	9 % (n = 32)
5–11	7 % (n = 31)	9 % (n = 32)
12 or more	3 % (n = 13)	4 % (n = 13)
Unknown	18 % (n = 80)	15 % (n = 55)

a. Percentages are weighted, n values unweighted.

One of the primary goals of fire drill training is to make occupants aware of the location of the emergency exits. Ninety-three percent (n = 490) of respondents who reported participation in a fire drill were instructed about the location of the nearest stairwell as part of the drill. However, of the respondents who reported being shown a stairwell, 82 percent (n = 432) did not enter or use the stairwell. Seventeen percent (n = 92) reported that they did use the stairs during a drill, while approximately 1 percent (n = 5) reported not knowing. Overall, more than half (51 percent [n = 415]) of the occupants had never used a stairwell in WTC 1 or WTC 2 prior to September 11, while 48 percent (n = 386) had used a stairwell. Two persons reported not knowing whether they had used the stairs previously.

Another goal of the fire drills was to introduce the floor warden system and evacuation procedures. Eighty-two percent (n = 528) of the occupants with fire drill training were aware that there was a floor warden for their floor. Approximately 70 percent (n = 557) of all occupants reported that they were aware of the evacuation procedures. When asked what those evacuation procedures comprised, however, answers varied significantly, including: wait in hallway for further instructions; do not use elevators, use stairs; meet at a designated site outside the building for a head count; or proceed down (varied number of) flights of stairs and wait. Further analysis of the understanding and implementation of the emergency procedures is under way.

Future Work

Significant additional analysis is presently under way. It is particularly important that results of questions related to the events, observations, and activities within the towers on September 11, 2001, be analyzed within the context of the findings coming from face-to-face interviews, focus groups, and other data collection activities. As with the work presented in this appendix, ongoing analysis of the telephone interviews will form the statistical basis for many significant recommendations, and there will be continued analysis of the information collected during the face-to-face interviews and focus groups.

NIST is utilizing existing computer egress models to better understand the evacuation experience on September 11, 2001. Three full evacuation scenarios are being considered: a typical full capacity building evacuation assuming the WTC tower is fully occupied—with one case considering only tenants and another case considering both tenants and visitors; a full capacity building evacuation of each WTC tower with aircraft impact damage; and a September 11, 2001, capacity evacuation from a WTC tower. NIST is using two classes of egress models in order to frame the evacuation questions: (1) partial behavior: simulates occupant movement and limited behavioral rules by including delay times, smoke effects, and occupant characteristics; and (2) behavioral: simulates movement and more comprehensive evacuation decisions and activities.

2.9 FIRE SERVICE TECHNOLOGIES AND GUIDELINES (PROJECT 8)

The purpose of this project is to build on work already done by the FDNY and McKinsey & Company by (1) fully documenting what happened during the response by the fire services to the attacks on the WTC up to the time of collapse of WTC 7; (2) identifying issues that need to be addressed in changes to practices, standards, and codes; (3) identifying alternative practices and/or technologies that may address these issues; and (4) identifying research and development (R&D) needs that advance the safety of the fire service in responding to massive fires in tall buildings.

2.9.1 Project Objectives

Project 8, Fire Service Technologies and Guidelines, has four tasks:

- **Task 1.** Collect emergency response data in cooperation with FDNY to document first responder fatalities, command and control procedures, and equipment performance. Records of interest include dispatch logs, recorded radio communications, run logs from surviving responding units, 9-1-1 records, data recorded by the FDNY, PANYNJ operations, and the NYPD, and fireground positioning of emergency apparatus. Information will also be sought on operations and function of communications systems, on-site emergency information systems, fire alarm panels, elevator control panels, standpipes and fire hoses, and other pre-positioned emergency equipment. In coordination with Project 7, Occupant Behavior, Egress, and Emergency Communications, oral history data will be collected from witnesses, those in control of emergency operations, and surviving first responders to the extent their oral history has not already been documented. Technical experts will review and conduct a fact-based analysis of the data.

- **Task 2**. Interpret the factual analysis to determine the effect on responder successes of factors such as:

  – The influence of building design (e.g., height, stairways, elevators, smoke control systems) on fire service command and control procedures, life saving operations, and safety of rescue personnel;

  – The influence of aircraft impact damage and fuel run-off on fire service command and control procedures, life saving operations, and safety of rescue personnel;

  – The impact of systems failures (e.g., communications systems, water supply, sprinklers, standpipes) on fire service command and control procedures, life saving operations, and safety of rescue personnel;

- Building occupant egress as related to fire service operations;
- The ability to fight large fires on the upper floors of tall buildings;
- The impact that the 1993 bombing of the WTC had on codes, standards, and procedures that affected first responders in tall buildings;
- Preplanning, training, and standard operating procedures (including command and control) at the time of the incident;
- Firefighter accountability, location, and tracking;
- Fire and emergency response protocols for tall buildings;
- The resources available for initial situation assessment and incident management, and practices for determining the possibility of structural collapse; and
- Communications and coordination of response activities with other authorities at the incident.

- **Task 3**. Identify alternative emergency response practices and technologies that may advance the safety and effectiveness of first responders, such as: knowledge/information systems for command and control decisions; elevator use by firefighters; firefighter tracking systems; interoperability of communication systems (occupants, firefighters, police, emergency management services); fire growth and smoke hazard prediction; structural safety monitoring, assessment and prediction; and simulation tools for training.

- **Task 4**. Report preparation. The results of this project will be synthesized into a report to describe the actions of the fire service and performance of their equipment; identify available alternatives related to fire service technology, training, and operational procedures; and identify R&D needs in support of their capability to protect the public, themselves, and vital physical infrastructure during extreme events.

2.9.2 Project Status

Task 1, Data Collection

Project 8 has collected or reviewed a large volume of data from the three primary departments that sent first responders to the WTC, FDNY, NYPD, and PANYNJ. NYC submitted data directly to NIST for analysis and also provided NIST with the opportunity to review other data related to the incident in their offices. The data collection and review process is largely completed. Data collected and review includes: documentary data, electronic data, and first-person face-to-face interviews.

Documentary data collected or reviewed included the following:

- Official lists of first responder fatalities
- FDNY and NYPD McKinsey & Company reports

- FDNY dispatch logs
- FDNY run logs of surviving units (CD12/CD15)
- FDNY standard operating procedures and policies
- Documents describing radio communications equipment and operations
- Review of approximately 500 interviews conducted by FDNY

The following electronic data have been collected or reviewed:

- Approximately 1,000 hours of radio and telephone communications for PAPD and other departments within the Port Authority
- NYPD Special Operations Division and Division 1 tape recordings
- NYC 9-1-1 Emergency Telephone Operator tapes
- FDNY Fire Dispatcher tapes

First-person interviews—To date, 108 face-to-face emergency responder interviews have been completed:

- 68 FDNY personnel including Commissioners, Chief Officers, Company Officers, Firefighters, and Emergency Medical Service personnel
- 24 NYPD personnel including Commissioners, Chief Officers, Aviation Officers, Emergency Service Unit personnel, and Police Officers
- 13 PANYNJ personnel including managers, directors, communications personnel, vertical transportation personnel, fire safety personnel, security personnel, PAPD Chief Officers, and Police Officers
- 3 other emergency responders including security personnel, fire safety personnel, and communications personnel

Task 2, Factual Analysis of the Collected Data

As documentary data were collected they were reviewed for information critical to understanding the emergency response at the WTC. This information has been filed relative to the source of information and the type of information. The documentary data are being compared with other data gained through communications records and recordings and first-person face-to-face interviews. The data are in the process of being coded for analysis using Atlas.ti software.

In addition, Appendix P contains results from a preliminary analysis of emergency responder communications. The objective of this analysis is to develop a better understanding of the role that emergency communications played during the WTC attack, and to quantify information related to communications effectiveness. Although there have been numerous reports of radio equipment failures during the emergency response at the WTC, and these are all being examined as part of the overall project, the only radio equipment system examined in this preliminary report is that of the FDNY WTC site high-rise repeater that was installed by the Port Authority. Many factors are associated with the ability of emergency communications to be successful. The following objectives were set in this report:

- To document radio and telephone communications operations
- To document radio communications readability or understandability
- To quantify radio communications traffic volume
- To understand the impact of traffic volume on communications readability and the transfer of information
- To identify communications associated with dispatch and arrival of responders
- To identify communications related to evacuation and emergency response operations
- To identify communications related to building conditions at the WTC and the impact of this information on the emergency response.

Results from this analysis are presented in Sections 2.9.3, Preliminary Results on Emergency Responder Communications; 2.9.4, Preliminary Findings; and Appendix P, Interim Report on Emergency Communications.

Appendix P also contains a detailed description of methods used for documenting and analyzing the communications data obtained from the various sources.

Task 3, Alternative Emergency Response Practices and Technologies

As the data analysis has progressed, issues critical to emergency responder operations are being identified. Most of the critical practices and technologies identified relate to Command and Control, Communications, and significant issues related to emergency responder high-rise building operations. This work is still in progress, and details concerning alternative practices and technologies will be addressed in the final report.

Task 4, Report Preparation

The efforts described above in Tasks 1, 2, and 3 continue. Data analysis necessary for final report preparations is ongoing. The final report outline has been drafted, and report preparation has been initiated.

2.9.3 Preliminary Results on Emergency Responder Communications

A more detailed analysis of radio equipment and systems operations is in progress, and it will be addressed in the final report. However, the following preliminary results on emergency communications provide some insight into the challenges associated with communications systems following the attack on the WTC.

All radio systems analyzed appeared to work well during the normal operations period just before the attack on the WTC. It was noted that Channel W of the PAPD was experiencing some difficulty with a handie-talkie radio transmitting a carrier wave from an open or keyed microphone. This disrupted communications on that channel. PAPD personnel were trying to correct the problem just before the first aircraft struck WTC 1. The problem continued after the attack occurred. NYPD also had a problem with an open microphone after the incident began. This occurred on the Special Operations Division (SOD) channel. Efforts made by NYPD personnel corrected the problem.

All radio communications evaluated experienced traffic volume surge load conditions as a result of the attack. Coupled with the increase of traffic load was a significant increase in a process known as “Doubling” or “Crossing” of radio signals. This condition occurs when more that one person attempts to transmit a message at the same time on the same radio frequency. Doubling results in the mixing of radio signals that seriously degrades the signal readability. This condition will often block all clear communications from getting through, except possibly from a high power base station.

2.9.4 Preliminary Findings

The following are preliminary findings based on the current status of emergency responder communications analysis:

- After the first aircraft struck WTC 1, there was an approximate factor of 5 peak increase in traffic level over the normal level of emergency responder radio communications, followed by an approximate factor of 3 steady increase in the level of subsequent traffic.

- A surge in communications traffic volume made it more difficult to handle the flow of communications and delivery of information.

- Analysis of the radio communications records received by NIST indicates that roughly one-third to one-half of the radio messages transmitted during these radio traffic surge conditions were not complete messages nor understandable.

- Preliminary analysis of the FDNY City-wide, high-rise Channel 7 (PAPD Channel 30) recording indicates that the WTC site repeater was operating.

- Communications records and interviews indicate that smoke and heat conditions on the top of the two WTC buildings prevented the NYPD helicopters from conducting safe roof evacuation operations.

- NYPD aviation unit personnel reported critical information about the impending collapse of the WTC towers several minutes prior to their collapse. No evidence has been found to suggest that the information was further communicated to all emergency responders at the scene.

Analysis of communications records and face-to-face interviews with emergency responders indicate that radio and telephone communications were a critical part of the emergency response operations. It is also clear from the evaluation that communications technology was not operating at a performance level adequate for handling emergency responder requirements at the incident.

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Chapter 3
UPDATE ON SAFETY OF THREATENED BUILDINGS (WTC R&D) PROGRAM

3.1 OBJECTIVES OF SAFETY OF THREATENED BUILDINGS (WTC R&D) PROGRAM

This program is designed to (1) facilitate the implementation of recommendations resulting from the World Trade Center (WTC) investigation, and (2) provide the technical basis for cost-effective improvements to national building and fire codes, standards, and practices. Under the program National Institute of Standards and Technology (NIST) will develop guidance and tools to assess and reduce building vulnerabilities and will support private sector organizations that develop building and fire codes and standards in the United States. Implementation of the results will better protect building occupants and property in the future, will enhance the safety of fire and emergency responders, and will increase confidence in the safety of commercial and public buildings.

Four general areas of research are targeted to support near- and long-term improvements to reduce the vulnerability of the structure, building occupants, and first responders to potential threats:

- Increased Structural Integrity
- Enhanced Fire Resistance
- Improved Emergency Egress and Access
- Building and Emergency Equipment Standards and Guidelines

3.2 BACKGROUND AND DESIRED OUTCOMES OF SAFETY OF THREATENED BUILDINGS (WTC R&D) PROGRAM

Building and fire codes in the United States exist, among other reasons, to ensure the safety of occupants in the event of anticipated excessive loads due to wind, earthquake, and snow, and the potential for severe fires. The tragic collapse of the WTC buildings in 2001 (along with the terrorist attacks on the Pentagon, Hart Senate Office Building, and the Murrah Federal Building) has focused the general public, governments at all levels, and the construction and building products industries on the need to understand the possible impacts of terrorist acts on building operations, structural integrity, and emergency response procedures, and on the need to develop economically justifiable strategies to mitigate the potential loss of life from possible future threats.

The standard test methods and building practices upon which current building and fire codes are based rank the performance of one material, component, or system against alternative designs, with the expectation that some minimum rating translates into a sufficient level of safety of the material, component, or system when installed in a building. Safety factors are used to account for our ignorance abut the magnitude of actual loads, and of the uncertainty in response of the complex building frame to these loads.

The prediction of failure modes in a closely-coupled building system is beyond our current capability, and standard test methods provide little information on the expected performance of the building should the mechanical or thermal load exceed a prescribed value.

In addition, building designers, operators, occupants, and first responders are faced with chemical and biological threats unforeseen as little as two years ago. How should heating, ventilation, and air-conditioning (HVAC) systems be designed and operated to contain a poisonous aerosol or gas? How has the behavior of occupants changed since September 11, 2001, in responding to an emergency? Should the same emergency egress and fire service access techniques and strategies be used in the case of a biological threat as for a fire? Can new technologies be developed, or design practices adapted, to increase the safety of the building occupants without undue economic burden on the owners/operators?

Additional research and development is being conducted in this program to answer questions like these, to provide guidance and tools to assess and reduce future vulnerabilities, and to better prepare facility owners, contractors, designers, and emergency personnel to respond to future disasters, naturally or intentionally initiated.

Increasing Structural Integrity—Structural integrity will be increased through the development and implementation of performance criteria for codes and standards, tools and practical guidance for prevention of progressive structural collapse. System design concepts, retarded collapse mechanisms, built in redundancy, and hardening structures though retrofit are being considered. Performance criteria for fire safety design and retrofit of structures is being developed through examination of five key factors: the suitability of standard fire resistance test methods; the role of structural connections, diaphragms, and redundancy in enabling load transfer and maintaining overall structural integrity; the effectiveness of alternative retrofit, design and fire protection strategies to enhance structural fire endurance; the fire behavior of structures built with innovative materials; and models to predict the fire hazard to structures from internal and external fires. Guidance on methods to enhance fire resistance of steel and concrete structures based upon our current state of knowledge is being developed as well.

Enhancing Fire Resistance—Fire resistant steels exist and are in use elsewhere in the world. More efficient and accurate tests for performance of steels under building fire conditions are needed and are being developed to help industry incorporate fire resistant steels into U.S. construction practice. Fundamental mechanical and thermal properties of fire protective materials are being measured. This requires the development of new test methods and instrumentation, and a data base that spans the full range of expected temperatures and mechanical loads. These data will supplement, or may even supplant the need for, the ASTM E 119 test in certain situations, and in any case are essential to the implementation of meaningful performance codes and design criteria.

Facilities do not yet exist that are suitable for demonstrating in a quantitative manner the improved performance of new materials, systems, and processes in their end-use within a building under actual fire conditions. Hence, simulations are required to bridge the fundamental data and the results of bench- and pilot-scale tests to the environment in which they would be exposed during severe fire conditions. The severity of a fire is dependent upon many parameters that are beyond the control of the building designer, especially when one considers the range of terrorist threats that are possible. The performance in a fire of non-structural elements such as walls and ceilings is directly linked to the structural integrity of the building because a collapsed wall, ceiling, or floor exposes more areas of the building to the fire while providing additional fuel and air upon which the fire can feed. The technical basis for accurate measurement methodology and simulation tools for the inclusion of fire resistant properties of walls and ceilings in performance-based fire safety design is being developed under this program.

Improving Emergency Egress and Access—By working with the primary stakeholders (elevator and construction industries, fire services, professional societies, and code making bodies), the role of elevators in providing access by the fire service to a fire in a high rise building is being greatly enhanced over current practice. The development of hardened fire service elevators and new emergency operation procedures/controls will also lead to improved egress capabilities from tall buildings, especially for mobility-impaired or injured occupants. However, the behavior of people in an emergency situation has been altered in unpredictable ways by the events of 9/11. Current egress models may be inappropriate and/or insufficient for the design and placement of doors and stairways and the control of elevator movement. Behavioral and engineering studies are being conducted, drawing on experts in academia and elsewhere, to enable the development of simulation tools that better capture the movement of people within a building under fire and other emergency situations.

Developing Building and Emergency Equipment Standards and Guidelines—Partnering with other federal agencies and American Society of Heating, Refrigerating and Air-Conditioning Engineers, Inc. (ASHRAE), NIST-developed indoor air quality (IAQ) simulation tools are being extended to analyze and guide the assessment and subsequent reductions in the vulnerability of buildings to chemical, biological, and radiological aerosols. Standard building information models that facilitate the simulation of building system behavior during adverse events are being developed to allow communication among IAQ controls and other building controls associated with, for example, security, transportation, energy, and fire alarm systems. A user-friendly tool is being developed for building owners and managers to aid in the selection of cost-effective strategies for the management of terrorist and environmental risks. Also, facilities are being established for science-based exposures for measurement of firefighter equipment performance attributes essential to support informed fire service procurement decisions.

3.3 ACCOMPLISHMENTS OF WTC R&D PROGRAM

Prevention of Progressive Collapse

Buildings that are designed according to modern building codes are not expected to collapse during their service life period. They are designed typically to resist traditional governing vertical loads and lateral loads such as wind and earthquake. This situation is changing, however, due to an increase in deliberate terrorist attacks. In general, a terrorist attack may lead to failure of a small part of a building. When an initial local failure causes the loss of gravity load capacity in the structural frame, the failure spreads from story to story, which may lead to the total collapse of an entire building or a disproportionately large part of the building. This type of collapse is defined as “progressive collapse” (see Fig. 3–1). At present, U.S. building codes do not provide explicit provisions to enhance the resistance to progressive collapse. In terms of magnitude and probability of occurrence, the traditional vertical and lateral design loads are quantifiable. In contrast, terrorist loads are difficult to quantify as to size, location, and the nature of the loads. Terrorist attacks may include thermal, impact, or blast loads. Thus, in order to improve resistance to progressive collapse, U.S. building code developers have attempted to incorporate into the codes structural redundancies by introducing prescriptive requirements for “structural integrity.” Changes are needed in the way buildings are designed and constructed so that resistance to progressive collapse is provided explicitly. Following the 2002 National Workshop, NIST is working jointly with the Multihazard Mitigation Council of the National Institute of Building Sciences and industry experts to produce a “Best Practices Guide” for mitigating progressive collapse of buildings for design professionals. This document will be published in FY2004. Subsequent research efforts will focus on the development of tools to assist design professionals in the design of new buildings against progressive collapse and methods to enhance the resistance of existing buildings to progressive collapse.

Figure 3–1. Ronan Point Collapse in 1968.

Fire Safety Design and Retrofit of Structures

Current building design practice does not consider fire as a design condition to predict and evaluate structural performance in the presence of an uncontrolled fire. Instead, fire endurance ratings of building members, derived from standard fire endurance tests, are specified in building codes. At the present time, there is no accepted science-based set of verified tools to evaluate the fire performance of entire structures under realistic fire conditions. Thus, there is an urgent and critical need to develop and implement verified and improved standards, technology, and practices that explicitly consider structural fire loads in the design of new structures and the retrofit of existing structures. A workshop was held recently in cooperation with the Society of Fire Protection Engineers to assess current fire safety practice and existing codes and standards, and to identify research gaps for an improved fire safety design and retrofit approach. The workshop was attended by national and international fire safety experts and provided the technical basis for the development of a national R&D roadmap for Fire Safety Design and Retrofit of Structures.

In addition, an evaluation has been performed of state-of-the-art numerical tools, including ANSYS and SAFIR, to assess their suitability for use in analyzing performance of structures under the combined fire and mechanical loadings. The evaluation process of these analytical platforms, which are rarely used in practice for fire safety design, is ongoing and necessary due to the complexity of structural systems, loading conditions, boundary conditions, and the highly nonlinear nature of material and structural behaviors. The effect of high thermal loading on structural performance of tested concrete columns, WTC steel connections, members, and subassemblies was examined. To better inform the modeling effort, a series of large-scale tests were conducted of steel components in a fire environment (see Fig. 3–2 and Fig. 3–3). The tested components included steel rods, columns, and open-web steel joists that were either left bare or had sprayed-on fire protective insulation material of varying thickness. Test fires were generated using liquid hydrocarbon fuels to produce medium-soot fires and high-soot fires, and the tests were continued until the temperature at any steel surface approached approximately 600 °C.

Figure 3–2. Insulated steel trusses, steel rod and steel column inside the NIST large-scale fire laboratory.

Figure 3–3. View of fire compartment from air exhaust outlet several minutes after the start of a fire test. Note: the flame impingement on the steel trusses and bar.

Fire Resistant Steel

Structural steel loses strength at building fire temperatures, leading to the need for fireproofing (see Fig. 3–4). Fireproofing adds costs and can be damaged or removed from the structure in a blast situation or unanticipated impact. In contrast, a new class of fire-resistant steels is specifically designed to retain more of the design strength at high temperature. These steels are being produced in Japan and Europe, and are now in use, either with or without additional fire protection. The use of fire-resistant steels leads to cost savings and schedule benefits during construction when application of fire protection can be avoided, and enhanced performance when protection is applied in the case of damage to the insulation. Unfortunately, the benefits of fire-resistant steel are not adequately tested under the standard U.S. structural fire standards (ASTM E 119), and thus, are not currently used in the U.S.

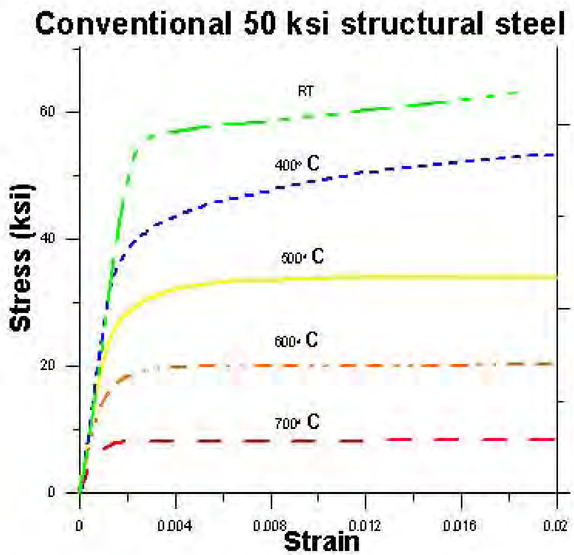

Figure 3–4. Stress-strain curves of a conventional structural steel, showing degradation of properties at high temperature.

A project has been initiated to ascertain which properties of steel are critical for efficient use of fire-resistant steels, such as high temperature strength and creep (see Fig. 3–5). Conventional steels and fire-resistant steels are being characterized to determine these critical properties. Our goals include both provision of accurate data on fire-resistant steels and development of quick and accurate tests for measuring relevant high temperature properties.

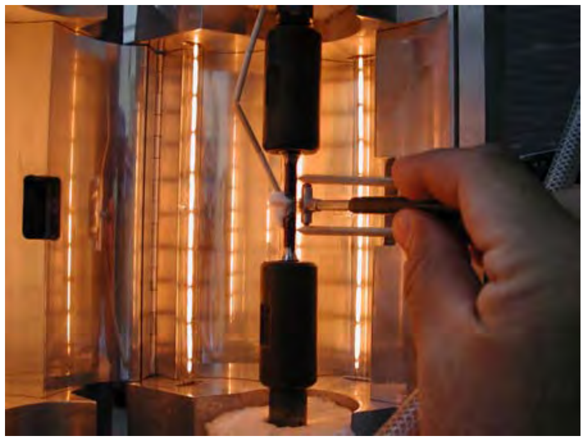

Figure 3–5. High temperature tensile tests to measure performance of structural steel at temperatures found in building fires.

Methodology for Fire Resistance Determination

Compartmentation is the cornerstone of limiting room-to-room and building-to-building fire spread. Standard fire resistance testing of wall/floor/ceiling assemblies provides an indicator of fire resistance and has proven valuable over time. However, these procedures have significant limitations that restrict their value for performance-based design and especially for high-risk occupancies. These limitations include: (a) standard time-temperature curves that may not be sufficient for all threats; (b) uniform heating, while many fires produce hot spots that may make the tests non-conservative; (c) single point thermal measurements; (d) pass/fail criteria, which make adaptation to other fire scenarios difficult; (e) documentation of the initial failure mode, but not the relative time to any successive modes, and (f) relative ratings, not absolute values.

Compartmentation is especially important in tall buildings, where the egress of numerous occupants can be a complex process, and barriers to the spread of flame keep the egress paths open, extend the time available for escape, and increase the safe time in places of refuge. For all these functions, it is necessary to know, in terms of real time, how long the interior partitions in a building will contain flames and smoke.

NIST has embarked on a course to provide such a methodology for inclusion in performance-based design of buildings. The research involves obtaining real-scale experimental data, modeling the behavior of partitions as they are driven to failure by fire, and developing recommendations for obtaining the input parameters from modifications of standard fire resistance tests such as ASTM E 119 and ISO 834. The initial work will focus on non-loadbearing walls of gypsum panels and steel studs, the most common interior construction in tall buildings. A continuing effort will extend the research to glass-panel walls.

The modeling effort will be done in three steps. First is a simple model for failure, beginning with crack initiation and propagation, continuing to the supporting structure, and finally to the fasteners and their failure points. This is now underway. The second component will be development of a detailed model of the partition materials to ascertain what additional data need to be obtained from the test method. The third component is development of a detailed model of a partition assembly for use by building design and engineering firms.

A series of real-scale compartment tests is providing information on the phenomenology of partition response and failure and also quantitative information to guide the model development (see Fig. 3–6). Various wall assemblies 2.44 m x 2.44 m were exposed to intense fires from the time of ignition to beyond flashover. Flux meters provided time histories of the energy incident on the walls. Thermocouples and infrared videos provided data on the transport of heat through the walls and on the progress toward perforation.

Figure 3–6. NIST measured the thermal behavior of gypsum/steel wall assemblies subjected to severe fire conditions.

Emergency Use of Elevators

This project is aimed at the development and implementation of protected elevators that can be used for fire department access and occupant egress during emergencies in tall buildings. The general strategy is to first incorporate into U.S. codes and standards a protected elevator system for fire department access. These are known in other countries as firefighter lifts, and there are existing requirements for these in at least 12 countries (as identified in a report by ISO TC178 on Elevators and Escalators). Once the U.S. fire services are satisfied that elevators are safe and reliable during fires, codes and standards would be changed to recognize protected elevators for occupant egress, secondary to, or integrated with, stairs.

The key technological advancement offered in the NIST strategy is the (new) concept of remote manual control. Here, the elevator system safety is monitored in real time by the fire alarm system and displayed on a standardized fire service interface developed jointly by NIST and the fire alarm industry, through the National Electrical Manufacturers Association (NEMA) and implemented in the 2002 edition of the

National Fire Alarm Code (NFPA72). This system addresses residual concerns held by the fire service and elevator industry even where such systems are utilized under existing codes and standards. The system might be further specified for accessible elevators required in U.S. and other building codes for access by people with disabilities but where the safety for use in egress during fires is still considered questionable.

Several technical papers have been written by NIST and presented at a recent international conference on Tall Buildings organized by International Council for Research and Innovation in Building and Construction (CIB) and Council on Tall Buildings and Urban Habitat (CTB&UH). NIST organized and chaired the speakers' session on emergency use of elevators, which included papers by two of the largest U.S. elevator companies (Otis and Kone). NIST also co-sponsored and presented papers at a workshop in March 2004, organized by American Society of Mechanical Engineers (ASME) and their A17 committee, who are responsible for the standard convening the safe use of elevators referenced in all U.S. building codes. NIST is working with the key representatives of the elevator industry and regulators represented on the A17 committee and with the product development engineers at Otis and Kone to implement the required technology and interfaces into their elevator controls, and on a novel approach to work out changes to the elevator control software for emergency operations protocols during fires. This approach would utilize NIST Virtual Cybernetic Building Testbed (VCBT) to allow numerous simulations of building fires to test the ability of the control software to adapt to conditions and to maintain safe operations.

Workshop on Building Occupant Movement During Fire Emergencies

NIST, in cooperation with the United Technologies Research Center, hosted a 2-day workshop focusing on the needed research on occupant behavior and movement during building emergencies. This workshop was motivated by a renewed interest in how buildings should be evacuated during fire emergencies and by the desire to provide a forum for the exchange of experiences among the fire and non-fire communities working on emergency egress. Sessions were held on codes and standards requirements for building evacuation, data needs for predictive building movement models, building movement strategies, and a roundtable discussion among selected government agencies. Participants included national and international experts in building occupant movement, representing the academic, consulting engineering, building products, and codes and standards communities. An outcome of the workshop will be the identification of areas where research is needed most to aid government agencies, industry and academic researchers in prioritization of their resource investments.

Guidelines and Technologies for Mitigation of Chemical, Biological and Radiological Aerosol Attacks

The increased attention to the potential vulnerability of buildings to airborne chemical, biological and radiological (CBR) agents has led to the need for better simulation tools to evaluate the transport and fate of such agents in buildings. NIST's longstanding expertise in airflow and contaminant transport modeling in buildings systems has been employed in many such analyses, and recently these capabilities have been extended via the release of version 2.1 of the CONTAM software. Among other enhancements, CONTAM is now able to use the output of exterior plume models as an input, such that outdoor contaminant concentrations from an exterior agent release can vary as a function of opening location on the building façade and time. This new capability allows users to link their exterior transport models to CONTAM and allow detailed analyses of the impact of an exterior release on indoor concentrations. In addition, CONTAM version 2.1 has improved models of particulate contaminants and has added fan and damper transients to the ability to simulate controls. The updated CONTAM model is now being used by an increasing number of researchers and practitioners in their evaluation of specific buildings and of technologies with the potential to increase building protection (see Fig. 3–7).

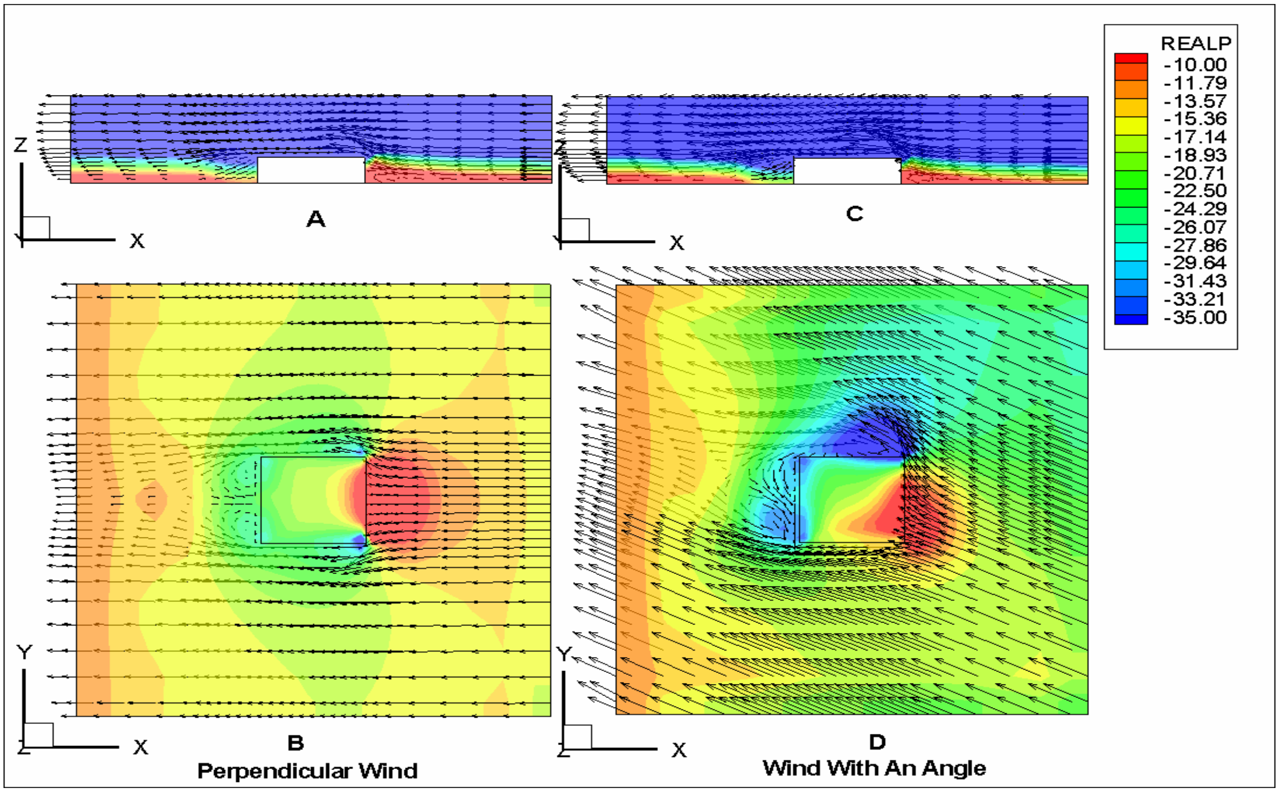

Figure 3–7. Exterior flow field as input to CONTAM model of building.

Cost-Effectiveness Tool for Managing Terrorist Risks in Constructed Facilities

Owners and managers of constructed facilities are faced with the task of responding to the potential for future terrorist attacks in a financially responsible manner. An economic tool is needed to direct limited resources to investments in mitigation strategies that will provide the most cost-effective reduction in personal injuries, financial losses, and damages to buildings, industrial facilities, and infrastructure.

The economic tool under development by NIST is a decision methodology, embedded in user-friendly, decision-support software, that helps building/facility owners and managers choose the most cost-effective mix of mitigation strategies. Three mitigation strategies are considered: (1) engineering alternatives; (2) management practices; and (3) financial mechanisms. The economic tool will provide decision makers with the basis for generating a risk mitigation plan that responds to the potential for future terrorist attacks in a financially responsible manner.

Early in 2002, NIST prepared a white paper outlining the tool development effort. NIST used the white paper to solicit stakeholder inputs, create opportunities for collaborative efforts, and form a technical working group of individual external subject matter experts. This has resulted in collaborative efforts with the Wharton Risk Management and Decision Processes Center, the Construction Industry Institute, ASTM International, and the U.S. Environmental Protection Agency. Safe Buildings Program. An expanded version of the white paper entitled “Economic Approaches to Homeland Security for Constructed Facilities” was delivered at the September 2002 CIB Meeting in Cincinnati as the invited Keynote Address.

Significant recent products include a prototype version of the software and a NIST Internal Report illustrating the methodology via a case study building. The prototype version of the software was completed and presented to the Steering Committee in September 2003. The prototype includes the software’s graphical user interface and linkage to database files and key reports. The beta version of the software is planned for completion in 2004; it will facilitate a variety of user-specified analyses. All analyses employed in the software will be consistent with ASTM standard practices. A case study report illustrating how to apply the life-cycle cost method (ASTM E 917) to a prototypical commercial building renovation project was published in NIST IR 7025. A subsequent technical report documenting the decision methodology is planned for publication in 2004.

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Chapter 4
UPDATE ON THE WTC DISSEMINATION AND TECHNICAL ASSISTANCE PROGRAM

4.1 OBJECTIVES OF THE WTC DISSEMINATION AND TECHNICAL ASSISTANCE PROGRAM (DTAP)

An industry-led dissemination and technical assistance program (DTAP) is the third part of the National Institute of Standards and Technology (NIST) response plan. The DTAP is designed to engage leaders of the construction and building community in assuring timely implementation of proposed changes to practices, standards, and codes. It also will provide practical guidance and tools to better prepare facility owners, contractors, architects, engineers, emergency responders, and regulatory authorities to respond to future disasters. The DTAP is an important component of the World Trade Center (WTC) Response Plan because it will facilitate the timely adoption and widespread use of proposed changes to practice, standards, and codes resulting from the WTC Investigation and the research and development (R&D) program.

4.2 BACKGROUND AND DESIRED OUTCOME OF DTAP

NIST is working closely with other government agencies; with the engineering and architecture professions; with the construction and manufacturing industries; with authorities having jurisdiction over building and fire code enforcement; and with national code-making organizations to target building safety issues (including those identified in the Federal Emergency Management Agency (FEMA) Building Performance Study) that (1) could be addressed by expeditious revision to national building codes, standards, and practices based upon current knowledge, and (2) would benefit from additional research. The mechanism by which this is being done is through contracts to the private sector. About $1 M has been spent to date since the start of the WTC Investigation. The objectives of each contract are described in the following section. Note that these contracts are over and above those in direct support of the Investigation and that the DTAP will continue beyond the completion of the WTC Investigation.

4.3 ACCOMPLISHMENTS OF THE DTAP

Contracts have been issued to the following organizations: the Civil Engineering Research Foundation (CERF), the National Institute of Building Sciences (NIBS), the National Conference of States on Building Codes & Standards, Inc. (NCSBCS), the Construction Industry Institute (CII), Integrated Manufacturing Technology (IMTI), the Society of Fire Protection Engineering (SFPE), and the Wharton School of Business.

National Workshop on Progressive Collapse (NIBS)

The Multihazard Mitigation Council of NIBS conducted a workshop in February 2004 that brought together national experts who have an interest in mitigating the threat of progressive collapse. The two-day workshop included presentation of ten white papers to frame the issues associated with mitigation of progressive collapse with respect to guidelines, codes, design of new buildings, and retrofit of existing buildings. The workshop also included breakout sessions on the topics of development of codes and guidelines, structural systems and analytical methods, and existing buildings. The breakout sessions resulted in identification of research needs and included estimation of costs to address each of those needs. The workshop also provided an opportunity for industry to review and comment on draft guidelines for retrofit of existing buildings and design of new buildings to resist progressive collapse.

Information Technology in the Building Regulatory Process (NCSBCS)

The National Alliance for Building Regulatory Reform in the Digital Age, a public-private partnership, will continue action on its agenda to stimulate economic recovery, enhance public safety, and increase the security of buildings. The agenda focuses on the use of information technology and the development and use by state and local jurisdictions of products, guidelines, model processes and procedures that enable jurisdictions to better respond to natural and manmade disasters and reduce the regulatory cost of construction by up to 60 percent. This effort includes support for the development and initial testing of a prototype secure database for first responders of as-built designs, evacuation plans, and other contact information.

National Alliance Interoperability Project (NCSBCS)

The objective of this project is to speed the development of technologies and requirements needed to advance the creation of a state-of-the-art integrated and interoperable building regulatory system by developing, in collaboration with New York City, an interoperability statement. This statement is to be included in their fall 2003 request for proposals for permitting software services and holding a national summit workshop with the software industry to identify common data requirements for currently available software component systems, common data/information formats, and recognized standards and best practices. The workshop will also identify actions that the industry and the National Alliance and state and local jurisdictions can take together to generate national standards for interoperable hardware and software for use in the building codes adoption, administration, and enforcement processes.

Strategies, Candidate Liaison Teams, and Actions to Conduct and Implement the NIST Response Plan (NIBS)

The NIST response plan and R&D program and the NIST outputs will be reviewed. Strategies will be suggested for achieving the private sector and state and local involvement needed to assure the likelihood of implementing each of the final products and for assessing their impacts in use. Synergistic benefits will be identified beyond those relevant to homeland security for which the outputs and strategies hold promise. The contractor will identify potential liaison teams and develop action plans for implementation of each product, or group of products. The liaison teams will include potential advocates, and those with serious concerns from public and private sector organizations most likely to be affected by the product.

Capital Projects Technology Roadmap (IMTI)

The baseline Capital Projects Technology Roadmap is being updated to address technological issues and solution paths related to homeland security and economic development. An outcome of this effort will be a detailed plan for the necessary R&D to support the deployment of technological solutions. Specific tasks include a workshop with FIATECH, follow-on meetings with technical experts, and review and teambuilding with top-level executives from the construction/capital facilities industry.

Benchmarking Homeland Security Construction Practices (CII)

The goal of this effort is to collect information on 9/11-driven security initiatives from industry leaders in the areas of chemical manufacturing, oil production and refining, natural gas processing and distribution, water treatment, and other critical industries needed to support the Nation's infrastructure. Information collected as part of a series of regional workshops and field site visits shall establish a basis for identifying best practices related to the security of capital facilities projects, and provide the basis for assessing the impacts of these practices on the key project outcomes of cost, schedule, and safety.

Best Practices Guidelines for the Mitigation of Progressive Collapse of Buildings (NIBS)

NIBS has begun to formulate a course of action that will increase the design and construction community's understanding of progressive collapse and provide practitioners with appropriate guidance. Draft guidelines have been completed and were reviewed by national experts during a February 2004 workshop. The draft is currently being revised, and a complete draft is planned for publication in September 2004. Following publication of the draft guidelines, a series of regional seminars is planned to educate practitioners and gather additional input that will be used to finalize the guidelines.

Workshop on Structural Fire Resistance (SFPE)

An international group of experts was convened in early October, 2003, to examine different technical aspects associated with structural fire resistance and to develop a detailed roadmap identifying research gaps to be filled to meet industry needs. Ten detailed white papers were presented by leading national and international experts on topics relevant to fire resistance of building structures. Workshop attendees supported the development of a best-practices manual for structural fire protection, including design and analysis tools, to add to the knowledge base and aid building officials in evaluating the adequacy of performance-based rather than “prescriptive” designs. The participants also developed a list of actions/research needs having the highest priority in developing best practices for fire design and retrofit of structures.

Accelerating Technologies/Systems for Fire Protection of Structural Steel in High Rise Buildings (CERF)

There are a number of objectives for this study. The first is to conduct a brief review of the types of materials and systems that are in use, in development or proposed for fire safety protection of structural steel in high-rise buildings. A second objective is to identify the performance requirements for such systems, including fire resistance, durability, impact and/or vibration resistance. A workshop was held in February 2004. In preparation for the workshop, white papers were developed on the topics cited above to frame the issues. Participants developed a prioritized set of recommendations to address technical and procedural/organizational issues.

Cost-Effective Risk Management (Wharton, CII)

The Wharton Risk Management and Decision Processes Center is expected to deliver in October 2004 a draft report covering (1) economic incentives for mitigation of consequences of extreme events (e.g., natural disasters and terrorism) and (2) procedures for estimating potential benefits and costs of alternative mitigation measures. The CII has developed a security rating index for industrial facilities. It is planned to be published by October 2004 in a report entitled *Best Practices for Project Security*.

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Appendix A
INTERIM REPORT ON THE ANALYSIS OF BUILDING AND FIRE CODES AND PRACTICES

A.1 INTRODUCTION

As stated in Chapter 2, one of the objectives of the National Institute of Standards and Technology (NIST) Investigation of the World Trade Center (WTC) disaster is to determine the procedures and practices that were used in the design, construction, operation, and maintenance of the WTC buildings. (For other objectives refer to http://wtc.nist.gov.) Since WTC 1, 2, and 7 were designed according to the New York City Building Code, it is important to understand how this code compared with contemporaneous building codes. This appendix summarizes the structural and fire provisions of the 1968 New York City Building Code and compares them with a number of building codes that existed when WTC 1, 2, and 7 were designed, as well as the 2001 edition of the New York City Building Code.

As a result of devastating fires in major cities of the U.S. such as Boston, New York, Chicago, and Baltimore in the late 1800s, the first model building code was developed by the fire insurance industry to minimize future fire losses. The National Board of Fire Underwriters (predecessor of the American Insurance Association) published the *National Building Code* in 1905. Subsequently, the Pacific Coast Building Officials Conference (predecessor of the International Conference of Building Officials) issued the *Uniform Building Code* (UBC) in 1927, the Southern Building Code Congress International Inc. published its *Southern Standard Building Code* (SBCCI) in 1946, and the Building Officials and Code Administrators, Inc. (BOCA) published the *Basic Building Code* in 1950. In the mid-1980s, the Basic Building Code was changed to the *BOCA National Building Code* (NBC). These regionally used model building codes were revised annually to incorporate developments in new materials and construction methods, and new additions were published every three years.

It should be pointed out that provisions in these model building codes establish minimum requirements to safeguard life, health and property and public welfare by means of regulations pertaining to the design, construction, and quality of materials, use and occupancy, and maintenance of buildings. When buildings are designed, constructed, and maintained according to building code requirements, they are considered to have met minimum requirements. While building code regulations address a number of objectives demanded by society, the primary objectives of building codes are structural stability and fire safety.

Before the issuance of the *International Building Code* (IBC) in 2000, which was published by the International Code Council (an amalgamation of the three regional code organizations), most local and state building codes in the U.S. were patterned after one of the three model building codes, UBC, SBCCI, or NBC. These model codes evolved in the mid-20th century to incorporate regional differences in construction materials and practices. When adopted by local jurisdictions, the model building code becomes a legal document and code provisions become mandatory laws. A number of major cities in the U.S. have developed their own building codes to meet their specific needs, such as San Francisco for earthquake resistant design and New York City for high-rise buildings design.

The National Fire Protection Association (NFPA) in the early 1900s initiated the development of a "life safety code" for safety of building occupants. This code, while not a building code, is frequently used as a supplement to the building codes. In 2002, NFPA also published a model building code known as the *NFPA Building Construction and Safety Code* (NFPA 5000).

The Port of New York Authority (PONYA) (whose name was changed to the Port Authority of New York and New Jersey [PANYNJ] in 1972) is an interstate agency that was established in 1921under a clause in the U.S. constitution that permits compacts between states. As an interstate agency, construction projects of the Port Authority are not required to comply with any building code. With respect to design and construction of the WTC towers, however, in 1963 the Port Authority instructed its consultants to prepare their designs of WTC 1 and WTC 2 to comply with the New York City Building Code (Levy 1963). Although it was not specifically stated in the letter to the architect, the 1938 edition of the Code was in effect at that time. In areas where the Code was not explicit or where technological advances made portions of the 1938 Code obsolete, the Port Authority directed the architect and consulting engineers to propose designs "based on acceptable engineering practice." The Port Authority also required the design professionals to inform the Planning Division of the WTC when such situations occurred. The Port Authority established a special WTC office that reviewed and approved plans, issued variances, and conducted inspections during construction instead of the city agencies and employees that would normally perform these duties.

The Port Authority further stated that all design concepts would be reviewed before the final design by the Chief Engineer of the Port Authority and by the appropriate New York City agencies. According to correspondence in 1975 from the architect-of-record for the WTC project, the New York City Building Department reviewed the design drawings of WTC 1 and WTC 2 in 1968 (Solomon 1975).

In 1965, the Port Authority instructed the architect and consulting engineers to revise their designs for WTC 1 and WTC 2 to comply with the second and third drafts of the New York City Building Code then being finalized and to undertake any design modifications necessary to comply with the new code provisions (Levy 1965). The new New York City Building Code (NYC BC 1968) was enacted by the City Council on October 22, 1968, approved by the Mayor on November 6, 1968, and became effective on December 6, 1968.

The Port Authority intended to lease space in WTC 1 and WTC 2 to tenants who would adapt their spaces to their own needs through a tenant alteration process. To maintain structural integrity and fire safety, the Port Authority issued a set of requirements for the alteration process. The first edition of the *Tenant Construction Review Manual* was issued in 1971, shortly after the first tenants occupied WTC 1 in December 1970 and before initial occupancy of WTC 2 in 1972. The manual contained the technical criteria to be used in planning alterations (architectural, mechanical, electrical, fire protection, and so forth) to Port Authority facilities. Included were applicable standards to be used by tenants and their agents and review criteria to be used by the Engineering Department of the Port Authority. Alteration designs were to be completed by registered design professionals, and at the completion of the work, as-built drawings were to be submitted to the Port Authority. The 1968 New York City Building Code was referenced, and specific code provisions were referenced in various checklists. The review manual was updated in 1979, 1984, 1990, and 1997, at which times changes that had been made to the New York City Building Code were incorporated. In 1998, the manual was replaced by the *Architectural and Structural Design Guidelines, Specifications, and Standard Details*, which dealt specifically with alterations to WTC 1 and WTC 2.

Unlike WTC 1 and WTC 2, which were developed and owned by the Port Authority, WTC 7 was developed on land owned by the Port Authority, but the building was owned by Seven World Trade Company and Silverstein Development Corporation, General Partner. It was designed and constructed as a "Tenant Alteration" project of the Port Authority. When WTC 7 was designed in the mid-1980s, the 1968 New York City Building Code with amendments was in effect. The Project Specifications for WTC 7 issued in 1984 required that the structural steel be designed in accordance with the then current New York City Building Code.

A.1.1 The New York City Building Code

The New York City Building Code is part of the Administrative Code of New York City. It is amended from time to time by Local Laws to improve safety requirements or to incorporate technological advances. New York City Council Members, at the request of any person or group, can introduce a bill to the Council for the purpose of amending the Building Code requirements. When passed by the Council and approved by the Mayor, the bill becomes a Local Law. Seventy-nine Local Laws were adopted between 1969 and 2002 that modified the 1968 Building Code. For example, Local Laws 5, 16, and 86 made important modifications to fire protection and life safety features of the 1968 Building Code.

To aid the implementation of and to clarify building code requirements, New York City issues "rules." These rules are initiated typically by City Government offices such as the Department of Buildings and the Department of Environment. These rules do not require enactment by the City Council, and new rules issued by the Building Commissioner can be put into effect expeditiously. The rules are part of the Building Code, and are required to be complied with for design, construction, and maintenance of buildings.

The 1968 New York City Building Code includes "Reference Standards." These include standard test methods published by the ASTM International, or design standards published by other consensus-based organizations. These reference standards may include modifications to the provisions in the published standards, or they may be stand-alone requirements developed by New York City.

A.1.2 Scope of Appendix

The 1968 New York City Building Code (NYCBC 1968) is compared with four other codes. They are: the 1964 New York State Building Construction Code (NYSBC 1964); the 1965 BOCA Basic Building Code (BOCA/BBC 1965); the 1967 Municipal Code of Chicago Relating to Buildings (MCC 1967); and the 2001 edition of the New York City Building Code (NYCBC 2001)

The 1964 New York State Building Construction Code was selected for comparison, as it would have been a governing building code outside the New York City limits. The 1965 BOCA Basic Building Code was selected, as it was typically adopted by local jurisdictions in the northeastern region of the U.S. The 1968 New York City Building Code is compared with the 1967 Municipal Code of Chicago to see whether there are any substantial differences in the structural and fire safety requirements of the two codes. In the late 1960s and early 1970s, several tall buildings were built in Chicago including the Sears Tower (110 stories) and the John Hancock Tower (100 stories). The 2001 edition of the New York City Building Code is compared with the 1968 version to examine the extent to which Local Laws have modified the code provisions, and in most cases, is only addressed in areas where changes have occurred between the two versions.

A provision by provision comparison was made between the 1968 New York City Building Code and these four codes. The code provisions that were compared are limited to the requirements related to structural stability, active and passive fire safety, and emergency egress. This appendix presents a summary of substantial differences noted in the comparison. This summary focuses on the following topics:

- Loads to be considered in the design of buildings;
- Requirements for materials, design, and construction;
- Fire protection requirements; and
- Egress requirements.

With respect to structural stability, no Local Law other than Local Law 17 (seismic provisions for new construction) has been adopted that modified the structural requirements of the 1968 New York City Building Code. Hence comparison between the structural requirements of the 1968 and 2001 New York City Building Code is not discussed here, with the exception of earthquake loads.

A.2 LOADS

A key aspect of any structural design is the loading that the structure is intended to support. Building codes provide minimum values for the different types of loads that are considered in typical building designs. The designer is permitted to use larger values for these loads, but is not permitted to use smaller values without approval by the building official. This section compares the specified loads in the codes that were compared. Similarities and differences are noted.

A.2.1 Dead Loads

Dead loads refer to loads that are permanently present in a building. They include, for example, the weight of the structural components, the weights of permanent partitions, the weights of floor and wall finishes, and the weights of service equipment that is part of the building (elevator equipment, plumbing, electrical, heating, air conditioning, and ventilation systems). Weights of the structural components are computed from the sizes of the members and the densities of the material, and codes typically provide default density values for different materials. The dead loads of partitions and walls are typically prescribed in terms of weight per unit area of wall, and the weight per unit length of wall or partition is determined from these prescribed values and the heights of the partitions or walls. Floor finishes and ceilings are typically specified in terms of a uniform load per unit area of floor or ceiling. Table A–1 gives examples of the minimum values of dead load prescribed in Referenced Standard RS 9-1 in the 1968 New York City Building Code and in Appendix J of the 1965 BOCA Basic Building Code. There are no corresponding provisions in the 1964 New York State Building Construction Code or the 1967 Municipal Code of Chicago. Typically, the designer is permitted to use weights based on available data that are greater than the specified minimum values, but the designer is not permitted to use lower values without approval of the Code Official.

Table A–1. Examples of dead loads given in NYC Building Code and BOCA Code.

	NYC	BOCA
Walls and Partitions		
Hollow concrete block – 8 in. thick	53 psf	50 psf
Clay tile, nonload bearing – 8 in. thick	34 psf	36 psf
Plaster partition, metal studs & lath, gypsum plaster both sides	18 psf	18 psf
Floor Finishes		
Resilient flooring	2 psf	2 psf
Hardwood flooring 7/8 in. thick (1 in. for BOCA)	4 psf	4 psf
Cement, 1 in. thick	12 psf	12 psf
Ceilings		
Suspended acoustical tile	2 psf	–
Suspended metal lath and gypsum plaster	9 psf	10 psf
Miscellaneous Materials		
Marble	168 pcf[a]	168 pcf
Concrete (normal density stone or gravel)	144 pcf	144 pcf
Reinforced concrete (normal density)	150 pcf	150 pcf

a. Note that the units in the 1968 New York City Building Code are given incorrectly as "psf."

According to the 1968 New York City Building Code, weights from service equipment (plumbing stacks, piping, heating, ventilating, and air conditioning (HVAC), etc.) are to be included in the dead load (C26-901.2)[1]. The weight of equipment that is part of the occupancy of a given area is to be considered as live load (see next section). The 1964 New York State Building Construction Code and the 1967 Municipal Code of Chicago do not have a provision in this regard. The 1965 BOCA Basic Building Code has a similar provision but without citing specific types of service equipment as in the New York City Code.

The 1968 New York City Building Code requires that weights of partitions be considered in two ways: (1) using line loads at locations shown on plans or (2) using the equivalent uniform load given in Reference Standard RS 9-1. The stipulated equivalent uniform load depends on the partition weight, for example, if a partition weighs 201 plf to 350 plf, it may be taken into account by designing for a uniform load of 20 psf. The uniform loading approach, however, is not permitted in certain situations for which actual partition weights must be used. Equivalent uniform loads must be used in areas where the locations of partitions are not shown on plans, or in areas where partitions can be relocated. The 1964 New York State Building Construction Code does not have a specific provision in this regard. The 1967 Municipal Code of Chicago prescribes a minimum partition load of 20 psf. The BOCA Basic Building Code requires consideration of the actual weight of the partitions or an equivalent uniform load of at least 20 psf.

[1] Refers to section number in the 1968 New York City Building Code.

A.2.2 Live Loads

Live loads are those resulting from the use and occupancy of the building, and include loads such as weights of occupants, furniture, filing cabinets, safes, mechanical equipment, and other items that the structure is called upon to support. Live loads are specified in terms of weight per unit of floor (or roof) area or in terms of concentrated loads. The values specified in codes are based largely on load survey data, experience, and judgment.

Floor Live Loads

In general, values of minimum uniformly distributed live loads specified in codes are organized on the basis of use or occupancy of spaces and there is no consistency in the names of these use categories. Thus comparison between codes is not straightforward. Table A–2 gives some examples of minimum uniformly distributed live loads for floors. It is seen that there is general agreement in the values of these selected minimum uniform live loads specified by the four codes.

Table A–2. Examples of minimum uniformly distributed live loads

	1968 NYC	1964 NYS	1967 Chicago	1965 BOCA
Office space	50 psf	50 psf	50 psf	50 psf
Restaurant	100 psf	100 psf	–	100 psf
Lobbies	100 psf	100 psf	100 psf	100 psf
Stairways	100 psf	100 psf	75–100 psf[a]	100 psf
Rest rooms	40 psf	60 psf	–	–
Hospital operating room	60 psf	60 psf	40 psf	60 psf
School classroom	40 psf	60 psf	40 psf	–

a. Depends on occupancy, for example, 75 psf for business, 100 psf for schools.

The codes also specify concentrated live loads placed so as to result in maximum stresses.

Live Load Reduction

There is a low likelihood that the full design floor live loads will be present on all floors of a building at the same time. In addition, the likelihood that the complete area on any one floor is loaded with the design load decreases as the floor area increases. To account for these factors, building codes permit "live-load reductions" in calculating the design loads for primary members (columns and girders) that support the roof and floors. The codes use several methods for live-load reduction (CTB&UH 1980):

1. *Percentage Method*—In this method, the live load reduction increases by a certain percentage with increasing numbers of floors, with a limit on the maximum value of reduction (typically 50 percent).

2. *Tributary Area Method*—The live load is reduced as the accumulated tributary area that is supported by a member is increased. The limiting value depends on the ratio of live load to dead load. The type of occupancy affects whether a reduction is permitted.

3. *Live Load to Dead Load Ratio*—The permitted reduction depends on the ratio of live load to dead load, provided that the dead load is greater than the live load.

The 1968 New York City Building Code uses the tributary area method and permits the percentage method as an alternative for columns, piers, and walls. The 1964 New York State Building Construction Code and the 1967 Municipal Code of Chicago use the tributary area method for beams and girders and the percentage method for columns and walls. The 1965 BOCA Basic Building Code uses a tributary method that is similar to the New York State Code.

Figure A–1 compares the reduced live load for columns, walls, and piers on the basis of the percentage method for three of the codes. It is seen that the permitted reductions are similar with the exception of the roof and top floor, where the 1968 New York City Building Code and the 1967 Municipal Code of Chicago are more conservative (less reduction permitted) than the 1964 New York State Building Construction Code.

Table A–3 compares the reduced live loads for beams and girders for the different codes. For the 1968 New York City Building Code, the reduced value of live load for a given contributory area depends on the live load to dead load ratio, with lower values permitted for lower live load to dead load ratios. For the 1964 New York State Building Construction Code and the 1965 BOCA Basic Building Code, the values shown in the table are based on a reduction factor of 0.08 %/ft^2. The lowest reduced value, however, is limited to 40 percent or

$$100\ \% \frac{3.33\frac{L}{D} - 1}{4.33\frac{L}{D}} \tag{A.1}$$

whichever is larger, where *L/D* is the live load to dead load ratio. As the ratio of live load to dead load increases, less live load reduction is permitted. A comparison of the values in Table A–3 shows that the 1967 Municipal Code of Chicago did not permit as large a reduction in live load for the same contributory area as the other codes.

A.2.3 Wind Load

The effect of wind on buildings is taken into account by the building codes by specifying a uniform pressure to be applied horizontally to a building. These pressures are to be applied in any direction so as to obtain the most critical loading condition.

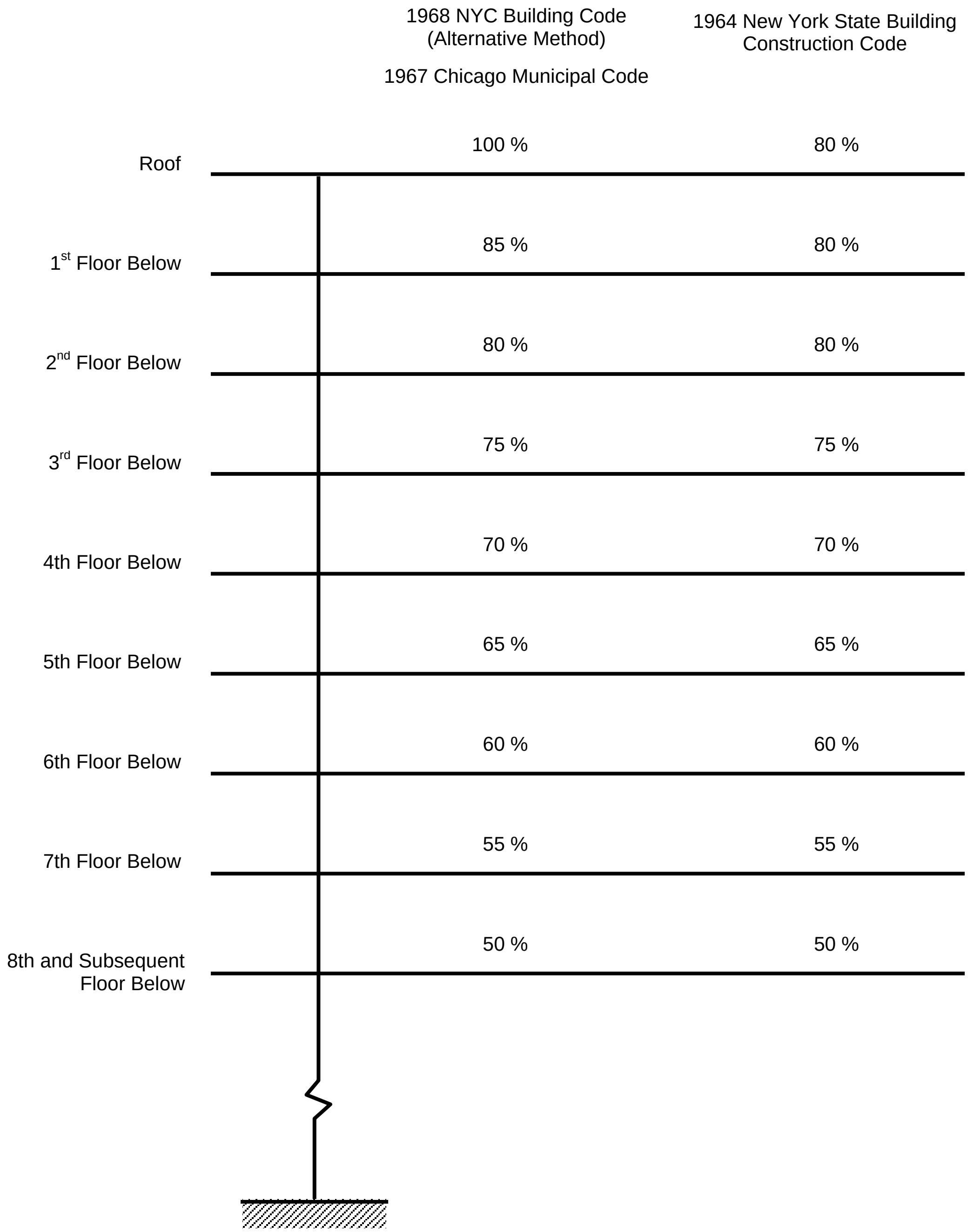

Figure A–1. Wind load pressure elevation.

Table A–3. Reduced live load for beams and girders.

Contributary Area (ft^2)	1968 NYC Building Code	1967 Chicago Municipal Code	1956 NY State and 1965 BOCA Codes
100 or less	100 %	100 %	100 %
100–149	100 %	95 %	100 %
150–199	80 % to 85 %[a]	95 %	84 % to 88 %[b]
200–299	80 % to 85 %[a]	90 %	76 % to 84 %[b]
300–449	60 % to 75 %[a]	85 %	64 % to 76 %[b]
450–599	50 % to 70 %[a]	85 %	52 % to 64 %[b]
600 and more	40 % to 65 %[a]	85 %	40 % to 52 %[b]

a. Permitted value depends on live load to dead load ratio; less reduction permitted with higher ratio.
b. The lowest value is limited to 40 percent or 100 percent (3.33 L/D –1)/(4.33 L/D), whichever is greater.

The pressure due to wind varies with the square of the wind speed, and wind speed increases with height. Thus building codes specify minimum design wind pressures that increase with elevation. The variations of pressure with height, however, are not the same among the building codes compared. Figure A–2 compares the specified wind pressure versus height relationships for the four codes that were compared. Several observations are noted:

- For buildings up to 600 ft in height, the 1964 New York State Building Construction Code prescribes the largest wind pressures.
- The 1967 Municipal Code of Chicago prescribes the lowest wind pressures for buildings up to 900 ft in height.
- The 1968 New York City Building Code and the 1965 BOCA Basic Building Code provide similar wind pressures for buildings up to 700 ft in height; for taller buildings the BOCA Code specifies larger pressures.

For a building height of 1,370 ft (the approximate heights of WTC 1 and WTC 2), the wind pressure distribution specified by the 1965 BOCA Basic Building Code would result in the largest shear force and overturning moment at the base of the building.

The 1968 New York City Building Code permits the designer to use wind pressure values, other than specified minimums, on the basis of wind tunnel tests and with approval of the building official. The following wording is provided in Section 6 of Reference Standard RS 9-5, “Minimum Design Wind Pressures.”

> In lieu of the design wind pressures established in sections 1 and 2 of this reference standard, and subject to review and approval of the commissioner, design wind pressures may be approximated from suitably conducted model tests. The tests shall be predicated on a basic wind velocity of 80 mph at the 30 ft level, and shall simulate and include all factors involved in considerations of wind pressure, including pressure and suction effects, shape factors, functional effects, gusts, and internal pressures and suctions.

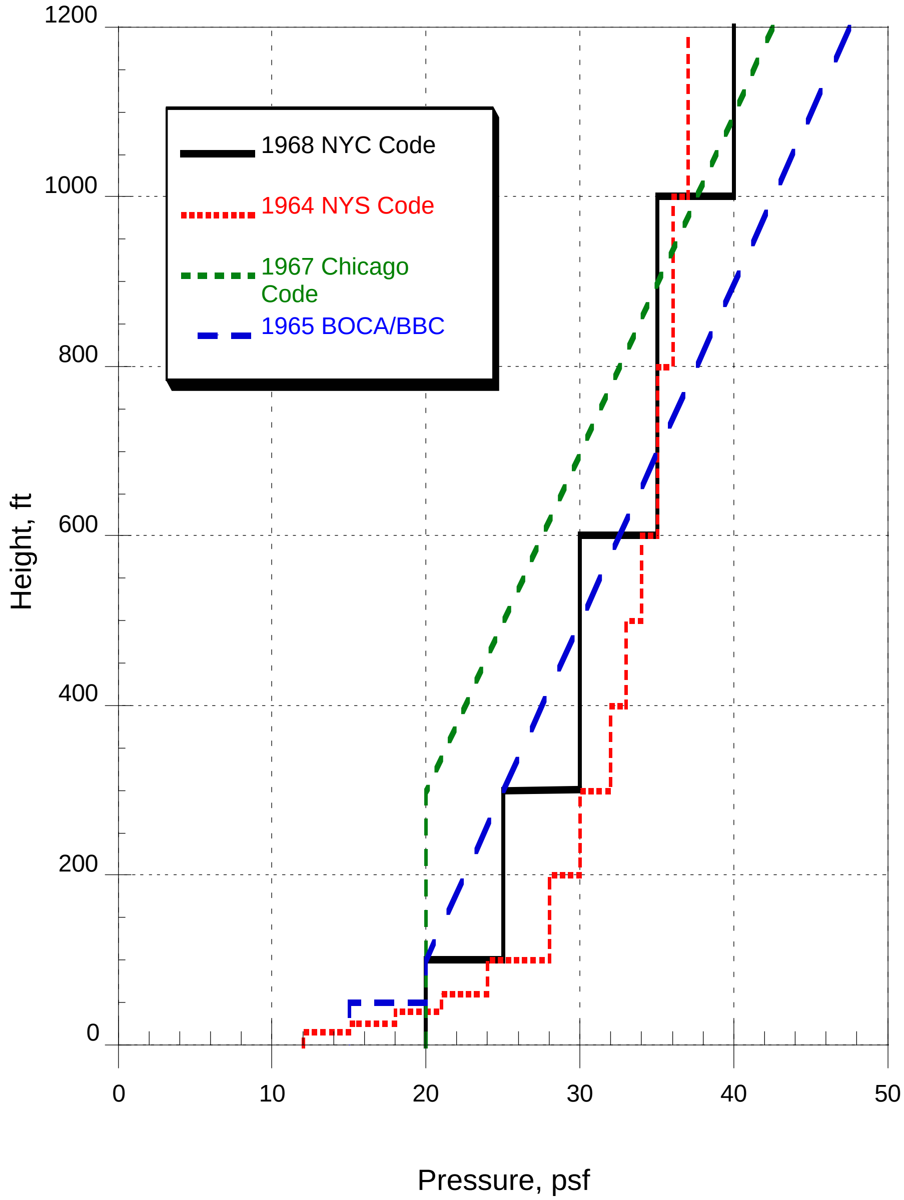

Figure A–2. Wind load pressure versus elevation.

Thus the 1968 New York City Building Code presumes a wind with a speed of 80 mph measured 30 ft above the ground. The 1964 New York State Building Construction Code, on the other hand, states that the prescribed wind loads "are based on a design wind speed of 75 mph at a height of 30 ft above grade level." Both the 1965 BOCA Basic Building Code and the 1967 Municipal Code of Chicago do not specify the design wind speed.

A.2.4 Earthquake Load

The 1968 New York City Building Code does not have provisions for earthquake loads. Among the contemporaneous codes that were compared, only the 1965 BOCA Basic Building Code has earthquake load provisions. These are contained in Appendix K-11 of that Code and were adapted from the 1962 edition of the Uniform Building Code.

The 2001 edition of the New York City Building Code contains seismic design provisions from the 1988 edition of the Uniform Building Code (UBC 1988), including the 1990 Accumulative Supplement. These provisions were put into effect in 1996 as a result of Local Law17 (passed in 1995). Significant modifications to the 1988 Uniform Building Code were made, and described in Reference Standard RS 9-6.

One modification is to the paragraph on "Minimum Seismic Design," which is modified to read:

> The following types of construction shall, at a minimum, be designed and constructed to resist the effects of seismic ground motions as provided in this section:
>
> > new structures on new foundations;
> >
> > new structures on existing foundations; and
> >
> > enlargements in and of themselves on new foundations.
>
> Buildings classified in New York City occupancy group J-3 and not more than three stories in height need not conform to the provisions of this section. The Commissioner may require that the following types of construction be designed and constructed to incorporate safety measures as necessary to provide safety against the effects of seismic ground motions at least equivalent to that provided in a structure to which the provisions of the section are applicable:
>
> > new buildings classified in occupancy group J-3 and which are three stories or less in height; and
> >
> > enlargements in and of themselves where the costs of such enlargement exceeds sixty percent of the value of the building.

In the subdivision on "Criteria Selection" the following paragraph was added:

> Seismic Zone. The seismic zone factor, Z, for buildings, structures and portions thereof in New York City shall be 0.15. The seismic zone factor is the effective zero period acceleration for S_1 type rock.

Other significant amendments include consideration of soil liquefaction that is not to be found in the Uniform Building Code.

A.2.5 Other Loads

The 1968 New York City Building Code has provisions dealing with types of loadings not considered in the other codes that were compared. Two examples are “thermal forces” and “shrinkage.” Paragraph C26-905.7 deals with thermal forces and includes the following requirement:

> …For exterior exposed frames, arches, or shells regardless of plan dimensions, the design shall provide for the forces and/or movements resulting from an assumed expansion and contraction corresponding to an increase or decrease in temperature of forty degrees F for concrete or masonry construction and sixty degrees F for metal construction….

Paragraph C26-905.8 on shrinkage includes the following requirement:

> The design of reinforced concrete components shall provide for the forces and/or movements resulting from shrinkage of the concrete in the amount of 0.0002 times the length between contraction joints for standard weight concrete, and 0.0003 times the length between contraction joints for lightweight concrete….

A.2.6 Distribution of Loads

Another topic that is contained only in the 1968 (and 2001) New York City Building Code is the distribution of loads, which is covered in Article 7 of Sub-chapter 9. Section C26-906.1 deals with vertical loads and states:

> “Distribution of vertical loads to supporting members shall be determined on the basis of a recognized method of elastic analysis or system of coefficients of approximation. Elastic or inelastic displacements of supports shall be considered and, for the distribution of dead loads, the modulus of elasticity of concrete or composition [composite] sections shall be reduced to consider plastic flow. Secondary effects, due to warping of the floors shall be considered.”

Section C26-906.2 deals with distribution of horizontal forces. Because this section provides important information in the design assumptions to be used in the design of high-rise buildings, several key sections are repeated here,

> The following provisions shall apply to superstructure framing only, and shall not apply to structures wherein horizontal loads are transmitted to the foundation by staycables, arches, non-rectangular frames, or by frames, trusses, or shear walls not oriented in vertical planes.
>
> **(a) Distribution of horizontal loads to vertical frames, trusses and shear walls. -** Horizontal loads on the superstructure shall be assumed to be distributed to vertical frames, trusses, and shear walls by floor and roof systems acting as horizontal diaphragms. The proportion of the total horizontal load to be resisted by any given vertical frame, truss, or shear wall shall be determined on the basis of relative rigidity, considering the eccentricity of the applied load with respect to the center of resistance of the frames, trusses, or shear walls. For vertical trusses, web deformations shall be considered in evaluating the rigidity.

(b) Distribution of horizontal loads within rigid frames of tier buildings. - (1) ASSUMPTIONS. - The distribution of horizontal loads within rigid frames of tier buildings may be determined on the basis of a recognized method of elastic analysis or, subject to limitations in paragraph two of this subdivision, may be predicated on one or more of the following simplifying assumptions:

a. Points of inflection in beams or columns are at their midspan and midheight, respectively. The story shear is distributed to the columns in proportion to their stiffnesses.

b. The change in length of columns due to axial effects of the horizontal loads may be neglected.

c. Vertical column loads due to horizontal forces are taken by the exterior columns only, or are resisted by the columns in proportion to the column distances from the neutral axis of the bent.

(2) LIMITATIONS. -

a. For buildings over 300 ft in height, the change in length of the columns, due to the effects of the horizontal loads, shall be evaluated or the framing proportioned to produce regular movements of the successive joints at each floor so that warping of the floor system may be neglected.

b. Simplifying assumptions used in design shall be subject to approval by the commissioner for any of the following conditions or circumstances:

1. For buildings over 300 ft in height or for buildings with a height-width ratio greater than five.
2. At two-story entrances or intermediate floors.
3. Where offsets in the building occur.
4. Where transfer columns occur.
5. In any similar circumstances of irregularities or discontinuities in the framing.

A.3 MATERIALS, DESIGN, AND CONSTRUCTION

Subchapter 10 of the 1968 New York City Building Code is entitled "Structural Work" and it provides minimum requirements for materials, design, and construction of all structural elements in buildings. This summary reviews design standards, materials, load combinations, and load tests.

A.3.1 Design Standards

Design standards refer to those documents that are used to proportion the structural elements and their connections. The principal structural materials in the WTC were concrete and steel, and the design standards are those produced by the American Concrete Institute (ACI) and the American Institute of Steel Construction (AISC). The ACI produces the standard known as ACI 318, *Building Code Requirements for Reinforced Concrete,*[2] and the AISC produced the following:

- Specification for the Design, Fabrication and Erection of Structural Steel for Buildings (AISC 1963)
- Specifications for Structural Steel Buildings–ASD and Plastic Design (AISC 1989)
- Load and Resistance Factor Design Specifications for Structural Steel Buildings (AISC 1993)

Table A–4 summarizes the concrete and steel design standards adopted by the codes that were compared. The 1964 New York State Building Construction Code is a performance standard and does not adopt design standards by reference. Thus at the time the WTC Towers were being designed, the other two codes (Chicago and BOCA) referenced the same concrete and steel design standards as the New York City code.

Table A–4. Design standards for concrete and steel.

	1968 NYC Code	2001 NYC Code	1967 Chicago Code	1965 BOCA Code
Concrete	ACI 318-63	ACI 318-89	ACI 318-63	ACI 318-63
Steel	AISC 1963	AISC 1989 AISC 1993	AISC 1963	AISC 1963

The 1963 edition of ACI 318 permits reinforced concrete members to be designed by either the working stress (or allowable stress) method or by the ultimate strength method. The 1963 AISC specification, on the other hand, is based on allowable stress design. The design method affects the loads used in the design calculations.

A.3.2 Load Combinations

The loads prescribed by the codes are used in different combinations to assess the governing design condition. The codes distinguish between sustained loads and loads of short duration or infrequent occurrence. For allowable stress design, two approaches are used for dealing with these two categories of loads, as will be discussed. For ultimate strength design, the prescribed loads are multiplied by specified load factors. In either case, the designer considers all applicable load combinations and determines the most critical condition, which becomes the design basis for a particular element.

[2] In 1999, the title was changed to *Building Code Requirements for Structural Concrete.*

Allowable Stress Design

The 1968 New York City Building Code defines two categories of loads:

1. Basic loads, which include dead load, live load, and reduced live load where applicable; and
2. Loads of infrequent occurrence, which include wind load, thermally induced load, shrinkage induced load, and unreduced live load where live load reduction is permitted.

Stresses in structural elements cannot exceed the allowable values specified in the referenced design standards under the following load combinations:

- The sum of the basic loads multiplied by a factor equal to 1;
- The factored sum of one or more basic loads and one load of infrequent occurrence, where the load factor equals 0.75;
- The factored sum of one or more basic loads plus two or more loads of infrequent occurrence, where the load factor equals 0.6.

The 2001 New York City Building Code is similar with the exception that it includes earthquake load as another load of infrequent occurrence.

The other Codes that were compared use a different approach for dealing with loads of infrequent occurrence. The 1964 New York State Building Construction Code states that stress due to wind load may be ignored if it less than 1/3 of the stress due to dead load plus imposed load excluding wind load. If the stress due to wind load exceeds this limit, the allowable stress for the material is permitted to be increased by 1/3.

The 1967 Municipal Code of Chicago uses a similar approach and states: "For combined stresses due to dead, live, and wind load, the allowable stresses in materials may be increased 1/3, provided the section thus determined is at least as strong as that required for dead and live load alone. Snow load shall be considered a live load."

The 1965 BOCA Basic Building Code is similar except that wind load or earthquake load is considered along with dead load and live load (including snow load). The same 1/3 increase in allowable stress is permitted under wind or earthquake load. The BOCA Code also explicitly states that wind load is permitted to be neglected if it results in stress less than 1/3 the stress due to dead load plus live load.

Ultimate Strength Design

In the 1960s, ultimate strength design was standardized only for reinforced concrete. As shown in Table A–4, the three codes from the 1960s referenced ACI 318-63, which include the following load combinations to establish the design loads (U) for structural members:

1. For structures where wind and earthquake loads may be neglected, U = 1.5 D + 1.8 L.

2. For structures where wind load must be included, U = 1.25 (D + L) or U = 0.9 D+ 1.1 W, whichever produces the most unfavorable condition for the member.

3. For structures where earthquake loading is included, E shall be substituted for W in condition 2.

4. In structures where effects of shrinkage and temperature are included, the effects of such items shall be considered on the same basis as the effects of dead load.

The 2001 New York City Building Code refers to ACI 318-99, which includes many more load combinations to be considered. These are as follows:

1. For all structures, U = 1.4 D + 1.7 L.

2. For structures where wind load must be included, U = 0.75[1.4 D + 1.7 L + 1.7 W)] or U = 0.9 D + 1.3 W, whichever produces the most unfavorable condition for the member.

3. For structures where resistance to earthquakes must be included, the load combinations of condition 2 are used with 1.1 E substituted for W.

4. For structures where resistance to earth pressure (H) must be included, U = 1.4 D + 1.7 L + 1.7 H or 0.9 D + 1.7 H, whichever produces the most unfavorable condition.

5. For structures where resistance to fluid pressure (F) must be included, U = 1.4 D + 1.7 L + 1.4 F or 0.9 D + 1.7 F, whichever produces the most unfavorable condition.

6. For structures where resistance shrinkage and temperature (T) must be included, U = 0.75 (1.4 D + 1.4 T + 1.7 L) > 1.4 (D + T).

7. For structures where resistance to impact must be taken into account, such effects shall be included with live load L.

A.3.3 Alteration of Existing Buildings

The compared codes have provisions to address code compliance when existing buildings are altered. The provisions of all codes, other than the 1964 New York State Building Construction Code, are broadly similar. In general, whether the altered building or only the alternations need to comply with code requirements depends on the ratio of alterations to the total building expressed either in terms of cost or dimensions. When the ratio is low, even the alterations may not have to be in compliance with the code, provided stipulated conditions are met. The 1964 New York State Building Construction Code, however, requires that any addition or alteration regardless of building value shall be made in conformity with that code. It is silent as to the structure being altered. Table A–5 summarizes code provisions related to alterations.

Table A–5. Compliance requirements for alterations.

Code	Provisions
1968 New York City Building Code	**Alterations exceeding 60 percent of building value (in any 12-month period):** The entire building shall be made to comply with the requirement of the code. **Alterations between 30 percent and 60 percent of building value:** Only those portions of the building altered shall be made to comply with the requirements of the code. **Alteration under 30 percent of building value:** Those portions altered may, at the option of the owner, be altered in accordance with the requirement of the code, or altered in compliance with their previously required condition and with the same or equivalent materials and equipment, provided the general safety and public welfare are not thereby endangered.
2001 New York City Building Code	Same as 1968 Code, except that wording for alterations **less than 30 percent** of building values was changed to: "those portions of the building altered may, at the option of the owner, be altered in accordance with the requirements of this code, or altered in compliance with the applicable laws in existence prior to December sixth, nineteen hundred sixty-eight, provided the general safety and public welfare are not thereby endangered." In addition, certain alterations are required to conform to the code regardless of magnitude or cost. These include, among others: Alterations to standpipes, sprinklers, or interior fire alarm and signal systems; Alterations to equipment for heating or storing water; Sprinkler, alarm protection, and emergency lighting requirements for places of assembly.
1964 New York State Building Construction Code	**Addition or alteration:** Any addition or alteration, regardless of cost, made to a building shall be made in conformity with applicable regulations of the code.
1967 Municipal Code of Chicago	**More than 50 percent:** Such buildings and structures shall be made to conform to all requirements of the code that are applicable to new buildings and structures. **25 percent to 50 percent:** All new constructions shall conform to the requirements of the code for new buildings or structures of like area, height and occupancy. **25 percent or less:** Certain exceptions can be made that allow the use of materials that conform to the strength and fire resistance for the materials with which the building is constructed. Otherwise, all new construction shall conform to the requirements of this code for a new building.
1965 BOCA Basic Building Code	"In the reconstruction, repair, extension or alteration of existing buildings, the allowable working stresses used in design shall be as follows: 1. **Building extended:** If altered by an extension in height or area, all existing structural parts affected by the addition shall be strengthened where necessary and all new structural parts shall be designed to meet the requirements for buildings hereafter erected. 2. **Building repaired:** When the uncovered structural parts are found unsound, such parts shall be made to conform to the requirements for buildings hereafter erected. 3. **Existing live load:** When an existing building heretofore approved is altered or repaired within the limitation prescribed in Section 106.3 (alteration under 50 percent) and 106.4 (alteration under 25 percent), the structure may be designed for the loads and stresses applicable at the time of erection, provided that public safety is not endangered. 4. **Posted live load:** May be posted for original approved live loads."

A.3.4 Materials and Methods of Construction

The compared codes have requirements for the materials and construction methods. Each code makes distinctions in materials and methods that depend on the nature of inspection and conformance with standards.

The 1968 New York City Building Code prescribes testing and inspection requirements for all materials, assemblies, forms, and methods of construction. A distinction is made between materials and methods subject to “controlled inspection” and those that are not subject to controlled inspection. Materials and methods subject to controlled inspections “shall be inspected and/or tested to verify compliance with code requirements.” In general, activities related to controlled inspections “shall be made and witnessed by or under the direct supervision of an architect or engineer retained by or on behalf of the owner or lessee, who shall be, or shall be acceptable to, the architect or engineer who prepared or supervised the preparation of the plans.” On the other hand, materials and methods not designated for controlled inspection “shall be inspected and/or tested to verify compliance with code requirements by the person superintending the use of the material or its incorporation into the work…”

The 1968 New York City Building Code provides tables to indicate which materials and methods are subject to controlled inspections and which are not. Table A–6 includes excerpts from the requirements for inspection of materials and assemblies. A footnote to the table in the code states that “All structural materials and assemblies subject to controlled inspection shall be tested and/or inspected at their place of manufacture and evidence of compliance with the provisions of this subchapter shall be provided as stipulated in sub-articles 1003.0 through 1011.0.” Table A–7 is an excerpt of the inspection requirements for methods of construction. A footnote to the companion table in the code states that “All construction operations designated for controlled inspection shall be inspected by the architect or engineer designated for controlled inspection during the performance of such operation.”

The 1968 New York City Building Code required that the installation of “sprayed-on fire protection” of structural members (except those encased in concrete) be subjected to controlled inspection requirements, as defined above. There were, however, no specific provisions on what testing was required.

The 1964 New York State Building Construction Code and the 1965 BOCA Basic Building Code make distinctions between “controlled” and “ordinary” materials in reference to establishing allowable stresses. For example BOCA defines “controlled materials” as those that are “certified by an accredited authoritative agency as meeting accepted engineering standards for quality.” Ordinary materials are those that do not conform to the requirements for controlled materials.

The 1967 Municipal Code of Chicago specifies that all materials and methods used in the design and construction of buildings shall be classified as “controlled materials” or “ordinary materials.” According to the Chicago Code, “controlled materials” means a building, structure, or part thereof, which has been designed or constructed under the following conditions: (a) All controlled materials must be selected or tested to meet the special strength, durability and fire resistance requirements upon which the design is based. (b) The design, preparation of working drawings, including details and connections, the checking and approval of all shop and field details and the inspection of the work during construction shall be under the supervision of a registered architect or structural engineer (Section 69-3.1).

Table A–6. Excerpts of inspection requirements for materials and assemblies in 1968 New York City Building Code.

Material	Elements Subject to Controlled Inspection	Elements Not Subject to Controlled Inspection
Steel	None	All structural elements and connections
Concrete	Materials for all structural elements proportioned on the basis of calculated stresses 70 percent or greater, of basic allowable stresses. See Section 1004.0 for specific requirements relating to “quality control of materials and batching.”	(1) All materials for all structural elements proportioned on the basis of calculated stresses less than 70 percent or greater of basic allowable values. (2) Concrete materials for: (a) Short span floor and roof construction proportioned as per section 1004.8. (b) Walls and footings for buildings in occupancy group J-3. (3) Metal reinforcement.

Table A–7. Excerpts of inspection requirements for methods of construction in 1968 New York City Building Code.

Material	Operations Subject to Controlled Inspection	Operations Not Subject to Controlled Inspection
Steel	(1) Welding operations and the tensioning of high strength bolts in connections where the calculated stresses in the welds or bolts are 50 percent or more of basic allowable values. (2) Connection of fittings to wire cables for suspended structures, except where cables together with their attached fittings are proof-loaded to not less than 50 percent of ultimate capacity.	(1) Welding operations and the tensioning of high strength bolts in connections where the calculated stresses in the welds or bolts are less than 50 percent of basic allowable values. (2) All other fabrication and erection operations not designated for controlled inspection.
Concrete	Except for those operations specifically designated in this table as not subject to controlled inspection, for all concrete, the operations described in section 1004.5(a) shall be subject to controlled inspection.”	(1) All operations relating to the constriction of members and assemblies (other than prestressed concrete) which involve the placement of a total of less than 50 cubic yards of concrete and wherein said concrete is used at levels of calculated stress 70 percent or less of basic allowable values. (2) placing and curing of concrete for all: (a) short span floor and roof construction as per section 1004.8. (b) Walls and footings for buildings in occupancy group J-3. (3) Size and location of reinforcement for walls and footings in occupancy group J-3. (4) All other operations not described in Sections C26-1004.5(a).

A.3.5 Stability, Bracing, and Secondary Stresses

The 1968 and 2001 New York City Building Code are the only codes of those compared that include provisions for *stability*, *bracing*, and *secondary stresses*. The provisions are the same in the two editions of the code. Stability, in this case, refers to resistance to sliding or overturning of the building on its foundation. The New York City Building Code requires a factor of safety of 1.5 against failure by sliding or overturning. The required stability is to be provided solely by the dead load plus any permanent anchorage that is provided. Bracing refers to lateral support to prevent buckling of compression members (columns and walls). The New York City Building Code requires that the bracing be proportioned to resist a load of at least 2 percent of the total design compression load in the braced member plus any transverse shear load on the bracing member. Secondary stresses refer to stresses associated with transverse deflection of a member. In trusses, for example, secondary stresses arise because joints are not true pins and some bending is introduced, which results in transverse displacements of the individual elements. The New York City Building Code requires that secondary stresses in trusses be considered in designing the size of the individual elements.

A.3.6 Deflection Limitations

All five codes contain limits on vertical deflections of floor and roof assemblies. Except for the New York City Building Codes (both the 1968 and 2001 versions), the deflection limits relate to crack formation of plastered building components. The deflection is limited 1/360 of the span for plastered members and 1/240 of the span for non-plastered members. The New York City Building Codes refer to the reference standards for deflection limits in addition to the 1/360 of the span limit. For concrete members, ACI 318-63 specifies limits for both short- and long-term deflections of beams and one-way slabs. For steel members, the 1963 AISC Specification specifies deflection limits to avoid damage to plastered ceilings and to limit deflections of flat roofs.

A.3.7 Load Tests

Building codes generally allow load tests to ascertain the adequacy of load carrying capacity of structural members. Specifically building codes allow load tests or tests of in-place materials:

- To verify adequacy of structural design for a member or an assembly,
- To verify adequacy of partially completed construction,
- To prequalify structural members or assemblies before used in service,
- To verify adequacy of questionable completed structure, and
- To determine concrete strength by means of core tests.

The New York City Building Codes have provisions to cover all five categories. The New York State Code has provisions for (1) and (4). The Chicago Municipal Code has provisions for (1), (4) and (5). The BOCA/Basic Building Code has provisions for (1) and (2).

A.4 FIRE SAFETY

A.4.1 Fire Protection

As mentioned in A.1, in 1965, the Port Authority directed that the design of the WTC towers be updated to conform to the second and third drafts of the 1968 New York City Building Code then under development. However, since it was not known which proposals in the second and third drafts would be adopted into the final 1968 code, the strategy was to identify any proposed design provision that differed from the 1938 code requirement as a variance to be specifically approved by the Port Authority (Kyle 1966a and Kyle 1966b). The Port Authority established the World Trade Department, a special Port Authority office, to review and approve plans, to issue variances, and to conduct inspections during construction.

The 1968 New York City Building Code contained detailed fire safety provisions on a number of topics not addressed in the other codes of the time, most of which appeared in these other codes at later times. For example, while the contemporaneous codes contained requirements for flame spread ratings for interior finish materials, the 1968 New York City Building Code also included requirements for “smoke developed" ratings that did not appear in other codes until later.

As stated in Section A.1.1, the New York City Building Code is amended by “Local Laws” (LL) and refined or interpreted by administrative orders issued by the Building Commissioner. While there were some 79 Local Laws adopted between 1969 and 2002 that modified the 1968 code, those that contained significant modifications to fire protection and life safety requirements include LL54/1970, LL5/1973, LL26/1975, LL55/1976, LL33/1978, LL41/1978, LL84/1979, LL86/1979, LL16/1984 and LL16/1987. Of particular importance in this group are LL5/1973 (and LL86/1979 that changed the compliance dates for LL5/1973) and LL16/1984 because some of their provisions applied retroactively to existing office buildings.

A.4.2 General Code Provisions for Fire Safety

Fire safety of building construction is generally regulated through limits placed on the height and the area per floor as a function of the type and degree of fire resistance of materials used in the structural elements. These material characteristics are categorized as *types of construction*, e.g., Type I through V, and the associated limits are contained in “*heights and areas*” tables that are a cornerstone of most (prescriptive) building codes, worldwide.

The intent of building height limits is to restrict taller buildings to non-combustible structural members with the greatest fire resistance (as measured in the ASTM E 119 test method). The primary concern with combustible structural members is that they can become ignited by an exposing fire and can continue to burn (often in concealed spaces) even after the exposing fire has been extinguished, leading to collapse.

The other important height factor is the definition of a high-rise building. This is generally based on the height above which fire department ladders will not reach, requiring that fires be fought from inside. An interior attack is limited to hand-held hoses supplied from standpipes and working from interior stairways. Traditionally, high-rise buildings have been defined as those that exceed 75 ft or 6 stories above grade in height, but some newer codes increase this height to 100 ft, as modern fire department ladders are longer.

The intent of area limits is generally to limit property risk and to limit the size (area involved on any floor) of the fire to that which can be dealt with by the fire department with the number of people and equipment typical of an initial response.

A.4.3 Occupancy Classification

The building codes define categories of occupancy (which may have more than one sub-class). The group designations vary in different codes, the ones presented here are those used in the New York City Building Code. These are:

- High Hazard (Group A)
- Storage (Groups B-1 and B-2)
- Mercantile (Group C)
- Industrial (Group D-1 and D-2)
- Business (Group E)
- General Assembly (Group F-1 through F-4)
- Educational (Group G)
- Institutional (Groups H-1 and H-2)
- Residential (Groups J-1 through J-3)
- Miscellaneous (Group K)

Building codes use occupancy as a surrogate for risk factors that determine the level of performance needed. For example, occupancy is determined by a combination of factors such as types and quantity of combustible contents, common ignition sources, and typical occupant characteristics. Business occupancies (which includes office buildings) are considered among the lowest risk because they typically contain grades of furniture that constitute relatively low combustible loads, few ignition sources, and a population that is predominately adult, in good physical and mental condition (e.g., not using alcohol), and not sleeping. The most risky occupancies are High Hazard, in which are found highly flammable, toxic, or explosive materials, and Institutional (e.g., hospitals and prisons) in which occupants are likely to be incapable of unassisted egress.

A.4.4 Construction Classification

The model building codes classify building constructions into different "Types." Although there are some variations in categories, they are reasonably consistent.[3] The main categories are Type 1 (fire resistive), Type 2 (non-combustible), Type 3 (combustible), Type 4 (heavy timber) and Type 5 (ordinary).

Types 1 and 2 are constructed with non-combustible exterior and interior bearing walls and columns. Fire resistance ratings (see A.4.5) are greatest for Type 1, and Type 2 is any (non-combustible) construction not meeting Type I requirements. Type 3 is where exterior bearing walls are non-combustible and interior bearing walls and some columns may employ approved combustible materials. Type 4 is known as *heavy timber*, which utilizes large, solid cross section wooden members such as in post and beam construction. Type 5 is traditional wood frame construction. Common non-combustible structural elements use steel or reinforced concrete. Combustible structural elements are usually made of solid- or engineered-wood and laminates.

Combustibility of the materials in a structural element is determined in ASTM E 136 in which the material is placed in a furnace at 750 ºC (1,380 °F), which is a "typical" fire temperature. Some minor surface burning (e.g., from paint or coatings) is allowed in the first 30 s but there cannot be any significant energy release as indicated by more than 30 ºC (54 ºF) increase in the furnace temperature, and the test specimen cannot lose more than half its initial mass. Materials that pass are designated non-combustible and the rest are combustible.

Within each construction type, there are several sub-categories determined by the fire resistance ratings of the columns, beams, and floor supports. In some codes these sub-categories are identified by letters following the type (e.g., 1B or 3A) or by a set of three numbers that represent the fire resistance required (in hours) of the columns, beams, and floors, respectively (e.g., Type 1 [3,3,2]).

For unsprinklered office buildings, the following construction classes are permitted in the five building codes reviewed.

- Type 1A and 1B—NYC BC 68, NYS BC 64, BOCA/BBC 65 (Unlimited height)
- Type 1A, 1B, 1C, 1D—NYC BC 01 (Height limited to 75 ft)
- Type 1A only—Chicago BC 67 (Unlimited height)

It is noted that the 1938 New York City Building Code did not include Type 1B construction for office occupancies. The reasons for the inclusion of Type 1B construction for office occupancies into the 1968 New York City Building Code are not recorded (recordkeeping in the codes and standards development process was very poor prior to the Hydrolevel vs. ASME Supreme Court decision in 1982). The codes then and now tend to follow each other as champions of changes to one code usually try to change all of the codes. The 1950 edition of the Basic Building Code (BOCA) included a Type 1B construction class with unlimited height and area for business and low hazard storage occupancies without sprinklers. Among other model codes, the Standard Building Code (1946-47 edition, SBCCI) had a Type 2

[3] Construction type definitions varied among the model codes until an effort was expended in the 1970s by the Board for the Coordination of the Model Codes (BCMC) to eliminate unnecessary differences.

construction similar to Type 1B for business occupancies and buildings more than 80 ft in height, the National Building Code (1934 edition, NBFU) had a semi-fireproof similar to Type 1B for buildings above 75 ft, and the Uniform Building Code (1927 edition, ICBO) had a Type 2 similar to Type 1B for buildings above 75 ft.

The Basic Building Code (BOCA) would be expected to have the strongest influence on New York City since BOCA was the regional code used in the Northeast U.S. This may be why Type 1B construction was included in the 1968 New York City Building Code.

Mandatory sprinkler requirement for new high-rise buildings was first introduced in the New York City Building Code in 1984 (by Local Law 16), in BOCA in 1984, and in the Chicago building code (which allows a compartmentation alternative) in 1975. Before Local Law 16 was adopted, the 1968 New York City Building Code permitted Type 1A, 1B, 1C, and 1D construction for sprinklered office buildings of unlimited height. In the 2001 New York City Building Code, the minimum permitted construction classification for office buildings of unlimited height is Type 1C.

A.4.5 Fire Resistance of Structural Elements

The structural elements of a building are protected against failure in fire for a specified period as determined in the ASTM E 119 test. The intent of the fire rating requirements is for the structure as a minimum to withstand design loads (including fire) without local structural collapse until occupants can escape and the fire service can complete search and rescue operations.

Fire resistance requirements in the building codes are greatest for structural members that are essential to the stability of the building as a whole. These include columns and other major gravity load carrying members that connect directly to columns such as girders and trusses.

For various construction classes, the building codes specify different fire resistance ratings. The building codes reviewed specify fire resistance ratings for high-rise office occupancies as follows:

- Type 1A
  - Columns: 4 h (supporting more than one floor)
  - Beams: 3 h (floor construction)
- Type 1B
  - Columns: 3 h (supporting more than one floor)
  - Beams: 2 h (floor construction).
- Type 1C (for sprinklered buildings only)
  - Columns: 2 h (supporting more than one floor)
  - Beams: 1 ½ h (floor construction).

The choice among permitted construction classes for a particular building is made by the architect and/or the owner. Thus, an unsprinklered high-rise office building that was designed according to the 1968 version of the New York City Building Code could follow either Type 1A or 1B, and if designed subsequent to the passage of Local Law 16/1984 a high-rise office building would have to be sprinklered but it could follow Type 1C as a minimum classification. Similar reductions in the minimum required fire resistance ratings for sprinklered buildings are found in all national model building codes over this period as requirements for fire sprinklers, especially in high-rise buildings, have become common.

Type 1B, and eventually Type 1C, construction was permitted for high-rise office occupancies because this occupancy is considered low risk. Most other use groups in high-rise buildings were restricted to Type 1A, which is the construction type with the maximum structural fire protection defined in these codes.

Compartmentation and Sprinklers

Section 6 of Local Law 5 adopted by New York City in 1973, required the subdivision of unsprinklered space in new office occupancies and in existing offices over 100 ft in height by fire rated partitions. Local Law 5 was challenged in the courts and was eventually upheld, although the original compliance dates were amended by Local Law 86 (1979) so that full compliance was required by February 7, 1988.

After the passage of Local Law 5, the Port Authority implemented a program to retrofit sprinklers and to offer tenants the option of sprinklering or compartmentation consistent with Local Law 5 provisions. Sprinklering of WTC 1 and WTC 2 was undertaken in three phases: Phase 1 was the sprinklering of below grade spaces completed with the original construction. Phase 2 was begun after Local Law 5 was adopted and included the installation of sprinkler risers and other infrastructure, and the installation of sprinklers in corridors, storage rooms, lobbies, and smaller tenant spaces for tenants not selecting the compartmentation option. Phase 3 involved sprinklering the remaining tenant spaces, initially as tenants changed, and later on negotiated schedules. This process was underway when in 1984 Local Law 16 was adopted, which required sprinklers in new high-rise buildings including offices. Thus all floor spaces by February 8, 1988, had to either be subdivided in accordance with the compartmentation requirement or sprinklered. A 1997 report states that there were four floors and the sky lobbies (all in WTC 1) left to be sprinklered, and that the installation of sprinklers at these floors was underway (Coty 1997). In an October 1999 report, it is stated that sprinklering of the tenant floors was completed and sprinklering of the sky lobbies was "currently underway" (PANYNJ 1999).

Summary

Table A–8 summarizes key fire safety requirements for business occupancy in high-rise buildings (greater than 75 ft) as stipulated in the codes that were compared. In addition, the provisions from the 1938 New York City Building Code are provided for comparison. It is seen that, overall, the 1968 New York City Building Code was in accord with contemporaneous codes. Exceptions are the permitted construction classes and minimum fire ratings, which were more restrictive in the 1967 Municipal Code of Chicago.

Table A–8. Summary of fire safety provisions for business occupancy in high-rise buildings (> 75 ft).

	1938 NYC	1968 NYC	2001 NYC	1964 NYS	1967 MCC	1965 BOCA
Detection	No requirement	Smoke detectors to shut down HVAC fans to prevent smoke recirculation	Class E fire alarm system with voice commu-nication	Fire alarm system required; fire detection or sprinkler system is alternative	Not required	Fire alarm system required if not sprinklered
Suppression (Sprinklers)	No Requirement	Below grade only	Required if gross floor area >100,000 ft^2	Below grade only	Not required	Not required until 1984
Permitted Construction Class	1A	1A, 1B	1A, 1B, 1C with sprinklers	1A, 1B	1A	1A, 1B
Fire Separation (Compart-mentation)	3 h shaft enclosures; 1 h tenant separations (demising walls)	2 h shaft enclosures; 1 h tenant separations	2 h shaft enclosures; 1 h tenant separations; unsprinklered requires compart-mentation if >7,500 ft^2 or >15,000 ft^2 with smoke detectors	2 h shaft enclosures; 1 hr tenant separations	2 h hoistways; 1 h shaft enclosures; 2 h tenant separations every 10,000 ft^2	2 h shaft enclosures; 3/4 h tenant separations
Minimum fire resistance ratings:						
Ext. and int. bearing walls and columns supporting >1 floor	4 h	3 h	2 h	3 h	4 h	3 h
Floors including beams	3 h	2 h	1 ½ h	2 h	3 h	2 h

A.4.6 Means of Egress

The basic concept of occupant egress implemented in building codes involves the provision of a properly designed *means of egress* that is continuous and unobstructed from any point in the building to the outside. Proper design includes the width of the spaces and doors, direction of door swing, lighting and marking, protection from the fire and its effects, and geometry of stairs or ramps. Limits on travel distances to reach a means of egress and on common paths of travel, dead ends, and the provision of alternative means of egress if the primary path is blocked by fire are also basic concepts of egress design.

The means of egress described in building codes consists of three parts. The *exit access* is a corridor, aisle, balcony, gallery, room, porch, or portion of a roof over which an occupant must travel to reach the exit. The *exit* is a door leading to the outside or through a protected passageway to the outside, a smoke-proof tower, protected stairway, exit passageway, enclosed ramp, escalator, or moving walkway within a building. The *exit discharge* is the door to the outside, although some codes allow not more than half the exits to discharge onto a floor with an unobstructed path to the outside, and which is protected by sprinklers and a 2-h fire resistance separation from floors below.

Another concept found in the context of Means of Egress systems is that of *defense in place*. This is normally associated with occupancies where the occupants cannot escape such as hospitals or prisons, but may also be applied to refuge areas (sometimes called areas of rescue assistance) where people with disabilities await assistance or to areas in which occupants are being held temporarily while they await their turn to evacuate in a phased evacuation. Defense in place usually involves providing some protection against exposure to fire, heat, and smoke for the time needed to move these people to a safe place.

Egress System Design

The objective of the egress system design is to allow unimpeded evacuation of the building population without exposure to fire or smoke. Prescriptive building code regulations address this by specifying a population density (people per unit floor area) for each building use group, called the "occupant load factor." When multiplied by the floor area, the *occupant load* is obtained on which the egress system design is based (unless there is reason to believe that the actual load will be greater or the owner desires a greater allowance).

The means of egress is then designed to accommodate that occupant load on the basis of an egress width per occupant served, also specified in the building code. Values are specified for stairs and for other egress components, sprinklered and unsprinklered, and with special values for high hazard and institutional building occupancies to allow for higher egress speeds (high hazard) and greater number of wheelchairs or evacuation in patient beds (institutional), respectively.

The width of the egress system at each floor is sized to accommodate the number of occupants on that floor only. There is an additional requirement that the egress system width cannot become narrower in the direction of egress travel and beyond any convergence of two or more egress systems the capacity cannot be less than the sum of the capacities. These requirements are intended to account for the accumulation of flows from multiple floors.

For very tall buildings, it was recognized that the accumulating flow from a large number of floors would result in congestion in the stairways and a reduction of flow speeds. Widening stairways to increase the capacity has an economic consequence. Thus the concept of phased evacuation was developed where occupants are evacuated first from the three floors closest to the fire, while others wait their turn. Such systems require a voice communication system to manage the process by voice messages from a fire command center staffed by the fire service, and (e.g., in New York City) fire wardens on each floor directing the flow.

Prescriptive Egress Specifications

Traditional (prescriptive) building codes specify the design of egress systems by first, estimating the number of occupants in an area to be evacuated, second, determining the (combined) width of the exit system needed for that number of occupants, and third, dividing that width among the number of exits needed to achieve the travel distance limits. These codes further establish some minimum requirements for the number and width of exits and exit components. These minimum requirements seem to derive from a 1935 National Bureau of Standards (now NIST) publication titled “Design and Construction of Building Exits” (NBS 1935). A survey of buildings showed that the nominal 44-in. (2 units of width) exit stair was in common use, and a majority of buildings provided two such stairs. This report recommended that these be adopted as minimum requirements in building codes. This report also recommended the values for occupant load and capacity allowances discussed below.

For the most part, the egress design factors found in all of the model building codes are identical. In the IBC and NFPA 5000 model codes, design occupant densities (loads) range from 500 ft^2 (gross) per occupant (aircraft hangers, warehouses) to 5 ft^2 (net) per occupant (assembly, standing space). Common values are 100 ft^2 (gross) per occupant (business, industrial) or 200 ft^2 (gross) per occupant (residential). By multiplying occupant loads by estimated floor area, the number of people to be evacuated is obtained. These same values have appeared in the building codes for a long time.

The *means of egress* provisions in the (1968 through 2001) New York City Building Code use the “unit width” method of computing egress capacity. This method was previously found in all the model codes but was changed to the “inches per person” method in the late 1980s (e.g., the NFPA 101 Life Safety Code changed to the “inches per person” method in its 1988 edition). The major reason for the change was that the capacity of elements of the egress system was measured in 22 in. increments (called a unit of exit width) with remaining fractions only credited in half units (12 in. or more). Thus the “inches per person” method allows egress credit for exit elements that may have non-standard dimensions. The New York City Building Code (1968 through 2001) requires at least two exits per floor for business occupancy (Group E) in buildings taller than 60 ft and it permits 80 people per (22 in.) unit of exit width and 60 people per unit width on stairs. These are identical to the values found in the contemporaneous codes (1965 BOCA and 1968 NFPA 101) reviewed. For example, for a floor space of 30,000 ft^2, 5 unit widths of stairs would be needed. Thus three 44 in. wide stairs (the minimum permitted width) would meet the New York City Building Code requirement. New York City further permits scissor stairs (an arrangement where two stairs are intertwined in a single shaft) with openings located at least 15 ft apart (C26.602.3), which are prohibited in most other building codes.

Both the IBC 2000 and NFPA 5000 model codes, as well as most building codes based on them (but not 2001 NYC), now determine the capacity of the egress system by specifying the width per person for egress system components. With the exception of hazardous and health care occupancies, both the IBC (without sprinkler protection) and the NFPA 5000 Code (sprinklered or not) specify the same egress system width of 0.3 in. per occupant in stairways and 0.2 in. per occupant elsewhere. The IBC reduces egress capacity where sprinklered to 0.2 in. per occupant in stairs and 0.15 in. elsewhere. The egress capacity of the exit system is the smallest capacity of any component. For example, a 34 in. (clear width) door leading into a 44 in. (clear width) stair have capacities of 170 and 147 occupants, respectively. Thus the exit capacity is the smaller of the two, or 147 occupants. The minimum number of exits specified in both model codes is two for populations up to 500, three from 501 to 1,000, and four if over 1,000.

Finally, building codes specify maximum travel distances to an exit by occupancy and presence of sprinklers, and the same values are found in all the codes, including New York City. For example, in the 1968 New York City Building Code, the maximum travel to an exit for business occupancy is 200 ft for unsprinklered construction and 300 ft for sprinklered construction. The IBC 2000 code specifies 200 ft (unsprinklered) and 250 ft (sprinklered) for most occupancies, except for business which is allowed 300 ft if sprinklered. The NFPA 5000 code specifies travel distances without sprinklers of 100 ft (for hotels, apartments, and mercantile), 50 ft (for health care and educational), or 200 ft (for business, industrial, and assembly). When fully sprinklered, these increase to 200 ft (for hotel, apartments, educational), 250 ft (for mercantile, industrial, assembly) and 300 ft (business). While most buildings will require two or more exits, the travel distance requirement only applies to the distance from any point to the closest (single) exit. The distance to any other exit(s) is unregulated.

Elevators

Currently there are no U.S. building codes that permit elevators to be used as a means of occupant egress in emergencies, and ASME A17.1 (ASME 2000) requires signs at all elevators warning that they shall not be used in fires. There are some recent exceptions to this, but these are limited to special cases. For example, NFPA 5000 permits protected elevators as a secondary means of egress for air traffic control towers and the City of Las Vegas accepted elevators as a primary means of occupant egress from Stratosphere Tower based on a performance-based design (Bukowski 2003).

U.S. building codes (including New York City) require *accessible elevators* as part of a means of egress that may be used by the fire service to evacuate people with disabilities. These elevators must comply with the emergency operation requirements of ASME A17.1 (Phase II emergency operation by the fire service), be provided with emergency power, be accessible from an area of refuge or a horizontal exit (unless the building is fully sprinklered), and operate in a smoke protected hoistway. Phase II operation involves the use of an elevator by a firefighter for fire service access or for rescue of people with disabilities performed under manual control (with the use of a special key).

A.5 PRELIMINARY FINDINGS

The 1968 New York City Building Code was compared with contemporaneous building codes to determine whether there were significant differences in code provisions. In addition, the 2001 New York City Building Code was also compared to examine the changes resulting from the adoption of Local Laws and rules. The comparison was limited to provisions related to structural stability, fire safety, and egress. In general, it was found that the majority of the provisions were similar among the four codes that were compared. It was also found that the New York City Building Code was more advanced in certain areas than the other three contemporaneous codes (1964 New York State Building Construction Code, 1967 Municipal Code of Chicago, and 1965 BOCA Basic Building Code). The following summarizes the major findings of the code comparison.

A.5.1 Provisions Related to Structural Stability

Dead Load

The New York City Building Codes (both 1968 and 2001 editions) allow lower than 20 psf for uniformly distributed partition loads for partitions that weigh less than 200 plf, while the other codes compared prescribed a minimum uniform partition load of 20 psf. As a reference, a 10 ft high partition made of 1/2 in. thick gypsum wallboard on both sides of 2 by 4 wood studs spaced at 16 in. on center weighs about 60 plf. Except for the partition load provision, other dead load provisions in the five codes compared are similar.

Live Load

The five codes compared had similar minimum live load provisions. They all permit live load reduction for the design of columns and floor framing members. The amount of reduction allowed varies among the codes. While the 1967 Chicago Municipal Code allowed only a maximum of 15 percent reduction for floor-framing members (beams and girders), other codes allowed as much as 60 percent reduction (see Table A–3). The amount of reduction allowed for columns is about the same for all five codes (see Fig. A–1).

Wind Load

The general trends of the specified wind pressure distributions along the height of a building in the five codes compared are similar. However, specific design wind pressure values vary as much as 10 psf at specific heights. For tall buildings, like the WTC towers, the 1965 BOCA Basic Building Code would produce the largest shear force and bending moment at the base of the building.

The 1968 New York City Building Code allows determination of design wind pressure based on wind tunnel tests with approval by the Building Commissioner.

Earthquake Load

In the 1960s, only the BOCA Basic Building Code had the earthquake design provisions, which were based on the Uniform Building Code. The 2001 New York City Building Code contains the earthquake design provisions that are based on the 1997 Uniform Building Code.

Others Loads

The 1968 New York City Building Code is the only code among the contemporaneous codes compared that has provisions to consider loads due to thermal expansion/contraction of structural members and shrinkage of reinforced concrete members. The New York City Building Code also included language on the distribution of loads among structural members in a building.

Referenced Design Standards

The 1968 New York City Building Code and the other contemporaneous codes compared, except the 1964 New York State Building Construction Code, make reference to the same national design standards for steel and reinforced concrete.

Building Alterations

All codes compared, with the exception of the 1964 New York State Building Construction Code, had similar provisions on whether alterations were required to comply with the current building code. In general, alterations need to comply with the current code when the alteration is a significant portion of the building size or value (see Table A–5). The New York State code, on the other hand, required any alteration to comply with the code.

Construction Inspection

The codes that were compared include provisions related to inspection of materials and methods during construction. The specific requirements, however, are not similar. The 1968 New York City Building Code distinguishes between "controlled inspection" that require supervision by a design professional and those not so designated as controlled, which can be tested by the person superintending the use of the material. The New York City Building Code required controlled inspection of sprayed-on fire protection, but it did not specify required tests.

Bracing

Only the New York City Building Code among the codes compared included provisions on bracing of compression members. The New York code requires that members designed to brace compression members (for example, columns), be proportioned to resist 2 percent of the design compression load in the member being braced.

A.5.2 Provisions Related to Fire Safety

Construction Types

The building codes compared classify building constructions into different types depending on the combustibility of the construction materials. The New York City Code uses two types, noncombustible and combustible, whereas modern model codes have adopted five types: fire resistive (Type 1); non-combustible (Type 2); combustible (Type 3); heavy timber (Type 4); and ordinary (Type 5). These types are typically sub-divided into classes such as Type 1A and Type 1B.

The cornerstone of fire safety of construction is the "height and area" table that defines the limiting floor and area and building height for different construction classifications (1A, 1B, and so forth) and occupancy groups. In the case where a building could be assigned to more than one construction classification, the codes are silent on which classification should be used, and the selection of the building classification is at the discretion of the owner/architect.

Compartmentation

In 1973, Local Law 5 was adopted by New York City, which required that large, open floors in existing office building over 100 ft in height be subdivided by 1-hour separations into areas not greater than 7,500 ft^2. Areas could be increased to 15,000 ft^2 with 2-hour separations and smoke detectors. Unlimited floor areas were permitted where fully sprinklered.

Sprinklers

In 1985, Local Law 16 was adopted in New York City, which limited Type 1B construction to buildings of 75 ft or less unless they were fully sprinklered, for which there were no height or floor area limitations.

A.5.3 Provisions Related to Egress

The 1968 New York City Building Code contained similar requirements to the other contemporaneous codes, which were compared, for the number and capacity of exits and stairs and for the design occupant load. The New York City Code permitted scissor stairs (two stairs in one shaft separated by a fire-rated partition) with doors located at least 15 ft apart, whereas other building codes prohibited scissor stairs.

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INTERIM REPORT ON DEVELOPMENT OF STRUCTURAL DATABASES AND REFERENCE MODELS FOR THE WTC TOWERS

B.1 INTRODUCTION

The objectives of this project are to (1) develop structural databases for the primary structural components of the World Trade Center towers, (WTC 1 and WTC 2), (2) use the databases to develop reference structural analysis models that capture the intended behavior of each of the two towers, and (3) perform linear, static structural analyses to establish the baseline performance of each of the two towers under design gravity and wind loads. This appendix focuses on the tasks related to the first two objectives. The appendix reports on the work conducted by the firm of Leslie E. Robertson Associates (LERA), the firm responsible for the original structural engineering of the WTC towers, for the development of the structural databases and reference models. It also outlines the comprehensive review process for the structural databases and reference models that includes the rigorous in-house NIST review and third-party review by the firm of Skidmore, Owings, and Merrill (SOM).

Section B.1 presents an introduction and a brief description of the structural system of the towers. Section B.2 presents an outline and methodology used for the development of the structural databases for both towers from the original computer printouts of the structural documents, along with the relational databases that are used for the development of the reference structural models. Section B.3 presents the development of the reference structural analysis models for WTC 1 and WTC 2, including global tower models, typical floor models, and parametric studies needed for the development of the global models. Finally, Section B.4 outlines the in-house and third-party review process for the structural databases and reference models.

B.1.1 Description of WTC Structural System

Global Structural System

WTC 1 and WTC 2 each consisted of a 110-story above grade structure and 6-story below grade structure. The buildings, which were each approximately 207 ft by 207 ft square in plan and with story heights of typically 12 ft, rose to heights of 1,368 ft (WTC 1) and 1,362 ft (WTC 2) above ground. The exterior walls of the towers supported gravity loads and all lateral loads, and were constructed of steel closely spaced, built-up columns and deep spandrels. The core contained columns that supported the remainder of the gravity loads of the towers. The core area was approximately 135 ft by 87 ft in plan (refer to Fig. B–1). The distances between the rectangular core and the square exterior wall were approximately 36 ft and 60 ft. The areas outside of the core were free of columns and the floors were supported by truss-framing in the tenant areas and beam-framing in the mechanical rooms and other areas.

The primary structural systems for the towers included exterior columns, spandrel beams, and bracing in the basement floors, core columns, core bracing at the mechanical floors, core bracing at the main lobby atrium levels, hat trusses, and the floor systems.

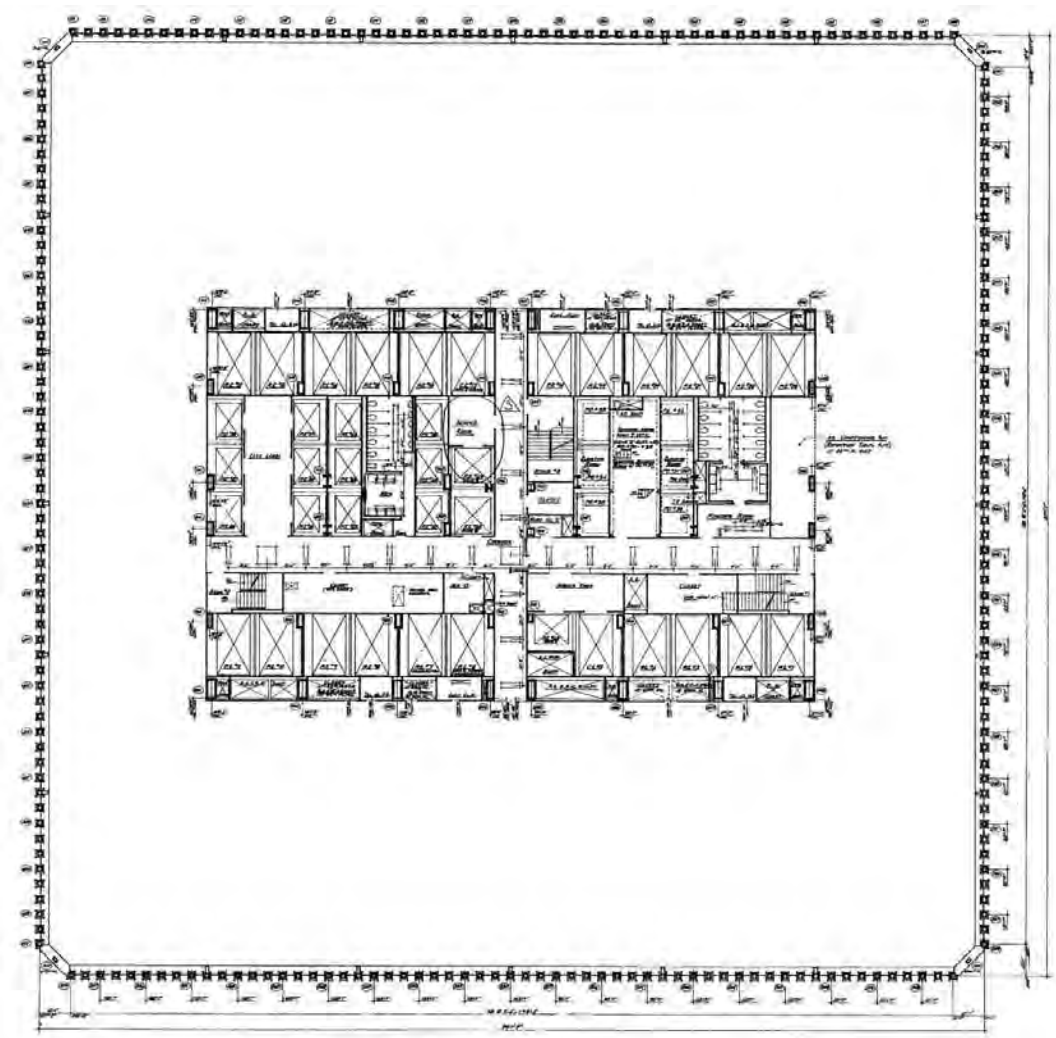

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Figure B–1. Typical WTC tower architectural floor plan (floor 26, WTC 2).

The exterior wall columns from the foundation level up to Elevation 363 ft were spaced 10 ft 0 in. on center, built-up of steels plates, and connected by spandrels. Bracing existed in the plane of the exterior wall between the Concourse Level and the foundation, (see Fig. B–2). Between Elevation 363 ft and floor 7, the single exterior wall columns spaced 10 ft 0 in. on center transitioned to three columns spaced at 3 ft 4 in. on center as shown in Fig. B–2 (see also Fig. B–19).

The exterior wall columns above floor 7 that were spaced 3 ft 4 in. on center, were built-up of steel plates, and were connected to each other by spandrel plates, typically 52 in. deep. The exterior columns and spandrels were pre-assembled into exterior wall panels, typically 3-columns wide by 3-stories tall (refer to Fig. B–22).

The core columns were typically built-up box members at the lower floors and transitioned into rolled structural steel shapes at the upper floors. The core columns were typically spliced at 3-story intervals at

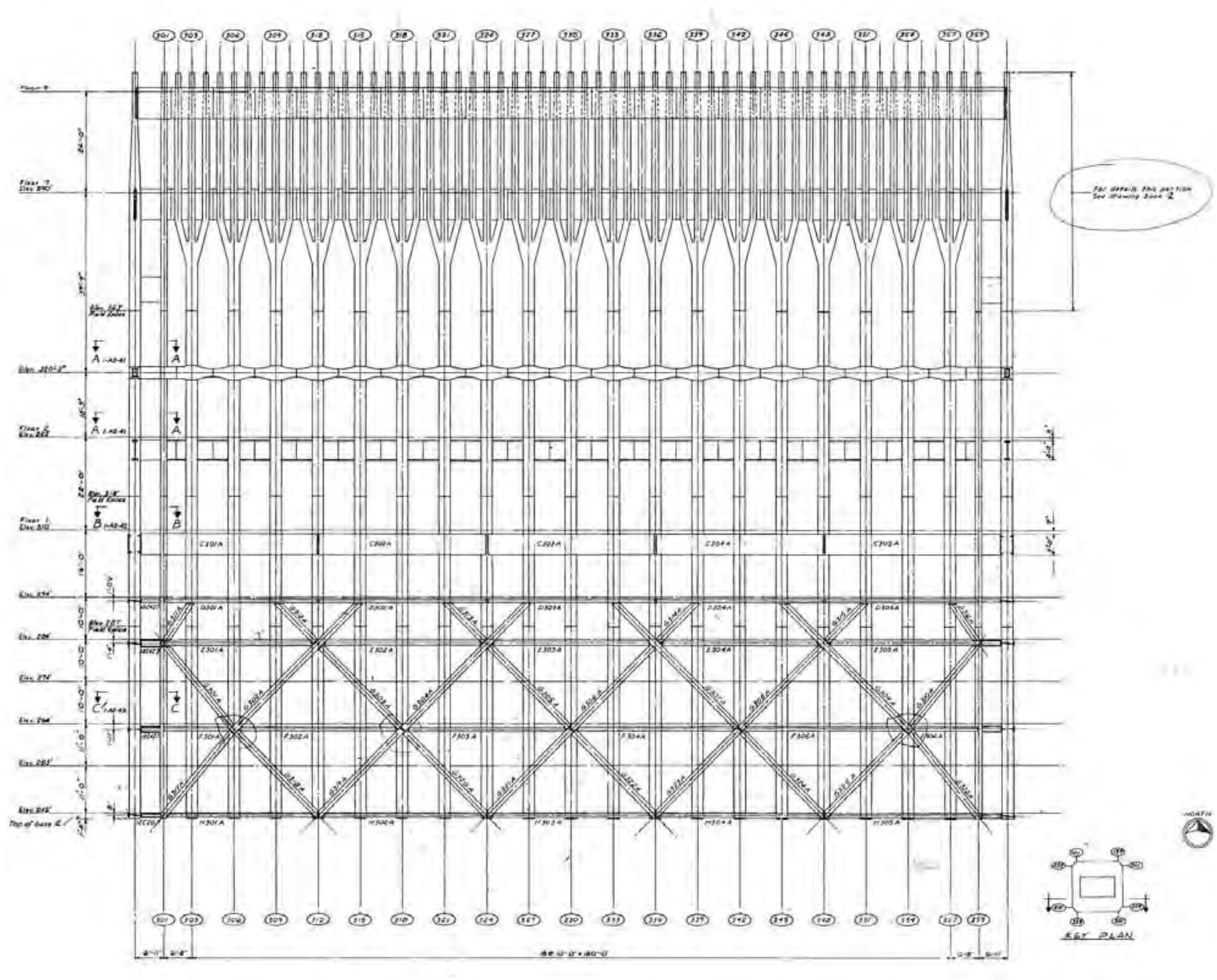

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Figure B–2. Typical WTC exterior wall, foundation to floor 9.

3 ft above floor level. Diagonal bracing of the core columns existed at the lobby atrium levels, the mechanical levels, and in the area of the hat truss.

At the top of each tower, hat trusses interconnected the core columns with the exterior wall panels and provided a base for the antennae. The vertical members of the hat trusses were wide flange core columns. The diagonals were primarily wide flange rolled sections with the exception of the end diagonals interconnecting the core to the exterior walls which were built-up box sections. The majority of the horizontal members in the hat truss system were wide flange and built-up box section floor beams. The members of the hat trusses were shown in the SA/B-400 series elevations (refer to Fig. B–3).

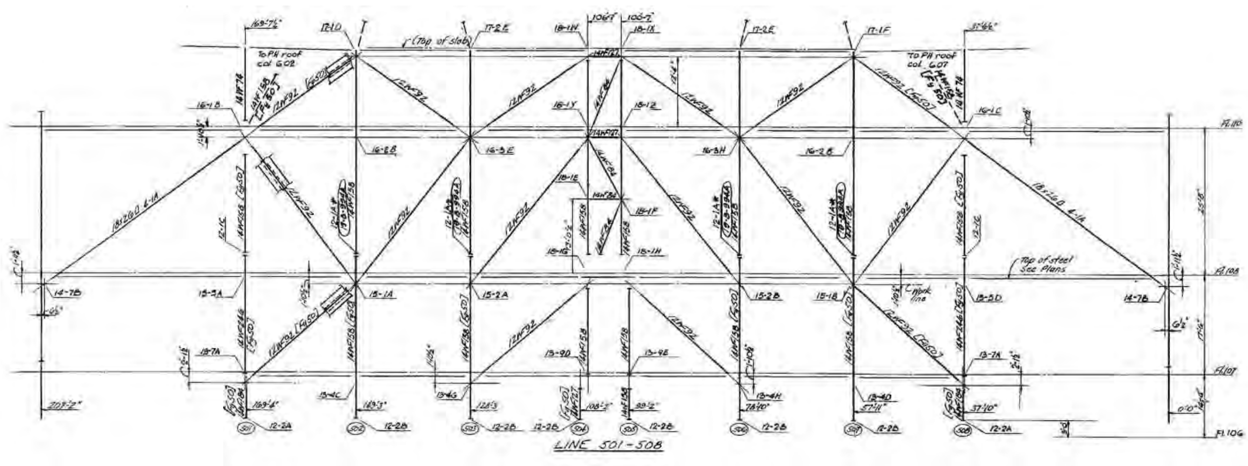

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Figure B–3. Typical WTC tower hat truss elevation (Drawing SA 401).

Floor Structural System

In the typical WTC tower floor plan, the area inside the core was framed with rolled structural steel shapes acting compositely with formed concrete slabs. The area outside the core was framed in either trusses (typical on tenant floors) or in rolled structural steel shapes (typical on mechanical floors).

Truss-Framed Floors—The majority of the floors of the WTC towers were tenant floors where the areas outside of the core were constructed of steel trusses acting compositely with concrete slabs cast over metal deck. The trusses consisted of double angle top and bottom chords with round bar webs and were designed to act compositely with the concrete slab. Composite action was achieved by the shear connection provided by the web bar extending above the top chord and into the slab. Two trusses were placed at every other exterior column line, resulting in a 6 ft 8 in. spacing between truss pairs. The typical floor consisted of three truss zones: a long span zone, a short span zone, and a two-way zone, (see Fig. B–4).

The floor trusses were pre-assembled into floor panels as defined in the contract drawings. The span of the trusses was about 36 ft in the short direction and 60 ft in the long direction. The floor panels included primary trusses, bridging trusses, deck support angles, metal deck, and strap anchors, all of which were defined by the contract drawings and specifications.

The floor truss panel types are indicated in the structural plans (see Fig. B–5) and the plans refer in turn to Drawing Book 7 for information regarding the components of the floor truss panels and to Drawing Book D for damper information. Drawing Book 7 provided panel by panel layout plans and elevations of each referenced truss. The section through a floor panel after the concrete was placed is illustrated in Fig. B–6.

Beam-Framed Floors—The typical locations of the beam-framed floors were the mechanical floors, the mechanical mezzanines, and the floors above the mezzanines (e.g., floors 41, 42, and 43). These floors were constructed using rolled structural steel shapes. The beam framing for the typical floor system was W27 beams in the long span region and W16 beams in the short span region. Typically, beam spacing was 6 ft 8 in. The steel beams acted compositely with the normal weight concrete slab on metal deck.

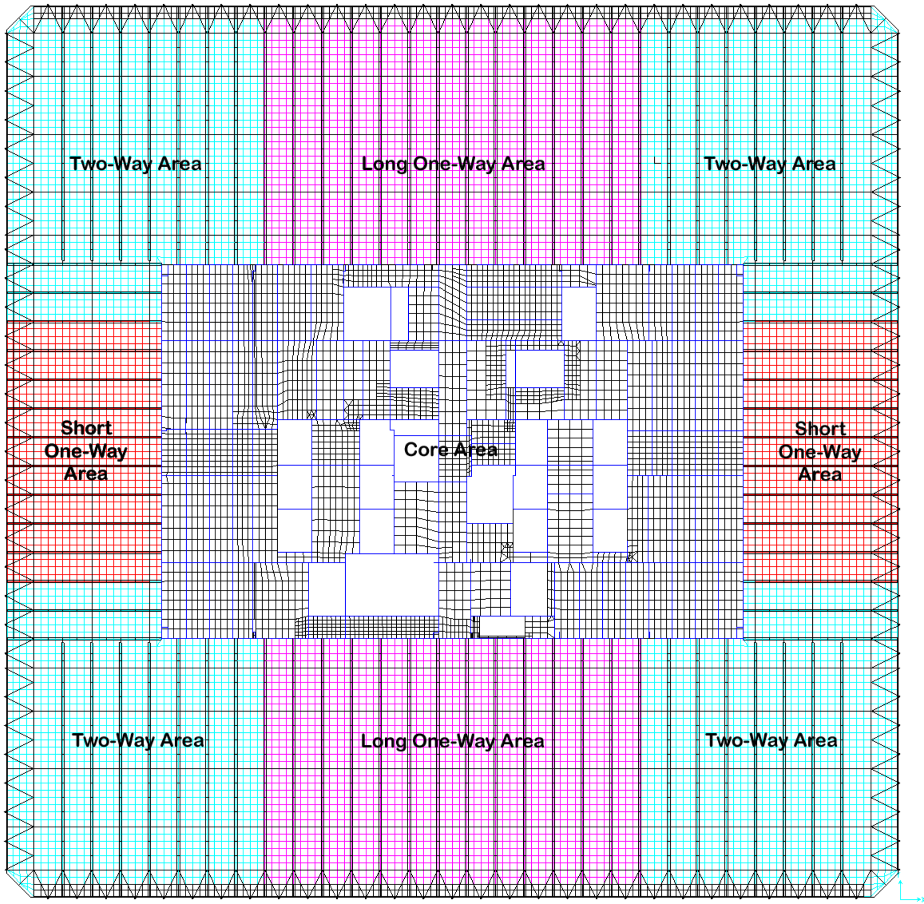

Figure B–4. Typical WTC floor truss framing zones.

The deck spanned in the direction of the primary beams and was supported typically at 6 ft 8 in. intervals by a 4C5.4 deck support channel. A 2 in. concrete topping slab was placed on top of the structural slab. The core area was framed similar to the core of the truss-framed floors, but the steel beams were typically larger, and the concrete slab was 6 in. deep. The beam-framed floors above the mechanical mezzanine had a 7 3/4 in. normal weight concrete slab on 1 1/2 in. metal deck, while the core slab was 8 in. normal weight concrete.

Beam-framing was added to truss-framed floors at levels which supported escalators or stairs in the areas outside of the core. The escalator floors occurred typically in the two levels directly above the mechanical rooms.

B.2 DEVELOPMENT OF STRUCTURAL DATABASES FOR THE TOWERS

This section outlines the development of the electronic databases for the major structural components of the WTC towers from original computer printouts of the structural documents. The structural databases are used to develop the reference structural models of the towers as outlined in Section B.3. Included in this section are an overview of the WTC towers' structural design documents, a description of the structural database contents, methodology for the development of the database, and a description of the relational database.

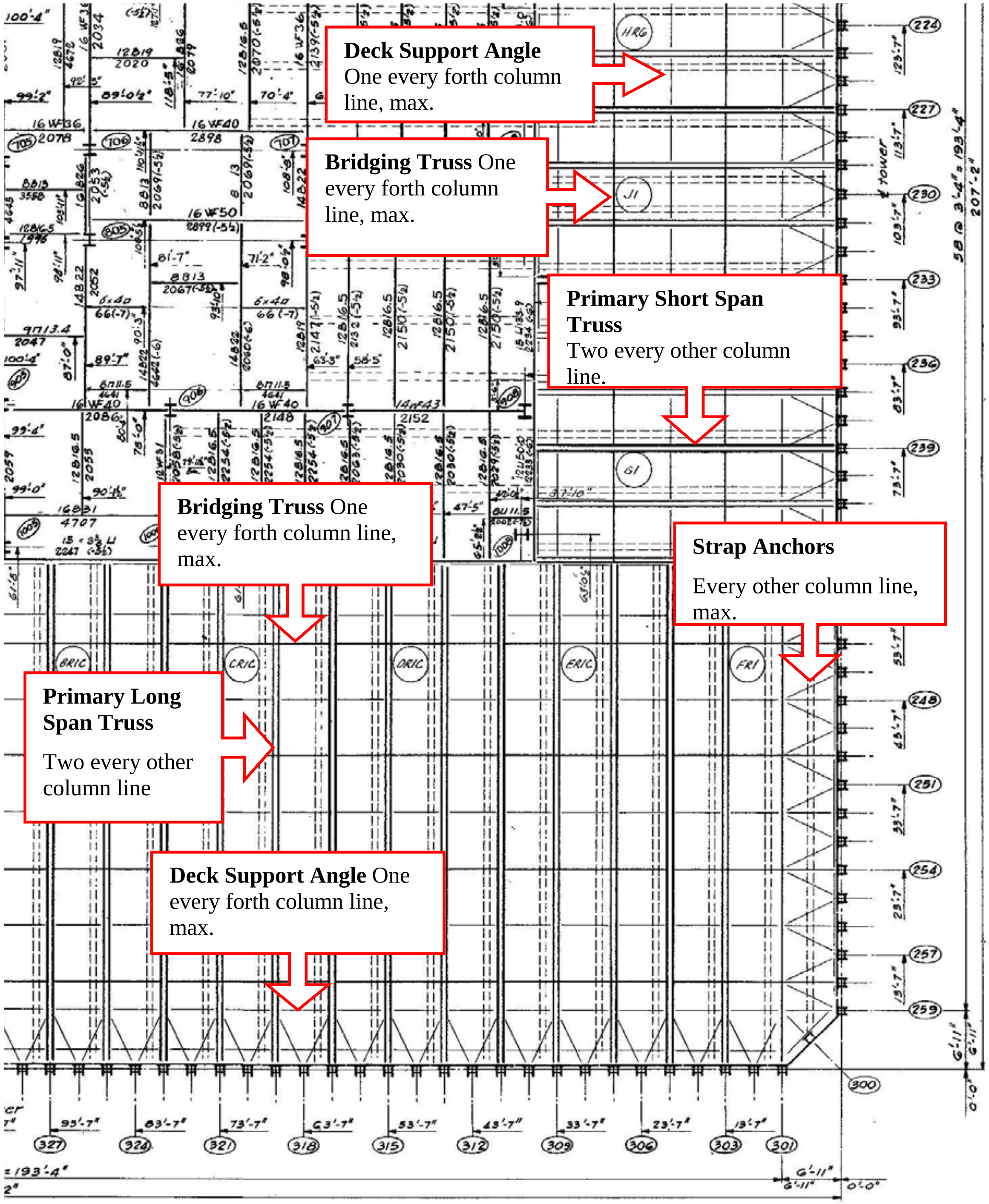

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Figure B–5. Part plan of floor 96 of WTC 1 (Drawing SA-104), components of typical truss framing system.

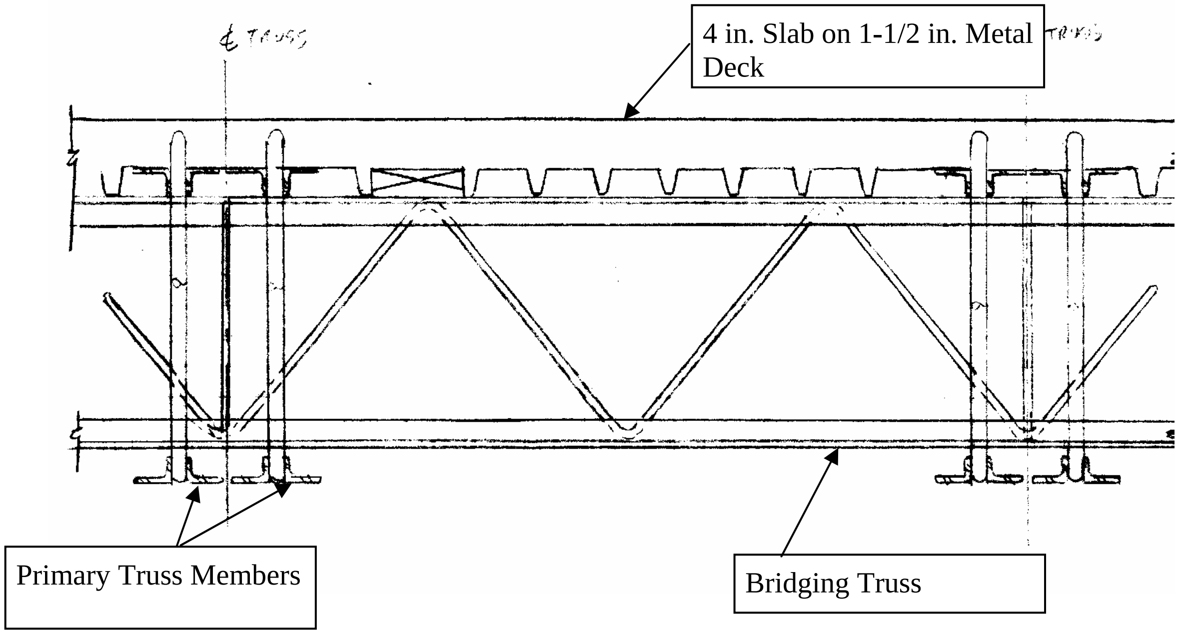

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Figure B–6. Part section typical truss floor panel.

B.2.1 Description of WTC Structural Documents

The WTC structural drawings were issued in two main formats: large-size sheets containing plan and elevation information and smaller book-sized drawings containing details and tabulated information. Throughout the WTC drawings, Tower A or WTCA denotes WTC 1 (north tower) and Tower B or WTCB denotes WTC 2 (south tower). The large size drawings always make reference to the structural drawing books through their notes, sections, and detail references. The structural drawing books for WTC 1 and WTC 2 include the following:

- Book 1 contains exterior wall information to elevation 363 ft. (Dates: 02/1967 to 12/1968, Approx. 213 pages).
- Book 2 contains exterior wall information elevation 363 ft to floor 9. (Dates: 04/1967 to 12/1967, Approx. 62 pages).
- Book 3 contains core column information. (Dates: 03/1967 to 09/1969, Approx. 137 pages).
- Book 4 contains exterior wall information floor 9 to floor 110. (Dates: 04/1967 to 10/1972, Approx. 1,080 pages).
- Book 5 contains the beam schedule. (Dates: 05/1967 to 08/1969, Approx. 292 pages).
- Book 6 contains connection details and core bracing. (Dates: 08/1967 to 05/1969, Approx. 1,060 pages).

- Book 7 contains truss floor panel information. (Dates: 10/1967 to 07/1969, Approx. 345 pages).
- Book 8 contains concrete notes and details. (Dates: 03/1968 to 07/1974, Approx. 926 pages).
- Book 9 contains roof area column splice details. (Dates: 05/1970 to 04/1971, Approx. 440 pages).
- Book 18 contains strap anchor and core truss seat information. (Dates: 10/1968 to 11/1969, Approx. 219 pages).
- Book 19 contains revisions after fabrication. (Dates: 08/1968 to 05/1975, Approx. 374 pages).
- Book 20 contains structural steel details. (Dates: 07/1968 to 03/1971, Approx. 41 pages).
- Book D contains damper details. (Dates: 03/1969 to 09/1971, Approx. 43 pages).

The remaining number books (Books 10, 11, 12, and 13) contain information about the sub-grade structure. Books 14, 15, 16, and 17 were never used.

Until fabrication was begun, the above drawings and drawing books (with the exception of Book 19) for the project were modified in keeping with the requests for changes by contractor(s) and early tenant modifications. The drawings were modified up until such time as the fabrication of elements commenced. At that time, Book 19 was introduced. It contained the information regarding ‘revisions after fabrication’.

LERA believes that the original structural drawings represent significantly accurate ‘as-built’ drawings for the towers. As tenant modification requests became large in scope, they became separate projects (e.g., the Fiduciary Trust Vault Project, see Section B.2.4). Tenant structural modifications designed by LERA were then documented in a single book of quarter-size plans referred to as the ‘WTC Tenant Structural Modifications Book’. Later tenant modifications were mostly archived on a job-by-job basis without a central accounting for all the changes. NIST has in its possession complete copies of all the drawings, drawing books, and modifications to the towers performed by LERA. In some instances modifications were made by the Port Authority of New York and New Jersey (PANYNJ) Engineering, such as additions to the mechanical levels. In other instances, tenant modifications were performed by other engineers. For these instances, LERA does not have record of the work completed. According to the PANYNJ, no record of structural work could be found so far for the additions to the mechanical floors made by the PANYNJ Engineering, and it is likely that they were lost with the collapse of the towers. Modifications made by other engineering firms include openings or closings of floor slabs and local reinforcement of floor segments to accommodate new loads. For these modifications, NIST has access to the documents related to the work.

The few modifications made by LERA to the components compiled in the WTC structural databases that will have an effect the global behavior of the towers are listed in Table B–1.

Table B–1. Modifications to members of the WTC database (WTC-DB).

Item	Description	Tower	Element	Floor	Element Effected	WTC-DB Modified	Archived
1	Core column reinforcing	WTC 1 and WTC 2	Numerous	98–106	Core columns	Book 3	Book 19
2	Fiduciary Bank Vault	WTC 2	Col. 508B and Col. 1008B	45–97	Core columns	Book 3	LERA P209
3	Bombing of 26 February 1993 repair	WTC 1	Col. 324, bracing G313A and G304A	B-2 level	Perimeter column and bracing	NA	LERA P1003118
4	EXCO stair	WTC 1	Col. 901A	26	Core column	NA	LERA P1003249

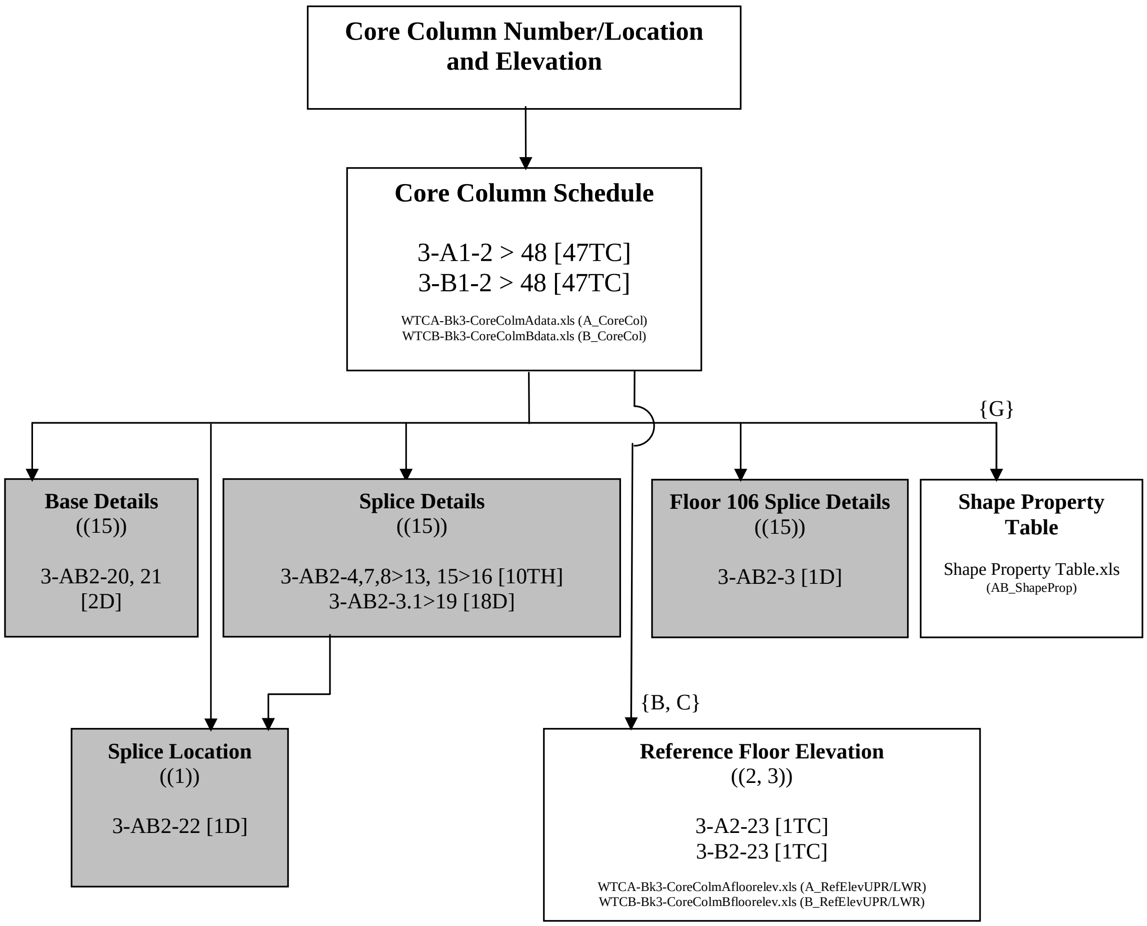
B.2.2 Overview of the WTC Structural Database (WTC-DB)

The original WTC design documents used the concept of limiting the need for repetition in documenting the data shared between different elements with similar characteristics. The drawing book schedules refer to subsequent tables for information common to several lines of the same schedule. In an effort to minimize the amount of repeated information and thereby the data checking of the digital WTC-DB, the drawing book data within the databases created for this project were linked in a similar manner. In order to accurately follow the original flow of the drawing book links, flowcharts of the drawing books to be digitized were developed for this project. These flowcharts were used to organize the links of the digitized data within the relational database. An example of such flowcharts for Drawing Book 3 (core columns) is illustrated in Fig. B–7.

The WTC-DB contains the computer and hand-tabulated data for the major structural components from the original Drawing Books 1 through 5, including exterior walls, core columns, and beam schedule for the towers. Where information from Drawing Books 1 through 5 was modified by Drawing Book 19 and would affect the towers' modeling, the information is included in the database. In addition, some information from Drawing Books 6 (core bracing schedule) and 9 (beams in the hat truss region) has been included in the database files as it was utilized in the finite element modeling of the towers.

The drawing book tables are first digitized and stored in Microsoft Excel format files. The Excel files include several worksheets that describe the evolution of the data from the drawing book to the final database format, as well as additional information and notes for interpreting the data.

The WTC relational database links the Excel files and allows users to view and select data through query commands. The primary benefit of the relational database format is the ability to programmatically query the database for data required in assembling the structural models of the towers. The query routine allows multiple users the ability to review, extract, and export the basic data in any required form. The data can be manipulated using Structured Query Language (SQL) according to the desired output, for example the structure of the user's finite element model input file.



NOTES:
1. 4-AB-* Denotes Drawing Book 4, Tower A , B or AB, and page number
2. [*TC] Number of pages and type, where page types include:
   TC - Computer generated tables; TH - Hand written tables; D - Diagrams
3. *.xls Excel spreadsheet file name; (AB_***) database heading
4. ((*)) Reference note key from original Drawing Book information table.
5. {*} Relational database link (i.e. Excel column number) from previous *.xls file.
6. Figures of columns and panels are shown from inside of building looking out, unless otherwise noted.

Figure B–7. Drawing Book 3 flowchart: WTC 1 and WTC 2 core columns, foundation to floor 106.

B.2.3 Methodology for the WTC-DB Development

Data Entry

The tabulated portions of WTC Drawing Books 1, 2, 3, 4, 5, 6, and 9 were first scanned and stored in TIFF image format files. The image files containing the tabulated information were then opened in an Optical Character Recognition (OCR) program that converted the information into a text file. The OCR program was modified to allow for the filtration of unnecessary characters during the documents conversion process. In other words, the user could direct the program to block specific characters that are not on the actual page. As an example, if after reviewing a table, one recognizes that it uniformly contains numbers and only the characters "A, B, -", and "/" then the remaining characters can be frozen out by the software. This reduced the misinterpretation, as an example, of a 'Z' for a '2', or an 'O' for a 'zero'.

The raw text file was then opened in a word processing program, where it was compared with the original hardcopy drawing books. As needed, data columns were adjusted, and obvious errors were individually corrected. The 'cleaned' text file was then imported into a Microsoft Excel spreadsheet with column headings and proper alignment. When importing into Excel, "text" was the cell format used for handling of the scanned information to avoid the misinterpretation of fractions as dates (e.g., 3/8 as March 8). An Excel macro was written at this stage to convert the text fractions into number fractions. The final product of this stage was an Excel file that contains the information from the drawing book table.

Quality Control

Checking began during the OCR data entry process, where the files being entered and the OCR software interpretation were viewed simultaneously. This was considered a first check. Once the Excel file was complete, the file entered the 'second check' process.

The 'Second Check'—An engineer not involved during the OCR process performed a second, sample checking of the database in a random but methodical manner. For approximately once in four pages, every cell of data in the page was compared with the original drawing books. Discrepancies of the files were then either re-entered using OCR or were individually corrected to agree with the original books.

The 'Cross-Check Rectify' Check—After completing the 'second check,' the files were compared with the database provided by a consultant for the leaseholder of the WTC towers as part of an insurance litigation concerning the towers (provided to LERA as Government Furnished Information [GFI]) using a cross-check macro formula worksheet. Once compared, conflicting information appeared in a yellow highlighted cell displaying both sets of compared information in the "Calculation" macro formula worksheet. The cell was then reviewed and confirmed with the WTC drawing books. If errors were from the developed worksheet, data was rectified, and the yellow highlight in the 'Calculation' worksheet was then removed by comparing the files again. If errors were from the GFI worksheet, raw GFI data was not modified, but the cell was highlighted in blue to note that it has been reviewed. The files were then compared again, and the cell color in the 'Calculation' worksheet changed to blue. The process was repeated to remove all the yellow cells so that only blue highlighted cells remain. The worksheet 'ComparisonORIGINAL' was retained for the record of the original comparison, and the updated worksheet 'ComparisonFINAL' was retained for the record of the final comparison.

Final Review—Finally, the files were reviewed for completeness, formatting, and data units. A final check was made to find any numbers that may have been input as text letters. Following this review, the worksheet was used to develop the member section properties.

Cross Section Property Calculations

The next step was to calculate the cross section properties for the members included in the database. The section properties calculated included cross sectional area (A), moment of inertia (I), section modulus (S), plastic section modulus (*Z*), radius of gyration (*r*), and torsional constant (*J*) for both the major and minor axes (where applicable). The *Section Designer* function of SAP2000 Version 8 was used to calculate the cross section properties since it enables the program to perform more precise code checks, as the dimensions of each plate element that is part of the section would be input into the finite element model.

The current rolled shape database in SAP2000 represents the modern day rolling practices. The rolled shapes used in the construction of the WTC towers were from a different era and thus, had different properties in comparison to present day shapes. Therefore, a rolled shape database consistent with the time of construction was developed in this project. See Section B.2.5 for further discussion about the rolled shape database.

Relational Database Development

As discussed earlier, the original WTC drawing books were designed to avoid repeating identical information. The drawing book schedules, therefore, refer to other tables for information common to several lines of the same schedule. In keeping with the nature of the original drawing books and to minimize the data in the digital WTC-DB, the drawing book and section property data were linked using Microsoft Access.

The assembly of the relational database began with the mapping of the original WTC drawing book into flowcharts (see, e.g., Fig. B–7). The digitized drawing book data with the corresponding cross sectional member properties from the Excel-format files were then imported into the Microsoft Access database program and partitioned into tables. The tables were then joined using the links cataloged in the flowcharts. These tables were developed to provide the input files for the finite element modeling of the towers as illustrated in Section B.3.1.

B.2.4 Modifications to Database Elements

The majority of the original members and elements defined within the WTC-DB could be fully defined by the original data in the drawing books. As outlined in Table B–1, however, some modifications were made that are described in the following sections. Of the items outlined in Table B–1, items 1 and 2 have been included within the database.

Core Column Reinforcing at Floors 98 to 106

A number of core columns in both WTC 1 and WTC 2 were reinforced at floors 98 to 106. Book 19, pages 19–AB–974.1 through 4, shows that core columns 501, 508, 703, 803, 904, 1002, 1006, and 1007 from floors 98 to 106 in both towers were reinforced with steel plates. Three methods were used to attach the reinforcing plates to the wide flange columns: (1) the plates were welded to the flanges; (2) the plates were welded to the webs; and (3) the plates, which were parallel to the web, were welded to the flange ends. The plate information (width, thickness, length, and yield strength) was incorporated into the database tables of Book 3. Since the plates varied from floor to floor, the original column (defined over a three-story height) was split into typically three floor-by-floor sections and the designation of the column was appended to include either U(upper), M(middle), or L(lower) designation (refer to Fig. B–8). For floors 104 and 106, the columns are two-story columns. Hence, the columns at these floors had only U and L designations. The section property calculations included the contributions of the reinforcing plate at each level. For the built-up section property data, the reinforcing plate was considered to be applied to the column for its floor-to-floor height.

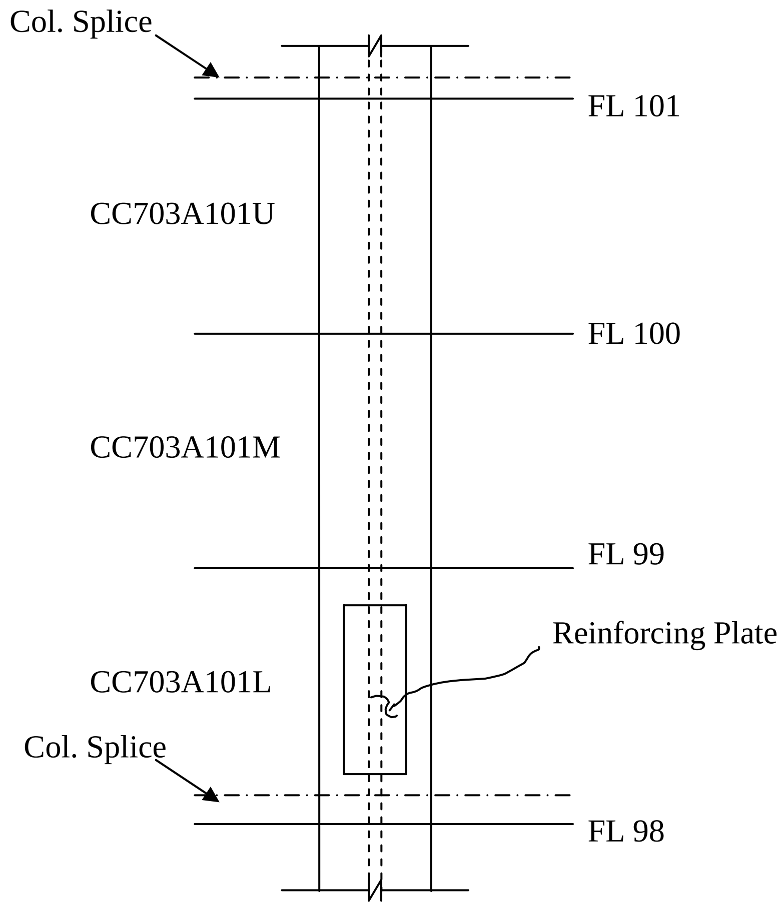


Figure B–8. Core column reinforcement.

Core Column Reinforcing Due to Construction of Fiduciary Trust Vault

The Fiduciary Trust Company added a concrete vault at floor 97 of WTC 2, which required reinforcing two corner core columns at the north end of the core. This work is included in the WTC Towers A and B Structural Renovation Drawings Reference Manual. The Fiduciary Trust Structural Drawing 765–S–A–4 shows that WTC 2 core columns 508 and 1008 were reinforced with steel plates from floors 45 to 97. The reinforcement consisted of plates welded to the flanges of the built-up box columns (floors 45 to 83) and the flanges of the rolled shape columns (floors 83 to 97). These reinforcing plate modifications and the reinforcing plates yield strength, F_Y, were added to the original Book 3 data contained in the WTC-DB. The database included plates that extended long enough such that they substantially affect the member properties of the column, e.g., the added plates increase the capacity of the columns. Where the plates appeared to reinforce only the column splice, they were not included in the database.

The reinforcing plate data were tabulated and incorporated into the database in the same manner as the plates discussed in the previous section, except that a second length designation was added to differentiate the length of the plates on the north and south faces of the columns, i.e., the column designation “LN” refers to the length of the plates on the north face of the column. Again, when calculating the built-up column properties, the plate was assumed to be continuous along the floor-to-floor height of the column. When the length of the reinforcing plate shown in the drawing was greater than the floor height, the plate was attributed to the two column segments. Where the plate extended over the entire height of the segment, the length was tabulated as the height of the column segment. The remaining length of plate was attributed to the other column segment.

Repair Due to the Bombing of February 26, 1993

The 1993 bombing resulted in structural damage to WTC 1, centered at exterior column 324 (south wall), B-2 level. The face of the column towards the explosion was slightly bowed, and the splice in the column developed a hairline crack. The column was reinforced locally to account for the loss of steel area. The bracing on either side was replaced with equivalent sections and attached in a similar manner as the originals. No modification to the WTC-DB has been made for this repair.

Tenant Alteration for an Interoffice Stair

A tenant alteration was provided for an interoffice stair between floors 25 and 26 in WTC 1. This work, adjacent to core column 901A, was performed by an engineering firm (other than LERA) and unknowingly resulted in the loss of a core column bracing strap (refer to Fig. B–9), leaving the column unbraced about its minor axis for two stories. The PANYNJ alerted LERA to the issue and asked LERA to review. The situation was reviewed by LERA, and the column stability was found to be adequate. No modification to the WTC-DB has been made for this modification. The effect of removing the strap is accounted for in the global model of WTC 1, see Section B.3.1.

Drawing Book Data Discrepancies

In the original WTC drawing book data, the following discrepancies were discovered by LERA:

- Book 1 page 1–B–15. For member number G311A, the inch portion of the length is listed as 3-1/18. Based on the comparison to similar bracing types in the area, this dimension was modified to be 3-1/8 in. in the WTC-DB.

- Book 3 page 3–A1–10. For core column 601A between floors 86 to 89 and 89 to 92, the column type is listed as 213. Type 213 is a column type which by definition has reinforcing plates, but for this location, no plate data was provided in the schedule. This, in combination with comparisons to similar columns in plan, led to modifying the column type to 111. This also applies to column 601B, page 3–B1–10 between floors 86 to 89 and 89 to 92.

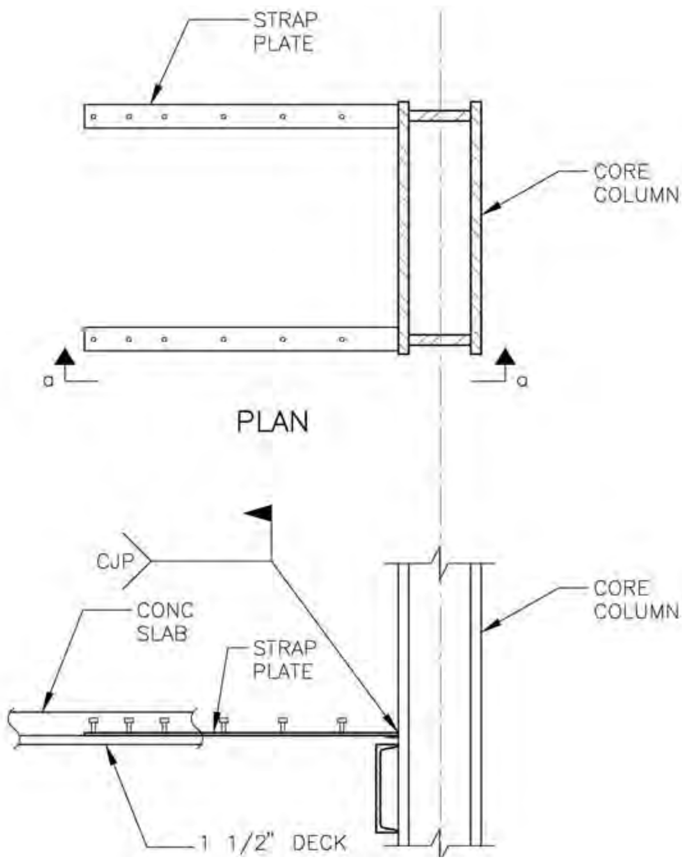


Figure B–9. Column section at original column strap detail (taken from drawing book 18, page 18–AB2–12).

- Book 3 page 3–B1–48. For column 1008B between floors 63 to 66, the yield strength, F_Y, is listed as 6 ksi in the table. Based on the yield strength of the columns above and below these floors, the yield strength was modified to be 36 ksi. For the same column number and floor segments, the lower splice detail number is listed as " 01G." Based on the lower splice detail number of the columns above and below these floors, the number was modified to be "301G."

- Book 3 page 3–B1–9. For core column 508B between floors 21 to 24, the length of plate 1, *W1*, is tabulated as 11.25 in. However, length *B* for this column is 22 in. and thickness *t2* is 5.5 in. *W1* equals *B* minus two times *t2* (see Fig. B–10). Hence, assuming *t2* was listed correctly in the table, *W1* was modified to be 11 in.

- Book 1 page 1–B–23 and 1–B2–19. The details for column types 1024, 1025, 5024, and 6025 listed in the tables are not explicitly shown in the drawing book. For these members, column shapes are assumed to be as shown in the typical details in page 1–B–19 for the 1000 series columns, 1–B–24 for the 5000 series column, and the 1–B–27 for the 6000 series column.

- Book 3 page 3–AB2–6. The column type 216 does not appear to be assigned to any member in the drawing book.

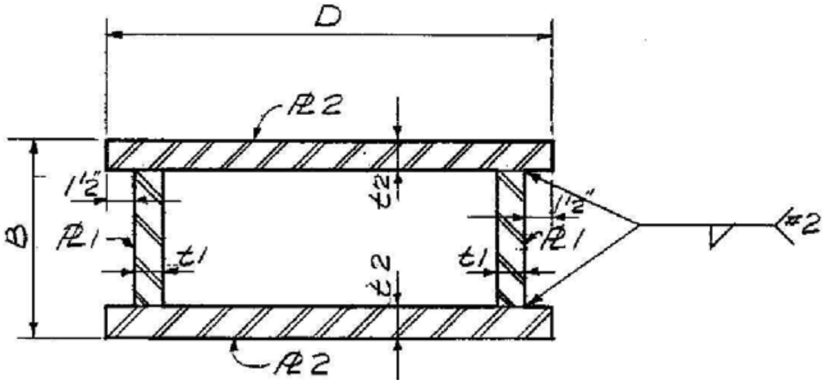


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Figure B–10. Core column series 300.

B.2.5 Section Property Calculations

SAP2000 *Section Designer* was typically used to calculate section properties for built-up sections. The sections were "built-up" within SAP2000 by defining plate dimensions and offsets from 0-0 location. Section orientations were defined with the X-X axis horizontal to the bottom of the original drawing book page as the detail is shown in the drawing book.

During the process of calculating properties there was an exception to this orientation rule. Core column members CC1007A104L, CC1002A104L, CC703A106L, CC1007B104L, CC1002B104L, and CC703B106L consist of a wide flange shape and web reinforcing plates. These members were input into SAP2000 rotated 90 degrees from the orientation shown in the details to utilize the default orientation of the wide flange section in *Section Designer*. Once the properties were calculated, the sections were placed in the WTC-DB following the orientation of the detail (i.e., the axis was shifted back 90 degrees).

When rolled shapes were used to create built-up sections, the rolled shapes database developed for this project was used to build the sections in SAP *Section Designer* as explained later in this section. The 200 series core columns (wide flange rolled columns reinforced with plates) are examples of members whose properties were calculated in this manner.

Member Designations

For member section property calculations and assembly of the finite element models, the members were named using the following general member designations. The member designations are listed in the Microsoft Excel files.

First character:

- Book 1—below tree–B
- Book 2—exterior wall tree–T
- Book 3—core columns–C

- Book 4—exterior columns and spandrels–E

Second character:

- C—column
- S—spandrel and below grade exterior wall spandrel, strut, or bracing

Third to fifth character: (third to sixth character for 4 digit column e.g., 1004)

- Column number

Sixth character:

- A—WTC 1
- B—WTC 2

Seventh character and above:

- Upper splice level—for core columns
- U(upper), M(middle), or L(lower)—column segment where reinforcing plates are added
- T or B—top or bottom of nonprismatic columns
- Detail letter (lowercase)—(where more than one section is calculated)
- F or C—face or center of nonprismatic spandrel
- Elevation—below tree spandrel elevations

Column Member Multiple Section Property Calculation

In the database, the following three types of column members had different cross sections along the length of the members:

- Exterior wall tree at level C in Drawing Book 2 (two different cross sections)
- Exterior wall tree at level E in Drawing Book 2 (three different cross sections)
- Exterior column type 300 (floor 9 to 106) in Drawing Book 4 (two different cross sections)

For these three member types, the section properties of the different cross sections were calculated and listed in the database tables. In an effort to minimize repeated information, the raw input data for all sections were only shown in the rows that correspond to the first cross section. For the second and third (if any) cross sections, the calculated data followed in the rows below. The constant raw data such as the column number were not repeated in these rows of the table, and thus the corresponding cells were left blank. Since the column number was used as a link for the development of the relational database, only the row containing the raw input data and the first cross section properties was returned in a query, and thus, the user must refer back to the Microsoft Access 'Tables' for the remaining section property information. The section names of the different cross sections along the member length were distinguished by the last one to two characters, which identified the cross sections where the section properties were calculated.

For example, exterior column EC339 (mechanical floors) tapers over a portion of the length of the member (refer to Fig. B–11). The section properties above and below the spandrel were calculated. The column section above the spandrel was called EC339, while the column section below the spandrel was called EC339cc. The suffix 'cc' denoted the section below the spandrel. Note that the raw dimensional data for EC339cc were not shown in the table, as the information was the same as for EC339.

Spandrel Member Multiple Section Property Calculation

In the database, the exterior columns below elevation 363 ft in column series 5000, 6000, and 7000 in Drawing Book 1 had corresponding spandrels shown in the details in Book 1. There were two types of spandrels for these members, tapered built-up box shapes and built-up I shapes. For the tapered built-up box shapes, the section properties of the different cross sections were calculated and listed in the database tables. The data were listed in the database files as described for columns with multiple cross sections. The section names of the different spandrel cross sections along the member length were distinguished by the last three to four characters, which identified the cross sections where the section properties were calculated.

For these exterior columns, there are spandrels at two elevations, 332 ft and 350 ft. At elevation 350 ft, the spandrels tapered, and as a result two cross section properties were calculated. The first section was at the face of the exterior column, and the corresponding section name had a Suffix F (face). The second section was at the center of the spandrel in between two exterior columns, and the corresponding section name had a Suffix C (center). The elevations and locations of the cross sections of the spandrels were shown in the figures in the "Cross Section" worksheets in the database Excel files.

For example, four different section properties were calculated for exterior column 6009 in WTC 1. The first section was the exterior column itself, and the section name was BC6009A. The other three sections, BS6009AB332, BS6009AT350C, and BS6009AT350F were for the spandrel sections. The suffix B332 in BS6009AB332 denoted the bottom spandrel at elevation 332 ft. Suffixes T350C and T350F in BS6009AT350C and BS6009AT350F, respectively denoted the top spandrel at elevation 350 ft, and the "C" or "F" identified the locations where the section properties were calculated, see Fig. B–12.

Section Property Calculation Comparisons

For all the members whose section properties were included in the GFI database, the cross sectional properties in the GFI data were compared with the data contained within the WTC-DB. Most section property results compared with good accuracy between GFI and the WTC-DB (within 1 percent). It was found that results from the calculations of the torsional constant, J, however, did vary. LERA in-house

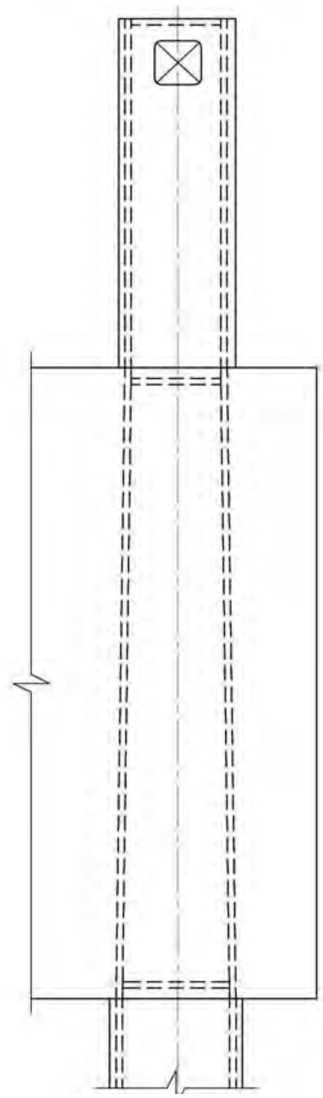


Figure B–11. Exterior column type 300, floor 9 to floor 106 (taken from drawing book 4, page 4–AB2–18).

programs were then used to confirm the accuracy of the *J* calculation. For core columns in WTC 1, SAP2000 generated values used in the WTC-DB were on average 8 percent larger than *J* values calculated using a LERA in-house program, while the results provided by the GFI database were on average 13 percent greater than LERA in-house program *J* calculations.

It was found that for box sections, *J* values calculated by the above equation matched the *J* values given by SAP *Tube Section*. However, for the same tube section, the *J* values given by SAP *Section Designer* were greater than *J* given by SAP *Tube Section*, even while all other properties were equivalent. According to Computers and Structures, Inc., the developer of SAP2000; the *J* values given by SAP *Section Designer* are more accurate as SAP *Section Designer* uses a finite element method to calculate the *J* values while an approximate equation is used in SAP *Tube Section*.

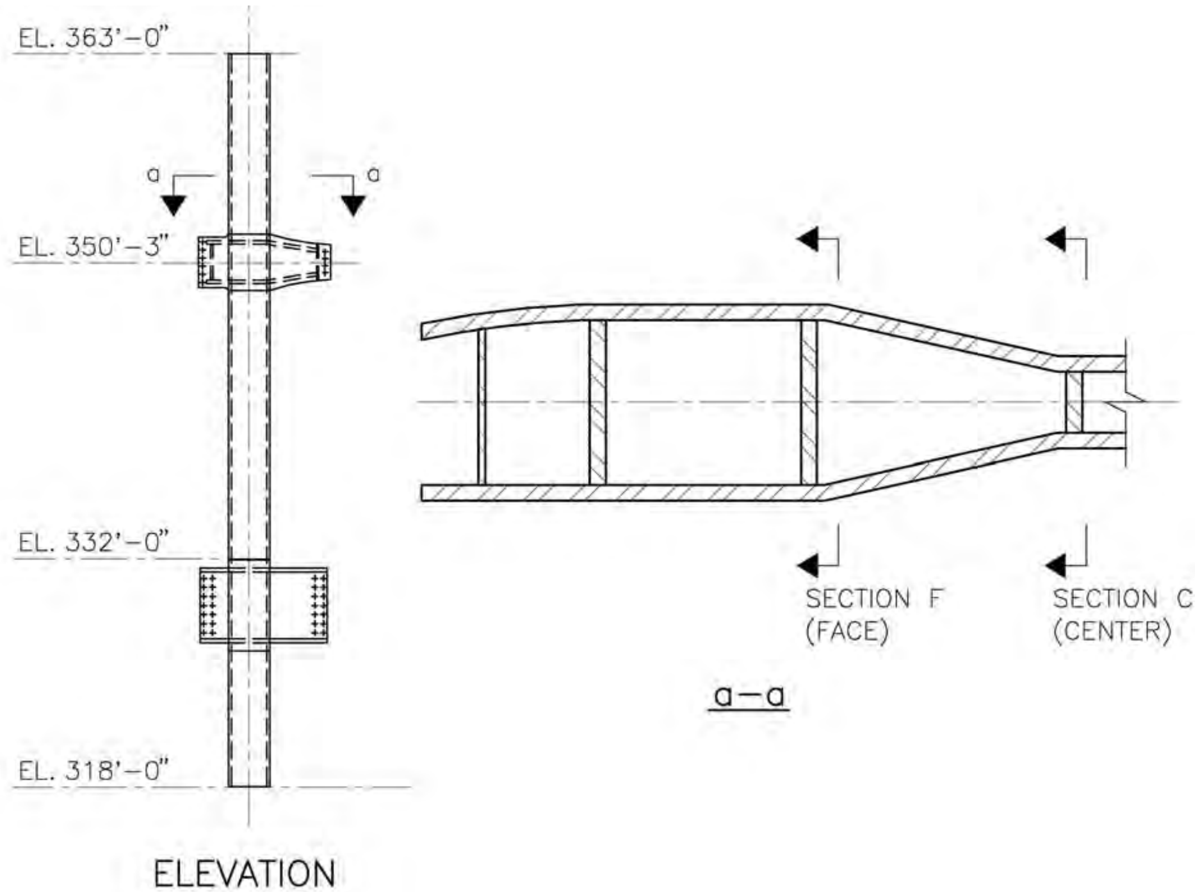
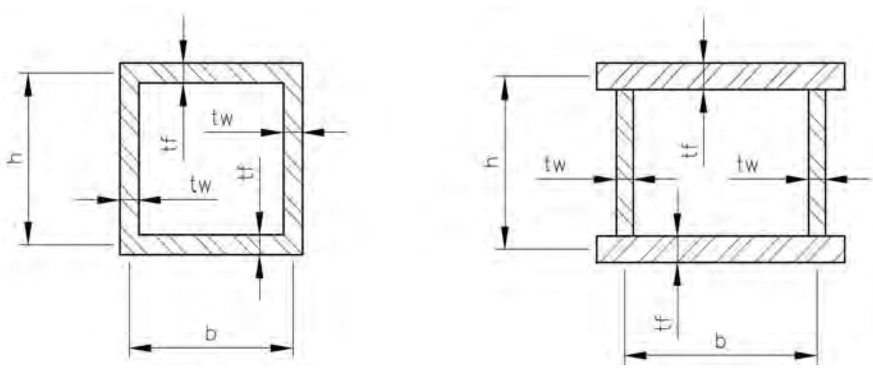


Figure B–12. Column type 6000 with tapered spandrel (taken from drawing book 1, pages 1–A2–27 and 28).

The approximate equation used to calculate J values by the LERA in-house program for a built-up column or box section as shown in Fig. B–13 is as follows:

$$J = \frac{2(bh)^2}{\frac{b}{t_f} + \frac{h}{t_w}}$$



(a) (b)

Figure B–13. Box section and a built-up column.

In order to minimize the complexity of the model, where the member cross-section was of the type illustrated in Fig. B–13 (a), box column members were defined in SAP *Tube Section*. The remaining built-up box columns (similar to Fig. B–13 b) were defined in SAP *Section Designer*.

For members whose properties are not given in the GFI database, hand calculations or calculations by LERA in-house program were carried out to verify the results from SAP2000 *Section Designer* for at least one section for each member type.

In summary, it was found that SAP2000 *Section Designer* provided section properties in close agreement to LERA calculated properties. In most cases these properties also closely matched with the properties listed in the GFI database. In the cases where SAP2000 results disagreed with the GFI database, the results were reviewed and it was concluded that the SAP2000 calculation provided the correct properties. Therefore the section property results calculated using SAP2000 were used in the WTC-DB and the development of the finite element models of the towers.

Rolled Shape Database

While the majority of the primary members of the WTC towers' super-structure were built-up members, rolled shapes were also used. The rolled shapes specified in the drawings in a number of cases are no longer produced and therefore, are not included in the rolled shape database embedded within SAP2000. Therefore, a rolled shape database was developed using the old nomenclature and section properties. The result was a file named 'Shape Property Table.xls' and it contains three worksheets, 'Database', 'Excel Format', and 'WF Shape Properties from SAP'. The following is a discussion of their contents.

<u>Data contained in 'Database' and 'Excel Format'</u>—Drawing Books 3, 4, and 5 include reference to specific rolled shapes. The referenced shape names were extracted from the above books and assembled into a single reference database for rolled shapes. Most of the section properties were obtained from the Manual of Steel Construction, American Institute of Steel Construction (AISC), Sixth Edition, 1963 (AISC 6th Edition) with few exceptions where cross sections were not included in this edition. Examples of these exceptions include the following:

- Section properties of 14WF455 to 14WF730 were obtained from the Manual of Steel Construction–Load and Resistance Factor Design, American Institute of Steel Construction, Third Edition, 2001 (AISC-LRFD 3rd Edition).
- Section properties of 6CH12, 6CH15.1, 12CH40, 12CH45, and 12CH50 were obtained from the MC-shapes table in the AISC-LRFD 3rd Edition.
- Section properties of 18WF69 were obtained from the Iron and Steel Beams 1873 to 1952, the American Institute of Steel Construction, 1968. 16WF342 was assumed to have the same section properties of 16H342 tabulated in Iron and Steel Beams 1873 to 1952, the American Institute of Steel Construction, 1968.
- For 7x5 tube, Z_x, Z_y, and J were obtained from the AISC- Allowable Stress Design (ASD), 1989, 9th Edition.

- For 2L 3 1/2 × 3 × 1/2in. long leg back to back, the combined properties were taken from SAP's embedded rolled shape database.

Data contained in 'WF Shape Properties from SAP'—For the rolled wide flange shapes, an additional database was created in SAP2000 based on the tabulated shape dimensions from the AISC Manuals as discussed above. Computers and Structures, Inc. provided an MS Excel file named 'Proper.xls' with a macro that allowed the accurate calculation of the section properties for use within SAP2000. This information was then used by SAP2000 *Section Designer* to calculate section properties for built-up members comprised of wide flange sections and added plates.

For calculation of the properties with 'Proper.xls', dimensions of the webs and flanges, as well as the size of the fillet, were input into the spreadsheet. The macro then calculated the section properties based on the input information. The results were shown to be in good agreement with the original tabulated properties.

B.3 DEVELOPMENT OF REFERENCE STRUCTURAL ANALYSIS MODELS FOR THE TOWERS

This section outlines the development of the reference structural analysis models for each of the two towers. Included in this section are descriptions of the structural models, modeling techniques, parametric studies utilized in the development of the models, and a description of the methodology used in exporting data from the relational database presented in Section B.2 into the global models.

The main types of the models developed are as follows:

- Two global models of the major structural components and systems for the towers, one each for WTC 1 and WTC 2; and

- One model each of the typical truss-framed floor and typical beam-framed floor (mechanical level) within the impact and fire regions.

The models are all linear elastic, three dimensional structural analysis models developed using Computers and Structure, Inc.'s SAP2000 Software, Version 8. The models will be used to establish the baseline performance of each of the two towers under gravity and wind loads in the third phase of this project. In addition, these models serve as a reference for significantly more detailed models to be developed independently in other parts of the NIST investigation for the aircraft impact analysis (Project 2) and thermal-structural response and collapse initiation analyses (Project 6).

B.3.1 Global Models of the Towers

Three-dimensional structural analysis computer models of the 110-story above grade structure and 6-story below grade structure for each of the two towers were developed. The global models for the towers consist of the major structural components and systems required to establish the baseline performance of the towers under gravity and wind loads.

In establishing the modeling techniques for the global models, parametric studies were performed to evaluate the behavior of typical portions of the structure (Section B.3.4). In addition, once the models were completed, order-of-magnitude checks were performed for gravity load, wind load, and eigenvalue results to check the accuracy of the models. More refined checks will be done in the third phase of this project on baseline performance analysis.

Components and Systems in the Towers Global Models

The models included all primary structural elements in the towers including exterior columns, interior (core) columns, exterior wall bracing in the basement floors, core bracing at the mechanical floors, core bracing at the main lobby atrium levels, spandrel beams, hat trusses, and rigid and flexible diaphragms representing the floor systems as developed in Section B.3.4 of this report.

Coordinate System, Nomenclature, and Models Assembly Overview

The extent of the data required to assemble the tower models dictated that the relational database capability of the WTC-DB be used (see Section B.2.3). The methodology for the development of the models using the relational database is described in this section.

Coordinate System—The coordinate system for the model geometry was based on the column layout from the original drawings. Figure B–14 shows the location of the X and Y axes for the global models and the floor models. The Z coordinates were based on actual elevations of the towers. The original column numbers were used throughout the models for member identification.

Nomenclature—A standard nomenclature for joints, frame names, and section names for use in the models was established. The nomenclature enables the user to know quickly where in the building a section is located by viewing any given piece of the model. Joint names generally included the column number, tower letter, and floor level. Frame element names generally included the joint name at the 'j' end (second node). Section names were based on the section as described in the drawing book and were repeated for each steel yield strength assigned for that section. Alternatively, where the section was unique to a particular member in the building, sections were named based on the frame member.

As an example, most nodes (or joints) in the tower models were named according to the following format:

- Column number
- Tower letter (A for WTC 1 and B for WTC 2)
- Floor level
- S for column splice nodes only
- J for spandrel splice nodes only

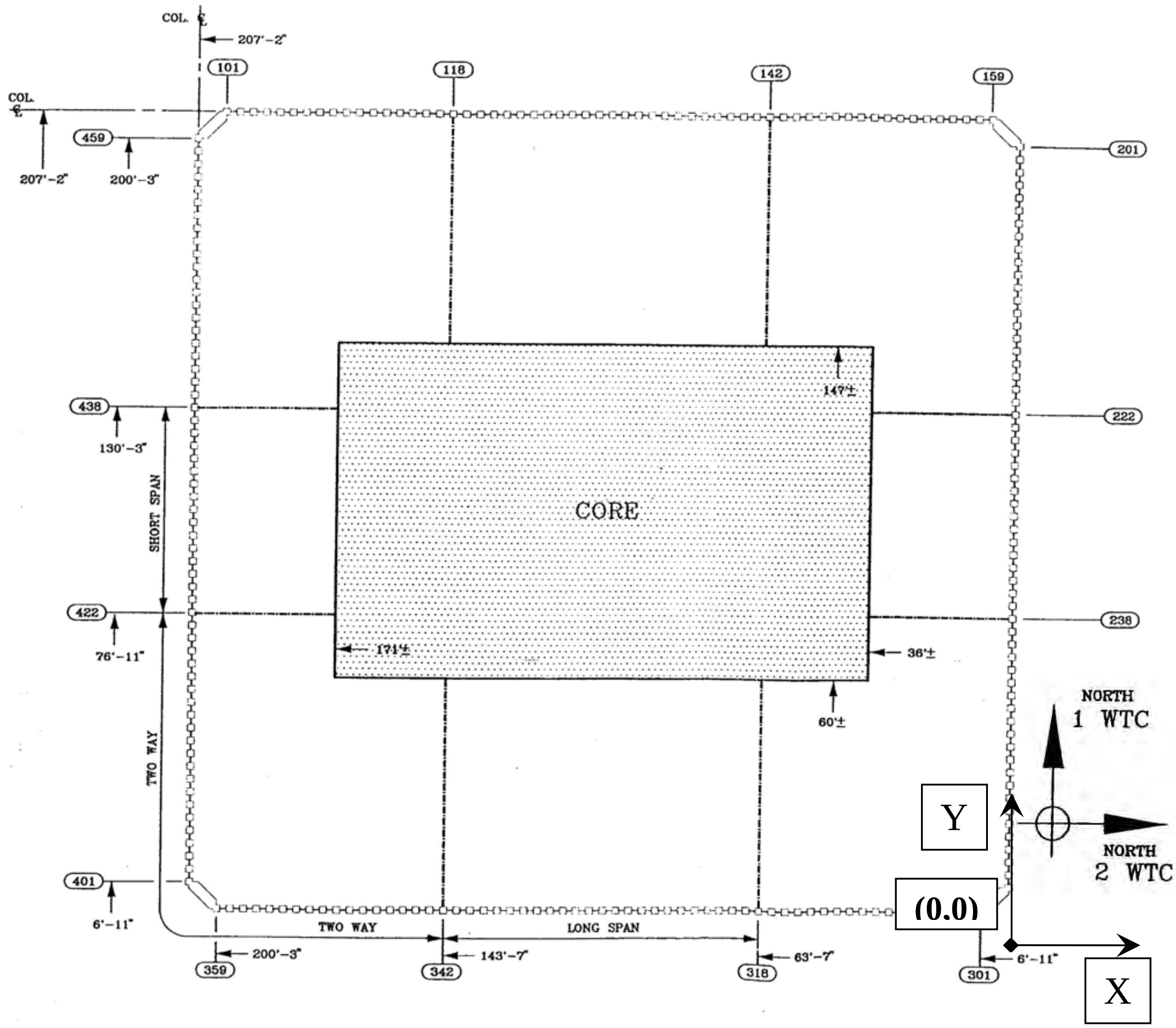


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Figure B–14. Global model coordinate axis location.

Figure B–15 illustrates the detailed frame and joint nomenclature for a typical exterior wall panel.

Model Assembly Overview—An overview of the assembly of the data into the tower model is described herein along with an expanded section on the programmatic assembly of the models.

Following a basic study of modeling techniques and testing of SAP2000, Version 8 input format and capabilities, it was determined that the best approach was to divide the model into six main parts and thenassemble them into a unified model. Manipulation of these individual parts was more efficient than attempting to build the whole model simultaneously.

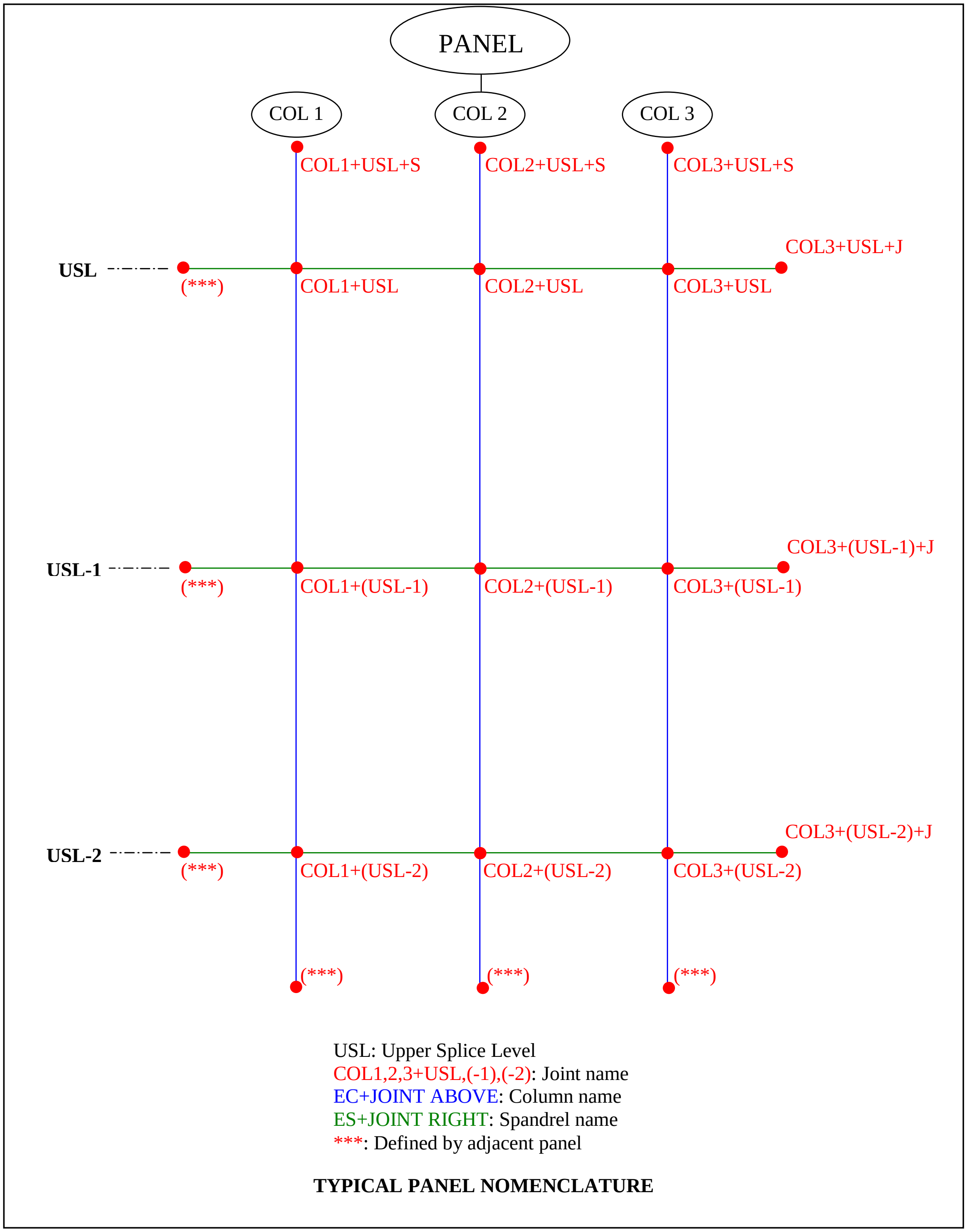

Figure B–15. Typical exterior panel nomenclature.

The six initial models were:

- Core columns
- Exterior wall, foundation to floor 7
- Exterior wall trees (floors 7 to 9)
- Exterior wall, floors 9 to 106
- Exterior wall, floors 107 to 110
- Hat truss

For the core columns and exterior wall at floors 9 to 106; most of the analysis input files were generated from queries of the WTC-DB. The other four parts of the model were assembled primarily in a more conventional manner.

Core columns and exterior wall panels (floors 9 to 106) were the greatest data-intensive challenges in the model development. Both areas included a large number of frame members and section and material property variations. The query files were used to gather the necessary data, and then simple computer programming was used to convert the data into the SAP input file format. Four main input tables for the SAP input file were developed programmatically:

- Joint coordinates table
- Connectivity-frame/cable table
- Frame section properties tables
  - Frame section properties 1—general
  - Frame section properties 5—nonprismatic
  - Section designer properties 04—shape I/wide flange
  - Section designer properties 05—shape channel
  - Section designer properties 11—shape plate
- Frame assignments table

The remaining data is added directly in the SAP model:

- Material properties
- Frame local axis
- Joint restraint

- Insertion point
- Constraint
- Gravity and wind load assignments

After the joint coordinates, connectivity, frame section properties, and frame assignments were complete for the six parts, the individual models were combined into a unified model. Rigid diaphragms, flexible diaphragms, core bracings, gravity loads, wind loads, and masses were then added to the unified model. After assembly of the model, the assignment of properties for selected model elements was spot-checked and the model was executed to verify its performance.

The development of the WTC 1 and WTC 2 models has been separate and consecutive endeavors. The lessons learned in the assembly of the WTC 1 model were applied to the development of the WTC 2 model. While there were only minor differences in the basic structural systems of the two towers, there were significant differences in section properties, material properties, and additional column transfers at the lower levels in WTC 2.

Isometric views of the complete WTC 1 model are illustrated in Fig. B–16. Elevations of the complete WTC 2 model are illustrated in Fig. B–17. A summary of the size of the global models of WTC 1 and WTC 2 is presented in Table B–2. The following presents the details of each of the six parts used in the development of the unified global models for WTC 1 and WTC 2.

Core Columns Modeling

Core column coordinates were tabulated based on the structural drawings. Column locations were typically referenced at their centerlines. Columns on lines 500 and 1000, however, were located in plan drawings along most of their height according to the face of the column to which the floor trusses frame (i.e., WTC 1 north face for 500 columns and south face for 1000 columns). The centerline of these columns was based on their dimensions given in the drawing books. Where these column centerlines varied along the height of the towers (typically 1 1/2 in. between three-story pieces), a representative location was chosen to define the column node. Thus, the column coordinate at floor 106 was used as a constant along the tower height because at this level, these columns align with the hat truss above.

The spandrel centerline elevation was selected as the representative floor elevation for exterior columns and used also for core columns. If there were no spandrels in exterior panels, reference elevations were used for the core columns.

There were over 5000 nodes in the core column model. This amount of data required that the *Interactive Database* input table be set up using a macro. These data were converted to text file format and later imported into SAP. Built-up sections were defined as *Section Designer* sections, and wide flange shapes were defined directly from “SectionWF1.pro” file (see Section B.2.5). All section names were identical to those in the database. Around 1280 *Section Designer* sections were defined in this model and imported through *Interactive Database* function of SAP2000 to the model.

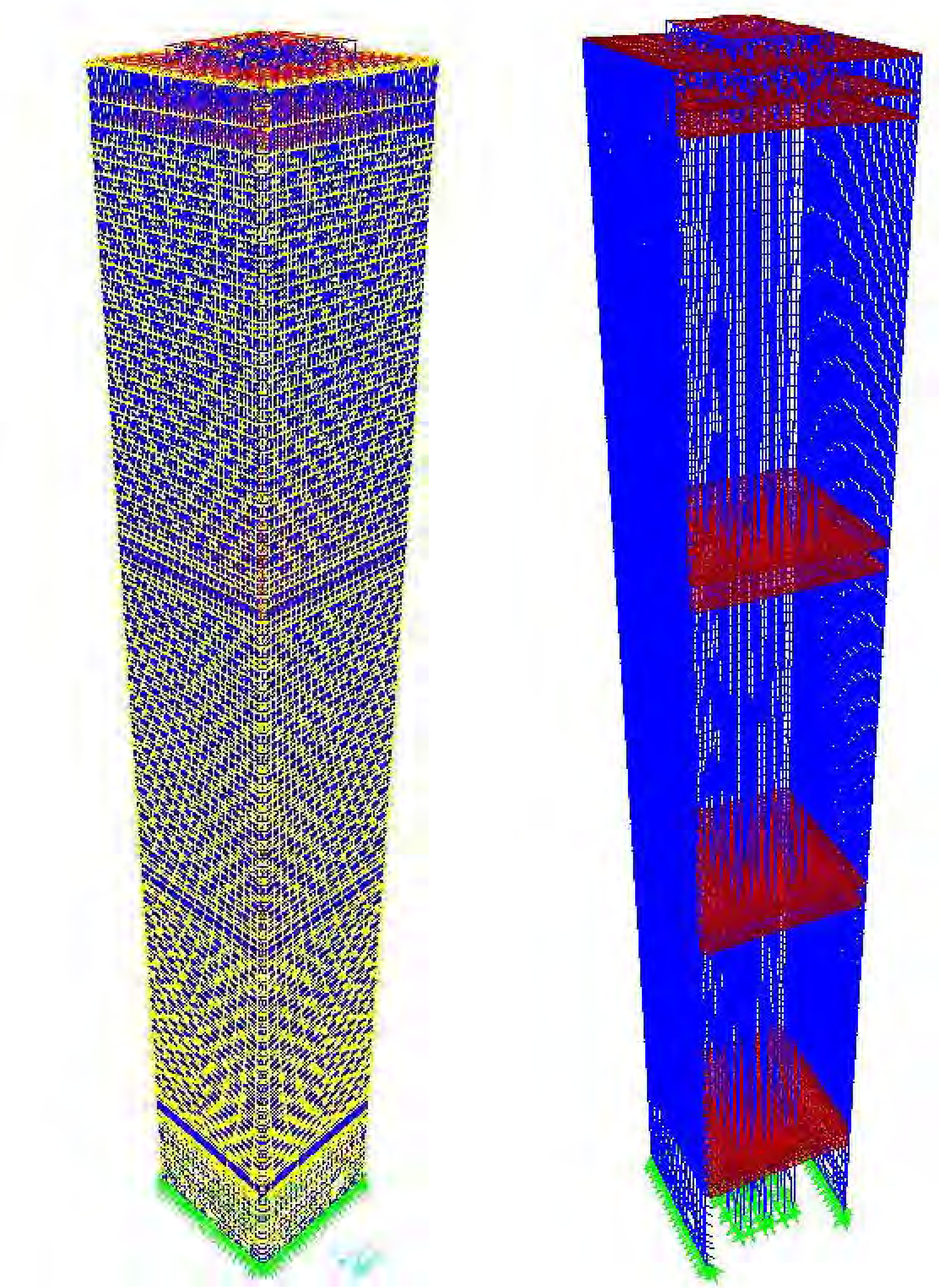

Figure B–16. Rendered isometric views of the WTC 1 model.

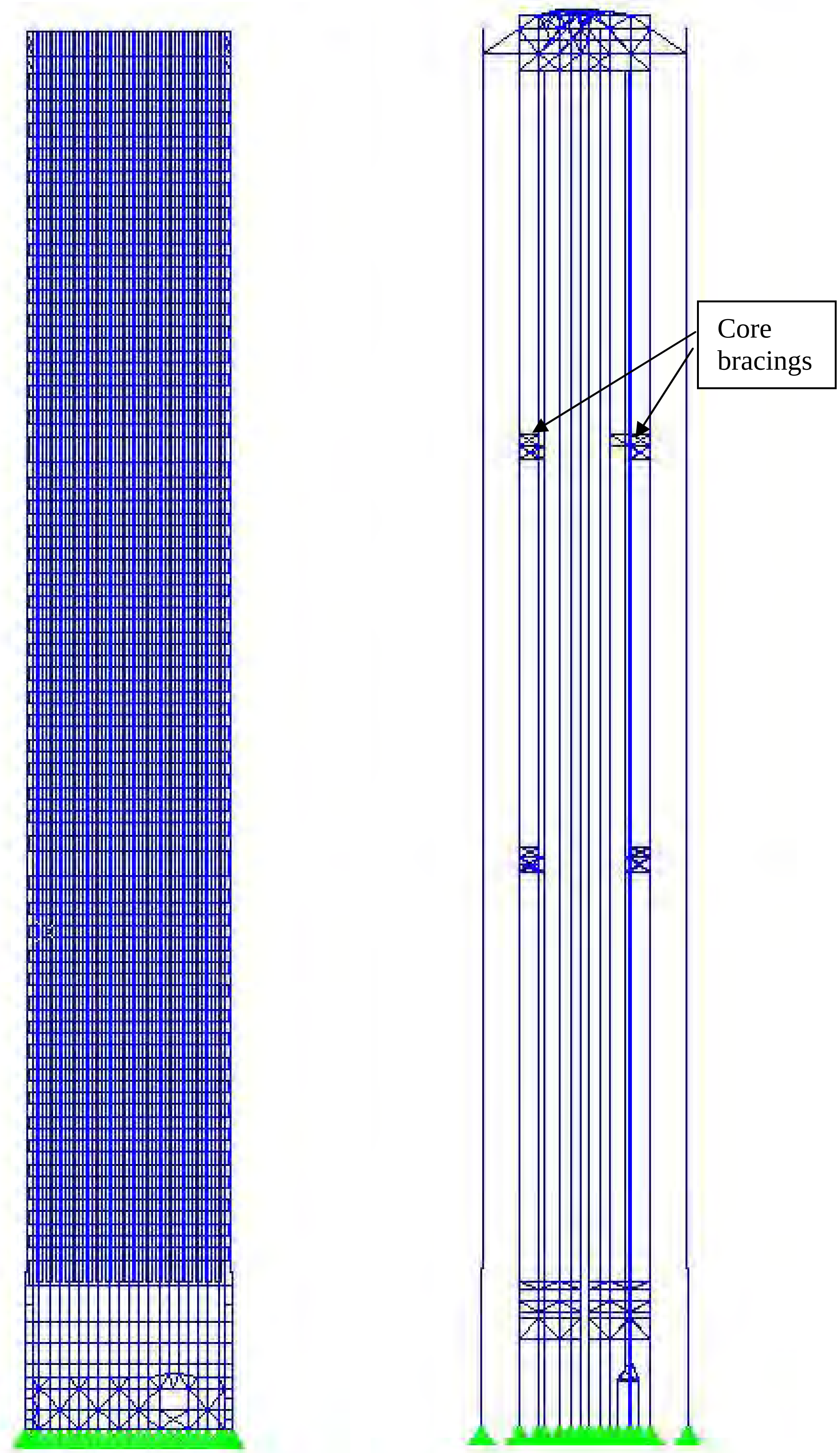

Figure B–17. Frame view of the WTC 2 model: exterior wall elevation and interior section illustrating the core columns, core bracing, and hat truss.

Table B–2. Approximate size of the reference structural models (rounded).

Model	Number of Joints	Degrees of Freedom	Number of Frame Elements	Number of Shell Elements	Total Number of Elements
WTC 1 global model[a]	53,700	218,700	73,900	10,000	83,900
WTC 2 global model[a]	51,200	200,000	73,700	4,800	78,500
Typical truss-framed model	28,100	166,000	27,700	14,800	42,500
Typical beam-framed model	6,500	35,700	7,500	4,600	12,100

a. Model does not include floors except for flexible diaphragms at 17 floors as explained later.

The core columns were defined as frame members spanning from node to node at the representative floor elevations. Splices in core columns occurred typically 3 ft above the floor level. In the models, however, the splice was considered to occur at the floor level, and nodes were only defined at these levels (i.e., typically at spandrel centerlines). Most three-story column pieces are unique, as tabulated in WTC-DB (Drawing Book 3). A section for each three-story piece was defined and then assigned to each of the three frame members that make up that column. Using the SAP shading feature to graphically show the section on the model, each frame was rotated to its proper orientation based on the structural drawings.

In the as-designed drawings, there were strap anchors connecting the core columns to the concrete floor slab to provide lateral bracing for the column. At floor 26 of WTC 1 the straps at column 901 were removed during a renovation project that was engineered by a firm other than LERA (see Section B.2.4). The loss of the straps at this location has been included in the model by releasing the column from the diaphragm in the direction of the straps.

Exterior Wall, Foundation to Floor 7 Modeling

The models of the exterior wall up to elevation 363 ft were developed manually, assigning joints and members connectivity as shown in the drawings. The elevation drawings show that below elevation 363 ft, columns were typically spaced at 10 ft and braced with spandrels and diagonals. Joints were defined at all locations where diagonals braced the columns. However, when coordinates were not given in the drawings, joint coordinates were determined based on the geometry of the diagonal. Details in WTC Drawing Book 1 show that the column-diagonal intersections had continuity. Joints at elevation 253 ft (level B-5) were defined only where the diagonals connect to the columns, since the tower floor did not frame into the exterior spandrels at that floor.

Where noted in elevation drawings, spandrel centerline elevations were used to define joint coordinates. Additionally, joints were defined at the spandrel splice midway between two columns at elevation 350 ft 3 in. (floor 3) and at elevation 329 ft 3 in. (floor 2) to allow for section type transitions.

The majority of the elements at these levels were defined as *Section Designer* sections, except for box shapes which were defined as “Box/Tube”. Channel shapes were defined directly from “SectionWF1.pro” file (see Section B.2.5). All section names were identical to those in the database. Around 200 sections were defined in this model using the *Interactive Database* function of SAP2000, which was used to import data into SAP2000.

Typical columns were connected from bottom to top and typical spandrels were connected from left to right. Frame names followed the nomenclature description presented earlier. The SAP2000 program allows assignment of rigid zone factors to frame end offsets to account for the overlap of cross sections. At the intersection of columns and spandrels, 100 percent rigidity for the column and the spandrels were assigned due to the large size of both columns and spandrels. Using the SAP shading feature to graphically show the section on the model, each frame was rotated to its proper orientation based on the structural drawings.

Refer to Fig. B–18 for a frame view and rendered view of the exterior wall (foundation to floor 9) of the WTC 1 model. The figure also shows the core columns and core bracings.

Exterior Wall Trees (Floor 7 to 9) Modeling

The panels of the exterior wall between elevation 363 ft and elevation 418 ft 11 1/2 in. are called exterior wall trees. At the exterior wall trees, the typical exterior wall columns transitioned from a spacing of 10 ft to a spacing of 3 ft 4 in. A typical exterior wall tree panel is shown in Fig. B–19. Each panel was divided into five different levels; level B, C, D, E, and F. For each panel in the model, the three exterior columns from above elevation 418 ft 11 1/2 in. continued down to level D. At that level, the three columns were connected by a horizontal rigid element to become one member, which extended down to elevation 363 ft.

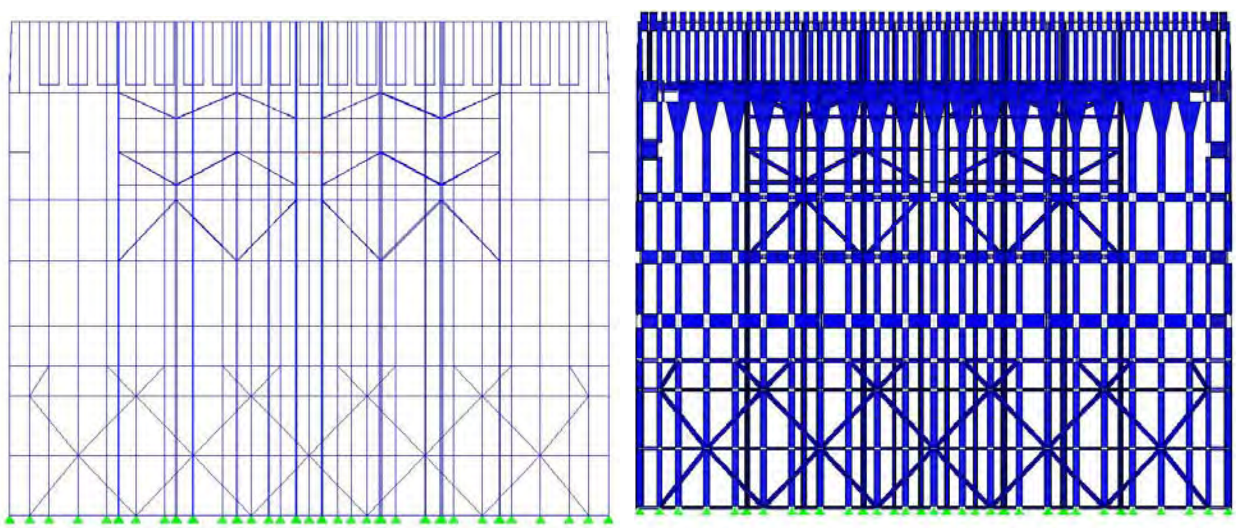

Figure B–18. Frame view and rendered view of the WTC 1 model (foundation to floor 9).

In the model, the tree was also the location where the column insertion point transitioned from the inside face (at the spandrel) of the upper column to the centerline of the lower column. Between levels B and D (see Fig. B–19), the location of the spandrel transitioned from 6 1/2 in. offset from the exterior column reference line to the center of this reference line. Within the floor 9 spandrel, the exterior columns taper; however, in the model, the tapering of the columns was not included because frame end length offsets were assigned to the columns to account for the rigidity of the spandrels.

Through the height of level C, the box-shaped columns tapered (Fig. B–19). In the model, non-prismatic members were used to model the tapering columns. The columns start to taper at the bottom of the spandrel at level B, and cease to taper at the top of the spandrel at level D. The dimensions of the columns at the spandrel edges were defined in the drawing book. In the model, the column extended from the centerline of the spandrel at level B to 1 ft below the top of the spandrel at level D (see discussion for level D below). Therefore in order to obtain the correct section properties along the length, the dimensions of the section at the joints were interpolated based on the dimensions of the section at the spandrel edges shown in the drawing book. The section properties of the tapering column were assumed to vary linearly between the two sections. Frame end length offsets were assigned to the columns to account for the rigidity of the spandrel at level B and the one foot dimension at level D.

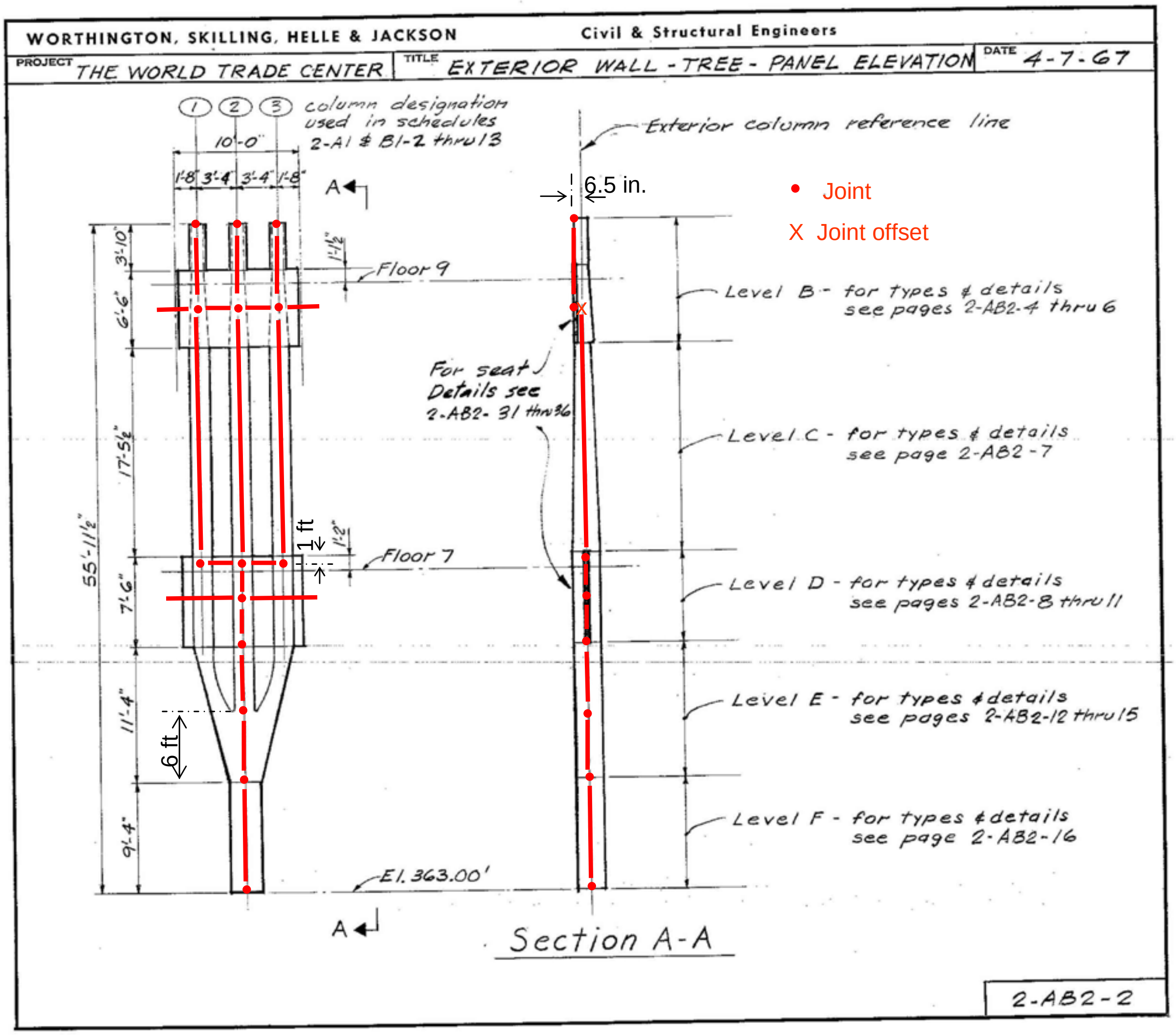

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Figure B–19. Exterior wall tree panel (taken from Drawing Book 2, page 2-AB2-2).

At level D, two transitions occurred in the model. The first transition was for the exterior columns, where the three columns coming down from level C were connected by a horizontal rigid element to become one member at the bottom of the tree. This frame member consisted of the three exterior columns and the spandrel plate. Another horizontal member of the same section properties with the spandrel plate was also defined and connected between the neighboring exterior wall trees. This member connected the neighboring exterior wall trees and provided lateral bracing for the columns. Frame end length offsets were assigned to the spandrel to account for the overlap of the spandrel plate with the frame member, which also included the spandrel plate. The transition of the three members into one member was assumed to occur at one foot below the top of the spandrel at level D to account for the fact that the spandrel becomes engaged with the exterior columns after being connected to the exterior columns for a certain distance. Hence, the joints were defined at one foot below the top of the spandrel at level D.

There was a second transition at level D (Fig. B–19). The nodes for the exterior wall columns were typically defined at 6 1/2 in. offset from the exterior column reference line. But for the joints at and below level D in the exterior wall tree, the joint coordinates were defined along the exterior column reference line. As a result, for the column member that framed between the nodes at levels B and D, a joint offset of 6 1/2 in. was assigned at the top of the member, while no offset was assigned at the bottom. The column therefore remained a vertically straight element while being connected to nodes that were not aligned vertically.

At level E, the exterior columns tapered and had two different types of cross section (Fig. B–20). For each panel, the exterior column transitioned from Section b–b in Fig. B–20 into a box-shaped column (Section c–c in Fig. B–20). The location of the transition between the different types of cross section varied for different column types from 5 ft 8 in. to 6 ft 4 in. measured from the bottom of level E. In the model, the transition was assumed to be at 6 ft measured from the bottom of level E. For each panel, the exterior column at level E was modeled as two nonprismatic members. The top section of the first non-prismatic member consisted of three box-shaped columns and a middle plate, while the bottom section was a box-shaped column (Section c–c in Fig. B–20). The properties were assumed to vary linearly between the two sections. The second nonprismatic member was a tapering box shaped column (Section c-c in Fig. B–20), and again, the properties were assumed to vary linearly between the two sections. At level F, the exterior wall tree columns were prismatic box-shaped columns.

The final model of a typical tree is illustrated in Fig. B–21.

Exterior Wall (Floor 9 to 106) Modeling

In plan, column and spandrel members connected at nodes located at the outside face of the spandrel, 6 1/2 in. from the exterior column reference line (see Fig. B–22). The columns were offset horizontally, or ‘inserted’ at this node, using an insertion point located at the centerline of plate T3. Insertion points were not adjusted for spandrel thickness. With this modeling, gravity and wind loads can be applied at the spandrel location.

In elevation, the columns and spandrel members connected at the spandrel centerline, typically 12 1/2 in. below the reference floor elevation (Fig. B–22). The spandrels were then located correctly without the need for offsets to be defined. The effect of applying loads at both the spandrel centerlines and the reference floor elevations was studied, and it was found that it has a negligible difference in spandrel stresses.

For typical exterior wall panels (i.e., three columns wide by three stories high), nodes at five elevations were defined. The models included nodes at the three representative floor levels (defined at the spandrel centerlines) as well as the upper and lower column splices. Diaphragms were assigned to all nodes at floor levels where concrete slabs exist, to represent the high in-plane stiffness of the concrete floor slabs.

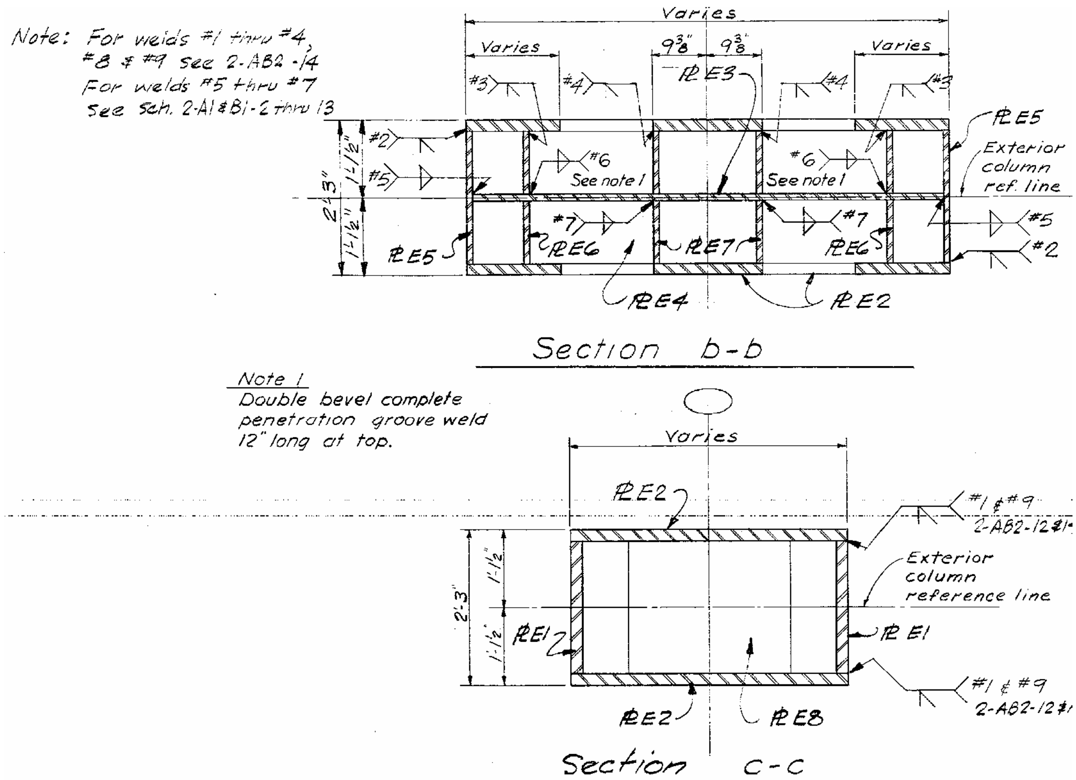

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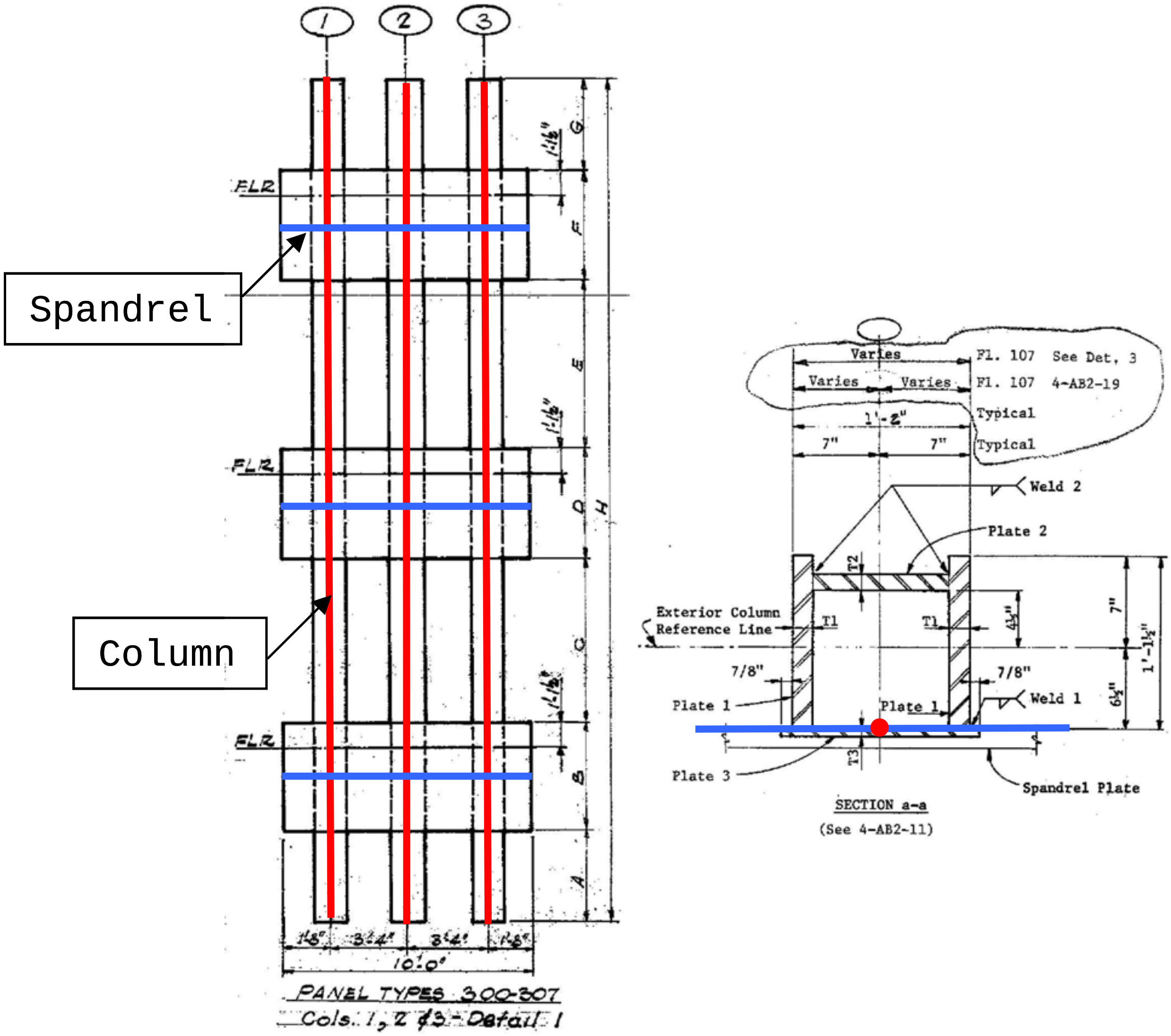
Figure B–20. Exterior wall tree: as-built cross sections for level E (taken from Drawing Book 2, page 2-AB2-13).

Original drawing used with permission from PANYNJ.

Figure B–21. Frame view and rendered view of an exterior wall tree.



Original drawing used with permission from PANYNJ.

Figure B–22. Typical WTC tower exterior wall panel.

The SAP2000 program allows assignment of rigid zone factors to frame end offsets to account for the overlap of cross-sections. In the global model, 50 percent rigidity for the column and 100 percent rigidity for the spandrels were assigned for the typical exterior wall panels to match the lateral deflection of the detailed shell model of the panel based on the parametric study results (see Section B.3.4). It was also found that, due to the relatively large depth of the spandrels and the close spacing between the columns, the spandrels contribute to the axial stiffness of the columns in the panels. This contribution was estimated to range from 20 percent to 28 percent increase in the vertical stiffness of the panels. Therefore, a frame property multiplier for the exterior wall column's cross-sectional area was used to provide a 25 percent increase in columns' axial stiffness (see Section B.3.4).

For exterior wall corner panels, 25 percent rigidity for the column and 50 percent rigidity for the spandrels were assigned based on the parametric study results (see Section B.3.4). Also, an area modifier was used to provide a 25 percent increase in the axial stiffness of the two continuous columns of the corner panels (Section B.3.4). No modifier was used for the 100, 200, 300, and 400 series intermittent columns.

Exterior column types were defined in Drawing Book 4. A few types (100 series typical, 300 series at mechanical floors, and 400 to 500 series at corners) repeated extensively throughout the building, with steel yield strengths that vary from 36 ksi to 100 ksi. Since SAP does not allow for the assignment of material properties at the member assignment stage, the number of different steel strengths was determined for each exterior column type, and sections were defined for each. The section name included the section number and the yield strength as tabulated in the drawing books.

Typical spandrels and corner panels were defined as rectangular shape and *Section Designer* section with stiffener, respectively. The top and bottom stiffener of each corner spandrel were included in both the parametric study and the global models. The detail shows that the stiffeners were 6 in. plates of thickness matching plate T2 in the corner column.

Exterior Wall (Floor 107 to 110) Modeling

Spandrel depths varied at floors 108 and 110. A weighted average of spandrel depth was determined in order to define the average centerline elevation of the spandrels and, therefore, the node elevation for the entire floor.

For the 7 × 5 structural tube sections that were used in these floors, sections from the current AISC Manual were assigned, and modification factors of 1.04 were applied to the section properties. The modifiers were used to match the section properties from the 6th Edition AISC Manual.

The exterior wall members from floors 107 to 110 were typically rolled shapes with F_y =42 ksi or F_y =50 ksi. Where not shown in the drawings as F_y =50 ksi, F_y =42 ksi was used.

Hat Truss Modeling

In both WTC 1 and WTC 2, a truss system referred to as a ‘hat truss’ was constructed between floor 107 and the roof. The hat truss system was intended to support the load of the antenna on top of the tower and to interconnect the exterior walls to the core. The hat truss was made up of eight trusses spanning perpendicular to the long-direction of the core and six trusses spanning perpendicular to the short-direction of the core (refer to Figs. B–23 and B–24).

Frame members between floors 107 and 110 were assigned to the model according to plan and elevation drawings of the hat truss. Node locations were set to coincide with the centerline of spandrels at the exterior wall. Columns, diagonals, and beams were included in the model. All columns and diagonals shown in drawings SA/B–400 through SA/B-404 were included in the model. Floor beams that did not participate in the hat truss system were not included in the model, unless they were used to transfer truss chords or core columns. Flexible floor diaphragms were used in this area.

Coordinates were generally not given at floor 109, as this level does not contain a complete concrete floor slab. The geometry of the diagonals, columns, and beams was used to determine the location of the node where the diagonal would intersect floor 109. Unless otherwise noted in the drawings, diagonals and

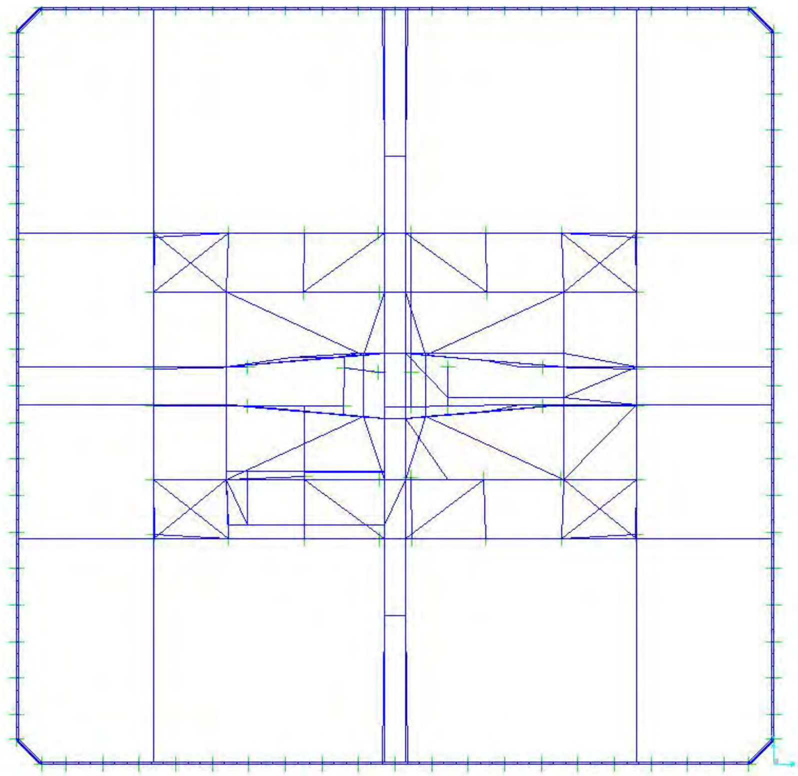


Figure B–23. As-modeled plan of the WTC 1 hat truss.

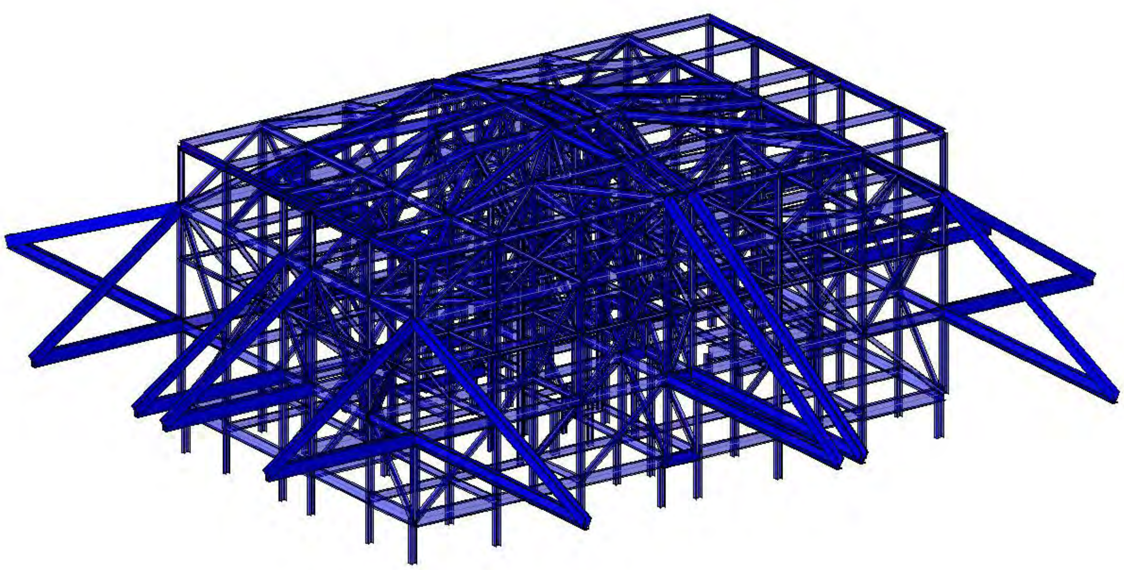


Figure B–24. Rendered 3–D model of the WTC 1 hat truss (prior to assembly in the unified model).

columns were assumed to be non-composite and floor beams were assumed to be composite. Hat truss diagonals, main chords, and main columns were modeled with continuous joints. Hat truss beams, however, had pinned ends.

Flexible and Rigid Floor Diaphragm Modeling

For most floors, rigid diaphragms provide for a sufficiently accurate representation of the flow of forces and deformations for global structural response. This is a customary engineering practice. In cases where the flow of forces and deformations would be affected significantly by the use of rigid diaphragms, the floors were modeled as flexible diaphragms.

The floor models described in Sections B.3.2 and B.3.3 were used to develop the flexible diaphragm stiffness utilized within the global models. Section B.3.4 outlines the study for the determination of the in-plane diaphragm stiffness of the detailed floor models, using that in-plane stiffness to arrive at an equivalent shell element floor model. The equivalent shell element floor was used to represent the in-plane floor stiffness in the global model. The shell elements attached to all exterior wall columns and core columns.

Flexible diaphragms were used at the floors of the towers in the core of the atrium area, in the mechanical floors, and in the floors of the hat trusses. The floors modeled using flexible diaphragms are floors 3, 4, 5, 6, 7, 9 (atrium levels); 41, 42, 43, 75, 76, 77 (mechanical levels); 107, 108, 109, 110, and roof (hat truss region) of both towers.

Initial Verification of Global Models

Several steps were taken to verify the model input. SAP2000 Version 8 offers a ‘shading’ option once a model has been built with frame section assignments. This allows the user to view the members as the program has interpreted their input. The shading option was helpful for using section-designed shapes, and for verifying the orientation (i.e., local axes) of members. Note that shading is not correct when two *Section Designer* sections are used in non-prismatic members, so orientations for these sections were verified by reviewing their local axis member properties. The work was independently reviewed by engineers not associated with the initial model development.

Once the models were completed, checks for gravity loads, wind loads, and eigenvalue results were performed. The overall performance of the tower models under these loads was found to be reasonable by checking deformations, stresses, reactions, etc. More refined checks will be done as part of the third phase of this project on baseline performance analysis. The natural periods will be calculated using mass properties estimated from realistic loads on the towers as part of the baseline analysis. Calculated natural periods will be compared with the measured periods of WTC 1.

B.3.2 Typical Truss-Framed Floor Model—Floor 96A

In order to select the typical truss-framed floor within the expanded impact and fire zones of both towers, the drawings for floors 80 to 100 were reviewed to identify structural similarities. It was found that floor 96 of WTC 1 (96A) represented the typical truss-framed floor in the expanded region for WTC 1 (floors 89A to 103A). The lone exception in this region of WTC 1 was floor 92 which had an increased dead load capacity required for the support of secondary water lines.

Floor 96A was also representative of the typical truss-framed floor in the expanded region for WTC 2 (floors 74B–88B). Specifically, floor 96A was similar to the truss framing at floor 74B and floors 84B through 88B. Floors 78B and 79B were sky lobby and upper escalator floors, respectively. Both contained long span trusses which were similar to floor 96A, but also contained beam-framed floor construction in the entire short span area (where the escalators were located). Floors 80B through 83B had beam framing in place of a single truss panel in the short span area, while the remaining area contained trusses which were similar to floor 96A.

Based on the above discussion, floor 96 of WTC 1 was selected as the overall representative truss-framed floor for the majority of the expanded impact and fire zone in both towers and is described in the following sections (see Fig. B–25). An isometric view of the typical truss-framed floor model is illustrated in Fig. B–26. Table B–2 includes a summary of the size of the 96A floor model. The following presents the major structural systems and components of the truss-framed floor model.

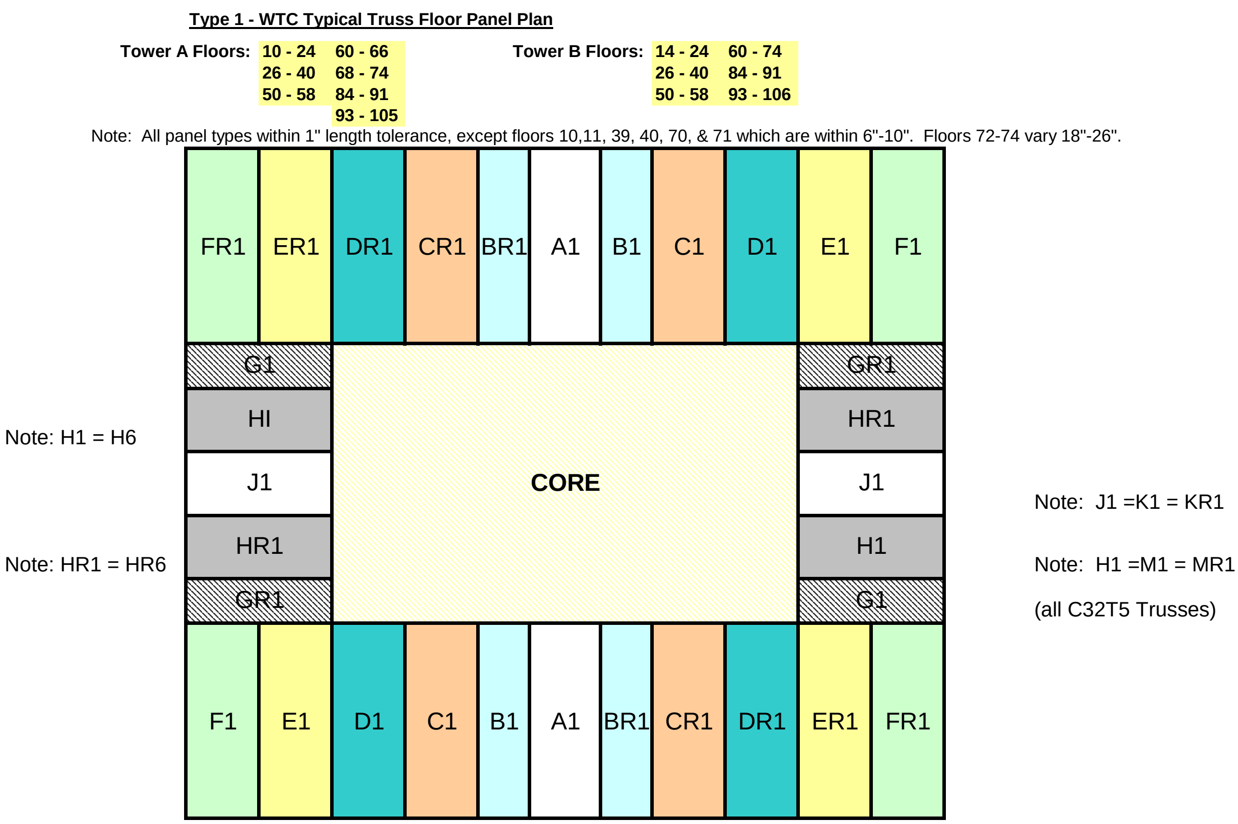


Figure B–25. Typical truss-framed floor panels arrangement.

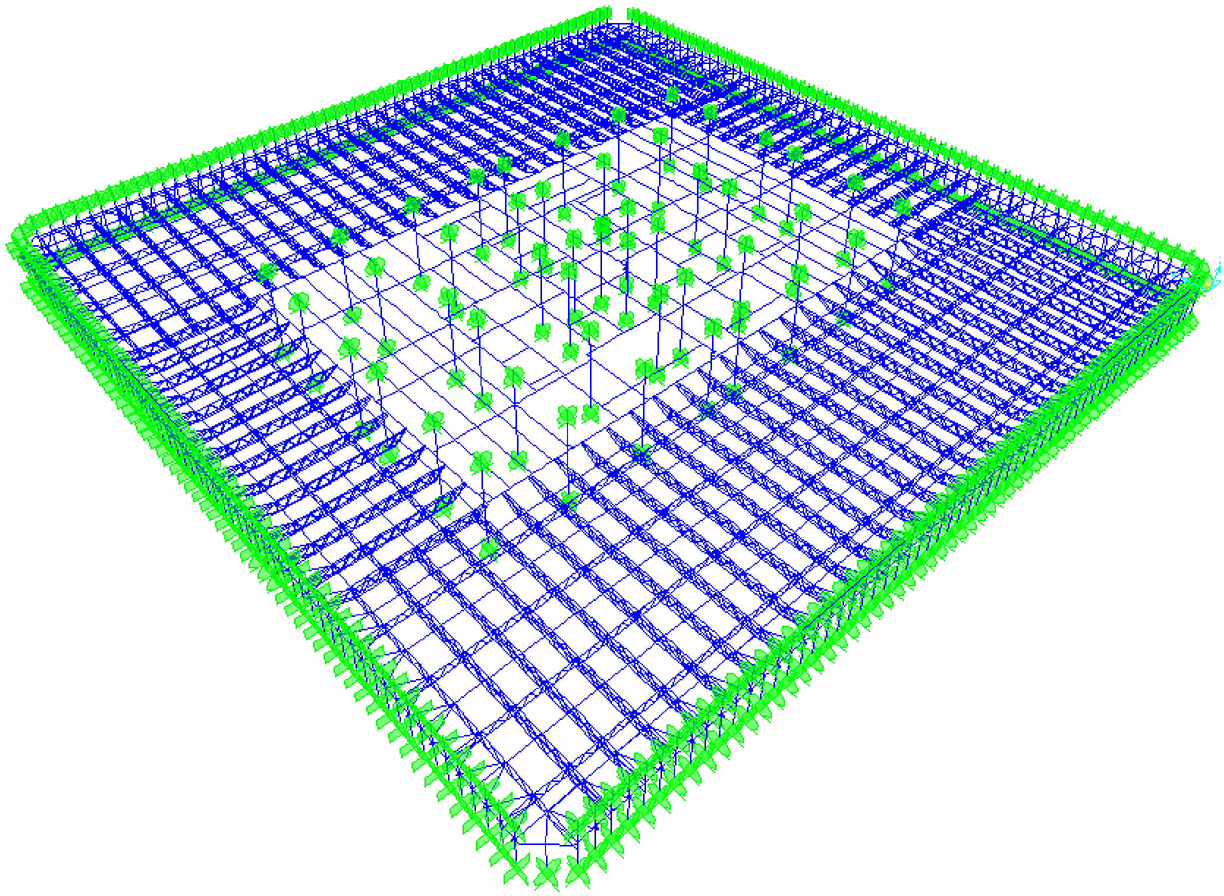


Figure B–26. Typical truss-framed floor model (floor 96A), slab not shown.

Primary Trusses

The primary trusses consisted of double angle top and bottom chords which were 29 in. out-to-out of the chords. The trusses acted compositely with a 4 in. concrete slab on 1 1/2 in. metal deck. For a typical long-span truss, C32T1, the top chord consisted of two angles 2 by 1.5 by 0.25 in., short legs back-to-back (SLB), and the bottom chord consisted of two angles 3 by 2 by 0.37 in., SLB. The distance between the centroid of the two chords was calculated to be 28.05 in. The distance from the centroid of the top chord to the neutral axis of the transformed composite slab with top chord was calculated to be 1.93 in. The sum of (28.05 + 1.93) is 29.98 in. (Fig. B–27). In the model, therefore, 30.0 in. was taken as the typical distance between the top and bottom chords for both short- and long-span primary trusses.

In the long-span truss zone, the two individual primary trusses, which were part of the same floor panel and attached to the same column, were separated (typically) by a distance of 7 1/8 in. At the joint between panels, the distance between the abutting long-span trusses was 7 1/2 in. Therefore in the model, 7 1/2 in. was used as the spacing between all long span primary trusses. In the short-span truss zone, two individual trusses which attached to the same column were separated by a distance that varied between 4 7/8 in., 5 in., and 5 1/4 in. In the model, the typical spacing between all short-span double trusses was 5 in. The long span trusses in the two-way zone had an as-modeled length of 58 ft 10 in. while the long span trusses in the one-way zone had an as-modeled length of 59 ft 8 in.

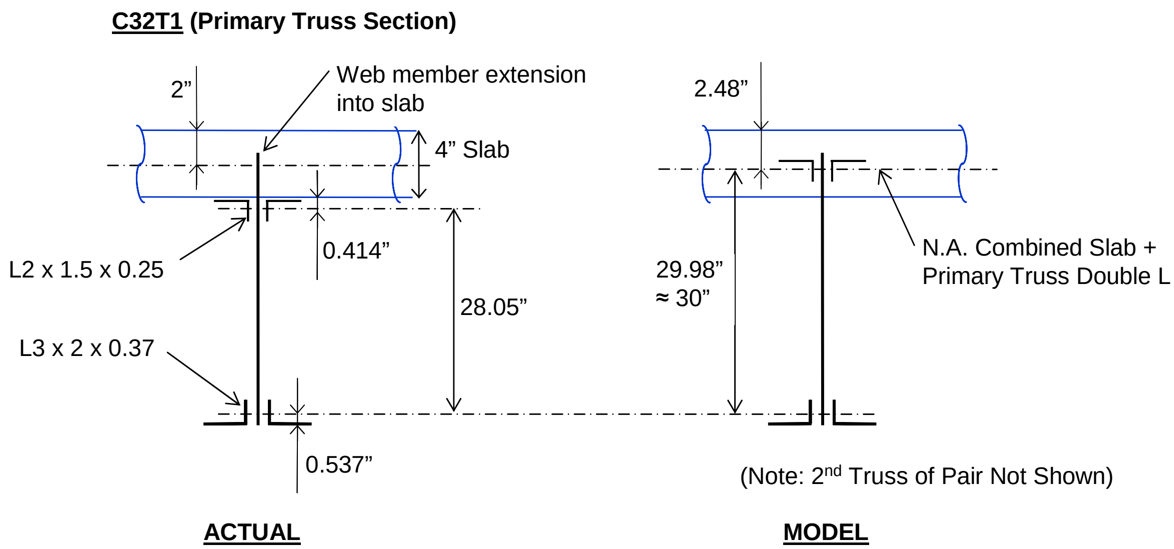


Figure B–27. Typical primary truss cross-section, as-built and as-modeled transformed truss work points.

The diagonal web bars for the primary trusses were most often 1.09 in. diameter bars. Therefore, for double angle shapes in the primary trusses, 1.09 in. is taken as the distance between the two angles. This holds true for primary trusses where bar diameters varied between 0.92 in. and 1.14 in.

The as-built truss diagonals had end fixity, but were considered pinned for the analysis. Pinning the diagonals is conservative and provides an upper bound of the gravity load stresses. To mitigate the effect of the pinned member approach, end length offsets were used for the truss diagonals to compensate for the difference in the as-built diagonal unbraced length and the model unbraced length. The as-built unbraced length for a typical diagonal in a primary truss was 32.4 in., while the modeled member length was 36.05 in., and therefore, an end offset of 1.8 in. was used at both ends. Similarly, for the bridging trusses, the actual unbraced length for a typical diagonal of a bridging truss was 29 in., while the modeled length was 30.66 in. Therefore, an end offset of 0.83 in. was used at both ends. A rigid zone factor of 100 percent is used for all offset zones.

In the model, the deck support angles, typically 3 by 2 by 0.75 in. were located in the same plane as the combined truss top chord and composite slab centroid.

Bridging Trusses

The bridging trusses were 24 in. deep, edge-to-edge, with double angle chords. For a typical bridging truss, 24T11, the top and bottom chords consisted of two angles 1.5 by 1.25 by 0.23 in., SLB. The distance between the centroid of the two chords was 23.26 in. The distance used as the offset between the top and bottom chords for all bridging trusses was taken as 23.25 in. (Fig. B–28). The distance between the work points of the top chord of the bridging truss and the top chord of the primary trusses and equivalent slab plate for 24T11 was calculated to be 3.39 in. This distance was selected for all bridging trusses to be 3.375 in. As in the as-built structure, the bridging truss was not connected along its length to the slab shell elements in the model. At the intersection of the top chords of the primary and the bridging trusses, the intersection was modeled using vertical rigid links, connected in turn to the slab shell elements representing the concrete slab.

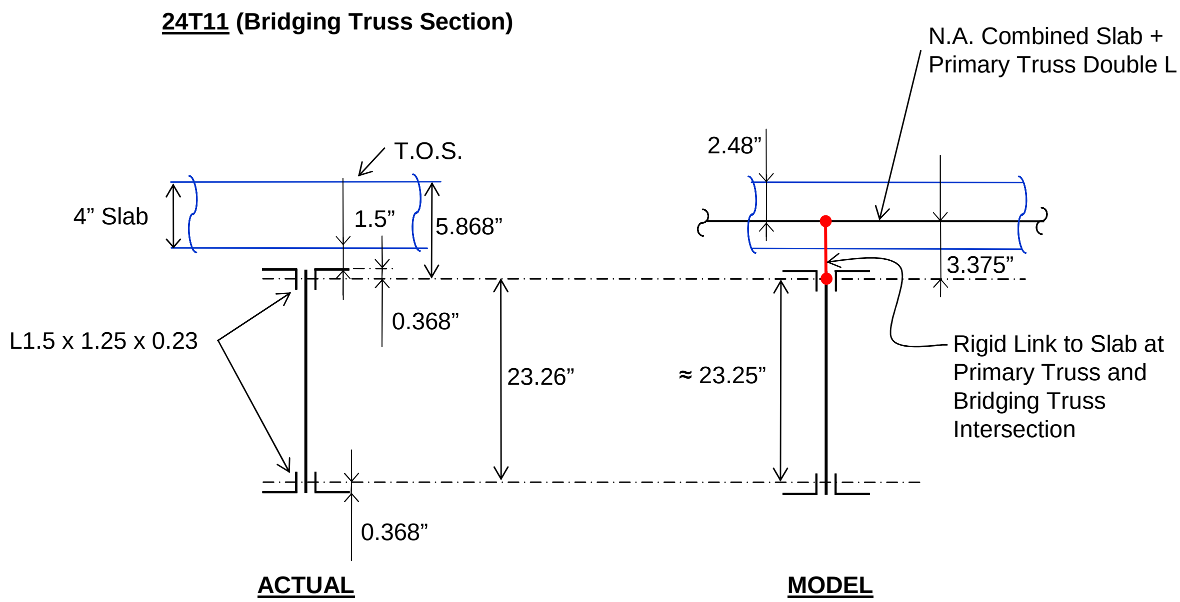


Figure B–28. Typical bridging truss cross–section, as-built and as-modeled transformed truss work points.

The bottom chord of the primary trusses was connected to the bottom chord of the bridging trusses along the length of the primary trusses only on column lines 111, 149, 311, and 349. The connection consisted of double angles 2 by 1 1/2 by 0.25 in. These connection angles were included within the model.

For bridging trusses in the model, a 0.75 in. angle gap was used for trusses with web bar diameters that varied between 0.75 in. and 0.98 in.

Truss Member Cover Plates

In 30 percent of the floor area, truss members were supplemented with cover plates. The members with additional plates included top chords, web members, and most typically bottom chords. Section properties were calculated with SAP *Section Designer*. The primary truss top chords were reinforced with an additional set of double angles at truss end connections. At these locations, the work points for the section were located at the centroid of the composite double angle and concrete slab.

The Laclede shop drawings indicated plates 3/8 in. by 3 in. connecting the bottom chord of the primary truss pairs together at each end and where intersected by a bridging truss. These plates were included in the model.

Viscoelastic Dampers

Viscoelastic dampers were located where the bottom chords of the long span, short span, and bridging trusses intersected the exterior columns. The dampers were defined in Drawing Book D. The dampers resisted static and quasi-static loads (such as gravity loads) at the time of load application. Immediately following load application, the dampers shed load until the stress in the dampers was dissipated. A placeholder element was located in the model at the damper location.

Strap Anchors

Exterior columns not supporting a truss or truss pair were anchored to the floor diaphragm by strap anchors. These strap anchors were connected to the columns by complete penetration welds. The strap anchors were then connected to the slab with shear stud connectors and to the top chords of the trusses by fillet welds. The straps were included in the model and located in the plane of the centroid of the composite top chord. Also, in the model the work points intersected with the centerline of the column and used a rigid link to attach back to the spandrel (see Fig. B–29).

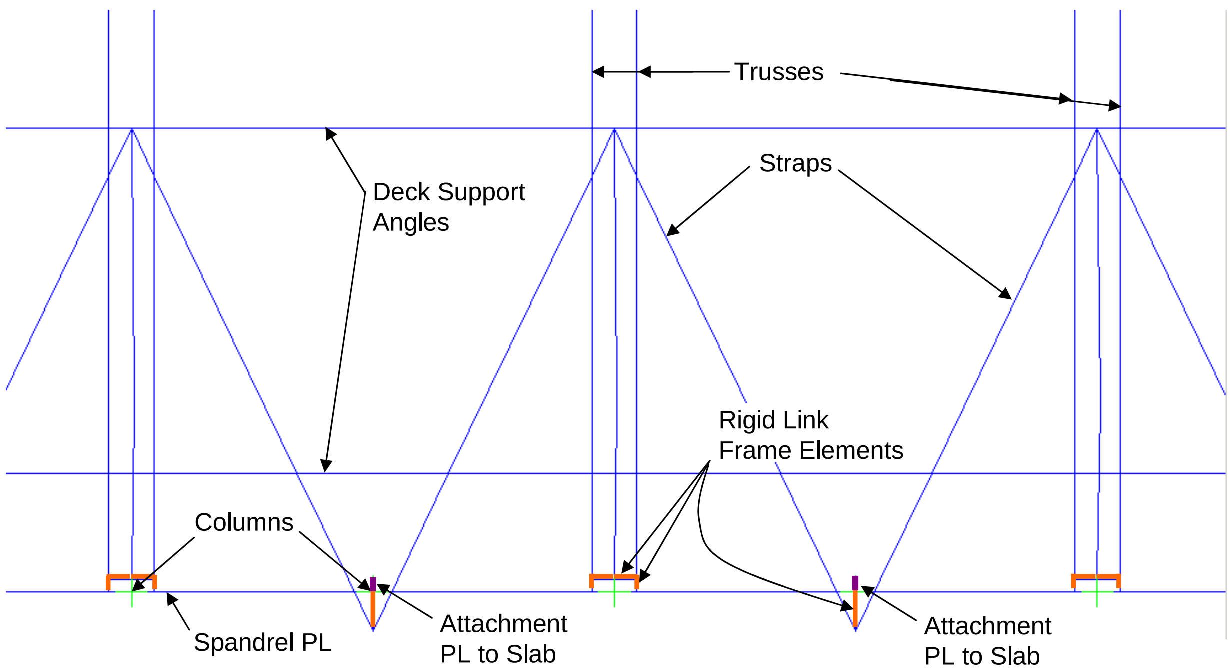


Figure B–29. Strap anchors modeling, slab not shown.

Concrete Slab and Metal Deck

Outside the core, the primary trusses acted compositely with the 4 in. concrete slab on 1 1/2 in. metal deck. In the model, the average depth of the slab plus deck was modeled as 4.35 in. The concrete slab consisted of lightweight concrete with a self-weight of 100 pcf and a design compressive strength, f'_c= 3,000 psi. The concrete modulus of elasticity, E_c, used for modeling is 1,810 ksi, and the calculated modular ratio, $n=E_s/E_c$, is taken as 16, where E_s is the steel modulus of elasticity. These values are consistent with those included within the WTC Structural Design Criteria Book.

Typically, inside the core, the beams acted compositely with a 4 1/2 in. formed concrete slab. The concrete slab consisted of normal weight concrete with a self-weight of 150 pcf and a design compressive strength, f'_c= 3000 psi. The concrete modulus of elasticity, E_c, used for modeling was 3,320 ksi and the calculated n ratio, E_s/E_c, was taken as 8.7.

The floors of the WTC towers had an in-floor electrical distribution system of electrified metal deck and trench headers. The effects of the in-slab trench headers were accommodated by reducing the slab shell element thickness. A 1 ft 8 in. wide shell panel (the typical truss-floor shell mesh size) was reduced in thickness from 4.35 in. to 2.35 in. or 1.35 in. at the trench header locations per drawing SCA–109 (Floor 96A Structural Concrete Floor Plan).

Initial Verification of the 96th Floor Model

Several steps were taken to verify the model input. SAP2000 Version 8 offers a 'shading' option once a model has been built with frame section assignments. This allows the user to view the members as the program has interpreted their input. The shading option was helpful for using section designed shapes, and for verifying the orientation (i.e., local axes) of members. The work was independently reviewed by engineers not associated with the initial model development.

Once the model was completed, checks were performed for gravity loads. All superimposed dead loads and live loads included in the model were based on WTC Design Criteria; self weight is accounted for by SAP2000. To justify the modeling assumptions, several studies were performed to compare stress results to hand calculations for representative composite sections. Hand calculations estimate deflections and member stresses for a simply supported composite truss under gravity loading. For the composite truss sections, the steel stress results were within 4 percent of those calculated by SAP2000 for the long-span truss and 3 percent for the short-span truss. Deflections for the beams and trusses matched hand calculations within 5 to 15 percent.

B.3.3 Typical Beam-Framed Floor Model—Floor 75B

As described in Section B.3.2 for truss-framed floors, the structural drawings were reviewed to identify structural similarities between the beam-framed floors within the expanded impact and fire zones of both towers. It was found that floor 75 of WTC 2 (75B) represents the typical beam-framed floor in the expanded impact zone for WTC 2 (floors 74B to 88B). There were no beam-framed floors within the expanded impact zone of WTC 1.

Floors 75 and 76 of WTC 2, lower and upper mechanical equipment (MER) floors, respectively, were typical of the lower three mechanical equipment floor pairs in both towers (floors 7 and 8, 41 and 42, and 75 and 76 for both WTC 1 and WTC 2). Floor 77 of WTC 2, a lower escalator floor, was a beam-framed floor similar to the lower floor of the mechanical equipment floor pairs, i.e., floor 75B.

Based on the above discussion, floor 75 of WTC 2 was selected as the overall representative beam-framed floor for the expanded impact and zone in both towers and is described in the following sections (see Fig. B–30). An isometric view of the typical beam-framed floor model is illustrated in Fig. B–31. Table B–2 includes a summary of the size of the 75B floor model. The following presents the major structural systems and components of the beam-framed floor model.

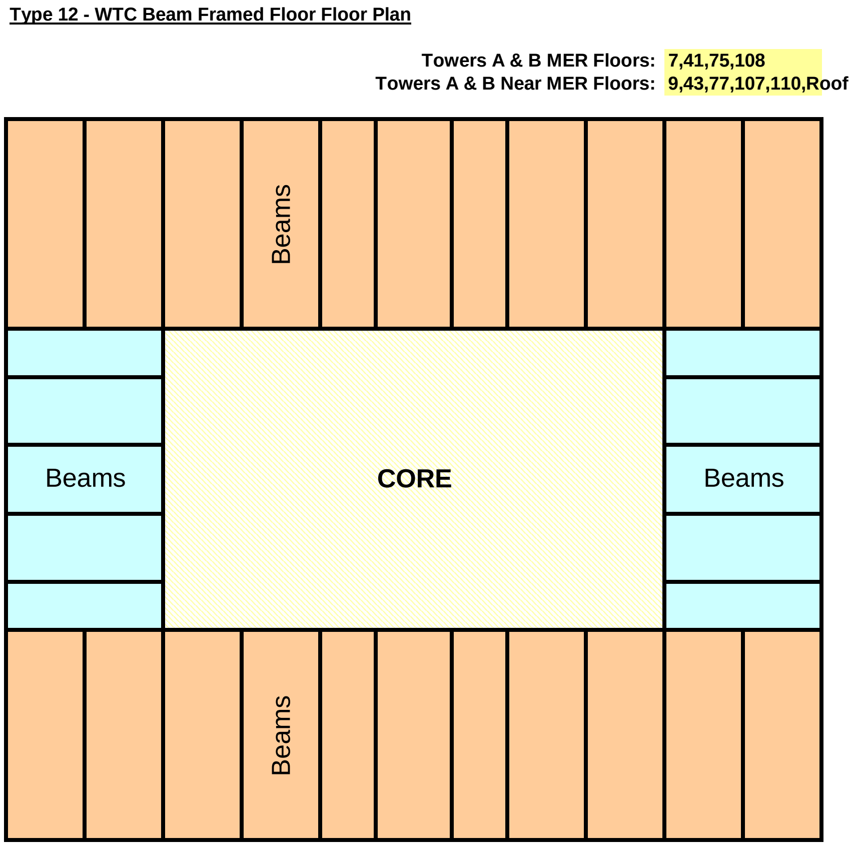


Figure B–30. Typical beam-framed floor arrangement.

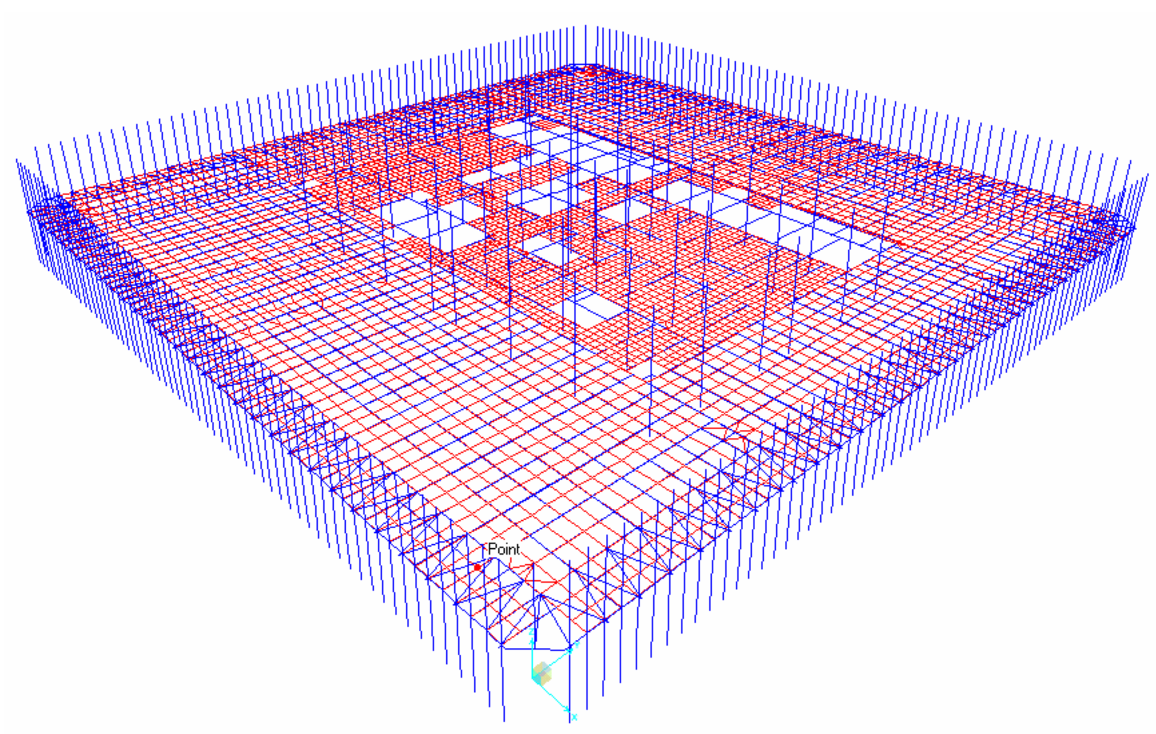


Figure B–31. Typical beam-framed floor model (floor 75B).

Composite Beams

The beams in the model were located at the elevation of the centerline of the concrete slab. The insertion point for the beams was set at the beam top flange, and then the beam was offset down by one-half the thickness of the slab. The beam was rigidly linked with the slab to simulate the composite action. This option provided for accurate estimation of the composite stiffness of the floor.

For beams with cover plates, the properties were calculated by SAP *Section Designer*, and the slab, beam, and reinforcing plates were rigidly linked.

Horizontal Trusses

Exterior columns which did not support a beam were connected to the floor for bracing purposes by horizontal trusses. These exterior horizontal trusses were anchored to the columns with complete joint penetration welds. The horizontal trusses were then connected with shear stud connectors to the slab. The truss angles (typically 4 by 4 by 5/16 in.) were then connected to the top flange of the beams. In the model, the work points intersected with the centerline of the column and used a rigid link to attach back to the spandrel. The truss members were located in the plane of the centroid of the composite top chord (see Fig. B–32).

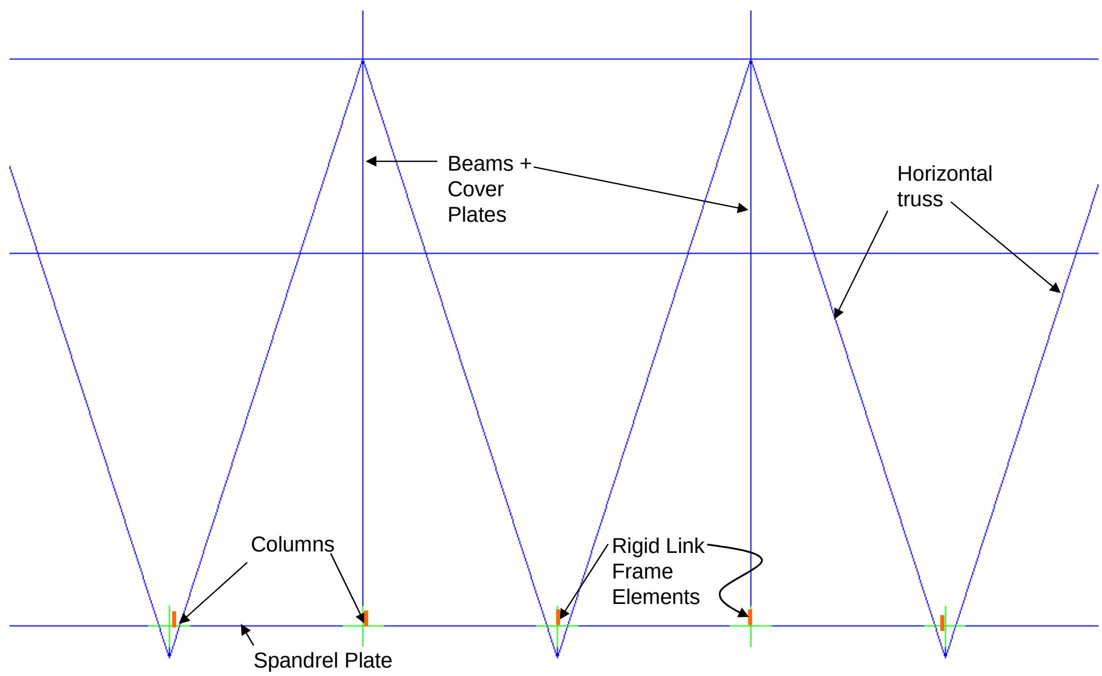


Figure B–32. Horizontal truss modeling, slab not shown.

Concrete Slab and Metal Deck

Outside the core on the mechanical floors, the beams acted compositely with a 5 3/4 in. concrete slab on 1 1/2 in. metal deck. The average cross-sectional depth of the slab in the model was taken as 6.1 in. The concrete slab consisted of normal weight concrete with a self-weight of 150 pcf and a design compressive strength of typically f'_c= 3,000 psi. The concrete modulus of elasticity, E_c, used for modeling is 3,320 ksi and the calculated modular ratio, n, is taken as 8.7.

Typically, inside the core, the beams acted compositely with a 6 in. formed concrete slab. The concrete slab consisted of normal weight concrete with the same properties as concrete outside the core.

The mechanical floors had a 2 in. maximum depth topping slab both inside and outside the core. The topping slab stiffness was not included in the models, but the weight will be accounted for in the baseline analysis.

Viscoelastic Dampers

Viscoelastic dampers were located below the bottom flange of the beams where the beams intersected the exterior columns. Similar to the 96 floor model, a placeholder element was located in the model at the damper location.

Initial Verification of the 75 Floor Model

Similar to the 96 floor model, the 'shading' option in SAP2000 was used to view the members as the program has interpreted their input. The shading option was helpful for using section designed shapes, and for verifying the orientation (i.e., local axes) of members. The work was independently reviewed by engineers not associated with the initial model development.

Once the model was completed, checks were performed for gravity loads. All superimposed dead loads and live loads included in the model are based on WTC Design Criteria; self weight is accounted for by SAP2000. To justify the modeling assumptions, several studies were performed to compare stress results to hand calculations for representative composite sections. Hand calculations estimate deflections and member stresses for a simply supported composite beam under gravity loading. The model yielded accurate steel stress results compared to hand calculations—around 1 percent for both short and long span beams. Where the beams were built-up with reinforcing plates, it was found that SAP *Section Designer* shapes were not calculating the stresses correctly, so instead, separate beam and plate elements drawn over each other were inserted. This method yielded very accurate steel stress results—between 1 percent and 2 percent for both short and long span beams.

B.3.4 Parametric Studies

Modeling techniques employed in the development of the global models of WTC 1 and WTC 2 are consistent with, but often more advanced than, the techniques typically employed in the analysis and design of high-rise buildings. As such, building components were idealized so that overall performance was replicated while appropriately reducing the computational requirements. The following describes the studies undertaken to establish the idealizations used in the models including typical exterior wall panels, exterior corner panels, and flexible floor diaphragms.

Exterior Wall Columns/Spandrel Typical Panels (Floors 9 to 106)

A parametric study of typical three-column, three-spandrel exterior wall panels from the face of the towers (floors 9 to 106) was performed using two modeling methods (see Fig. B–33). The first model was a detailed shell model where each plate of each column or spandrel was specifically modeled, and the second was a simplified frame model. The parametric study assumes that the shell model best represents the as-built panel performance, and therefore, it was used to tune the performance of the frame model which was used throughout the global model (see Section B.3.1). The objectives of the study were to (1) match the axial stiffness of the frame model with the detailed shell model under gravity load and (2) match the inter-story drift of the two models by modifying the rigidity of the column/spandrel intersections in the frame model.

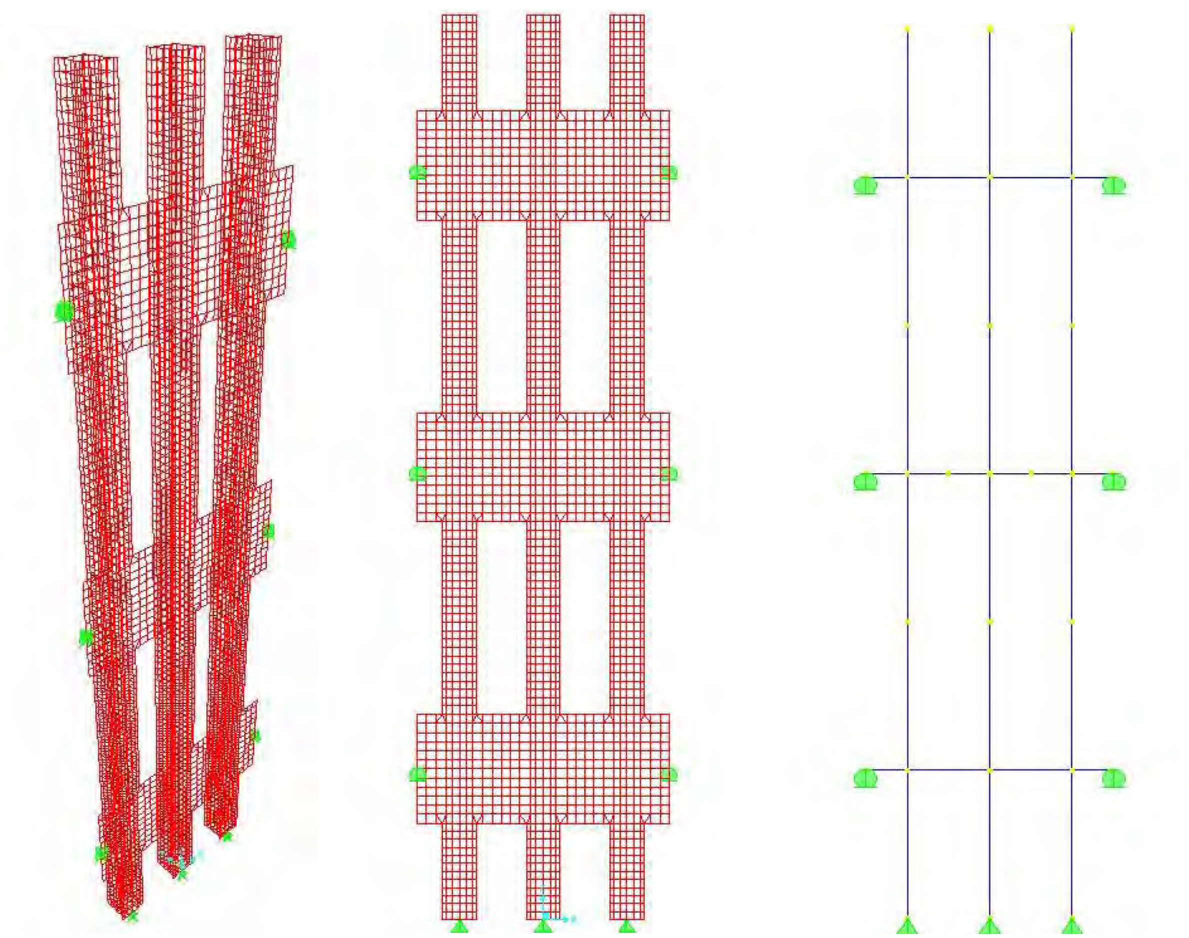


Figure B–33. Shell element and frame models of typical exterior wall panel.

For the axial stiffness of the simplified frame model of the panel versus the detailed shell model, results of loading both models vertically indicated that the shell model was stiffer than the equivalent beam model due to the contribution of the spandrel beams to the columns' axial stiffness. This is due to the rigidity of the spandrel beams and the proximity between the columns. The parametric study on a wide range of panels over the height of the towers showed that the vertical stiffness of the columns in the bottom third of the towers should be increased by a factor in the range of 25 percent to 28 percent, and the columns in the middle and upper thirds of towers should be increased by a factor in the range of 20 percent to 28 percent. Based on these figures, 25 percent increase of axial stiffness of exterior columns was selected as a reasonable representation for the panel vertical stiffness over the height of the towers between floors 9 and 106 (see Section B.3.1).

For studying the lateral deformation of the exterior panels, panel properties were taken from three different areas of the building. These include floors 79 to 82, 53 to 56, and 23 to 26. Internal column stiffeners were included in the shell model. The deformations at points A, B, I, and II (see Fig. B–34) were studied for three different panel locations and their respective spandrel and column thickness. The top most columns were connected via a rigid link and loaded in the plane of the panel and perpendicular to the column with a 100 kip load.

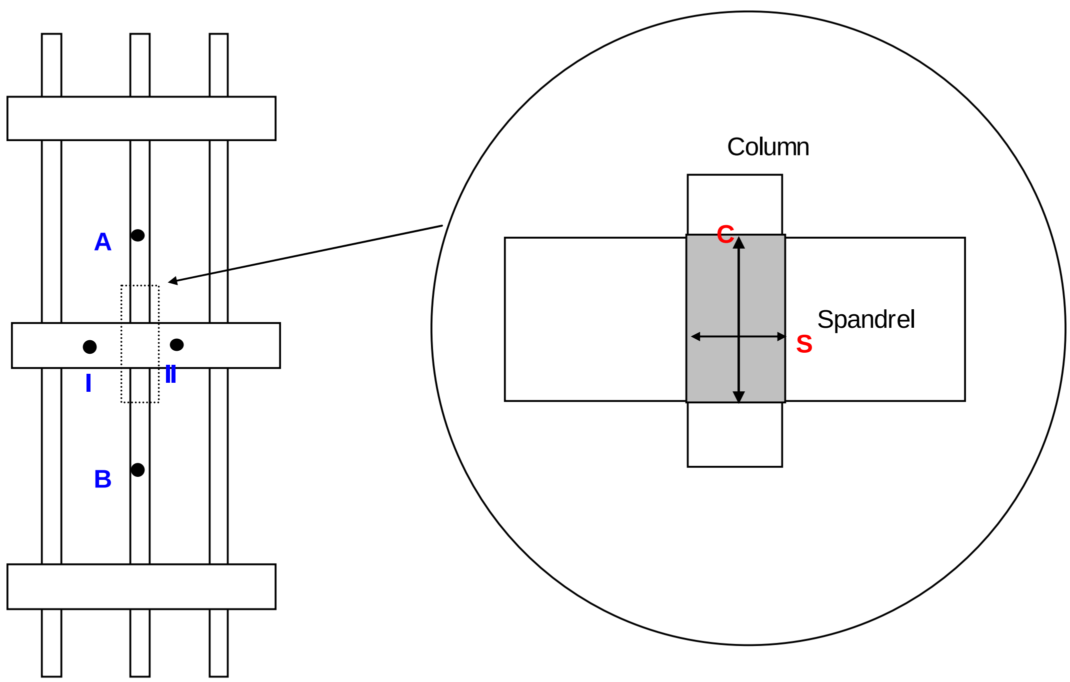


Figure B–34. Column and spandrel rigidity of typical exterior wall panel.

The lateral displacements found for the shell and frame models of typical exterior wall panels with varied column and spandrel intersection rigidities are reported in Table B–3. The study found that 50 percent column rigidity and 100 percent spandrel rigidity in the frame model produced deflection results consistent with the shell model.

Table B–3. Lateral displacement (in.) for the shell and frame models of typical exterior wall panel with varied column and spandrel rigidities.

	Lateral displacement (in)			
	Floor 79-82			
	Shell model	Frame model (Rigidity)		
		No rigidity	C:50%, S:100%	C:100%, S:100%
A	0.60	1.04	0.59	0.35
B	0.28	0.52	0.29	0.18
I	0.45	0.78	0.44	0.26
II	0.45	0.78	0.44	0.26
	Floor 53-56			
	Shell model	Frame model (Rigidity)		
		No rigidity	C:50%, S:100%	C:100%, S:100%
A	0.26	0.43	0.27	0.18
B	0.12	0.22	0.14	0.11
I	0.19	0.32	0.2	0.15
II	0.19	0.32	0.2	0.15
	Floor 23-26			
	Shell model	Frame model (Rigidity)		
		No rigidity	C:50%, S:100%	C:100%, S:100%
A	0.21	0.37	0.21	0.12
B	0.1	0.18	0.1	0.06
I	0.16	0.28	0.16	0.09
II	0.16	0.28	0.16	0.09

Exterior Wall Columns/Spandrel Corner Panels (Floors 9 to 106)

A parametric study was performed of an exterior wall corner panel typical over each corner of the towers from floors 9 to 106. Similar to the exterior typical panels, to account for the contribution of the spandrels into the axial stiffness of the columns, it was found that an area modifier to provide a 25 percent increase in the axial stiffness of the two continuous columns of the corner panels is suitable for modeling the columns' axial stiffness. No modifiers were needed for the 100, 200, 300, and 400 series intermittent columns.

The panel from floor 53 to 56 was selected to be representative with two additional columns attached on either side. The objective of the study was to match the inter-story drift of a detailed shell model and a simplified frame model of the corner panel by modifying the rigidity of the column/spandrel intersections in the frame model. For this parametric study, the panel was straightened to simplify the study and to isolate the behavior of interest (see Fig. B–35). The deformations at points T1, T2, B1, B2, and M2 (Fig. B–36) were studied for representative column and spandrel plate dimensions. The top most columns were connected via a rigid link and loaded in the plane of the panel and perpendicular to the column with a 100 kip load.

The lateral displacements calculated for the shell and frame models of the typical exterior wall corner panel with varied column and spandrel rigidities are reported in Table B–4. The study indicated that 25 percent column rigidity and 50 percent spandrel rigidity in the frame model produced deflection results consistent with the shell model.

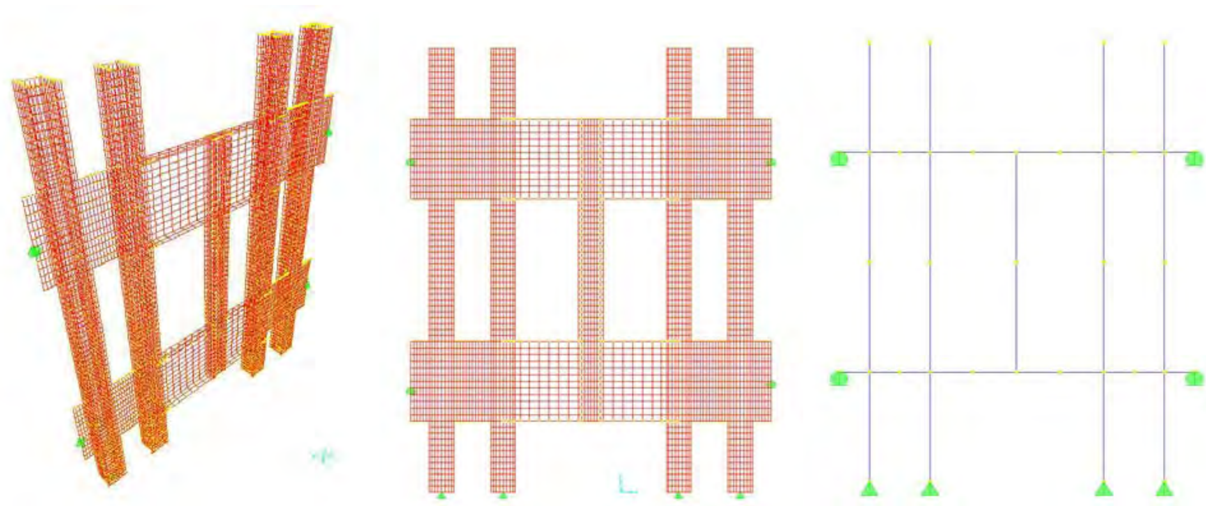


Figure B–35. Shell element and frame models of typical exterior wall corner panel.

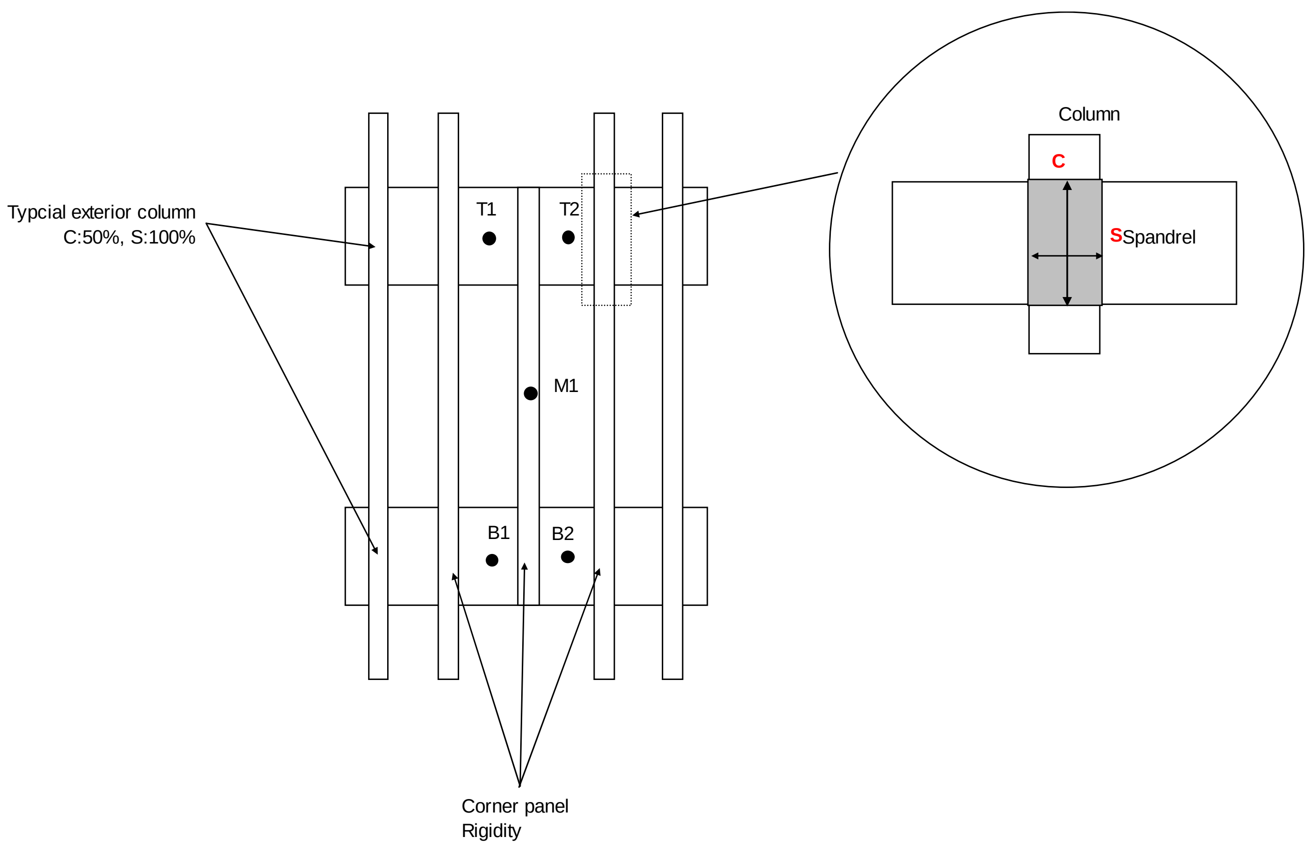


Figure B–36. Column and spandrel rigidity of typical exterior wall corner panel.

Table B–4. Lateral displacement (in.) for the shell and frame models of typical exterior wall corner panel with varied column and spandrel rigidities.

	Floor 53-56			
	Shell model	Corner panel rigidity		
		No rigidity	C:25%, S:50%	C:100%, S:100%
T1	0.227	0.236	0.222	0.152
T2	0.227	0.236	0.222	0.152
M1	0.149	0.154	0.149	0.102
B1	0.084	0.072	0.077	0.053
B2	0.084	0.072	0.077	0.053

As part of the in-house NIST review of the reference structural models (see Section B.4), a detailed shell element model of original corner panel (not straightened) was analyzed under lateral loads to test the accuracy of the simplified frame model with 25 percent column rigidity and 50 percent spandrel rigidity calculated above. Both the detailed and simplified models were loaded as shown in Fig. B–37. The deflections calculated from the frame model were consistent with those estimated from the shell model, indicating that the rigidities estimated using the straight model (Fig. B–35) accurately represent the actual corner panel behavior.

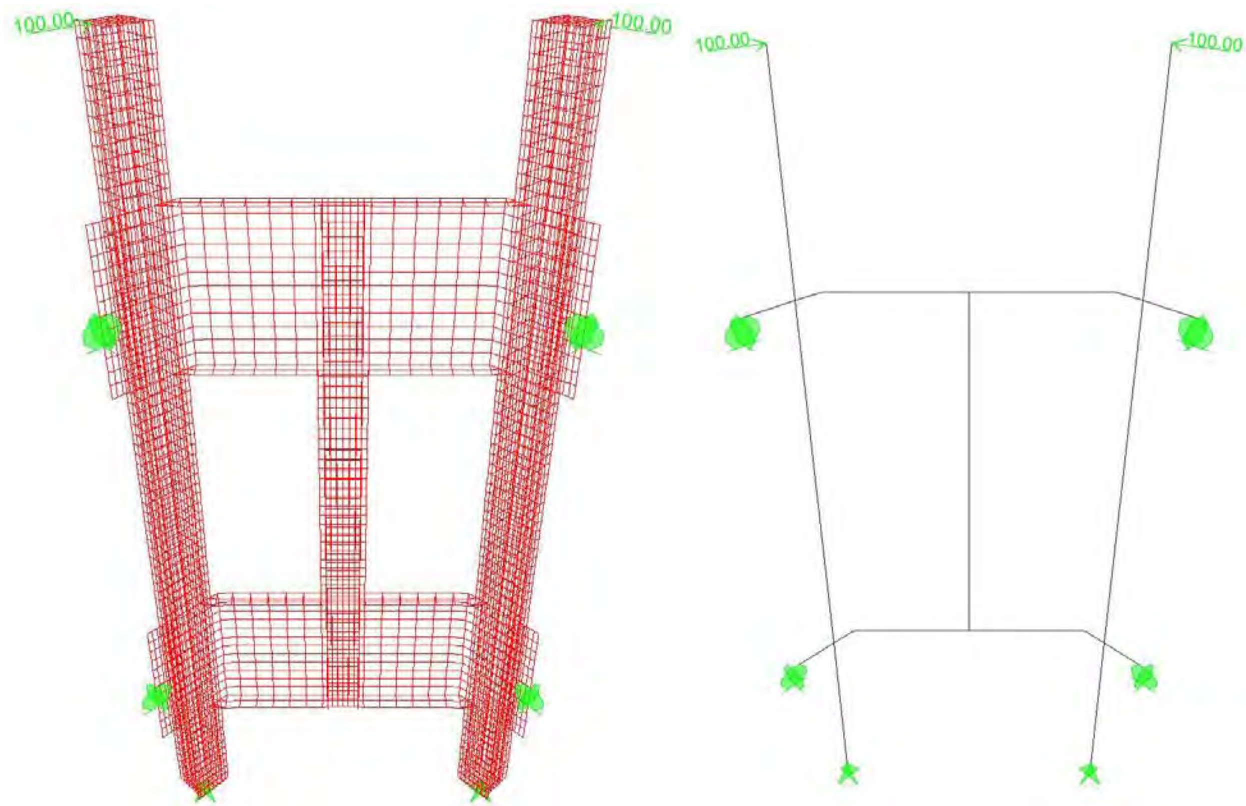


Figure B–37. Detailed and simplified model of the exterior wall corner panel.

Flexible Floor Diaphragm

The floor models developed in Sections B.3.2 and B.3.3 were used to develop the flexible diaphragm stiffness used within the WTC 1 and WTC 2 global models. The in-plane diaphragm stiffness of the detailed floor models was determined and used to arrive at an equivalent shell element floor model. This flexible shell element floor model is then inserted in the global models at specific floors to capture the in-plane flow of forces and deformations. These flexible diaphragms were not used throughout, as the rigid diaphragms in the majority of floors provided for a sufficiently accurate representation of the flow of forces and deformations while keeping manageable the model's computational requirements. In the global models, flexible diaphragms were used at the beam-framed floors 3, 4, 5, 6, 7, 9, 41, 42, 43, 75, 76, 77, 107, 108, 109, 110, and roof of both towers.

Parametric studies were performed to compare the diaphragm stiffness of two different floor models for both the typical truss-framed floor and the beam-framed floor. The typical floor models were compared with the simplified equivalent models that duplicate the representation of the exterior wall columns, exterior wall spandrels, core columns, and their boundary conditions. The floor framing, both inside and outside the core was replaced by shell elements. The material properties of the shell model matched the properties of the concrete floor outside the core in the respective floor model.

The comparative floor models were loaded in the plane of the floors with a lateral load of 180 lb/ft. (equivalent to 15 psf over the 12 ft story height) on both the windward and leeward faces. The column base supports were released for the exterior wall columns along the loaded faces and for all core columns to allow lateral translation only in the direction of loading.

The comparative models were executed to assess the horizontal deflection of the floor on both the windward and leeward sides of the model and for the case where the lateral loads were applied non-concurrently along the 100 face and 200 face of the tower. Both the total horizontal deflection of the slab and the relative displacement between the windward and leeward sides were compared between the models. The shell thickness was modified to match the in-plane stiffness determined by the detailed floor models.

The deformations from the lateral load case using the 96 floor model of WTC 1 are illustrated in Fig. B–38, while Fig. B–39 shows the deformations of the simplified floor model. Fig. B–40 shows the lateral deflection of the north and south sides of the floor model under lateral load applied in the north direction using the detailed and equivalent floor models.

B.4 REVIEW OF THE STRUCTURAL DATABASES AND REFERENCE MODELS OF THE TOWERS

NIST has implemented a rigorous and comprehensive review procedure to mitigate potential conflicts of interest and to ensure the integrity and objectivity of the deliverables of this project, including the structural databases and reference models. The review procedure includes an in-house NIST review as well as a third-party review by the firm of SOM. The following summarizes the results of these reviews for the developed structural databases and reference models.

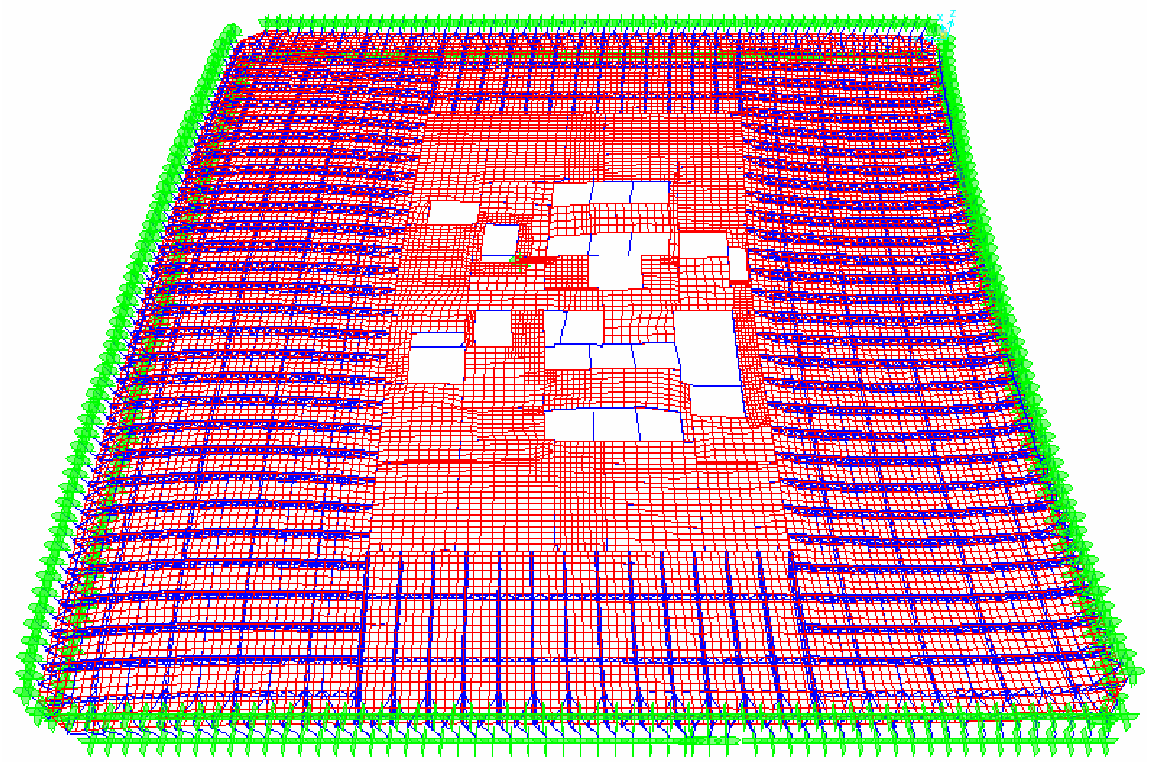


Figure B–38. Deflection of typical truss-framed floor model due to lateral loading (exaggerated scale).

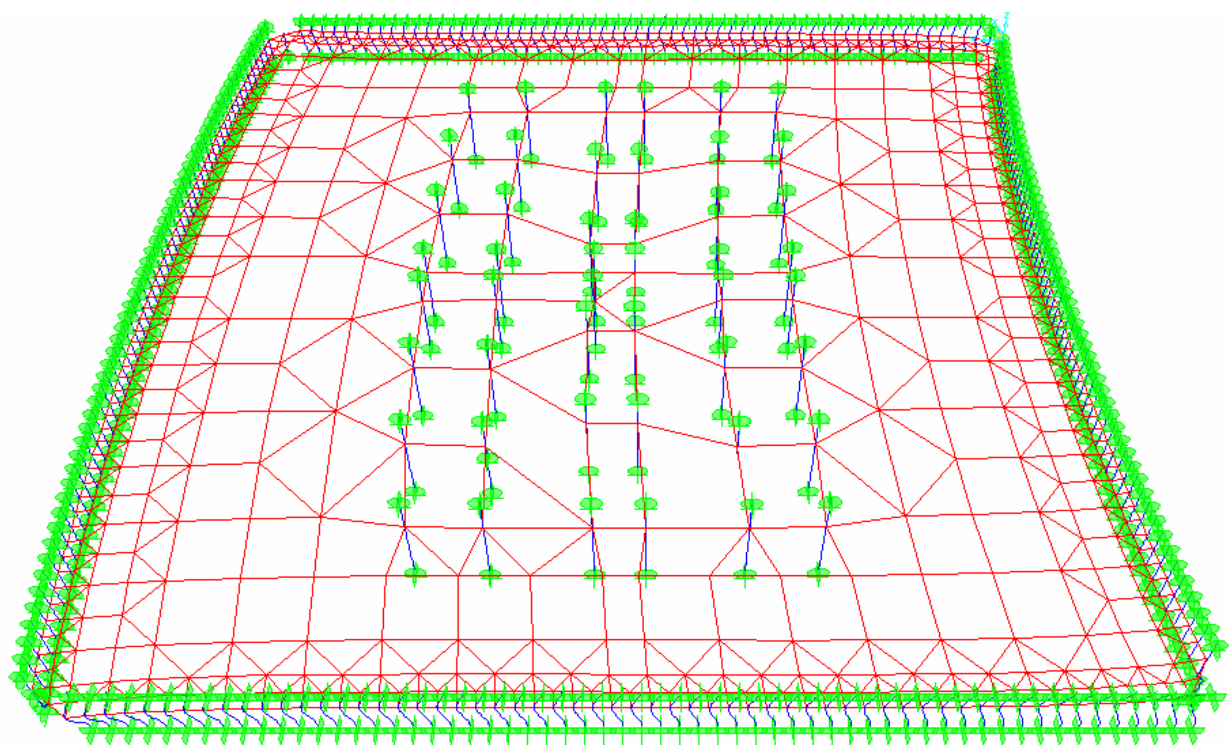


Figure B–39. Deflection of equivalent floor model due to lateral loading (exaggerated scale).

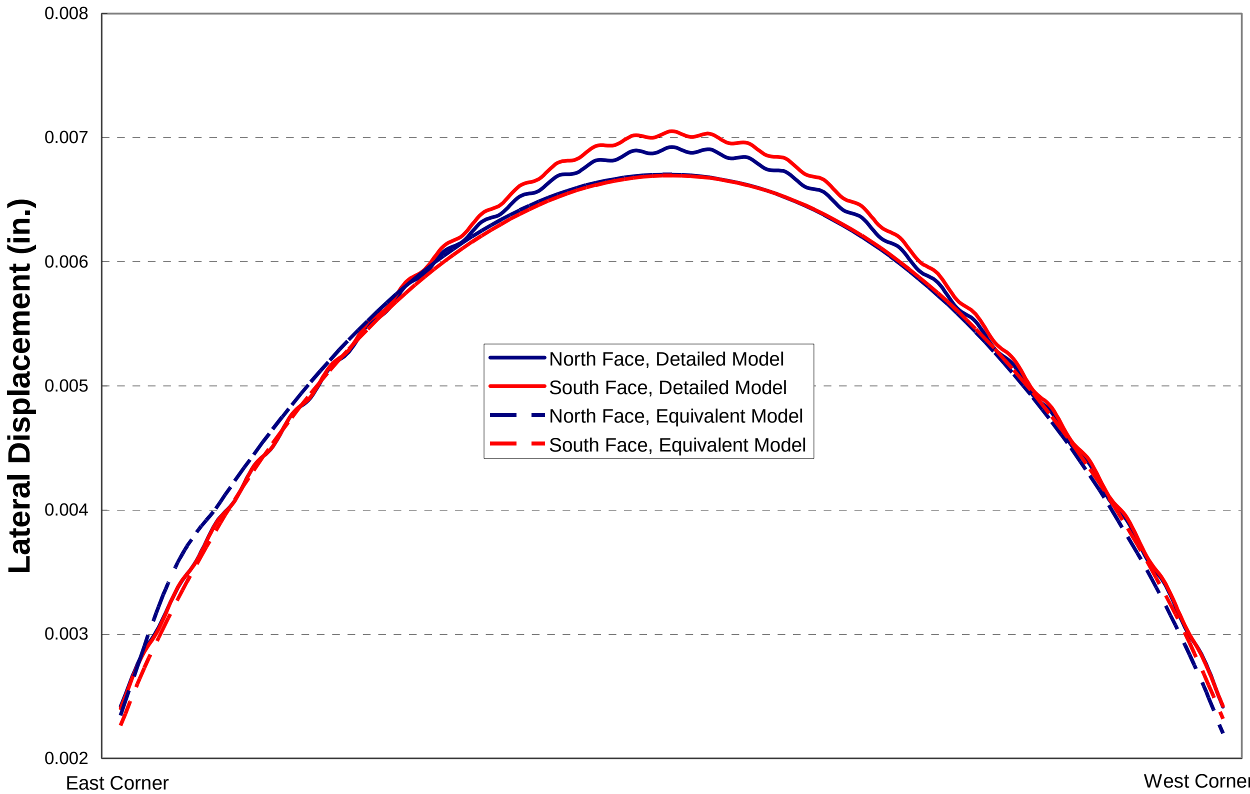


Figure B–40. Deflections of the north and south faces of the floor for the detailed and equivalent floor models.

B.4.1 Review of the Structural Databases

The third-party review by SOM included random checks of the digitized structural databases and cross section property calculations. The review indicated no discrepancies between the developed databases and the original drawing books. Also for cross section property calculations, the review indicated a good agreement (within 1 percent) between the properties in the developed databases and those estimated by SOM. Special attention was given to the calculation of the torsional constant, *J* (see Section B.2.5). It was found that using the software ShapeBuilder, version 3.0 which uses a finite element approach for the calculation of *J* a good agreement was obtained between the *J* values in the developed databases and those estimated by SOM.

The in-house NIST review included the following steps: (1) line-by-line review of all database files, (2) random checks on the developed databases by the project leader, and (3) calculation of all cross section properties and comparison with those in the developed databases. The review indicated minor discrepancies between the developed databases and the original drawing books. For cross section property calculations, good agreement was obtained between the properties in the developed databases and those estimated by NIST. The discrepancies between the developed databases and the original drawing books were reported to LERA, who implemented the changes and modified the databases accordingly. Consequently, the structural databases have been approved by NIST and are being made available for other phases of the NIST investigation.

B.4.2 Review of the Reference Structural Models

The third-party review by SOM included: (1) random checks of the consistency of the developed reference models with the original structural drawings and drawing books, and (2) verification and validation of the models, including reviewing assumptions and level of detail and performing analyses using various loading conditions to test the accuracy of the models. The review indicated that the developed models are consistent with the original design documents. The review indicated that, in general, the modeling assumptions and level of detail in the models are accurate and suitable for the purpose of the project. The SOM review identified two areas where the models need to be modified. The first is the effect of additional vertical stiffness of the exterior wall panels due to the presence of the spandrel beams (see Sections B.3.1 and B.3.4). The second area is the modeling of the connections of the floor slab to the exterior columns of the 75B floor model (Section B.3.3), where this connection appeared to be fixed while the connection should be modeled as pinned.

The in-house NIST review included: (1) checks on the consistency of the developed reference models with the original structural drawings and drawing books, and (2) verification and validation of the models, including reviewing assumptions and level of detail and performing analyses using various loading conditions to test the accuracy of the models. The review indicated minor discrepancies between the developed reference models and the original design documents. Similar to the third-party review, the in-house NIST review identified the proper modeling of the vertical stiffness of the exterior wall panels and the accurate modeling of the floor slab connections to the exterior columns in the 75B floor model as areas that need to be modified in the models.

In addition, NIST conducted a workshop for NIST investigators and contractors to review the reference structural models developed by LERA. The workshop attendees included experts from LERA (two experts); SOM (two experts); Teng and Associates (one expert, outside experts on probable structural collapse); Professor Kasper Willam (outside expert on thermal-structural analysis); Professor David M. Parks (outside expert on computational mechanics for aircraft impact analysis); Applied Research Associates (two experts, contractor on analysis of aircraft impact into the WTC towers) as well as all key investigators from NIST (17 experts). The purpose of the workshop was to discuss the methodology, assumptions, and details of the developed reference models. The minutes of the workshop are being prepared and will be made public. The feedback from the workshop was included in the final review of the models.

The discrepancies between the developed models and the original design documents, as well as the areas identified by both the third-party and in-house review for modification, were reported to LERA, who implemented the changes and modified the models accordingly. Consequently, the reference structural models have been approved by NIST and are being made available for other phases of the NIST investigation.

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Appendix C
INTERIM REPORT ON ANALYSIS OF AIRCRAFT IMPACT INTO THE WTC TOWERS

C.1 INTRODUCTION

One of the objectives of Project 2, Baseline Structural Performance and Aircraft Impact Damage Analysis, of the National Institute of Standards and Technology (NIST)-led investigation into the collapse of the World Trade Center (WTC) towers is to analyze the aircraft impact into each of the two towers. The purpose of this analysis is to provide the following: (1) estimates of the damage to structural systems due to aircraft impact—including exterior walls, floor systems, and interior core columns; (2) estimates of the aircraft fuel dispersal during impact; (3) estimates of accelerations and deformations as a function of time in each of the two towers due to aircraft impact to be used for estimating damage to fire proofing; and (4) a database of the major fragments of the aircraft and destroyed structural components of the towers to be used for estimating damage to the mechanical and architectural systems inside the towers. The analyses, thus, establish the initial conditions for the fire dynamics modeling in Project 5, Reconstruction of Thermal and Tenability Environment, and the thermal-structural response and collapse initiation analysis in Project 6, Structural Fire Response and Collapse Analysis.

The impact analyses are conducted at various levels including: (1) the component level, (2) the subassembly level, and (3) the global level to estimate the probable damage to the towers due to aircraft impact. The analyses also include simplified and approximate methods. Analysis of uncertainties using the component, subassembly, global, and simplified analyses will also be conducted to assess the effect of variability associated with various parameters on the damage estimates. NIST is working with experts from Applied Research Associates (ARA), Inc., to conduct the impact analyses. This appendix summarizes the progress made to date on this project. Emphasis is on the models development and component level analyses.

Section C.2 outlines the development of constitutive models describing the actual behavior of the various materials included in the aircraft impact analysis. The materials in this section include WTC steels, reinforced concrete, and aircraft materials. Special emphasis is also placed on the modeling of weldments and bolts. Section C.3 presents the status of the development of the aircraft model, including the engine and airframe. This section also provides an analysis of the fuel distribution at the time of aircraft impact. Section C.4 provides details on the component level analyses performed to date, including exterior and core columns, column and spandrel connections, and floor segments under the impact of an aircraft engine or a segment of a wing. A summary and preliminary findings are provided in Section C.5.

C.2 MATERIAL CONSTITUTIVE MODELING

An important requirement for high fidelity simulation of the aircraft impact damage is the development of constitutive models to represent the actual behavior of the structure under the dynamic impact conditions of the aircraft. The primary materials that need to be considered for the component analyses are the several grades of steel used in the columns, spandrels, and floor trusses and beams of the WTC towers, the concrete floor slabs, and the aluminum airframe structure of the Boeing 767 aircraft. All of these materials play a significant role in the aircraft impact damage analyses. These materials also display significant nonlinear rate-dependent deformation and failure behavior.

The analysis of the aircraft impact damage is being performed with the LS-DYNA finite element code (LS-DYNA 2003). LS-DYNA has an extensive library of more that 130 different constitutive models and is capable of accurately reproducing the important material behaviors required in this analysis. Material models currently available in LS-DYNA have been utilized for the analyses using material data from tests conducted by NIST or available in the public domain.

Secondary materials of interest include the nonstructural aircraft components and masses such as fuel, seats, interior trim, cargo, and luggage. Furthermore, a significant part of the mass of the WTC towers is material not included in the primary structural steel frame that includes windows, nonstructural walls, partitions, furniture and other building contents, flooring, mechanical equipment, and insulation. Selection of the constitutive modeling for these secondary material behaviors has not yet been performed. However, the strength of these materials is relatively small compared to the structural materials and simple description of their constitutive behavior will be adequate.

C.2.1 WTC Tower Steel Constitutive Models

Various constitutive models are available in LS-DYNA that can capture the nonlinear behavior of the steel under impact conditions including strain rates effects and failure. The primary constitutive model applied to date is the Piecewise Linear Plasticity model. This model is sufficient to model the nonlinear dynamic deformation and failure of the steel structures. A tabular effective stress versus effective strain curve can be used in this model with various definitions of strain rate dependency.

The constitutive model parameters were developed for each grade of steel used in the construction of the WTC towers based on engineering stress-strain data provided by Project 3 of the NIST-led investigation on Mechanical and Metallurgical Analysis of Structural Steel. The approach to developing the constitutive parameters for each grade of steel was:

- Convert the engineering stress-strain curve to a true stress versus true strain curve. The conversion process is described below and is valid up to the onset of necking in the specimen.

- Extrapolate the true-stress-strain curve beyond the point of necking onset.

- Perform iterative finite element analyses of the tensile test and adjust the true stress-strain curve extrapolation and failure strain until the necking behavior and failure point are accurately captured. The primary criterion is the quantitative agreement of the measured and calculated engineering stress-strain behavior in the softening region beyond maximum stress. These analyses require a fine mesh resolution in the specimen to accurately model the large strain deformation response during necking.

- Perform a final finite element analysis of the material test using a coarser mesh resolution (medium mesh corresponding to the mesh resolution applied in the component analyses). Adjust the failure criteria (strain at failure) to obtain failure at the same engineering strain level.

The advantage of this approach is that the measured nonlinear material behavior up to failure is accurately captured in the constitutive model. In addition, the simulation of the material testing provides a validation that the constitutive model parameters are defined accurately and that the model can reproduce the measured response for the test conditions.

The tensile tests performed by NIST applied the ASTM 370 test standard (ASTM Designation A 370-03a 2003). Example finite element models of a rectangular test specimen with the fine and medium mesh resolutions are shown in Fig. C–1. This specimen type was used for all of the tests on WTC exterior column materials. A similar figure of the round bar specimen models used for testing of the WTC core column steels is shown in Fig. C–2. The typical element length used in the gage section for the fine meshes is approximately 0.015 in. and for the medium meshes is approximately 0.10 in. The use of the specimen models to develop the constitutive model parameters is described in the following subsections.

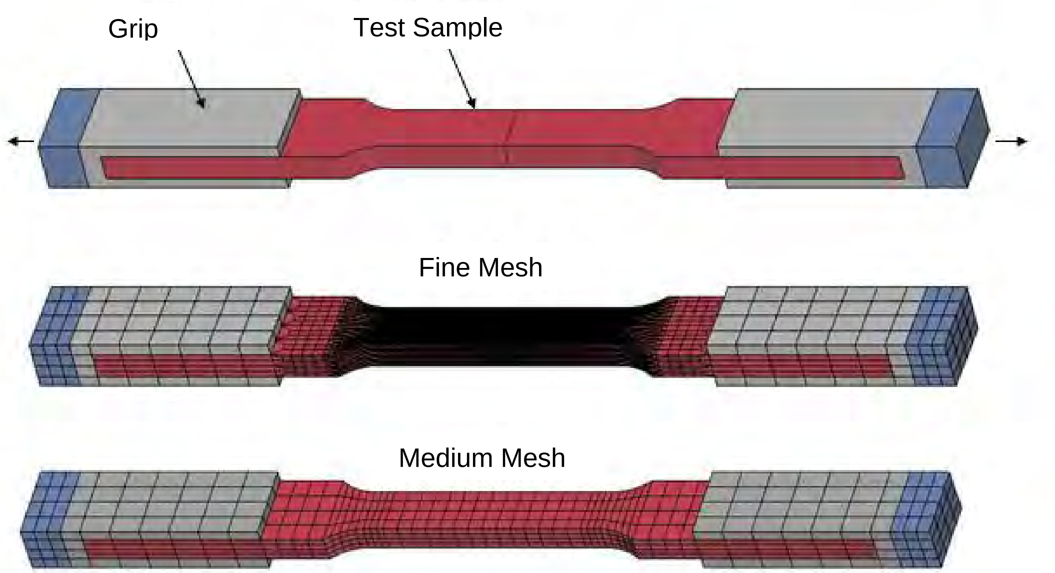


Figure C–1. Example finite element models of the ASTM 370 rectangular tensile specimen.

True Stress and True Strain Corrections

In most tensile tests, a plot is generated of the load measured at the cross head of the testing machine against the displacement of the specimen. A plot of engineering stress versus strain can be generated from this plot by dividing the load by the original cross sectional area of the specimen and the displacement by the original length of the specimen. If the changes in area and length are small during the test, these measures give a good indication of material behavior. However, in reality, the cross section changes (shrinks) significantly during the test and the engineering stress does not yield the “true” stress in the cross section. Similarly, the engineering strain is not representative of the material behavior, especially when a general 3-D state of strain exists. As a result, the engineering stress decreases as some materials approach failure, implying a weakening of the material. In reality, the stress in the cross section is increasing due to the reduction in the cross sectional area (i.e., necking).

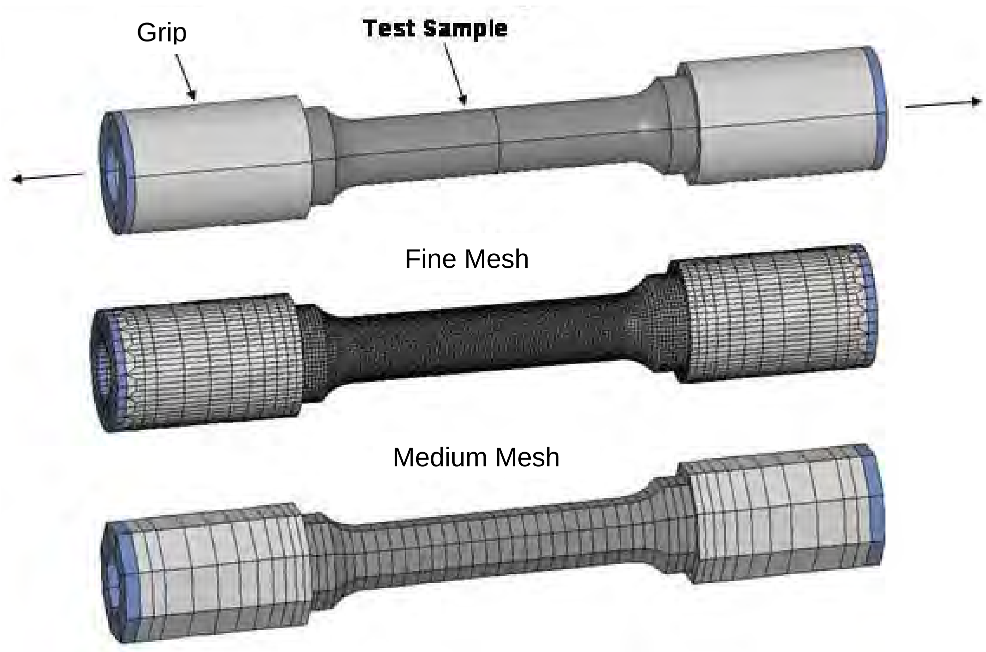


Figure C–2. Example finite element models of the ASTM 370 round bar tensile specimen.

There are several different ways to measure stress and strain based on the coordinate system used. Some are based on material (Lagrangian) coordinates and some on spatial (Eulerian) coordinates. These give rise to terms such as "Green" and "Almansi" strain tensors. These are important in writing a computer code to solve large strain problems. An alternate approach is to define a "true" or "natural" stress and strain. The true stress is based on the load divided by the actual cross sectional area of the specimen and is equal to the engineering stress multiplied by a term to correct for the change in cross section.

$$\sigma_T = \sigma_{eng}(1+e) \tag{1}$$

where:

σ_T = true stress

σ_{eng} = engineering stress

e = engineering strain

The natural or true strain is defined as

$$\varepsilon_T = \ln(\frac{l}{l_o}) = \ln(1+e) \tag{2}$$

where:

ε_T = true strain

This definition comes about from defining the incremental true or “natural” strain as the current “change in length” divided by the current length, or

$$d\varepsilon_T = \frac{dl}{l} \tag{3}$$

This is in contrast with the definition of engineering strain that references the change in length, Δl, divided by the original length, l_0, or

$$e = \frac{\Delta l}{l_0} \tag{4}$$

Development of Steel Constitutive Properties

In this section, examples are provided to illustrate the methodology for the development of the steel constitutive models and typical results. Figure C–3 shows an example of the measured engineering stress-strain behavior for the 75 ksi perimeter column steel. Four tests were performed by Project 3, and the data clearly indicates anisotropy in the behavior introduced by the roll forming process (longitudinal tests L1 and L2 versus transverse tests T1 and T2 in the figure). This particular grade of steel had a larger anisotropy than seen in most of the other steel grades. Whenever anisotropy was observed, the material had greater ductility for specimens aligned with the rolling direction. In addition, the largest effects of the anisotropy were seen in the behavior after the onset of necking.

The first step in the constitutive model development process is to obtain a true stress-true strain curve. The typical approach is to select a representative test and perform the data conversion process described earlier. In this example, the data from test L1 was used to create the true stress-strain curve shown in Fig. C–3. This true stress-strain curve is then approximated by a piecewise linear curve in tabular form as shown in Fig. C–4. This tabular curve is the input used to specify the mechanical behavior in the constitutive model.

The final step is to apply the tabular true stress-strain behavior in the constitutive model to simulate the tensile test as shown in Fig. C–5. If necessary, the extrapolation of the true stress-strain behavior is adjusted until the simulation matches the measured engineering stress-strain response including necking and failure (the portion of the stress-strain curve beyond the maximum engineering stress). A comparison of the calculated and measured tensile behavior for the 75 ksi perimeter steel is shown in Fig. C–6. In this example, the constitutive model was developed as an average between the longitudinal and transverse properties. Test results conducted by Project 3 on the tower steels indicated that the stress-strain behavior is very similar in the longitudinal and transverse directions up to the onset of necking. The difference between the longitudinal and transverse properties is in the ductility, where the average ratio of the longitudinal to transverse strain to failure in the exterior column steels is about 1.22:1. The approach used in the constitutive modeling is to use an average between the longitudinal and transverse properties and ignore the orthotropic characteristics of the material in the impact analyses. The effects produced by the variation of ductility between the longitudinal and transverse directions will be assessed as part of the uncertainty analyses.

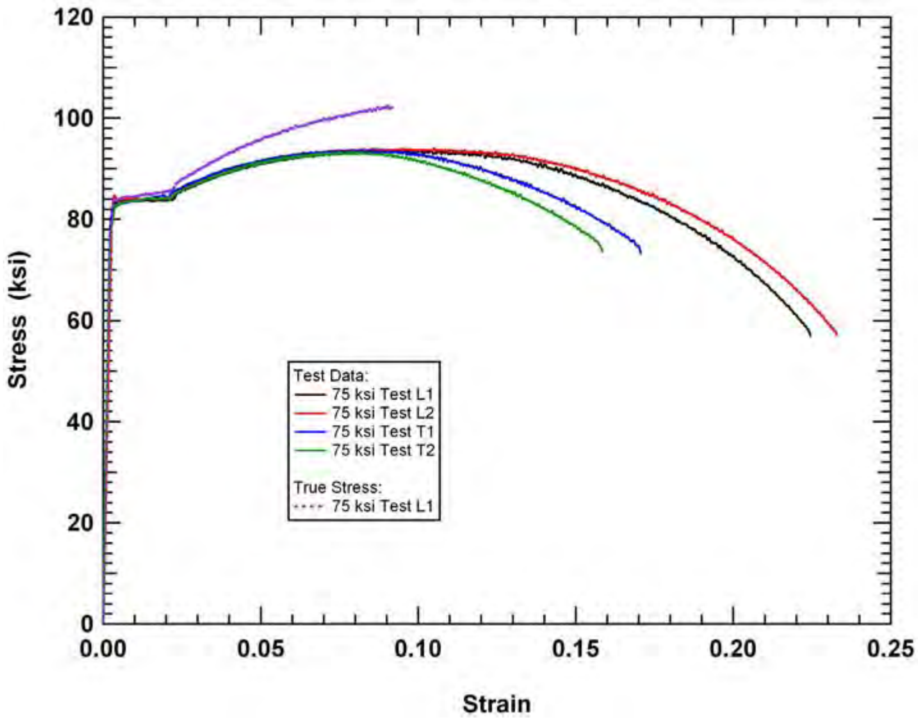


Figure C–3. Test data and true stress-strain conversion for the 75 ksi steel.

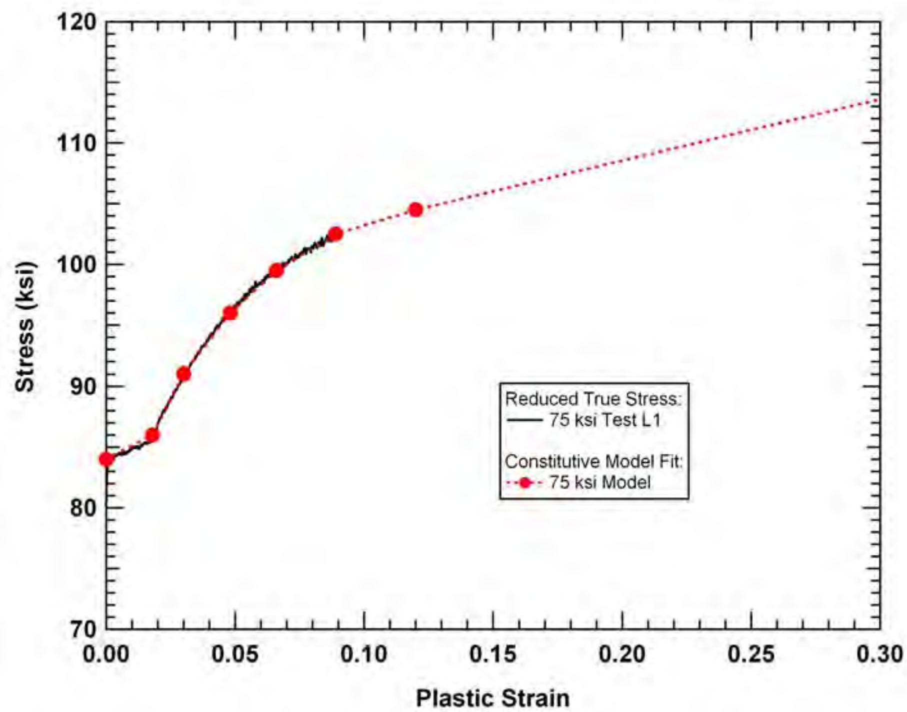


Figure C–4. Tabular true stress-strain constitutive model curve for the 75 ksi steel.

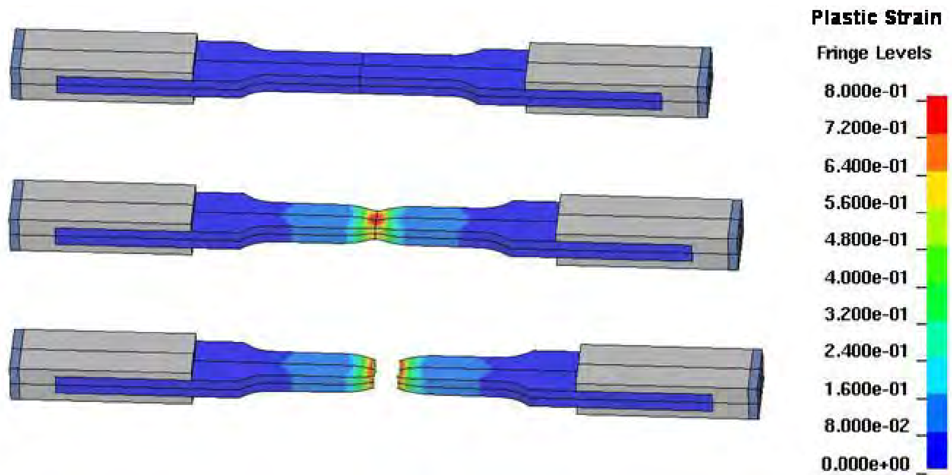


Figure C–5. Calculated tensile test response with necking for the 75 ksi steel.

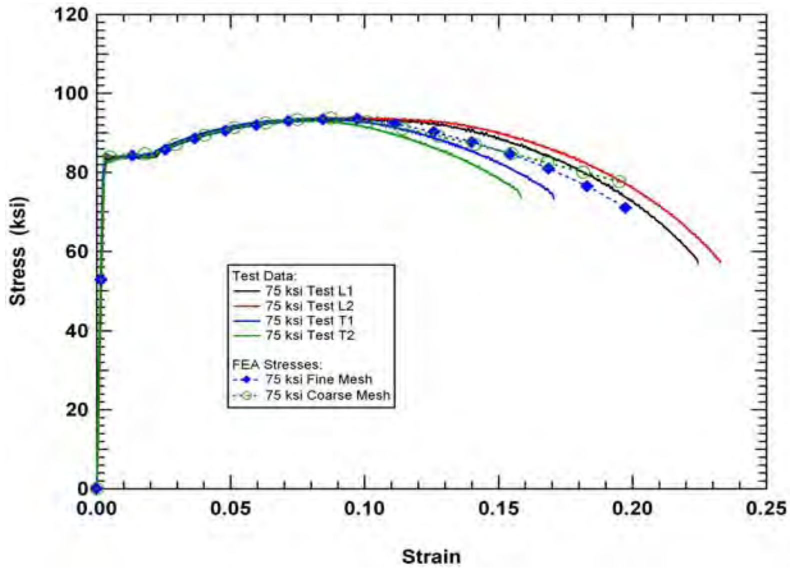


Figure C–6. Comparison of measured and calculated engineering stress-strain curves for the 75 ksi steel.

The resulting true stress-true strain behavior incorporated into the constitutive model is representative of the steels tested in this project. However, there were multiple sources (suppliers) of steel used in the construction of the WTC towers. Project 3 developed synthetic stress-strain curves for each grade of steel based on several data sources. A comparison of this synthetic curve with the constitutive model behavior is shown in Fig. C–7. There are observable differences in the curves such as the representation of a yield point behavior in the constitutive model. However, the differences are not sufficiently large to produce a large variation in the calculated structural impact behavior. In addition, the effect of material strength variation on impact damage will be assessed in the uncertainty analyses.

The above procedure was applied to develop constitutive models for all of the WTC tower steels for which test data was provided by Project 3.

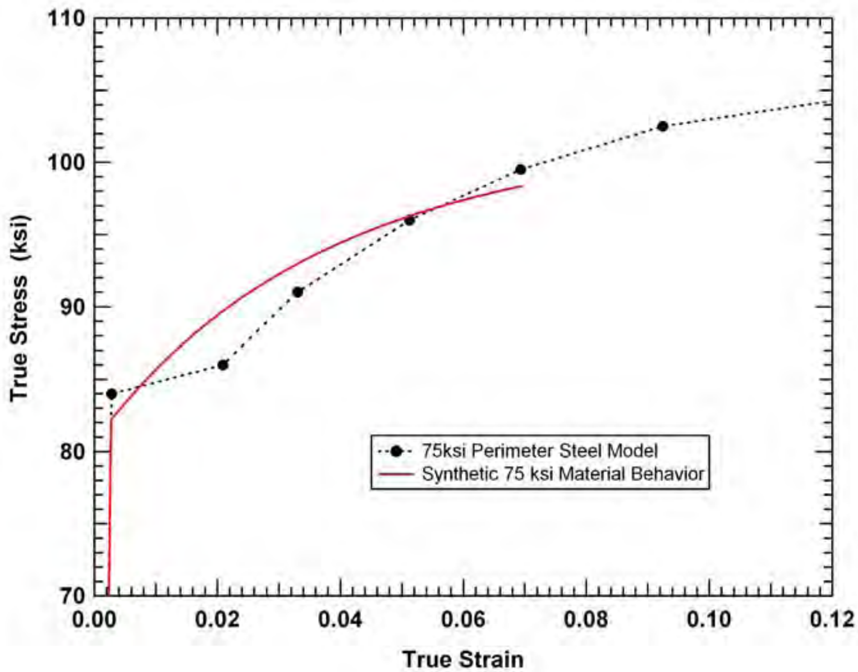


Figure C–7. Comparison of the constitutive model and synthetic steel behaviors for the 75 ksi steel.

Summary of Steel Constitutive Properties

A summary of the true stress-strain curves used in the constitutive models for the various WTC tower steels are summarized in Fig. C–8. Similarly a comparison of the true stress-strain constitutive curves with the synthetic stress-strain curves developed by Project 3 for the various exterior column steels is shown in Figures C–9 and C–10, and for core column steels in Fig. C–11. In general, the true stress-strain curves developed from the test data and the synthetic curves developed from multiple sources agree to within approximately 10 percent. This variation in measured and expected material strengths will be used as part of the future uncertainty analyses.

Strain Rate Effects in Steel Constitutive Models

Strain rate effects on the steel yield strength are included in the constitutive model for tower steels with the Cowper and Symonds rate effect model. The functional form for the rate effects on strength is governed by the equation:

$$\sigma_y\left(\dot{\varepsilon}\right) = \sigma_y\left[1+\left(\frac{\dot{\varepsilon}}{C}\right)^{1/p}\right] \qquad (5)$$

Where σ_y is the yield strength, $\dot{\varepsilon}$ is the strain rate, and C and p are the Cowper and Symonds parameters.

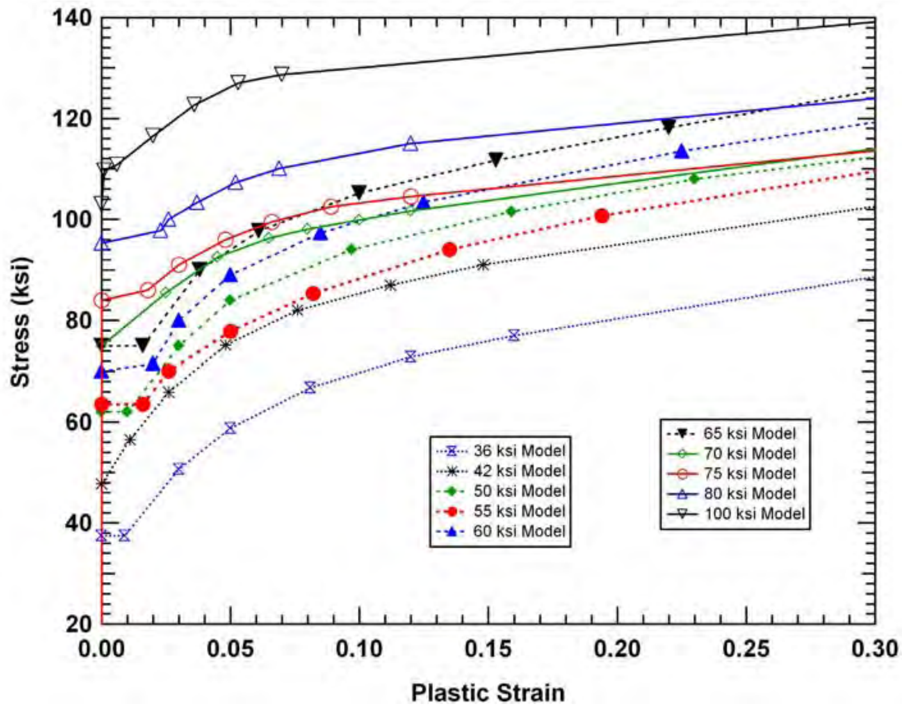


Figure C–8. Tabular true stress-strain constitutive model curves.

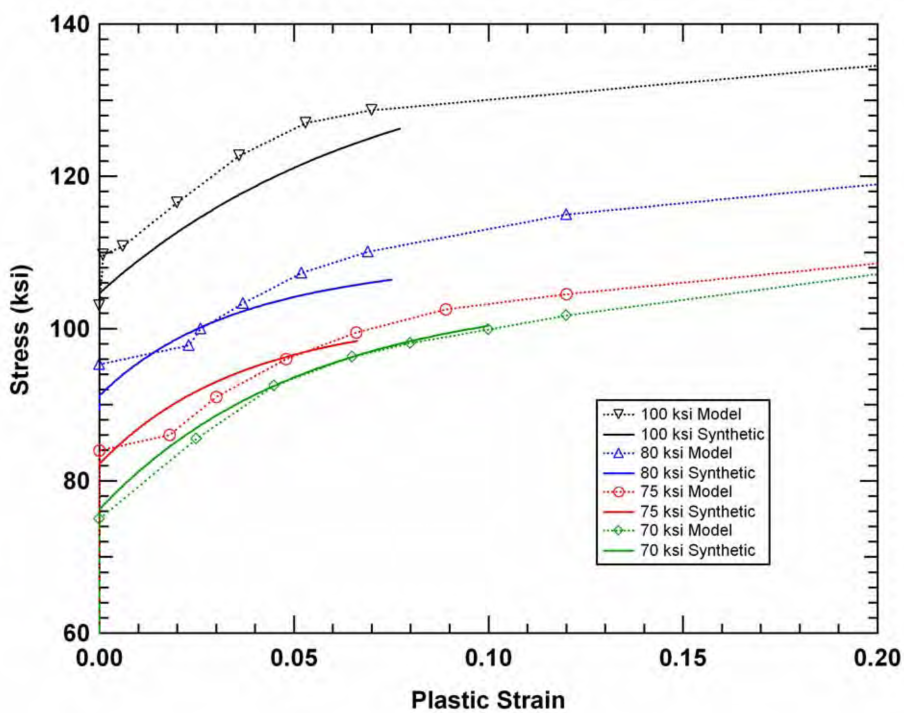


Figure C–9. Comparison of the constitutive model and synthetic steel behaviors for the higher strength perimeter steel curves.

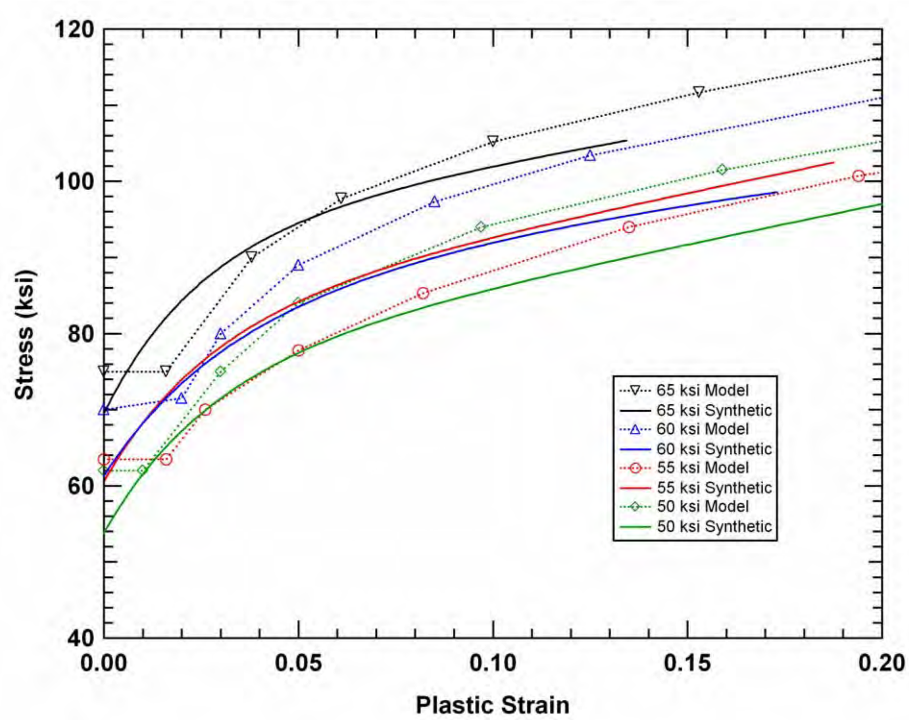


Figure C–10. Comparison of the constitutive model and synthetic steel behaviors for the lower strength perimeter steel curves.

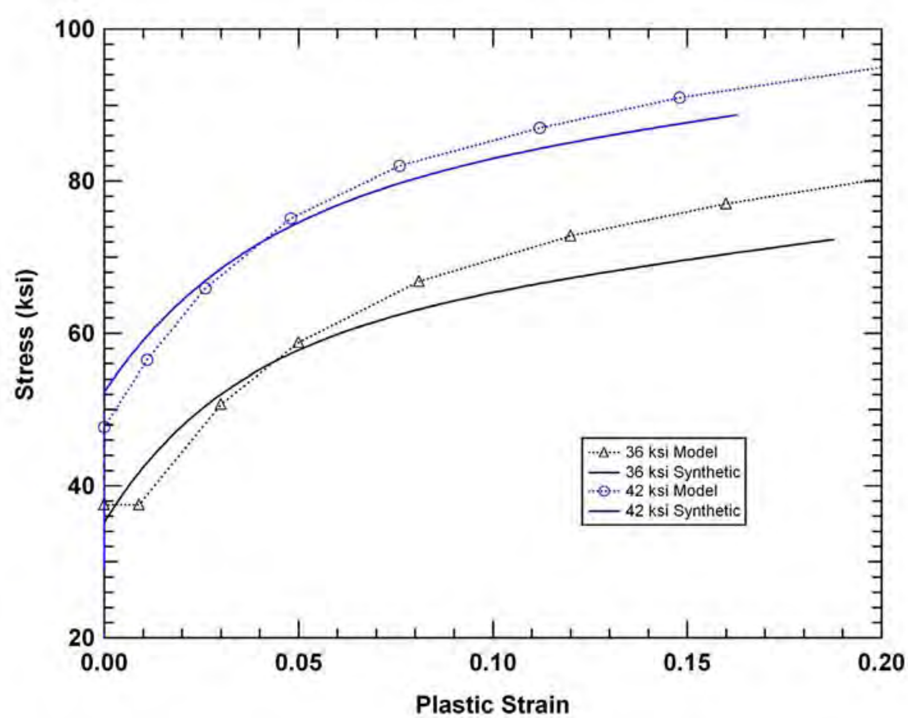


Figure C–11. Comparison of the constitutive model and synthetic steel behaviors for the core steel curves.

A series of high-rate characterization tests was performed on tower steels by Project 3. In addition to quasi-static tests (performed at a rate below 0.001 s^{-1}), a series of high rate tests was performed primarily at strain rates between 100 s^{-1} and 1000 s^{-1}. The Cowper and Symonds model parameters *C* and *p* were then fit to the test data and provided in the following functional form for a strain rate in s^{-1} and a yield strength in ksi:

- $\mathrm{Log}(C) = -7.55 + 0.324\sigma_y - 0.00153(\sigma_y)^2$
- $p = 6.7824$

The specific values used for each of the different tower steels are summarized in Table C–1. The 36 ksi and 42 ksi steels are materials used in the core columns and the remaining steels are used in the exterior columns. The resulting rate effects used in the constitutive modeling of tower steel based on Equation (5) are compared to the measured high-rate test data for the 50 ksi, 75 ksi, and 100 ksi tower steels in Fig. C–12. The comparison shows that the Cowper and Symonds model is capable of reproducing the rate effects for the range of data available.

Table C–1. Material constitutive parameter table–WTC tower steels.

Material Yield Specification	Young's Modulus	Poisson's Ratio	Strain-Rate Coefficient (C)	Strain-Rate Exponent (p)	Brick Element Failure Strain
36 ksi	29,700 ksi	0.288	7.900e+01	6.782	0.32
42 ksi	29,700 ksi	0.288	1.360e+05	6.782	0.32
50 ksi	29,700 ksi	0.288	4.220e+06	6.782	0.50
55 ksi	29,700 ksi	0.288	6.700e+06	6.782	0.64
60 ksi	29,700 ksi	0.288	3.950e+07	6.782	0.56
65 ksi	29,700 ksi	0.288	1.270e+08	6.782	0.51
70 ksi	29,700 ksi	0.288	1.270e+08	6.782	0.62
75 ksi	29,700 ksi	0.288	6.710e+08	6.782	0.56
80 ksi	29,700 ksi	0.288	2.440e+09	6.782	0.49
100 ksi	29,700 ksi	0.288	3.430e+09	6.782	0.53

Test results indicated that the influence of strain rate on the ductility of the tower steels did not follow a consistent trend. Several grades of steel had an increased ductility at high rates (more common for low strength steels), some had a reduced ductility at high rates (more common for high strength steels), and some showed no significant effect of strain rate on ductility. The approach used in the constitutive modeling is to ignore the changes in ductility produced by elevated strain rates. The effects of the variation of ductility over the expected range of strain rates will be assessed as part of the uncertainty analyses.

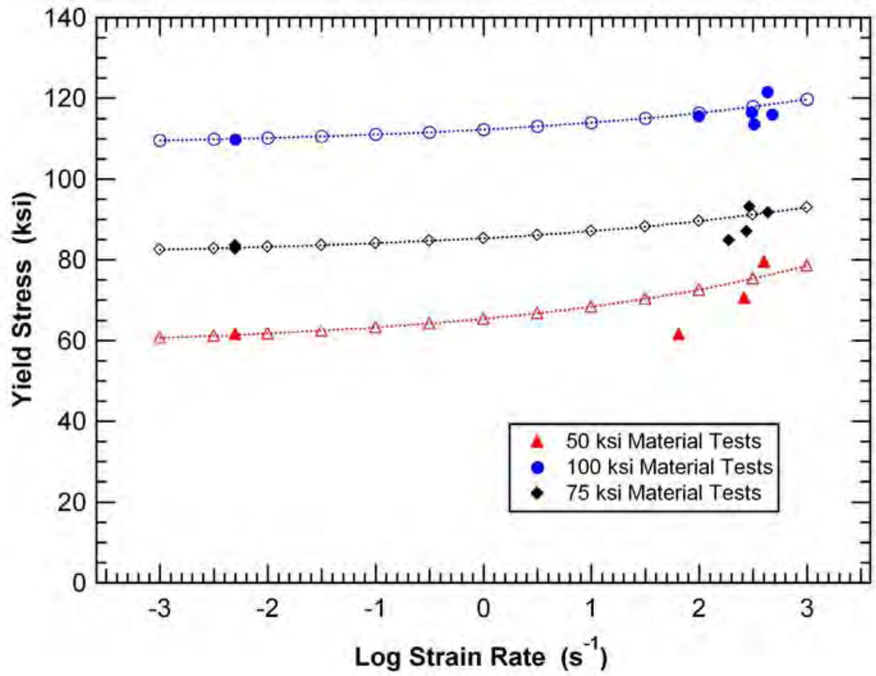


Figure C–12. Comparison of rate effects model and test data.

C.2.2 Failure Models

A challenge for calculating the aircraft impact response and damage to the WTC towers is the wide range of failure mechanisms that occur in both the aircraft and tower structures. These failures result from large scale deformations of the materials as well as from exceeding the strength of bolted, welded, and riveted connections. At the connection locations, complex behaviors are influenced by variations in material cross sectional geometry and material properties that produce large stress and strain concentrations.

The overall approach applied to modeling the impact damage and failure of the structures is to use engineering damage mechanics within the framework of the LS-DYNA finite element analyses. This analysis methodology is also referred to as local damage modeling and has been applied successfully in a wide variety of applications (e.g., Mudry 1985; Beremin 1983; Simons et al., 1999). Detailed fracture mechanics analysis of all the failures that occurred is beyond the scope of this project. The damage is calculated within each element in the impact simulations and damage development is based on local (element) quantities including plastic strain and stress state. When the specified failure criterion in an element is exceeded, the element is eroded (deleted) from the calculation. The erosion of elements allows for the propagation of failure through a structure.

Calculation of the failure of structural components is further complicated by the scale of the global impact analyses. Local damage modeling is often applied to the analysis of smaller components and failure initiation. In these applications, a relatively fine mesh can be applied, and damage regions around a local fracture can be resolved on a fine scale. For the global impact of an aircraft into a WTC tower, element sizes will have to be on average a few in. to maintain a model size at approximately 2 million to 3 million elements. At this resolution, the gradients around a fracture cannot be accurately resolved, and the damage criteria need to be adjusted to obtain the appropriate strength and ductility of the structures. In the following subsections, various local damage modeling techniques applied to estimate failure of the aircraft and tower structures are described.

Mesh Refinement Effects

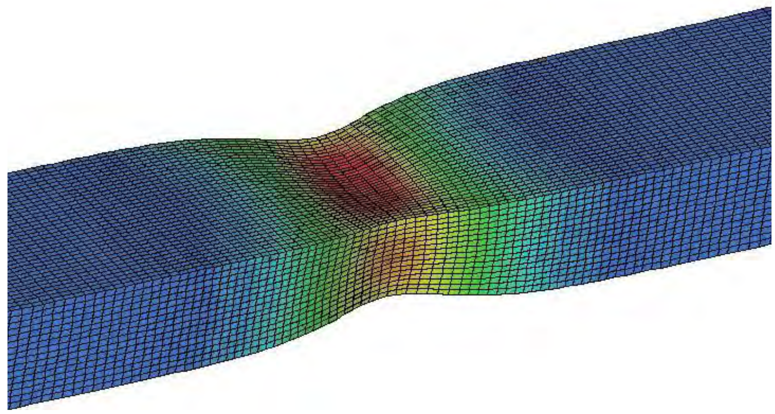
One of the significant modeling issues for the analysis of aircraft impact damage and failure is the effect of mesh refinement. The global impact analyses of the aircraft impacts into the WTC towers are very large analyses of complex structures and will require the refinement of the model to be reduced significantly from the detailed component analyses. As the mesh refinement is reduced, it is important to ensure that the damage mechanisms and extent of impact damage are properly captured.

A preliminary example of the effects of mesh refinement on the response was introduced earlier in the analysis of the material tests on the WTC tower steels. Figure C–1 showed both a fine mesh and a coarser mesh (referred to as medium mesh) version of the ASTM 370 rectangular tensile specimen for plate material. A comparison of the calculated necking behavior for two 75 ksi specimens immediately preceding failure is shown in Fig. C–13. The fine mesh is able to better resolve the strain gradients in the necking region and as a result has higher peak strain values at the same level of specimen displacement.

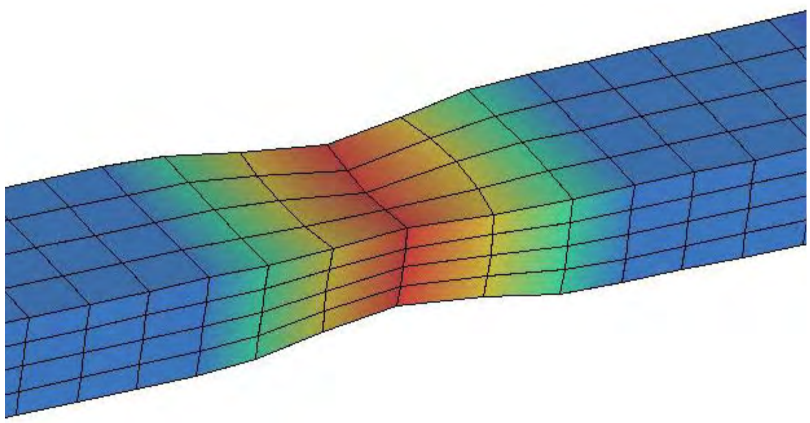
The effects of further reductions in mesh refinement required for global impact analyses can be demonstrated with the ASTM 370 tensile test example used in this section. The extreme limit for a coarse model of the tensile specimen would use a single shell element to model the entire specimen gage section. An example of coarse shell element model of the tensile specimen is shown in Fig. C–14. This coarse tensile specimen model would have a single stress and a single strain value for the gage section and does not have sufficient independent degrees of freedom to model the localization that occurs during necking. As a result, the critical plastic strain criterion for this single shell element model would be approximately equal to the engineering failure strain of 0.20 (see Fig. C–6).

The calculated engineering stress-strain behavior obtained with the three different mesh resolutions is shown in Fig. C–15. In these calculations, a maximum plastic strain criterion was used and the critical failure strain was shifted until each calculation failed at a value matching the average measured engineering failure strain. The corresponding critical plastic strains were 1.00 for the fine mesh resolution, 0.56 for the medium mesh resolution, and 0.18 for the coarse mesh resolution (shell element model).

The medium mesh resolution used in the above analysis of the tensile test corresponds to the mesh resolution applied in the exterior column component impact analyses described in Section C.4. Therefore, the critical failure strain of 0.56 would be carried forward to the detailed component analyses. The coarser shell element subassembly and global impact models are typically developed with shell elements and a resolution that might correspond to the coarse model shown in Fig. C–14. Therefore, the measured engineering elongation of the specimens is used for the critical strain. Additional modifications for regions with stress concentrations, such as along welds, is described later in this section.



(a) Fine mesh necking behavior (red = 75 percent plastic strain)



(b) Medium mesh necking behavior (red = 50 percent plastic strain)

Figure C–13. Calculated necking response in the 75 ksi tensile specimen.

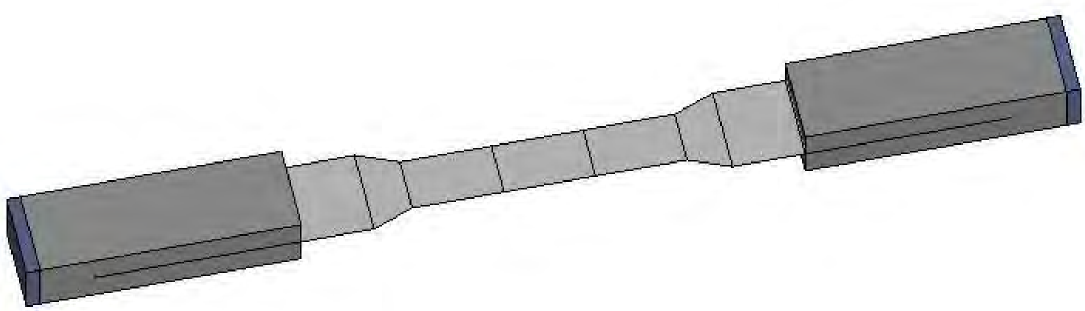


Figure C–14. Coarse shell element mesh for the 75 ksi tensile specimen.

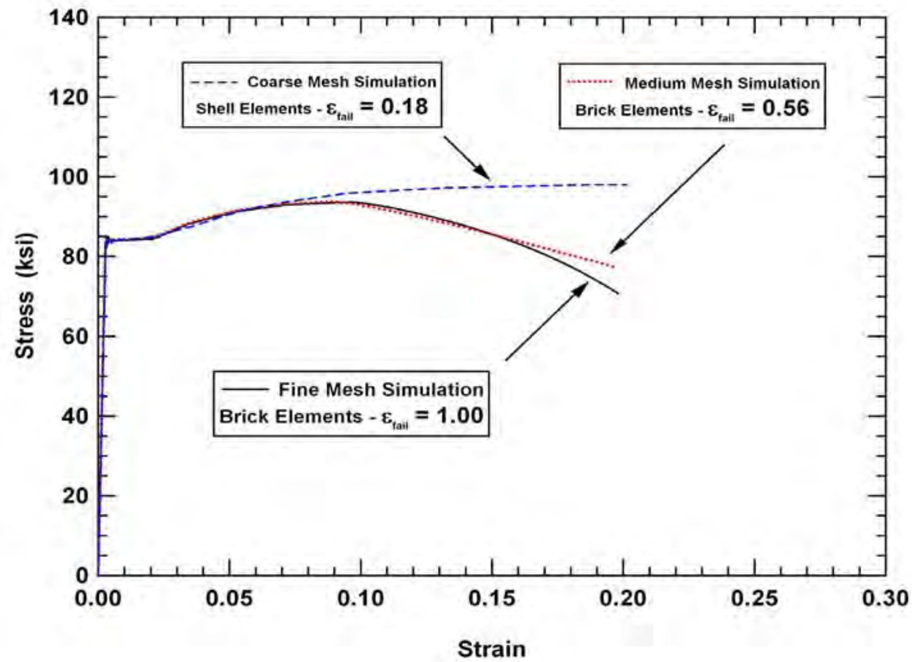


Figure C–15. Mesh refinement effects in the calculated 75 ksi tensile test.

Stress Triaxiality Effects

Using a local damage approach to model fracture with a damage model incorporating stress triaxiality effects has been successful in predicting the response and failure of welded steel structures to dynamic loads (Kirkpatrick, Giovanola, and Simons 1996). The strength of welded steel structures subjected to dynamic loads can be strongly influenced by the weld strengths and connection details. The following describes this approach to modeling fracture for impact damage to the WTC towers.

The damage analyses use a local damage approach within a finite element code to simulate the development of damage as the structure deforms. The fracture model is a simple form of a ductile fracture criterion (Mudry 1985). It assumes that failure of a material location occurs when the damage within a surrounding microstructural characteristic volume, V_{MIC}, exceeds a critical value. Mathematically, the damage function in the failure criterion can be written in the form:

$$D = \int \frac{d\varepsilon_{eq}^{p}}{\varepsilon_c(\sigma_{mean}/\sigma_{eq})} = 1 \qquad \text{over } V_{MIC} \approx (R_{MIC})^3 \qquad (6)$$

where:

D = normalized damage parameter

$d\varepsilon_{eq}^{p}$ = increment in plastic strain

$\varepsilon_c(\sigma_{mean}/\sigma_{eq})$ = critical failure strain as a function of the stress triaxiality

The stress triaxiality is defined as the ratio of the mean stress to the equivalent stress.

This critical strain function can be determined by a series of notched tensile tests with specimens of varying notch radii (e.g., Mackenzie, Hancock, and Brown 1977). V_{MIC} is a characteristic volume of the material, which can be interpreted as the critical microstructural process zone. In turn, R_{MIC}, the representative linear dimension of the volume V_{MIC}, can be associated with microstructural dimensions such as grain size or spacing of the microvoid nucleating inclusions. R_{MIC} is therefore a constant length dimension that can be used to introduce a scaling effect in the fracture simulations. In its present form, the model does not account for a possible strain rate sensitivity of the damage growth and has been calibrated using static data for the failure strain as a function of stress triaxiality.

The damage function in Equation (6) provides the failure criterion based on the accumulation of damage over the history of the material. However, this damage law requires an appropriate definition of a critical failure strain function. In this type of model, the performance of the damage law is typically better if the form of the damage law has a physical basis from observations of the microstructural failure processes. Previous studies have shown that the rate of void growth in the ductile fracture process is an exponential function of the stress triaxiality (Rice and Tracey 1969) in the form:

$$\frac{dR}{R_o} = 0.28\, d\varepsilon_{eq} \exp\left[1.5(\sigma_{mean}/\sigma_{eq})\right] \tag{7}$$

This type of void growth law can be used to develop a critical strain function of the form:

$$\varepsilon_c(\sigma_{mean}/\sigma_{eq}) = \alpha \left\{\frac{1.0}{\exp[1.5(\sigma_{mean}/\sigma_{eq})]}\right\} - \varepsilon_{shift} \tag{8}$$

where α and ε_{shift} are constants. The critical strain function in Equation (8), combined with the damage law in Equation (6), form the basis of the local fracture model.

The effect of stress triaxiality on damage development was measured by Project 3 for the core column steel materials using the specimen geometry shown in Fig. C–16. Only the core column materials were selected since the thickness of the components was sufficiently large to fabricate the notched round bar specimens. A series of 12 tests were performed using two different steel strengths and three different notch radii with a repeat test for each configuration. A summary of the notched round bar tests is given in Table C–2.

To determine the triaxiality in the three different notched round bar specimen geometries, models were developed of the specimens and the tests were analyzed. The models for the three different specimen geometries are shown in Fig. C–17. The models had a fine mesh (average element size of approximately 0.008 in.) capable of resolving the gradients in stress and strain across the specimen. It was found that the triaxiality and damage across the gage section is relatively uniform with a maximum along the axis of the specimen (failure is expected to initiate at the center of the specimen and propagate to the edges).

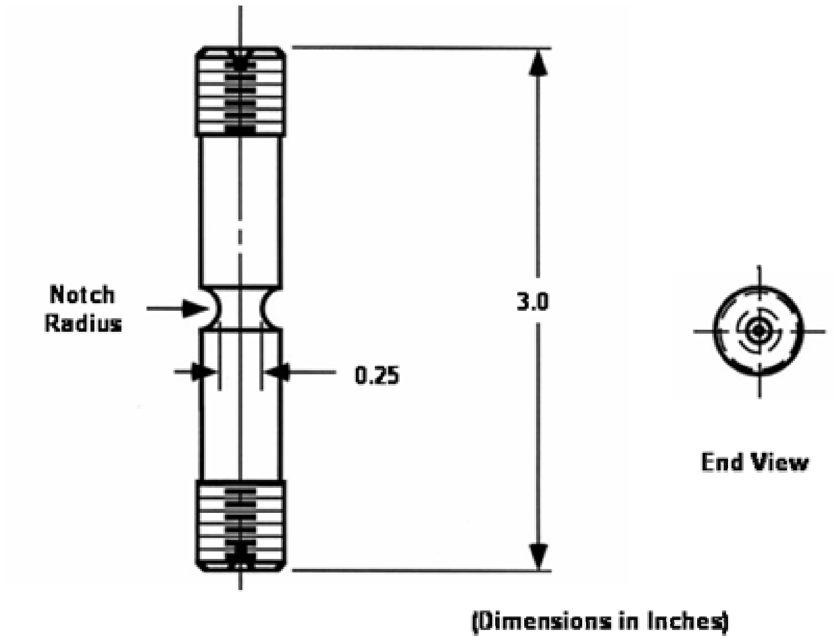


Figure C–16. Notched round bar specimen dimensions.

Table C–2. Summary of notched round bar tensile tests.

Specimen Number	Gage Dia. (in.)	Specimen Length (in.)	Notch Radius (in.)	Failure Dia. (in.)	Failure Strain	Stress Triaxiality
B6152-1-F1-1-1-05	0.2519	3.005	0.245	0.18646	0.2598	0.7
B6152-1-F1-1-1-06	0.2488	3.008	0.25	0.19223	0.2274	0.7
B6152-1-F1-1-2-01	0.2526	3.007	0.065	0.21466	0.1502	1.2
B6152-1-F1-1-2-02	0.2518	3.003	0.065	0.20518	0.1851	1.2
B6152-1-F1-1-2-03	0.2513	3.005	0.148	0.18571	0.2610	0.9
B6152-1-F1-1-2-04	0.2507	3.006	0.125	0.19837	0.2087	0.9
C71-1-F1-07	0.2506	3.005	0.245	0.19835	0.2085	0.7
C71-1-F1-08	0.2519	3.004	0.249	0.20325	0.1931	0.7
C71-1-F1-09	0.2487	3.005	0.125	0.21888	0.1199	0.9
C71-1-F1-10	0.2519	3.01	0.13	0.21818	0.1339	0.9
C71-1-F1-11	0.2514	3.005	0.064	0.23301	0.0732	1.2
C71-1-F1-12	0.2525	3.005	0.063	0.23919	0.0527	1.2

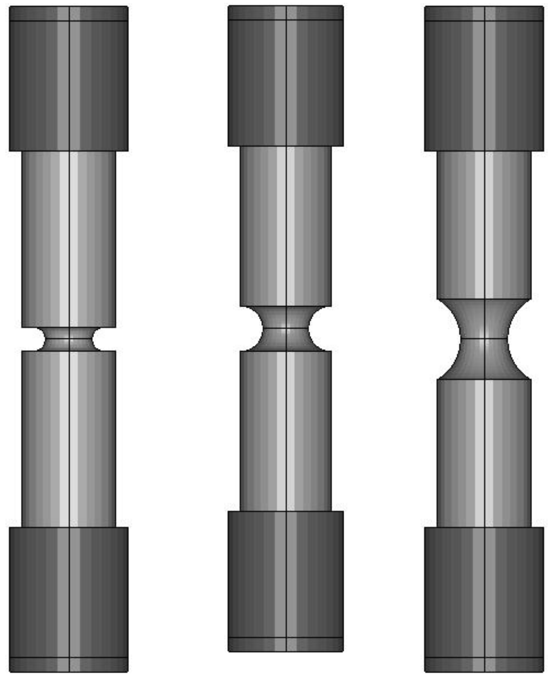


Figure C–17. Notched round bar tensile specimen models.

The calculated triaxiality at the specimen center as a function of damage, parameter D in Equation (6), for the three different notched round bar specimens is shown in Fig. C–18. The calculated responses for all three specimens show an initial spike in triaxiality and then settle to a constant stress triaxiality over a wide range of specimen deformation. The average value of triaxiality from these calculations is summarized in the last column of Table C–2.

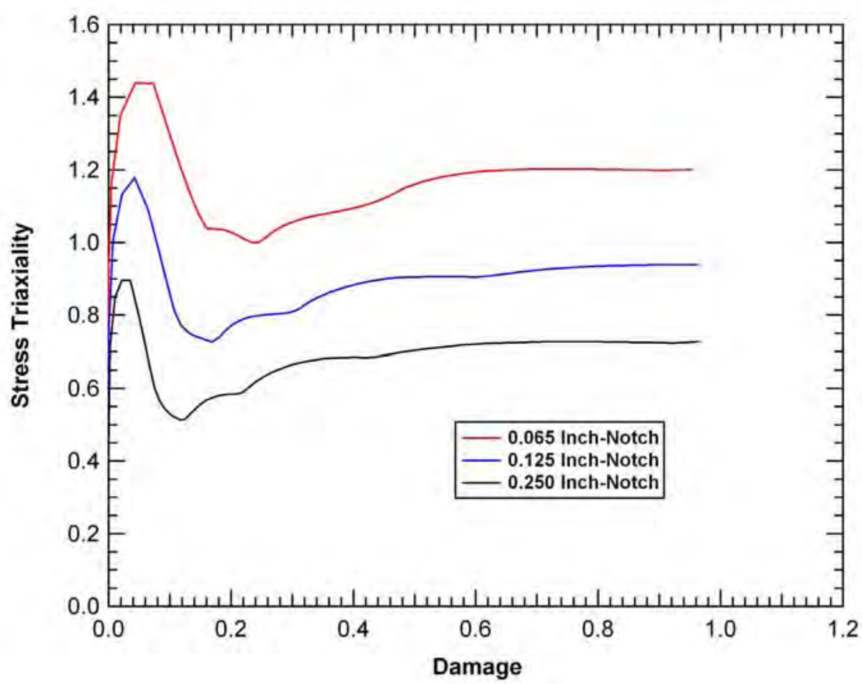


Figure C–18. Calculated notched round bar stress triaxiality.

A summary of the WTC steel notched round bar tests is shown in Fig. C–19. In addition, the figure shows failure criteria fit to the notched round bar data using Equation (8). For the WTC steels of interest, the presence of sulfide inclusions can be used to justify that the nucleation of voids occurs nearly instantaneously and ε_{shift} can be set to zero. The figure compares the C71-1-F1 tests with the failure criterion using a coefficient value α=0.5. Similarly the tests with the B6152-1-F1-1 steel (36 ksi yield) are compared to the failure criterion using a coefficient value α=0.90. Although neither set of material test is in perfect agreement with the failure criteria, the overall trends of the aggregate data set match and suggest that the form of the failure criteria is appropriate for the steels.

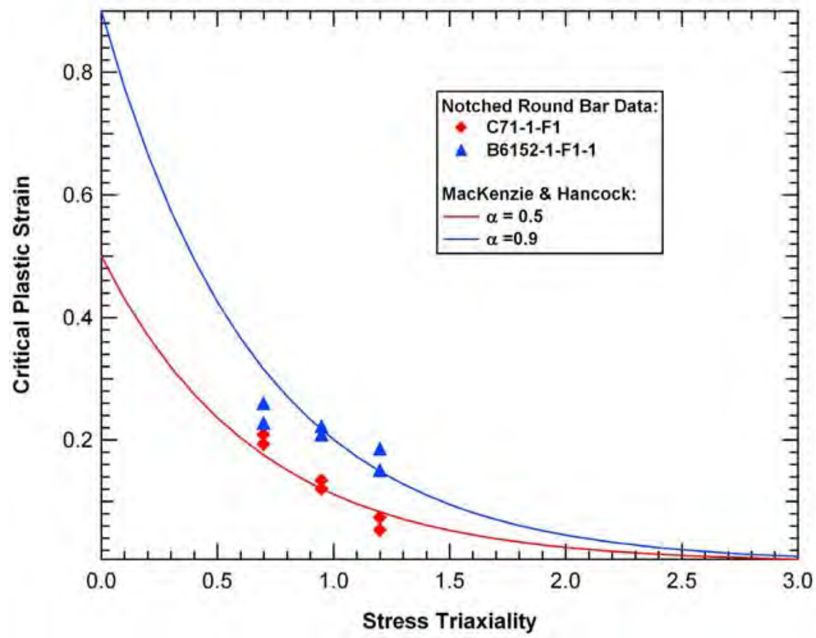


Figure C–19. Calculated critical plastic strain as a function of notched stress triaxiality.

A user-defined constitutive and failure model was developed and incorporated into LS-DYNA in this project. The model is identical to the piecewise linear constitutive model applied previously for the analysis of the WTC tower steels with the addition of the above stress triaxiality damage function incorporated. The simulations of the B6152 steel tensile and notched round bar tests were repeated with the user-defined constitutive model. The load-displacement relationships were compared to the previous analyses to validate the stress-strain relationships in the user-defined constitutive model. The calculated failure of the specimens is compared to the corresponding failure criteria in Fig. C–20. The agreement of the failure strains is a validation of the damage algorithm within the user-defined constitutive model.

In the following subsection, the user defined constitutive and damage model is applied to investigate failure of components in the WTC tower structure. A region of the WTC towers where the stress triaxiality is expected to be relatively high and play a role in the failure strength is the welded joints in the exterior column flanges and webs. This welded connection is analyzed below.

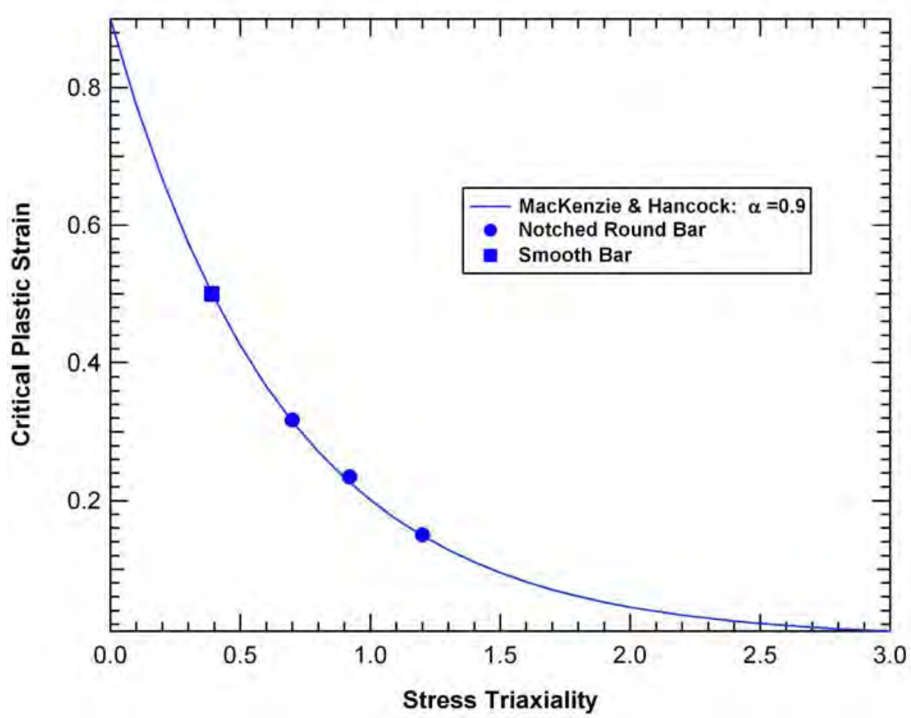


Figure C–20. Simulation of the notched and smooth round bar tests.

Weld Zone Constitutive Modeling

Photographs of the WTC towers immediately after impact, and inspection of the recovered exterior wall panels in the impact zone, have shown that failure along the weld zone was a characteristic feature of the impact damage. An example exterior column from the impact zone that has significant fractures along the weld zone for the outer web is shown in Fig. C–21. The amount of energy dissipated by these weld fractures is very small compared to the overall impact energy. As a result, the failure of the welds will have little effect on the subsequent damage to the interior structures and contents of the towers. However, to capture the damage mechanisms of the impacted exterior columns and to develop models and failure criteria for the global impact analysis, a failure model for the weld zone is required.

Modeling of the constitutive behavior for the weld and heat affected zone (HAZ) material is a challenging task due to the lack of significant material testing in these regions. To date, the data obtained on the weld properties consist of a micrographic characterization of an exterior column weld with microhardness indentation tests performed at various locations across the weld geometry. The specific weld geometry and microhardness characterization locations are shown in Fig. C–22. The corresponding hardness measurements across the base, HAZ, and weld materials are shown in Fig. C–23.

Figure C–21. Photograph of an exterior column with weld zone fractures.

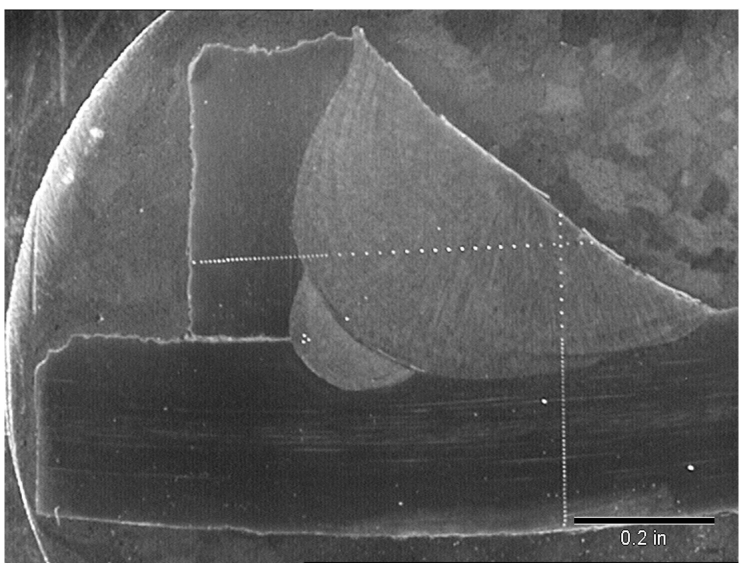


Figure C–22. Micrograph of an exterior column weld geometry.

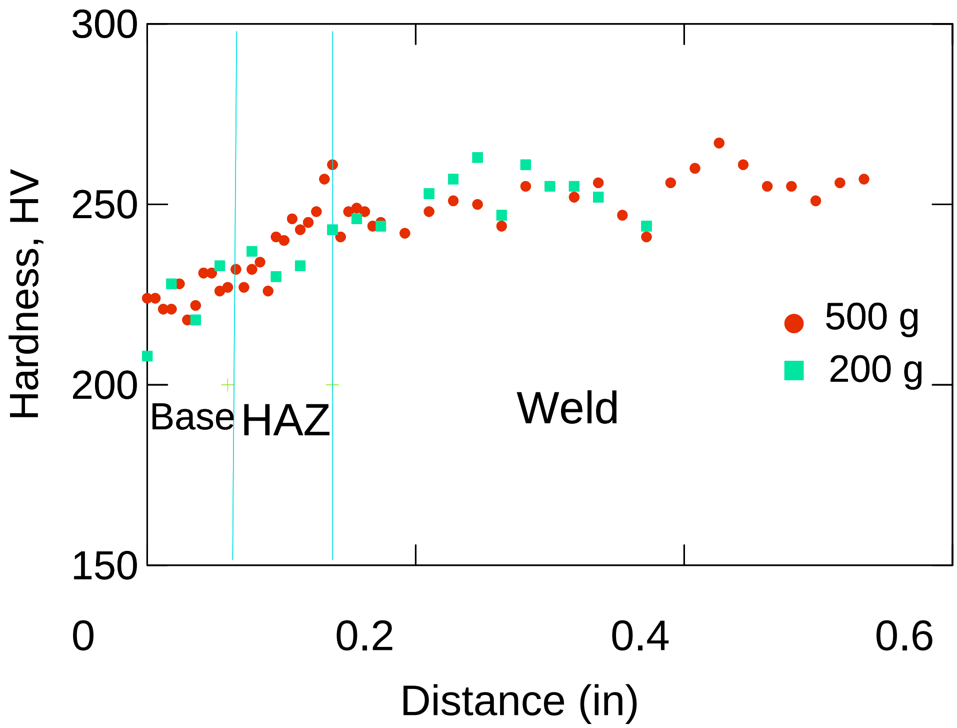
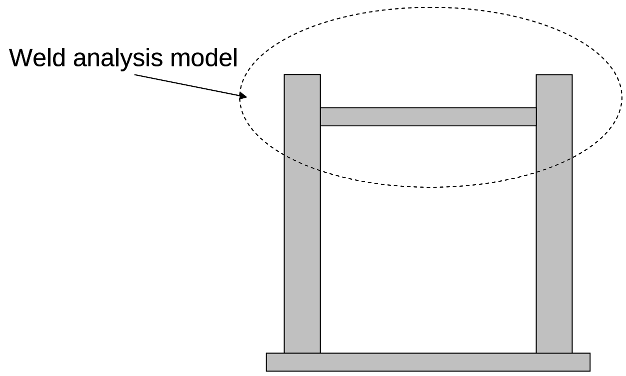


Figure C–23. Microhardness characterization of the weld and HAZ materials.

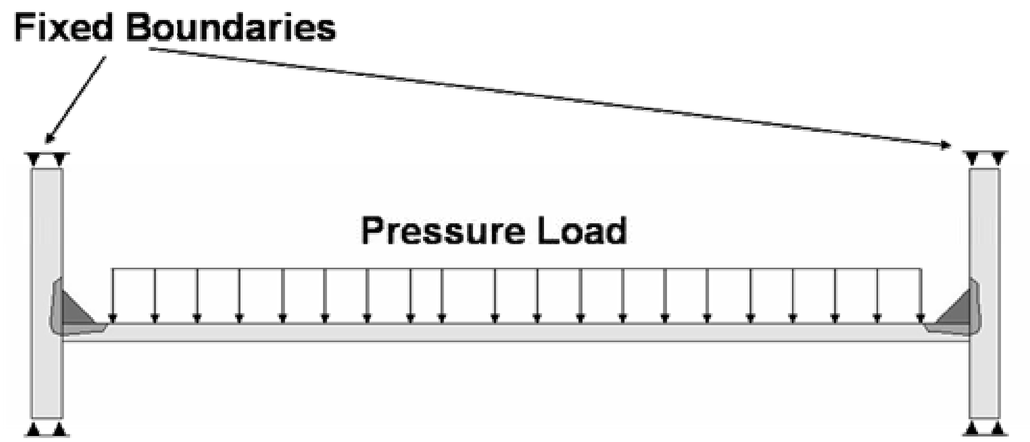
The microhardness measurements shown in Fig. C–23 were used to develop approximate plasticity behaviors for the weld and HAZ material regions. The approach used was to use the base material constitutive behavior and shift the flow stress of the HAZ and weld materials by 12 percent and 18 percent, respectively. These shifts correspond to the relative magnitude of the average measured hardness in each material region.

Analyses were performed to investigate the weld failure behavior using a detailed model of a simplified two-dimensional (2-D) weldment geometry representative of an exterior column side flanges and welded outer web as shown in Fig. C–24. A short duration pressure load is applied across the front web to introduce a dynamic loadcondition on the welded connection. The boundary conditions were approximated by fixing the displacements at the top and bottom of the side plates, seen in Fig. C–24 (b). The entire model is constrained to a plane strain condition. The weldment model includes weld and HAZ regions as shown in Fig. C–24 (c).

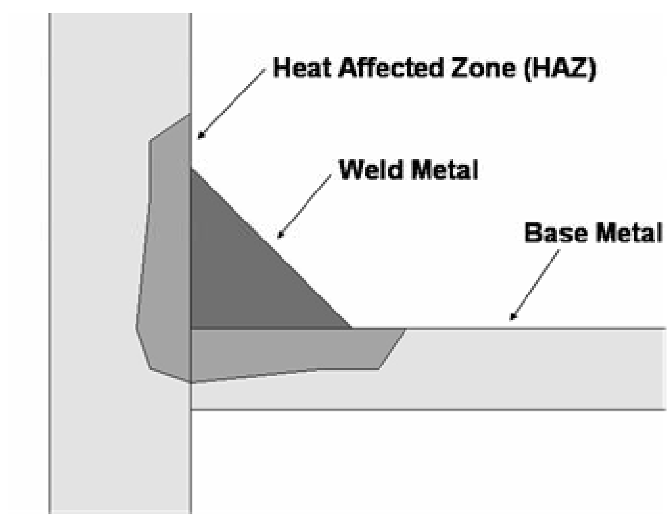
The base metal for the weldment is the 55 ksi steel. An approximate damage criterion was developed for the steel based on the results of the tensile tests on the material. The criterion is that of Equation (8) with a coefficient value α=0.92. This value was obtained by using an approximate value for the average triaxiality of the tensile specimen and the failure strain obtained from the previous analyses of the tensile tests described earlier in Section C.2.1. The estimate of the average triaxiality is approximate since the triaxiality is changing in the region of the necking prior to failure. The user-defined constitutive model with this estimated damage model was used to simulate the tensile test. The comparison to the data and previous analyses is shown in Fig. C–25. The approach underestimated the engineering failure strain by approximately 10 percent.



(a) Cross section of an exterior column



(b) Weld analysis model



(c) Weld region detail

Figure C–24. 2-D weldment model developed for analysis of failure behavior.

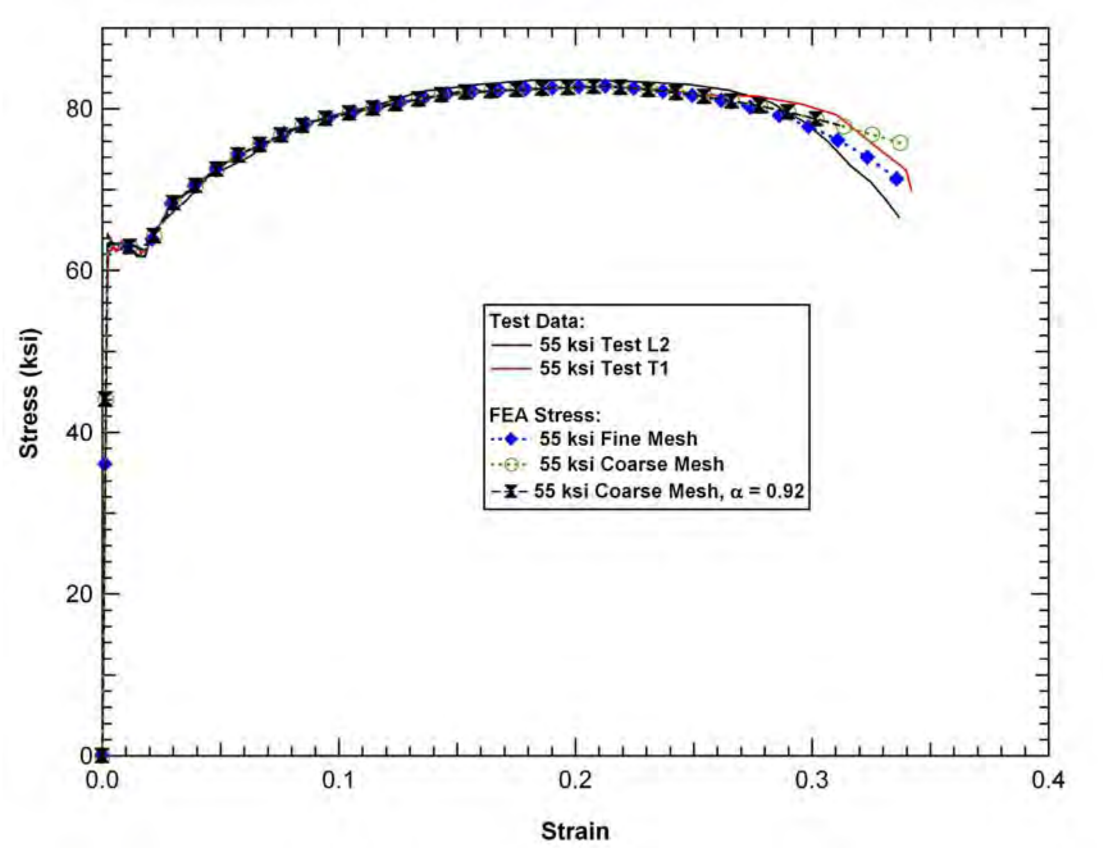
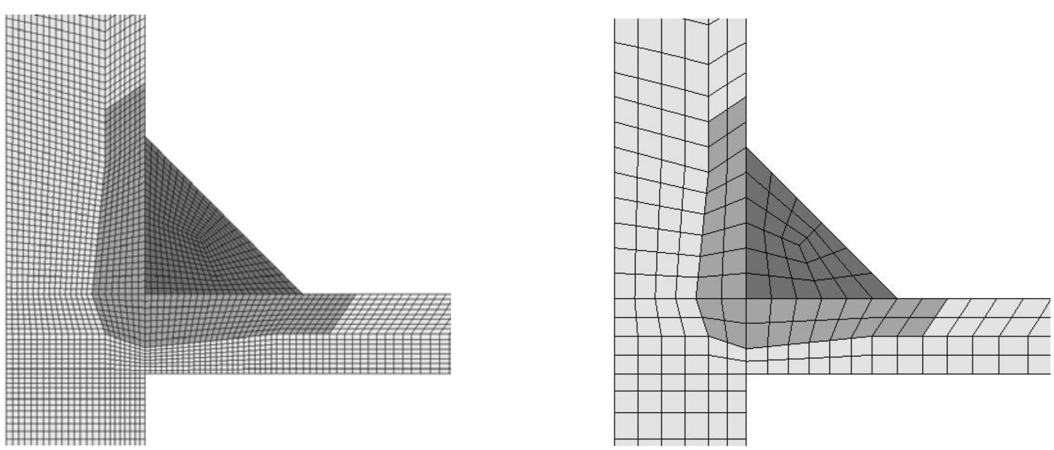


Figure C–25. Simulation of the 55 ksi steel tensile test.

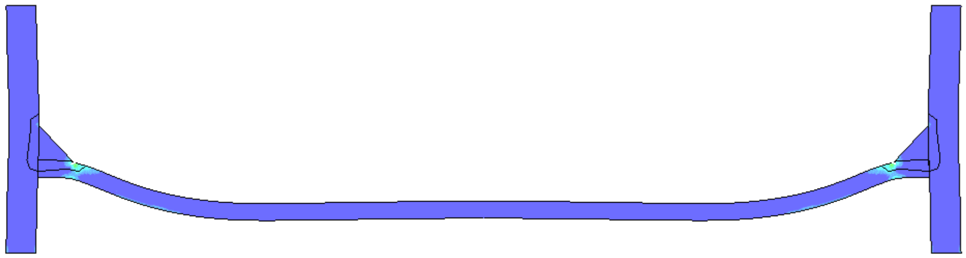
Two different mesh refinements were developed for the weldment model as shown in Fig. C–26. The model developed using the fine mesh has 10,240 solid brick elements, as shown in Fig. C–26 (a). The fine mesh model should have sufficient resolution to capture the gradients in stress and strain around the weld and HAZ. The model developed using the medium mesh has 702 solid brick elements, as shown in Fig. C–26 (b). This model was developed with a mesh refinement that could be applied to a three-dimensional analysis of a column for component impact analyses.



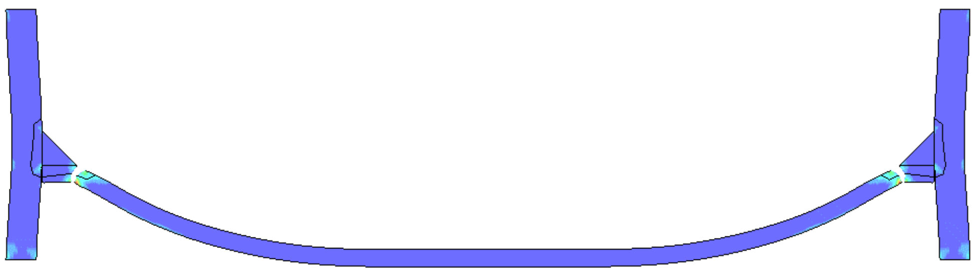
(a) Fine mesh resolution **(b) Medium mesh resolution**

Figure C–26. Mesh resolutions used in the 2-D weldment failure model.

An example fracture analysis of the weldment model using the fine mesh model is shown in Fig. C–27. The loading was provided by a rectangular pressure pulse with 0.10 ms duration and amplitude of 38 ksi (262 MPa). The failure initiates at the toe of the weld and propagates through the heat affected zone and outer web plate. This is the most common weld failure mode observed in the recovered WTC exterior columns. The pressure load results in a downward velocity of the front web. The motion at the ends of the web is restrained by the welds at the flanges and a moving plastic hinge behavior develops in the web. The deformations result in bending and tension at the weld location and eventually the local deformations initiate a fracture at the toe of the weld that propagates through the HAZ and web base metal. The details of the calculated fracture behavior are shown in Fig. C–28.

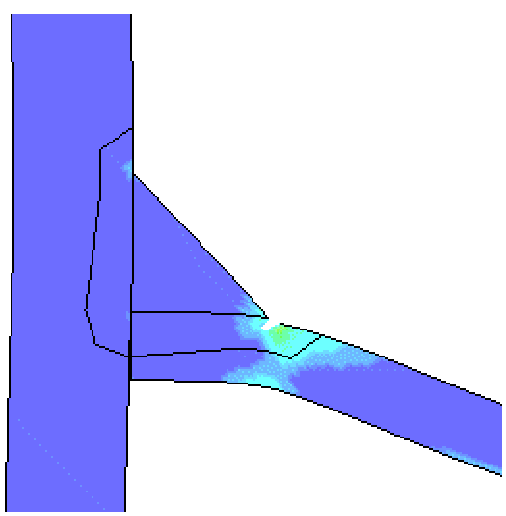


(a) Initiation of fracture

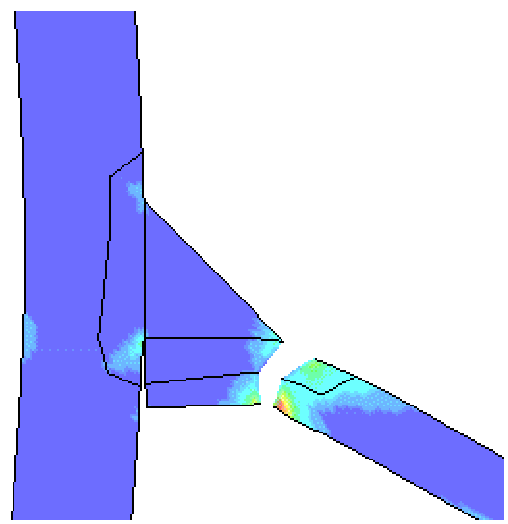


(b) Complete fracture through the web

Figure C–27. Calculated weldment deformations and failure.



(a) Initiation of fracture



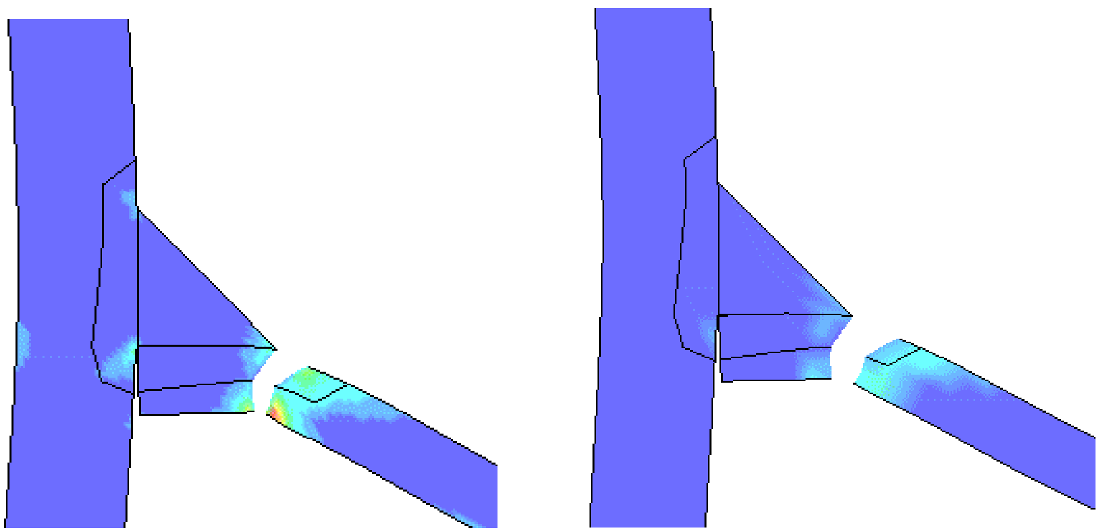
(b) Complete fracture through the web

Figure C–28. Detail of the calculated weldment failure behavior.

As described in previous analyses, the application of a coarser mesh resolution does not resolve the same gradients in stress and strain and the failure criteria need to be adjusted. The failure criteria parameters used in the two analyses are given in Table C–3. The comparison of the fracture behavior for the two mesh resolutions is shown in Fig. C–29. Similarly the kinetic and internal energies for the two calculations are compared in Fig. C–30. The comparisons show that both models have the same deformation and failure modes at very similar energy levels.

Table C–3. 2-D weldment model comparison.

Mesh Refinement	Fine	Medium
Element class	Brick	Brick
Base metal failure coefficient (α)	1.35	0.76
HAZ metal failure coefficient (α)	0.81	0.45
Weld metal failure coefficient (α)	1.10	0.62



(a) Fine mesh resolution **(b) Medium mesh resolution**

Figure C–29. Mesh refinement effects on calculated weldment failure behavior.

The remaining stage in developing the weldment failure model is to perform three-dimensional component impact analyses of the exterior column and develop a coarser shell element description of the weld region that can be applied in the global aircraft impact analyses. The problem analyzed is the drop test configuration shown in Fig. C–31. The drop test configuration shown has a 550 lb steel impactor with an impact velocity of 37.4 mph. The impactor is 12 in. wide and 5 in. across. The nose of the impactor has a reduced area that is 12 in. wide and 2 in. across with a one-half in. radius rounded edge around the impact face. The length of the column section represents a portion of an exterior column between spandrels.

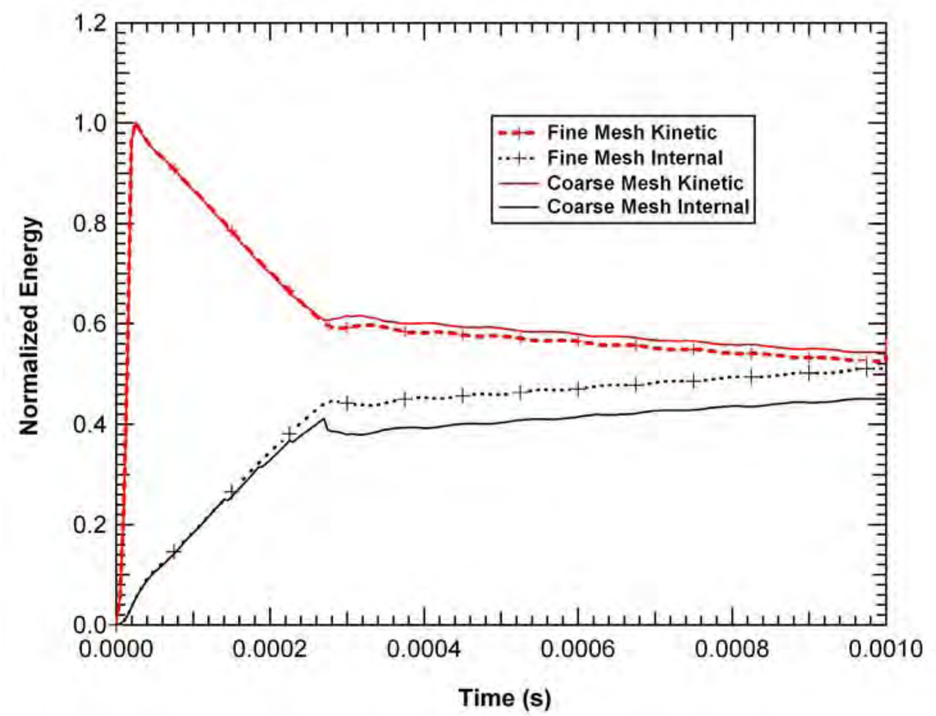


Figure C–30. Calculated energy balance for the 2-D weldment models.

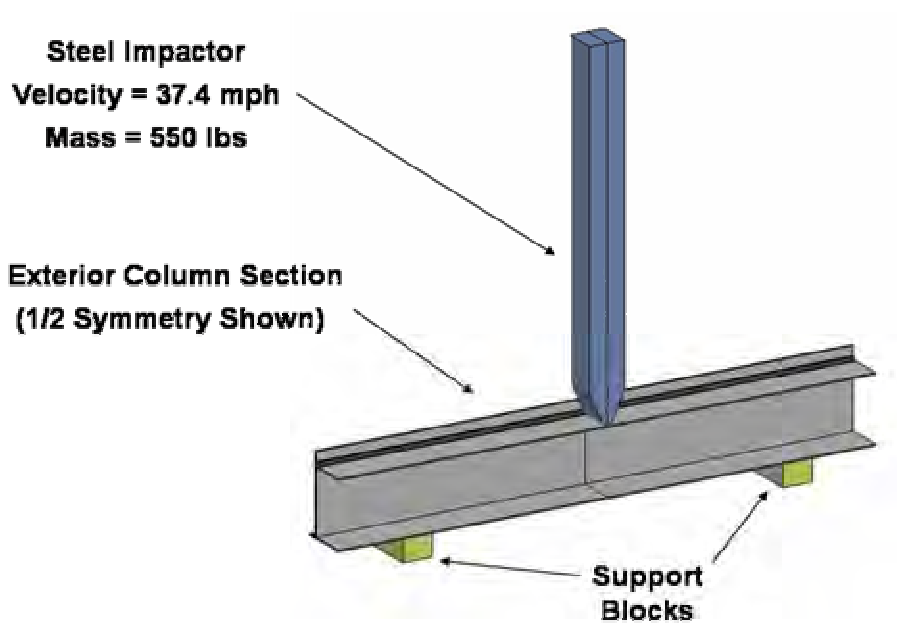
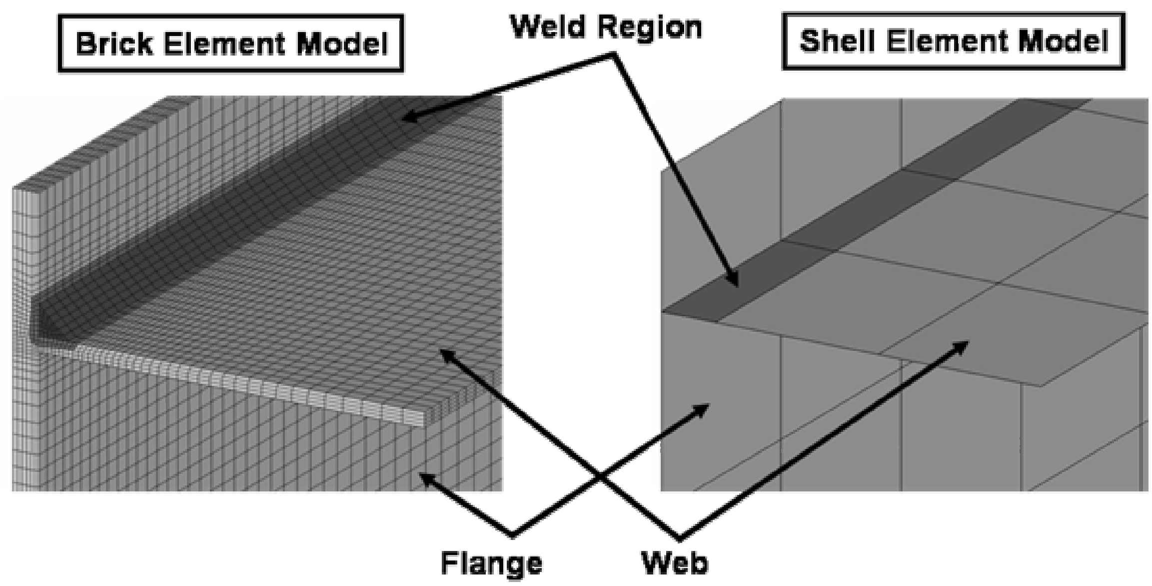


Figure C–31. Drop test model for column weld fracture analysis.

The two different column and weld models applied are shown in Fig. C–32. The medium mesh resolution was previously applied in the 2–D weldment fracture analyses. The half symmetry medium mesh column model contained 63,680 brick elements. The coarse shell element model has significantly fewer elements with approximately 4 in. elements to define the column and 1 in. wide elements in the weld zone. The half symmetry coarse shell element column model contained 144 linear shell elements. Obviously, the coarse shell element model is not capable of capturing the stress and strain gradients to the same extent as the brick element model. As a result, the shell element model applies the simpler critical plastic strain criterion available in the default piecewise linear constitutive model.



(a) Medium mesh resolution **(b) Coarse mesh resolution**

Figure C–32. Models developed for column weld fracture analysis.

The drop test was simulated first using the medium resolution brick element model. The results of the calculated impact response and failure were then used to develop an appropriate failure strain for the shell element model weld zone. The strain profile calculated in elements along the upper surface of the web in the weld fracture region is plotted in Fig. C–33. The figure shows a strong gradient in the calculated strains with the peak strains of approximately 15 percent plastic strain near the toe of the weld. A corresponding critical strain of 8 percent was selected for the corresponding one-in.-wide single shell element weld zone. This strain is indicated by the red line in Fig. C–33.

The comparison of the resulting impact behavior for the two models is shown in Fig. C–34. Both models have similar impact deformations and the lengths of the weld failures are in good agreement.

These analyses have been used to develop improved engineering fracture criteria in the weld region. There is still significant uncertainty in the details of the weld geometry and damage development in the weld region with variations in material properties and stress distributions. However, reasonable bounds on the effects of these variations will be assessed later in the uncertainty analyses.

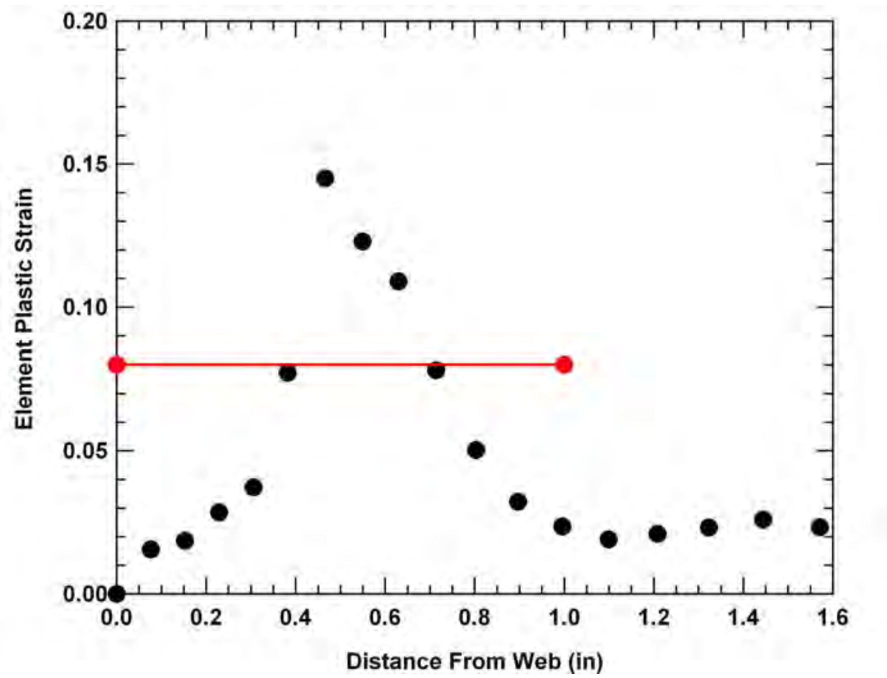
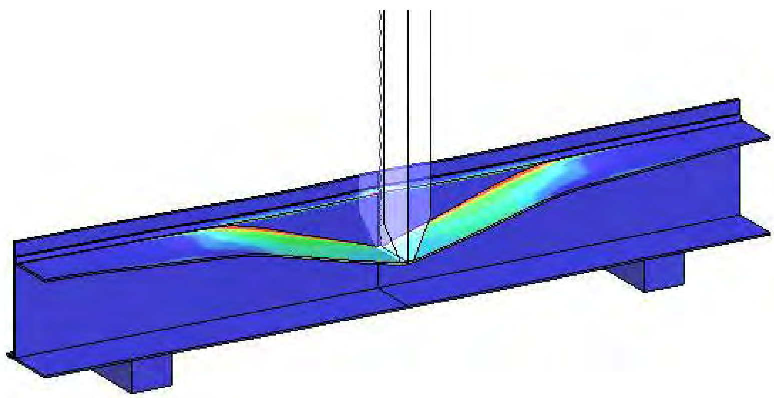
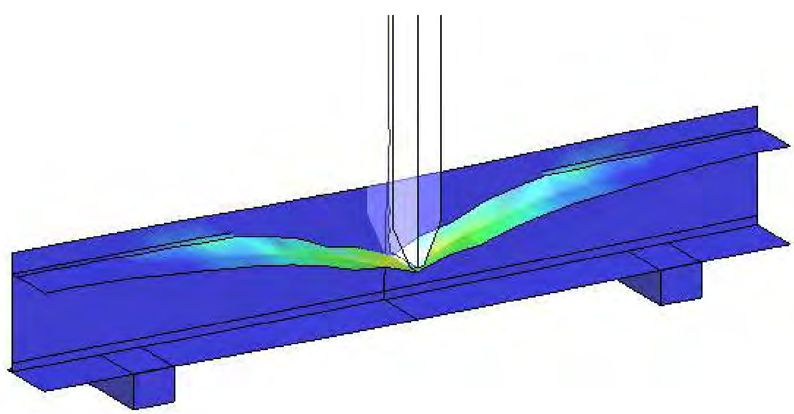


Figure C–33. Calculated strain profile for the weld zone.



(a) Medium resolution brick element model



(b) Coarse resolution shell element model

Figure C–34. Calculated drop test fracture behavior.

Bolt Material Constitutive Modeling

The primary bolts of interest for the impact analysis are those used at the connections between the exterior columns on the WTC towers. Within the impact zone, the connections are typically made using 0.875 in. in diameter steel bolts grade A325. Initially, there was no test data available that could be applied to develop a bolt model. The modeling approach was to develop a brick element bolt model and use it to develop a corresponding beam element bolt model for the majority of the impact analyses. A description of these bolt analyses are given in Section C.4.2, along with the component analyses.

Subsequently, a series of tests was performed by Project 3 on bolts recovered from the WTC towers. A summary of the bolt testing is given in Fig. C–35. The bolts were found to yield at a load of approximately 50 kip and have an ultimate failure load of approximately 68 kip. The measured elongation at failure is approximately 0.18 in.

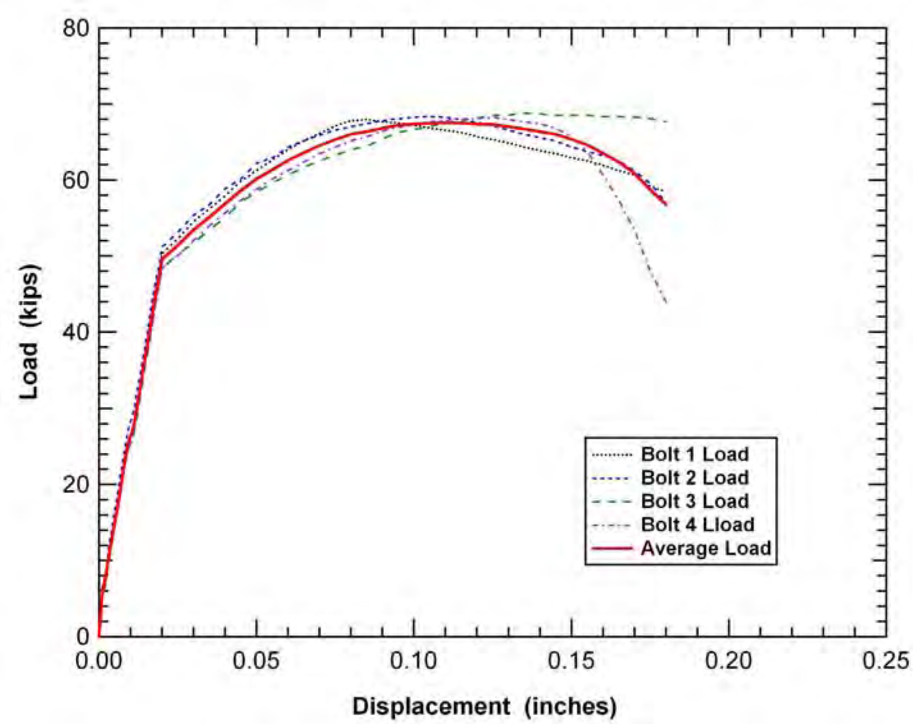


Figure C–35. Measured bolt load-displacement behavior.

The beam element model for the bolt, described in Section C.4.2, was compared to the bolt test data. The comparison showed good agreement in the strength of the bolt, but it also indicated that the beam model overestimated the ductility. This may be a result of not capturing the details of the stress concentrations in the region of the threaded connection and nut. The bolt test data was used to correct the ductility of the beam element bolt model and the resulting comparison of the model and data is shown in Fig. C–36. The bolt model shows a bilinear elastic-plastic behavior that is stiffer in the elastic region and yields at a higher stress level than the data. The inability of the simplified model to capture stress gradients in the regions of the bolt head, threads, and nut may cause this type of response. However, the overall strength and ductility of the model and test data as well as the strain energy capacity agree reasonably well that further model development was not required.

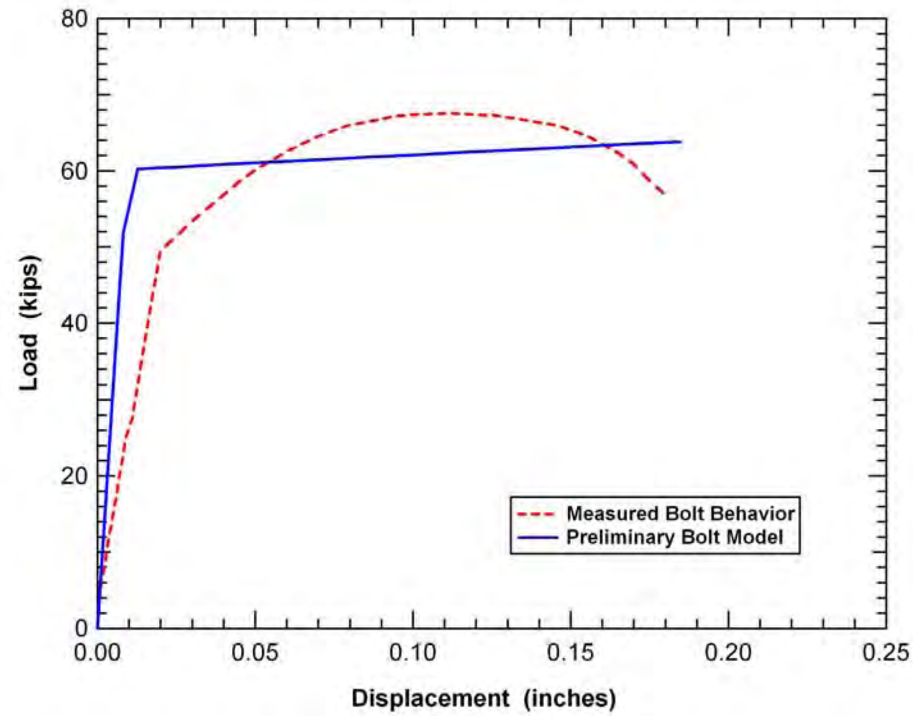


Figure C–36. Comparison of the measured and calculated bolt behavior.

C.2.3 Concrete Constitutive Models

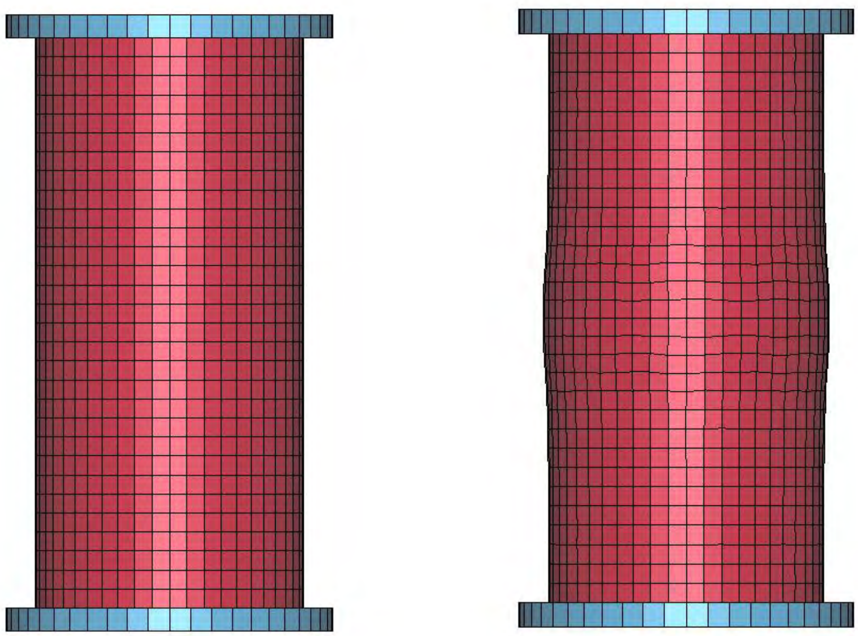
There are several concrete models in LS-DYNA. Each has different capabilities for modeling rate effects and nonlinearity associated with damage and failure behavior. Potential concrete models in LS-DYNA are Types 5 (soil and crushable/non-crushable foam model), 16 (pseudo-tensor concrete model), 25 (kinematic hardening cap model), 78 (soil and concrete model), and 111 (Holmquist-Johnson-Cook concrete model). An important factor in determining the behavior of a concrete structure in compression or bending is its lateral confinement. The concrete floor slabs in the WTC towers were not highly confined so a material model suitable for this loading condition is needed.

In this study, the ability to accurately model the low confinement damage and softening behavior is important. Damage caused by cracking in the concrete degrades the strength in the low confinement regime. Inclusion of this damage growth provides a more accurate representation of the stress-strain response. Based on this capability, the LS-DYNA material Type 16 (pseudo-tensor concrete model) is selected for modeling the concrete floor slabs. This model also accounts for the high strain-rate sensitivity of reinforced concrete.

As implemented in LS-DYNA, the pseudo-tensor model can be operated in two major modes: (1) a simple tabular pressure-dependent yield surface, and (2) a model with two pressure-dependent yield functions and a damage-dependent function to migrate between curves. The first option is well suited for implementing standard geologic material behaviors such as a Mohr–Coulomb yield surface with a Tresca limit and has been used successfully for the analysis of ground shock and soil-structure interactions under high confinement. The second option, applied here, allows for implementation of tensile failure and damage scaling, which are more dominant material behaviors at low confinement.

The pseudo-tensor model, as applied to the analysis of the lightweight concrete in the WTC towers, has two pressure-dependent yield functions. By defining suitable yield functions for the undamaged and fully damaged concrete and an appropriate tabular interpolation between the curves, the behavior of the damage under low confinement can be properly captured.

Material constitutive parameters for the pseudo-tensor model were developed for a 3 ksi compressive strength lightweight concrete. A simulation was performed of a standard unconfined concrete compression test to check the constitutive model behavior. The simulated behavior of the concrete specimen is shown in Fig. C–37. The calculated compressive stress-strain response is compared to measured compression data for 2.3 ksi and 3.8 ksi strength concretes in Fig. C–38 (Wischers 1978). Currently, the same material parameters are being used for the concrete in both the core (normal weight concrete) and truss floor (lightweight concrete) areas.



Initial Configuration **2 % Compression**

Figure C–37. Finite element analysis of the unconfined compression test.

C.2.4 Aircraft Constitutive Models

No material testing was performed to characterize the structural materials in the aircraft or develop the constitutive model parameters for these materials. Therefore, the constitutive and failure properties for the aircraft materials were developed from data available in the open literature. The principal sources of data for the airframe materials are the Military Handbook (MIL-HDBK-5F), 1987 and Aerospace Structural Metals Handbook (Brown et al. 1991). Additional sources of data are used to verify and supplement the information obtained from these primary data sources.

Complete engineering stress-strain curves were provided in the MIL-HDBK-5F for various 2024 and 7075 aluminum alloys that are commonly used in the construction of the 767 airframe structures. These

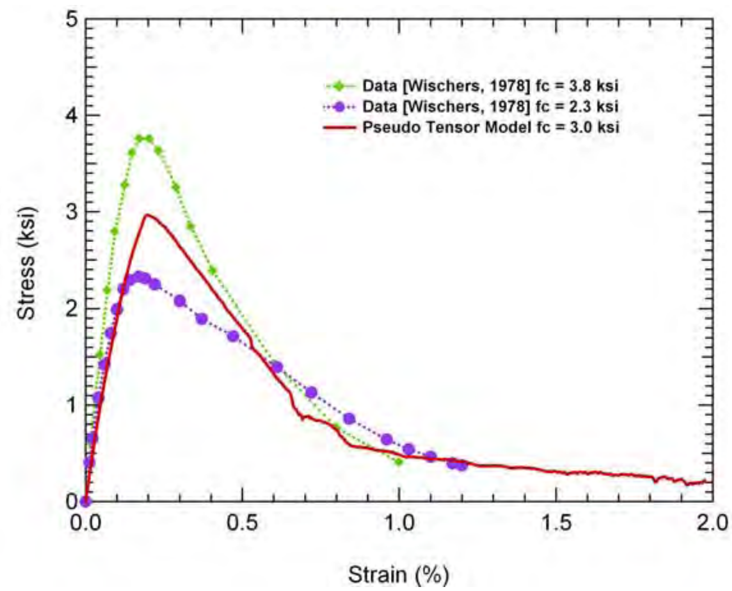


Figure C–38. Comparison of the calculated unconfined compression behavior with concrete compression test data.

curves were digitized for the various 2024 and 7075 alloys as shown in Fig. C–39 and Fig. C–40, respectively. Representative stress-strain curves were then converted into true stress and true strain as described earlier and used to develop tabular curves for constitutive models. The calculated true stress-strain curves and tabular constitutive model fits are shown in Fig. C–41 and Fig. C–42, respectively.

C.3 DEVELOPMENT OF AIRCRAFT MODEL

Development of the model of the Boeing 767-200ER aircraft is being accomplished through a three-step process. These are data collection, data interpretation and engineering analysis, and finally meshing of the structure. Data collection is nearing completion, and additional efforts will mainly focus on final details of non-structural contents and material properties for primary structural parts. Reviewing, organizing, and analyzing the data as well as meshing of the structure are also at the final stages.

C.3.1 Aircraft Data Collection

Data collection for the Boeing 767-200ER is nearing completion. Significant information on the aircraft structure and contents has been gathered from (1) documentary aircraft structural information, and (2) data from measurements on Boeing 767 aircraft.

C.3.2 Description of the Aircraft Model

The model for the Boeing 767-200ER is nearing completion. Certain components, such as the PW4000 engine shown in Fig. C–43, have been completed. Construction of the wings is in the final stage with only the inboard flaps and ailerons left to be modeled. Work has begun on the fuselage as well. Some details of the airframe model are shown in Fig. C–46 later. The LS-DYNA model of the aircraft is generated and meshed using the TrueGrid software (TrueGrid Manual 2001).

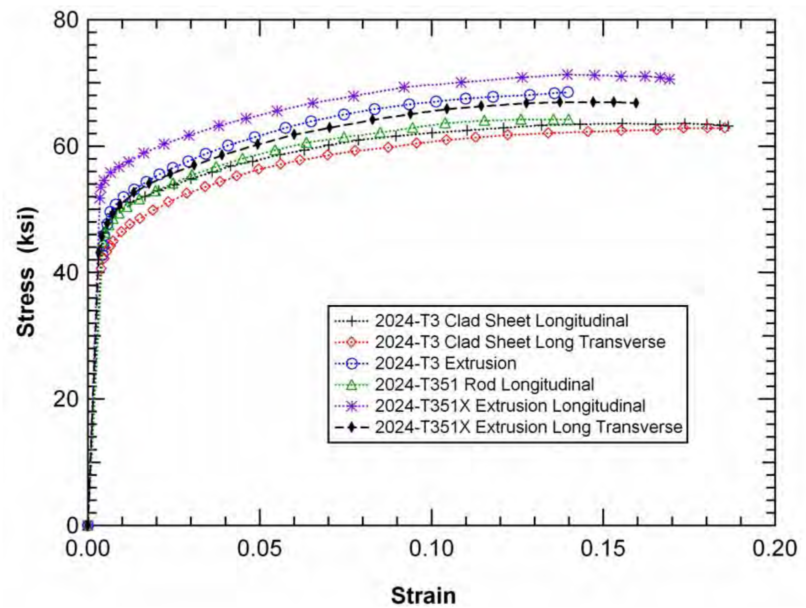


Figure C–39. Digitized engineering stress-strain curves for various 2024 aluminum alloys.

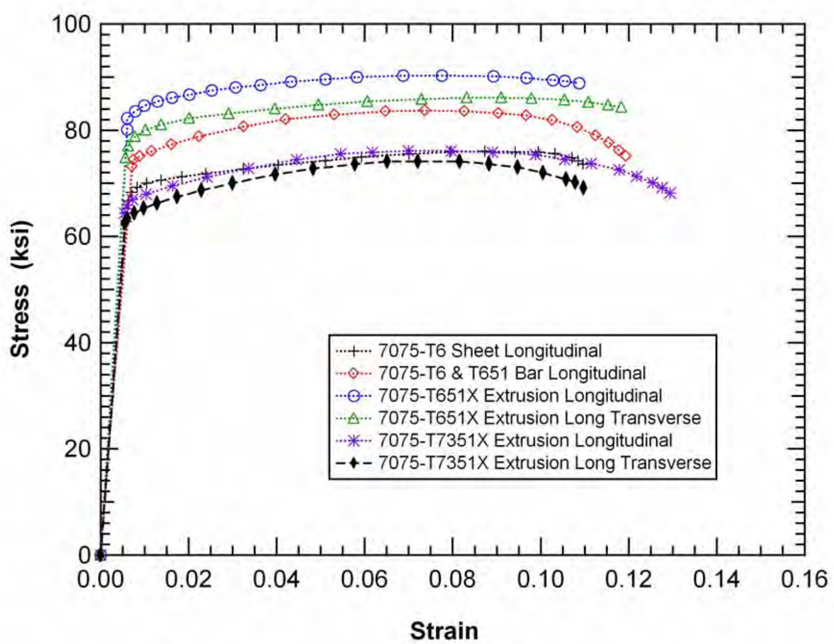


Figure C–40. Digitized engineering stress-strain curves for various 7075 aluminum alloys.

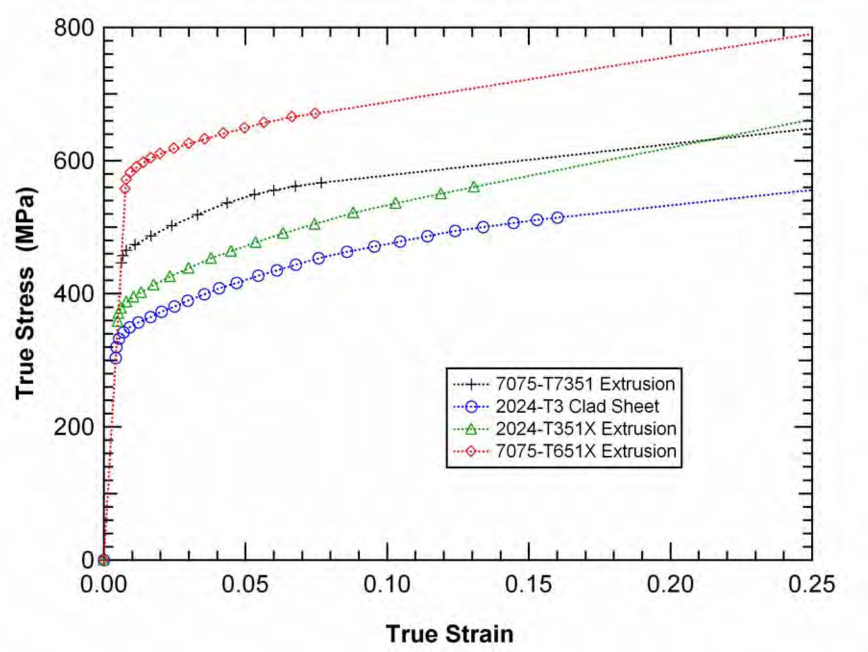


Figure C–41. True stress-strain curves developed for various aircraft aluminum alloys.

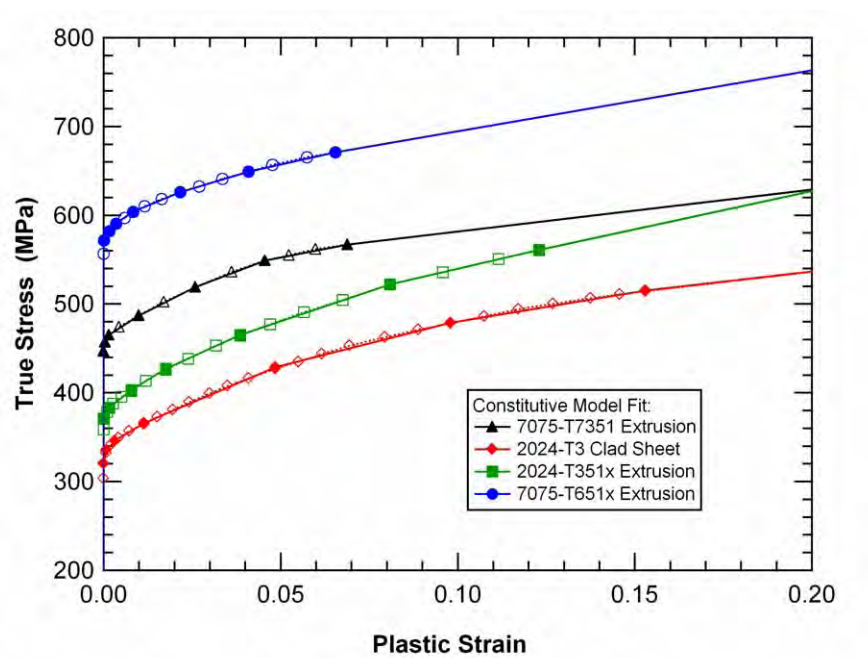


Figure C–42. Tabular true stress-strain curves developed for various aircraft aluminum alloys.

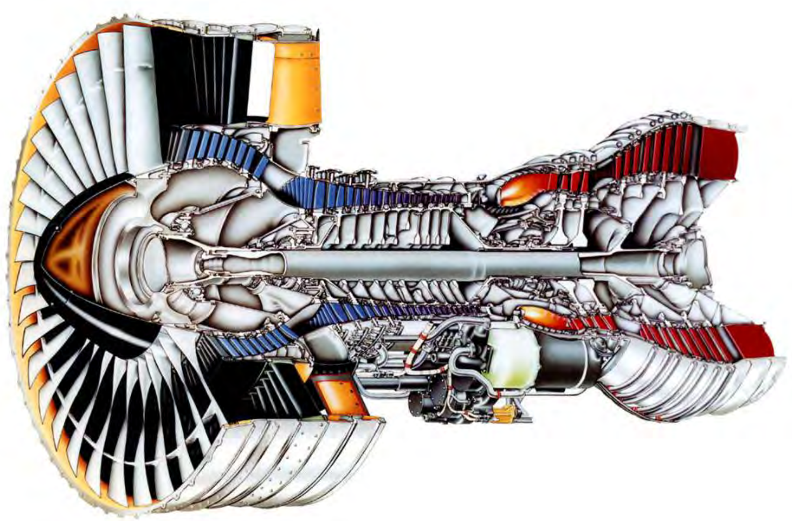


Figure C–43. Pratt and Whitney PW4000 turbofan engine.

Engine Model Development

The Pratt and Whitney PW4000 turbofan engine has a very complex structure as shown in Fig. C–43. The engine is an important component of the aircraft with the potential to produce significant impact damage to the WTC tower structures (e.g., fail core columns). As a result, special care was given to the development of the engine model to include all the details of the engine construction.

To develop a structural model of the engine, the primary structural components in the engine were identified and approximated with simplified geometry as illustrated in Fig. C–44. Known engine dimensions were used to determine the scale factor for the drawing. The simplified geometry of the engine structures could then be captured using a common digitization procedure. Once the engine's internal geometry was captured, the digitized geometry was imported into TrueGrid and used to generate surface definitions and part geometries for the engine model. The engine model was developed primarily with shell elements. The objective was to develop a mesh with typical element dimensions between one and two in. However, smaller element dimensions were required at many locations to capture details of the engine geometry. Brick elements were used for some of the thicker hubs and the roots of the compressor blades. The various components of the resulting engine model are shown in Fig. C–45. A summary of the elements used in the engine model is given in Table C–4.

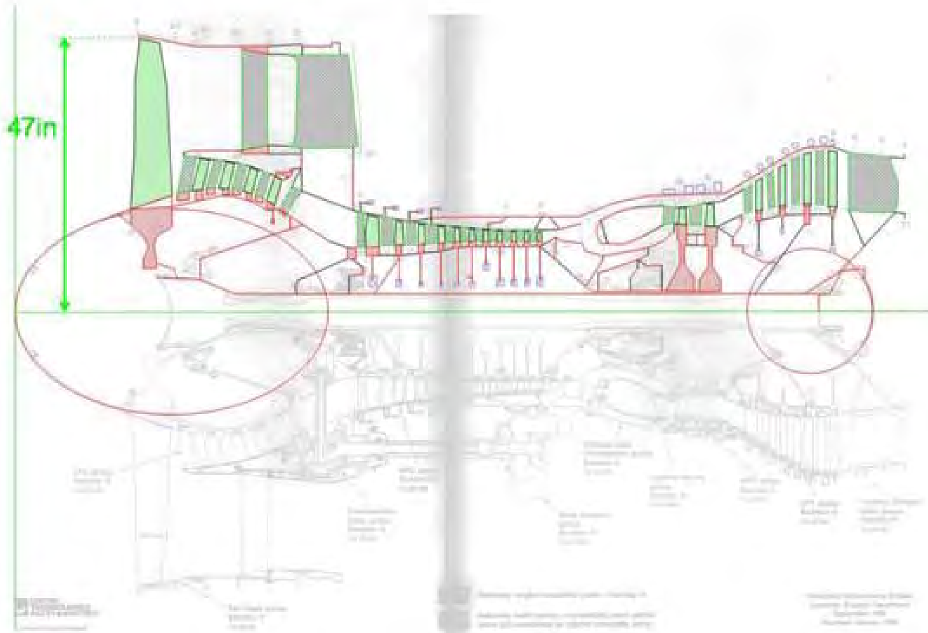


Figure C–44. PW4000 engine cross sectional geometry and simplification.

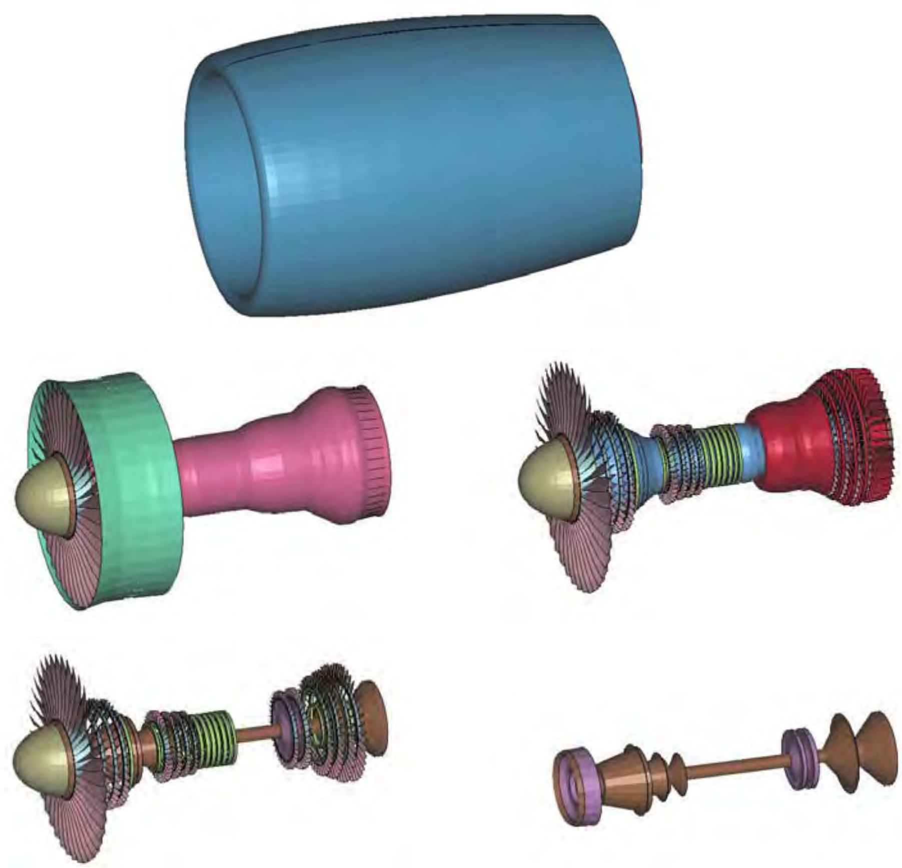


Figure C–45. Pratt and Whitney PW4000 turbofan engine model.

Table C–4. Engine model parameters.

	PW4000 Engine Model
Number of brick elements	9,560
Number of shell elements	54,788
Total nodes	101,822
Preliminary engine model weight	7,873 lb (3,571 kg)
Adjusted engine model weight	9,447 lb (4,285 kg)

After the known structural components of the engine were included in the engine model, the weight of the model was calculated at 7,873 lb (3,571 kg). The dry weight of the PW4000 engine is listed at 9,400 lb. The difference in weight potentially results from the nonstructural components (tubing, pumps, seals, bearings, etc) that were not captured in the model. To account for the difference, the density of all of the material models used for engine components was increased by 20 percent. This effectively smears the missing mass in proportion to the original mass distribution in the model. The resulting adjusted engine model mass is 9,447 lb (4,285 kg).

Information available from the Aviation Safety Network (http://aviation-safety.net/) indicates that American Airlines Flight 11 was powered by two General Electric CF6-80A2 engines, while United Airlines Flight 175 was powered by two Pratt and Whitney JT9D–7R4D engines. Review of these engines indicates that the PW4000 turbofan engine is very similar to the General Electric CF6-80A2 and the PW JT9D-7R4D engines. Comparisons of specific physical characteristics of the engines are given in Table C–5. In fact, the JT9D-7R4 and PW4000-94 are almost identical as they are in the same family of Pratt and Whitney aircraft engines. The PW4000 was labeled the "new technology JT9D" when it began replacing the latter engine on 767s built after 1987 where the PW4000-94 is 5.8 percent heavier than the JT9D-7R4 but produces up to 10 percent more thrust. Aside from an additional set of long stator blades and elongated exit nozzle, the CF6-80C2 is also of similar weight and dimensions to the PW4000. Due to these similarities, the PW4000 engine model will be used for all impact simulations.

Table C–5. Boeing 767 engine comparison.

Engine	Pratt and Whitney PW4000-94	Pratt and Whitney JT9D-7R4[a, b]	General Electric CF6-80C2[c, d]
Fan blade diameter	94 (in.)	94 (in.)	93 (in.)[e]
Length	153 (in.)	153 (in.)	161–168 (in.)[f]
Dry weight	9,400 (lb)	8,885 (lb)	9135–9860 (lb)

a. Reference value of 106 in. also found—may include cowling.
b. The "tail" of the CF6-80C2 is much longer than the PW4000. This potentially accounts for the additional 15 in. in length.
c. The CF6-80C2 has an additional set of long stator blades for the excess fan air that is not present in the PW4000.
d. The second stage compressor blades in the CF6-80C2 are closer to the central shaft than the PW4000 and do not appear to have counter weights.
e. The JT9D-7R4 and PW4000-94 are almost identical: (1) They are in the same family of Pratt & Whitney aircraft engines, and (2) the PW4000 was labeled the "new technology JT9D" when it began replacing the latter engine on 767s built after 1987.
f. The PW4000-94 is 5.8 percent heavier than the JT9D-7R4, but produces up to 10 percent more thrust.

Airframe Model Development

All significant structural components are being included in the airframe model of the Boeing 767-200ER. Figure C–46 shows the status of the initial version of the complete aircraft model. Detailed models of the empennage and landing gears, not shown in this figure, are shown in Figs. C–47 and C–48, respectively. Ribs, spars, rudder, and elevator have all been modeled in detail in the empennage. Tires and hubs, the main strut and truck, and support bracing have all been included in the landing gear model. The underside of the airframe in the model is shown in Fig. C–49 illustrating the position of the retracted main landing gear in the wheel well.

The models of the fuselage, empennage, and wing structures are developed using shell elements. The model is being developed in parameterized form where the mesh resolution is determined by a single element characteristic size parameter. This approach was selected early in the development to allow flexibility in the model size and resolution as the model development and impact analyses progressed. The objective was to develop a mesh with typical element dimensions between one and two in. for small components, such as spar or rib flanges, and element dimensions of 3 in. to 4 in. for large parts such as the wing or fuselage skin. A summary of the current model size is shown in Table C–6.

Figure C–50 shows the entire wing structure modeled to-date, including the center wing which attaches the port and starboard outboard wings. The wing stringers were not explicitly modeled to help reduce the size of the model. The stringers have a z-section geometry with typical dimensions of approximately one in. flanges and a two in. web with a thickness of approximately 1/8 in. These stringers run spanwise over the top and bottom of the wing ribs. In the model, an ‘effective’ wing skin has been used to account for the weight and strength of the riveted skin/stringer construction.

Wing Section Component Model Development

Two wing segment models have been developed to perform the component level and subassembly level analyses. The smaller segment, for component analyses, is shown in Fig. C–51. The large wing segment for subassembly analyses is shown in Fig. C–52. The wing section components are modeled with shell elements. The main spars, wing ribs, leading edge ribs, nosebeams, leading edge slats, and outboard flaps have been included in the model. Nonstructural components, such as hydraulic lines, and mechanical components, such as slat actuators, are not included in the model geometry. The density of the spars, ribs and other structural components have been increased to account for this nonstructural mass.

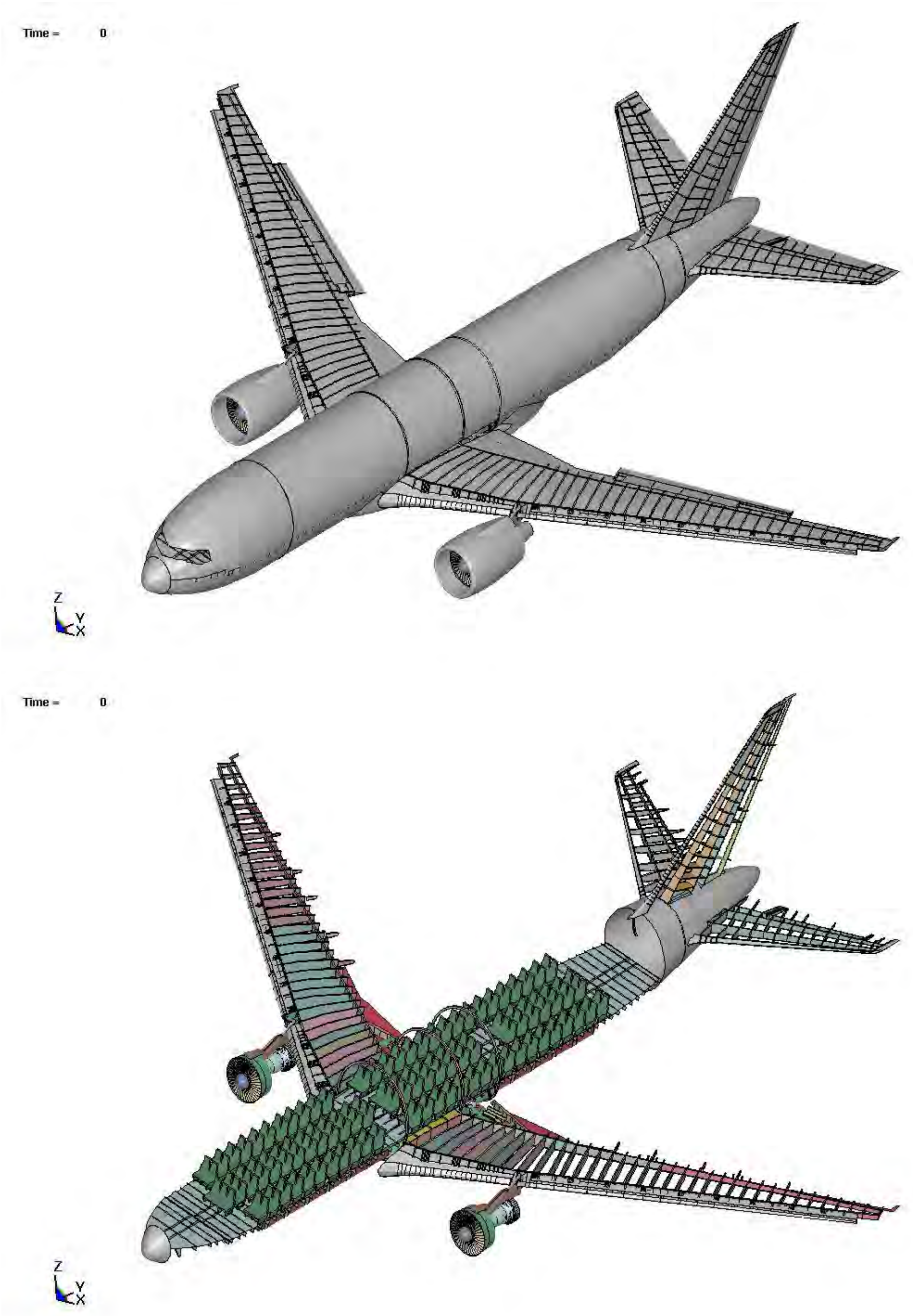
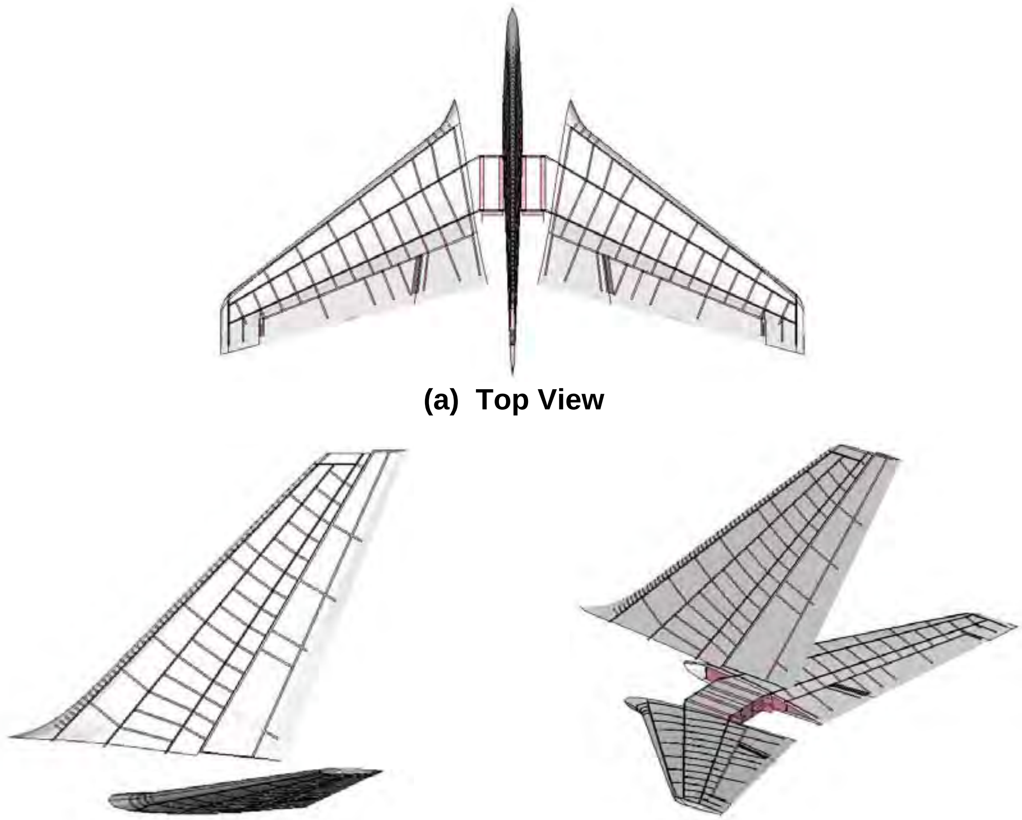


Figure C–46. Finite element model of the Boeing 767-200ER (under construction).



(a) Top View

(b) Side View **(c) Oblique View**

Figure C–47. Complete empennage for the 767 aircraft model.

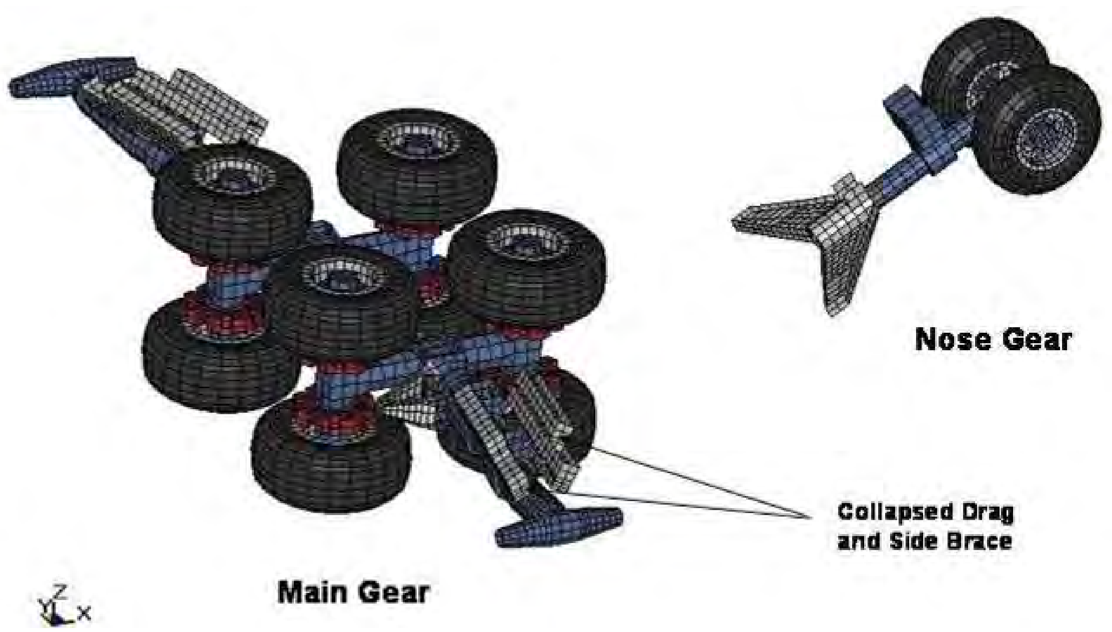


Figure C–48. Retracted landing gear components for the 767 aircraft model.

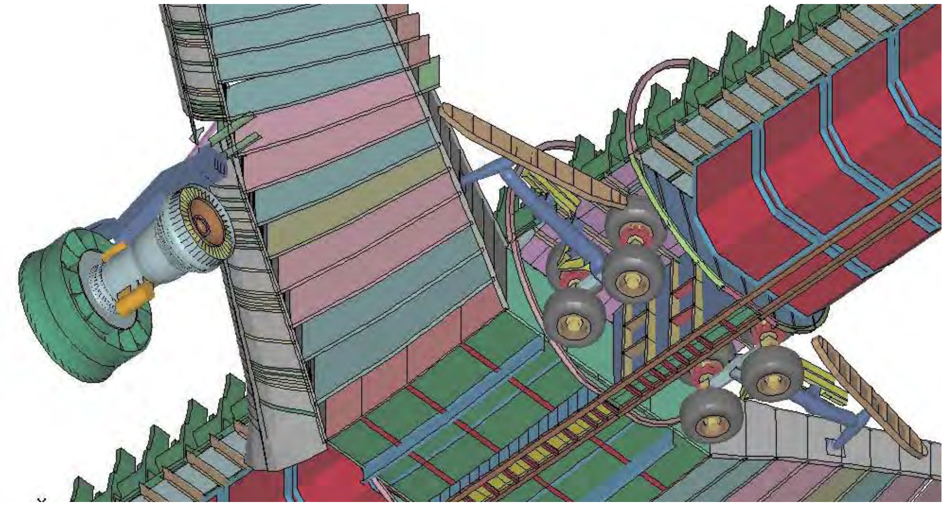
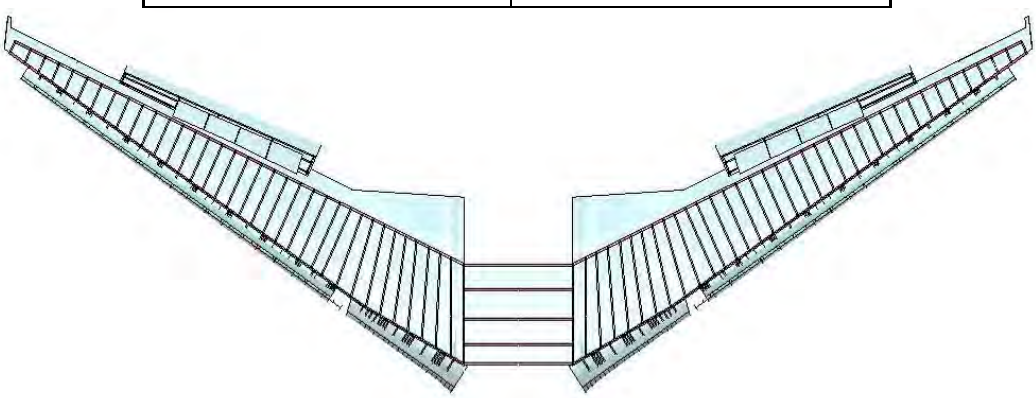


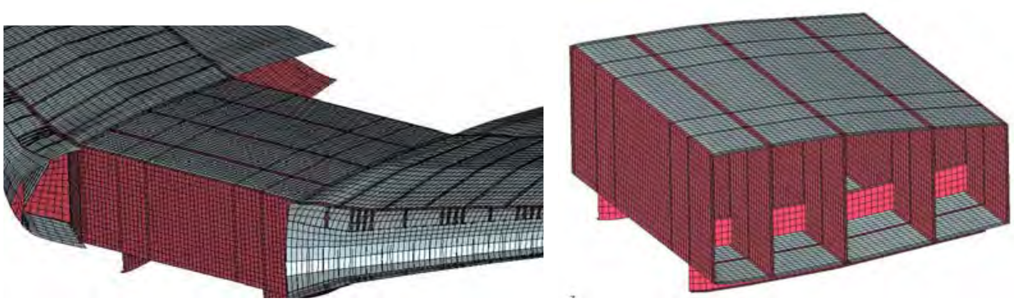
Figure C–49. Underside of the 767 airframe model (skin removed) showing the position of retracted main landing gear.

Table C–6. Summary of Boeing 767-200 aircraft model size (under construction).

	Number of Elements
Number of brick elements	70,000
Number of shell elements	562,000
Total nodes	740,000

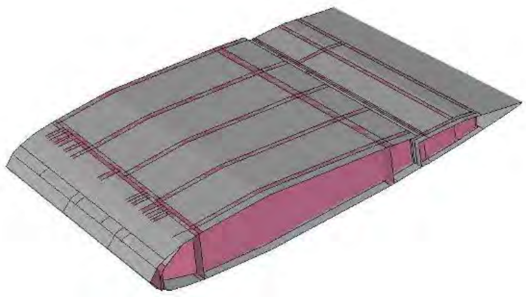


(a) Complete wing model

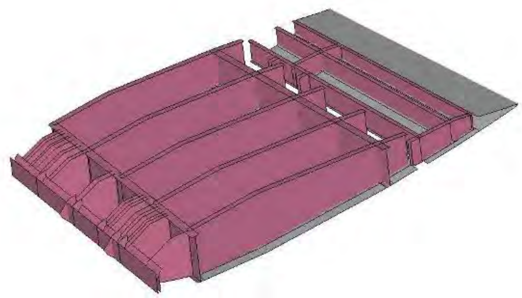


(b) Center wing structures

Figure C–50. Complete wing structures for the 767 aircraft model.

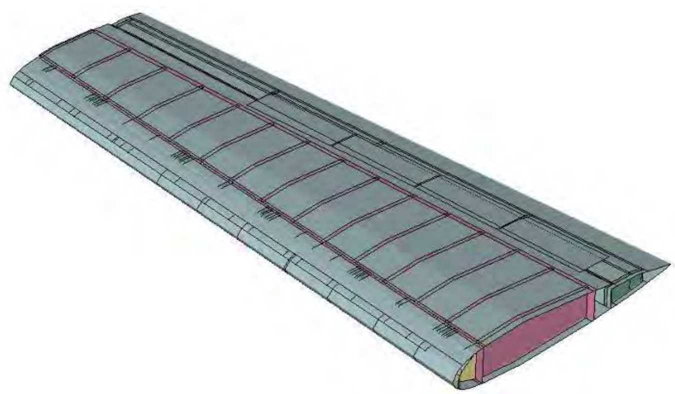


(a) Small Wing Section Model

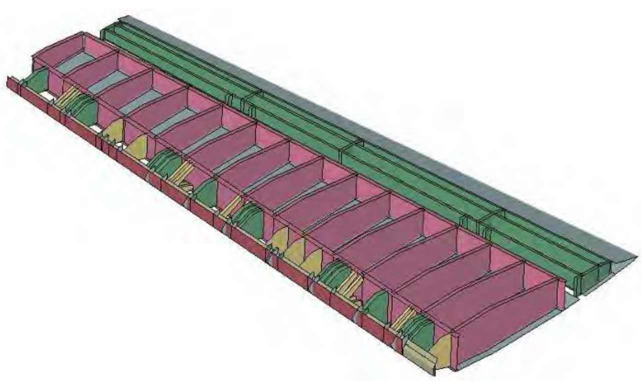


(b) Internal Structure (skin not shown)

Figure C–51. Small wing section model for component-level analyses.



(a) Large wing section model



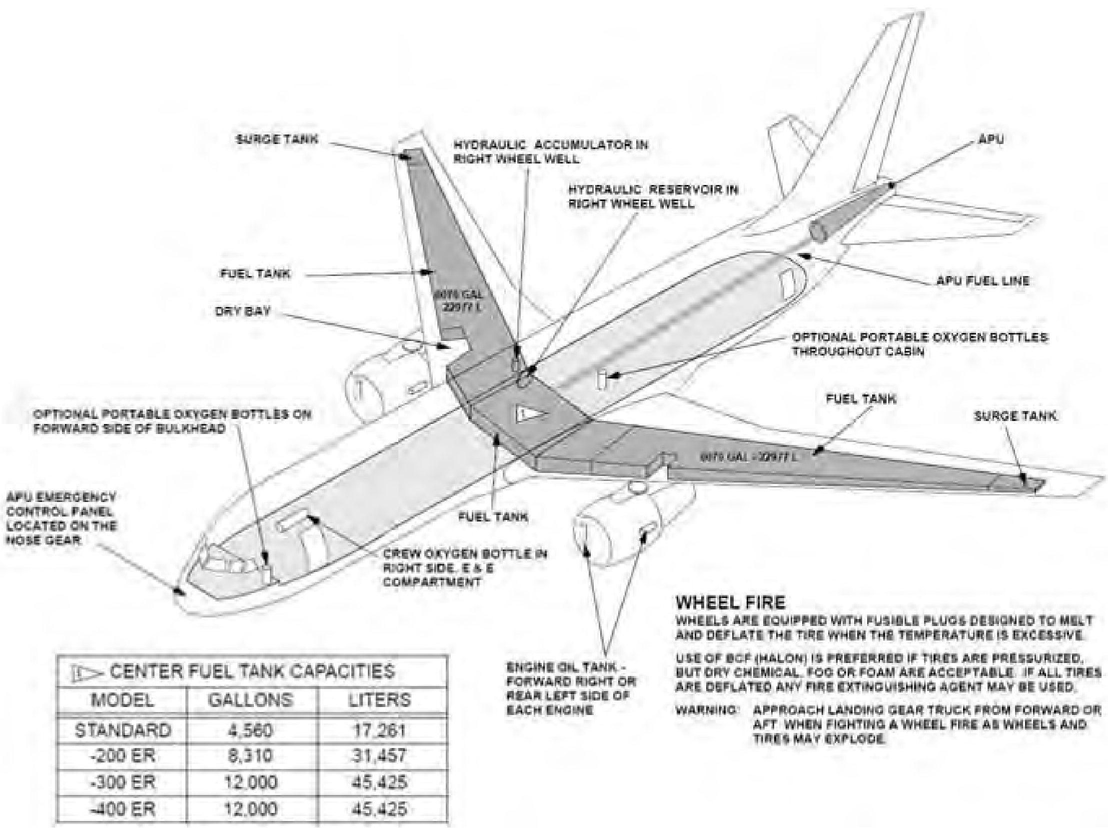
(b) Wing section internal structures (skin not shown).

Figure C–52. Wing section model developed for the subassembly impact simulations.

C.3.3 Analysis of Fuel Distribution at Impact

An important factor for determining impact damage and subsequent fire initiation is the distribution of the fuel in the aircraft at the time of impact. Both United Airlines and American Airlines have provided estimates for the quantity and distribution of fuel for UAL Flight 175 and AA Flight 11 at the time of impact.[1,2] UAL estimates that Flight 175 contained approximately 62,000 lb or 9,118 gal of fuel at impact with the "fuel evenly distributed between both main tanks."[1] American Airlines estimates that Flight 11 contained 66,081 lb or 9,717 gal of fuel at impact and "the fuel was evenly distributed between left and right wing tanks of the aircraft."[2]

Fuel tank locations and capacities for the Boeing 767 are shown in Fig. C–53. The Boeing 767 uses an integral fuel tank where the wing skin, ribs, and spars serve as the fuel tank. There are three classes of fuel tanks onboard the 767-200ER, a main tank, a surge tank, and auxiliary tanks. The auxiliary tanks consist of port, starboard, and center fuel tanks. All tanks are shown for the port wing in Fig. C–54 along with the associated internal structures. The main tank is from rib 3 to rib 31, the port and starboard auxiliary tanks are from the inboard closure rib to rib 3. The center auxiliary tank is between the port and starboard closure ribs and the surge tank is from rib 31 to rib 34. A dry bay is located above the engine at the forward part of the main tank between ribs 6 and 9. Baffle ribs are located at rib 5 and 18.



MODEL	GALLONS	LITERS
STANDARD	4,560	17,261
-200 ER	8,310	31,457
-300 ER	12,000	45,425
-400 ER	12,000	45,425

Figure C–53. Flammable material locations in a Boeing 767 aircraft (www.boeing.com).

1 Communication between United Airlines and NIST, September 5, 2003, "NIST WTC Flammable Contents Request."

2 Communication between American Airlines and NIST, August 12, 2003, "In re September 11 Litigation C&F Ref.: DTB/MH28079."

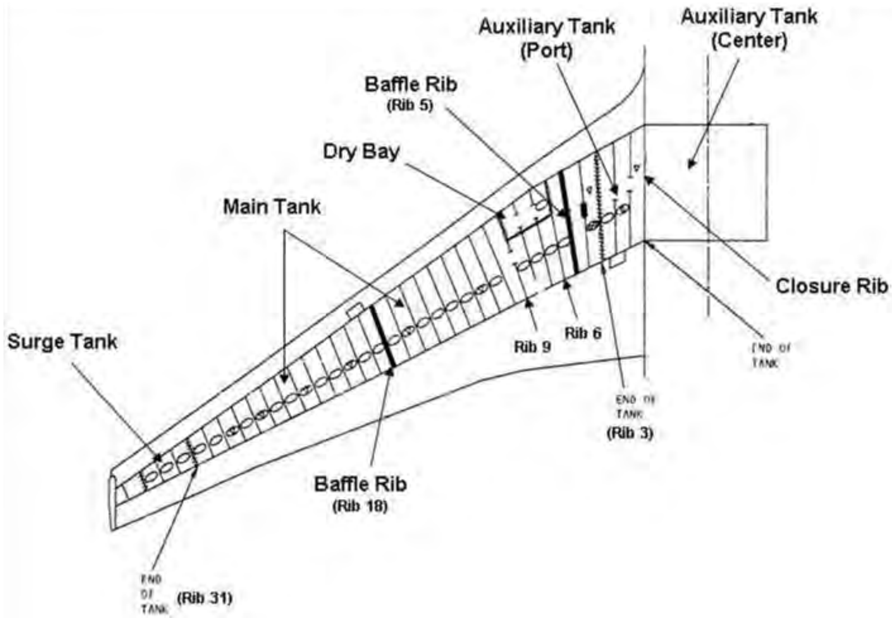


Figure C–54. Layout of fuel tanks in the Boeing 767 wing.

The functionality of the wing fuel tanks is such that a typical wing rib allows some fuel to flow along the wing but acts as a two-way fuel baffle to minimize fuel slosh. However, there are special rib designs that alter the position and flow of fuel within the tank. The ribs in the dry bay region, between ribs 6 and 9, include a fuel barrier running parallel to the rear wing spar. In addition, baffle ribs (ribs 5 and 18) include a series of fuel dams that act as a one-way valve allowing fuel flow inboard toward the sump areas (low point of tank). According to the statements from both airlines regarding fuel distribution, it is most likely that the surge tanks and all auxiliary tanks were dry at the time of impact.

Overall tank dimensions and geometry were estimated using the dimensions and approximated geometry shown in Fig. C–55. As shown in Fig. C–56, the front spar height is a good approximation for fuel depth in a full wing section. Using these approximate dimensions, the fuel tank capacity as a function of the distance along the wing buttock line (see Fig. C–57) was calculated. The maximum capacity of each main tank was calculated to be approximately 6,500 gal. The actual main tank capacity is 6,070 gal (Fig. C–53) so the calculated fuel capacity distribution was modified to match this maximum value, as shown in Fig. C–57. Notice that the main tank capacity inboard of baffle rib 18 is approximately the same volume as the fuel onboard each aircraft at the time of impact.

The exact location and distribution of fuel at the time of impact is complicated by the flight conditions prior to and at the time of impact. The terrorist pilots likely performed extreme flight maneuvers prior to impact causing most of the fuel to flow inboard. Extreme banking maneuvers with inappropriate trim could cause the fuel to flow inboard quickly. High loads on the wings due to the extreme flight regime at the time of impact would also cause fuel to flow inboard by increasing the dihedral angle of the wing. Since the baffle ribs restrict fuel from flowing outboard, it is reasonable to assume that all fuel that could flow inboard was actually inboard at the time of impact.

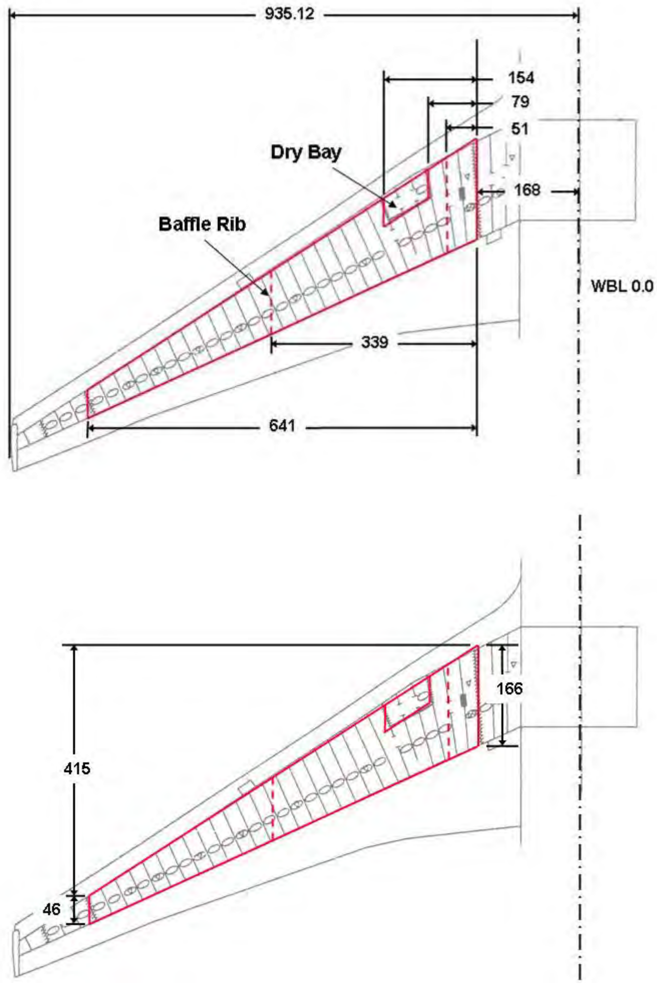


Figure C–55. Approximate fuel tank dimensions (in.).

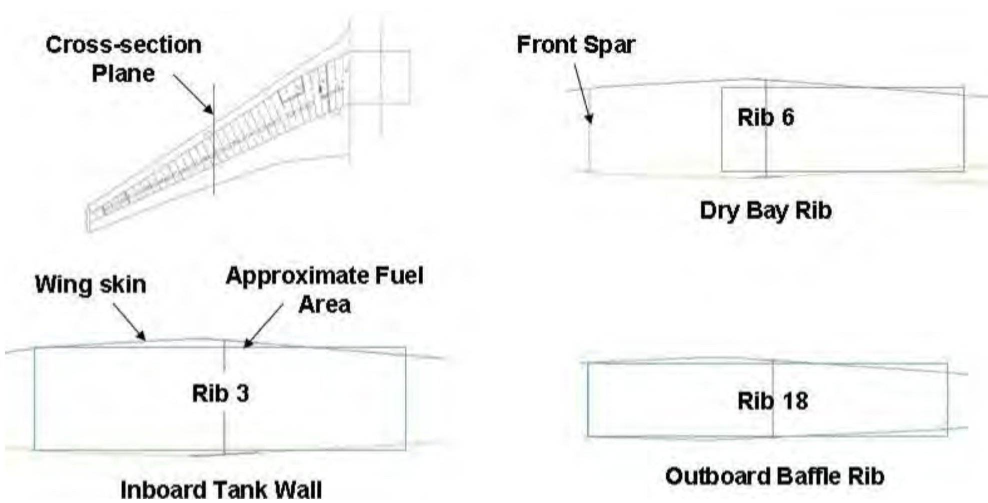


Figure C–56. Wing cross sections at various rib locations.

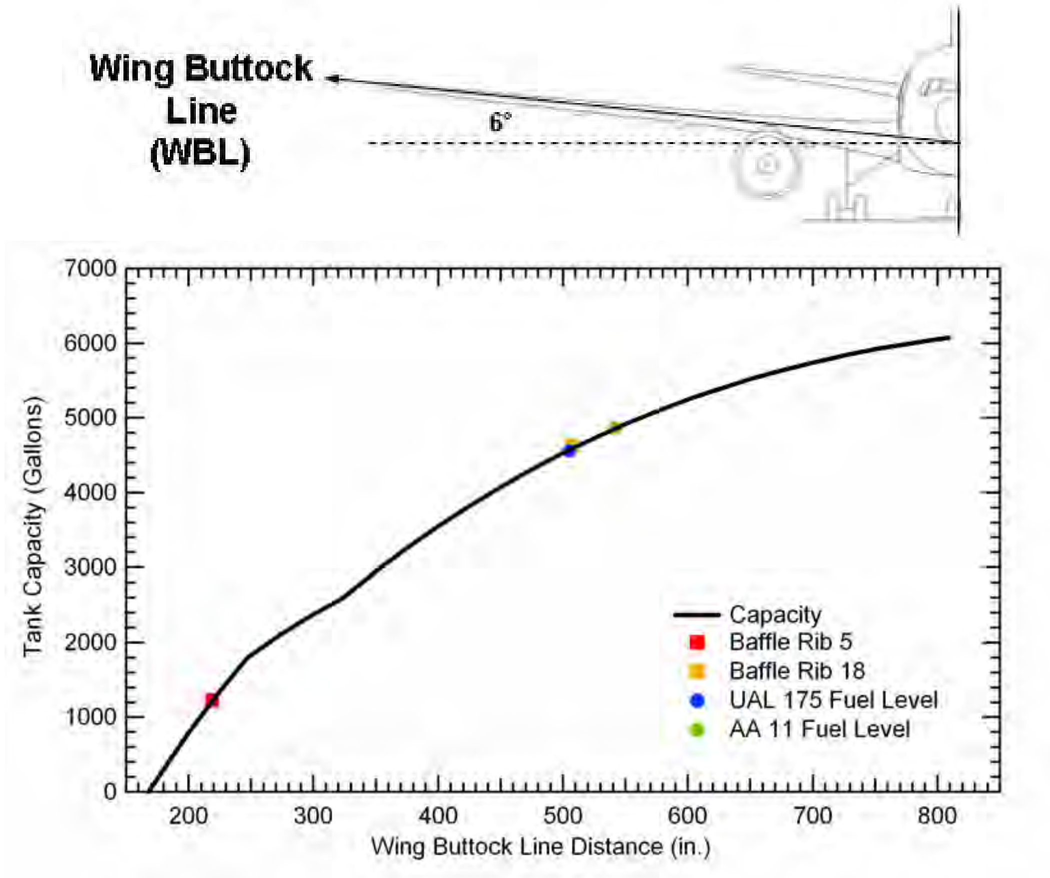


Figure C–57. Fuel tank capacity.

For simplicity, it was assumed that all fuel has moved inboard at the time of impact. Since the fuel tank capacity at the outboard baffle rib and the fuel onboard are approximately the same, a good first estimate is that the main tanks are full inboard of baffle rib 18 at the time of impact. A small amount of fuel is outboard of this rib for AA flight 11. This is shown graphically in Fig. C–58 for smooth and level flight with an undeformed wing shape. A full wing out to baffle rib 18 and dry outboard of this rib is taken as the nominal case in subsequent analyses.

C.4 COMPONENT-LEVEL ANALYSES

The primary objectives of component modeling are to (1) develop understanding of the interactive failure phenomenon of the aircraft and tower components and (2) develop the simulation techniques required for the global analysis of the aircraft impacts into the WTC towers. The approach taken for component modeling is to start with finely meshed, brick element models of key components of the tower structure and progress to relatively coarsely meshed beam and shell element representations that will be used for the subassembly and global models. This is done to develop reduced finite element models appropriate for high fidelity global impact analyses, as modeling each component with fine details would be too demanding from a computational standpoint and an inefficient use of resources.

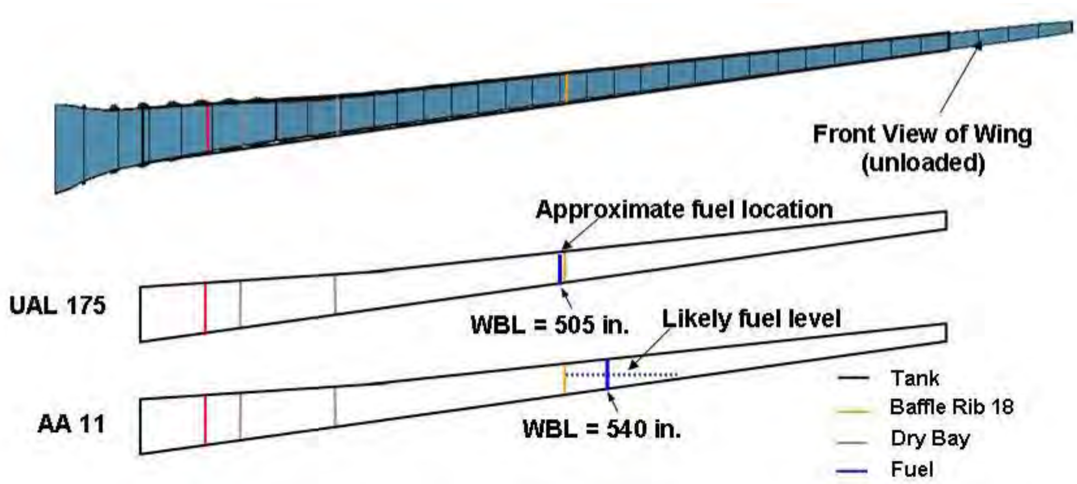


Figure C–58. Approximate fuel locations for smooth and level flight.

In addition to determining the optimal element size and type for global modeling, other key technical areas are addressed in the component modeling phase of the program. These issues include material constitutive modeling, treatment of connections, and modeling of aircraft fuel. The following component modeling scenarios were outlined at the start of the project:

- An exterior column impacted by an aircraft engine
- An interior column impacted by an aircraft engine
- An exterior column impacted by an aircraft wing segment with and without fuel

In addition to the above component impact analysis scenarios, a range of additional component analyses were identified that were considered important and helpful in developing the global impact models and analysis methods. These additional component analysis scenarios included:

- Bolted column end-plate connections with approximated dynamic loading
- Bolted spandrel connections with approximated dynamic loading
- Floor system with concrete slab impacted by an engine

The approach has deviated somewhat compared to the tasks above to maximize efficiency and to produce the most meaningful structural loading scenarios. Once preliminary calculations were performed it was found, for example, that the load generated by an impacting engine would totally overwhelm a single interior or exterior column. Simplified models based on this severe loading would match the detailed brick model but the subtle response from lesser loading might not be as accurate. To capture the more subtle response in the column components, wing sections with and without fuel were studied as impactors instead of engines. It was found that the empty wing section impact produced damage to the exterior columns that is near the failure threshold. Similarly, a fuel-filled wing section impacting both wide flange and box type core column resulted in damage near the failure threshold. As a result, these impact scenarios were used primarily for the exterior and core column component impact analyses.

Included in this appendix are the impact analyses for exterior columns, core columns, bolted column connections, bolted spandrel connections, and floor systems.

C.4.1 Analysis Methodology

The impact analyses were performed using the LS-DYNA finite element code (LS-DYNA 2003). LS-DYNA is a commercially available nonlinear explicit finite element code for the dynamic analysis of structures (LSTC 2003). The initial foundation of LS-DYNA was the public domain DYNA3D finite element code developed at the Lawrence Livermore National Laboratory. Since 1987, the code has been extensively developed and supported by the Livermore Software Technology Corporation (LSTC) and is used for a wide variety of crash, blast, and impact applications.

LS-DYNA has several unique capabilities for this project such as Arbitrary-Lagrangian-Eulerian (ALE) and Smooth Particle Hydrodynamics (SPH) algorithms that can be applied to the analyses of fluid-structure interaction and large-scale fracture and fragmentation of structures. These capabilities are critical for the analyses of the fuel tank breakup and dispersion of fuel inside the towers upon impact. The fuel and debris dispersion is crucial for assessing the impact loads inside the tower structures and the corresponding damage to the mechanical systems.

The impact analyses described in this report use a variety of capabilities and algorithms in LS-DYNA. A brief description of these capabilities is described in this section. A significantly greater detailed description of the analysis methods is provided in the LS-DYNA Theoretical Manual (1998).

The fine mesh detailed component analyses typically use 8-node solid hexahedron (brick) elements with single point integration. This is the most commonly used solid element type in LS-DYNA due to its computational efficiency. The biggest disadvantage of the single point integration is the potential for hourglassing or zero energy modes. There are several methodologies for controlling hourglass modes in LS-DYNA. The typical approach used in these impact analyses is to apply a viscous hourglass control where a viscous damping is introduced that suppresses the formation of hourglass modes but does not significantly influence the global modes.

The component impact analyses using solid elements typically have a fine mesh. As a result, damage and failure are included strictly through the constitutive algorithms. Damage criteria (such as maximum plastic strain) are tracked for each element within the constitutive model evaluation, and elements are eroded when the failure criteria are exceeded. This allows for a direct evaluation of damage and failure within the impact simulations.

The eroded elements allow for the initiation and extension of fracture in the model. Eroded elements no longer support any stress, and the strains in the eroded elements are no longer calculated. The associated mass of the elements remains with the nodes in the calculation. If adjacent elements have not reached the failure surface, the nodes remain attached to the structure. If all of the elements connected to a specific node have failed, the node becomes a free particle. Free nodes can either be eliminated from the calculation or remain in the calculation with associated inertial properties and potential for impacts against other structural components (free nodes remain in contact algorithms).

As the mesh refinement and model size are reduced, the components are typically modeled using Belytschko-Lin-Tsay shell elements. This is a four node shell element with single point integration. The Belytschko-Lin-Tsay element is a computationally efficient alternative to the Hughes-Liu element in LS-DYNA and again is the most widely used shell element formulation within LS-DYNA for crash, impact, and metal forming applications. Results generated with the Belytschko-Lin-Tsay element typically agree with those using the Hughes-Liu element. As used in the solid elements, the most common approach to introducing damage and failure for the shell elements is through the constitutive models and element erosion.

In specific applications, unique algorithms are required to introduce failure modes in the analysis. An example is the interface between the skin and internal frame structures of the aircraft. Rivets are used for the primary connection between the airframe and skin. The approach used to model this connection and failure during the impact event is the tied interface with failure. In this approach, interface segments (shell elements) are constrained to move together until a failure criterion is exceeded. The failure criterion is a quadratic combination of the normal and shear failure stresses at the interface. After failure, the segments are allowed to move independently but not allowed to penetrate each other (typical contact algorithm behavior).

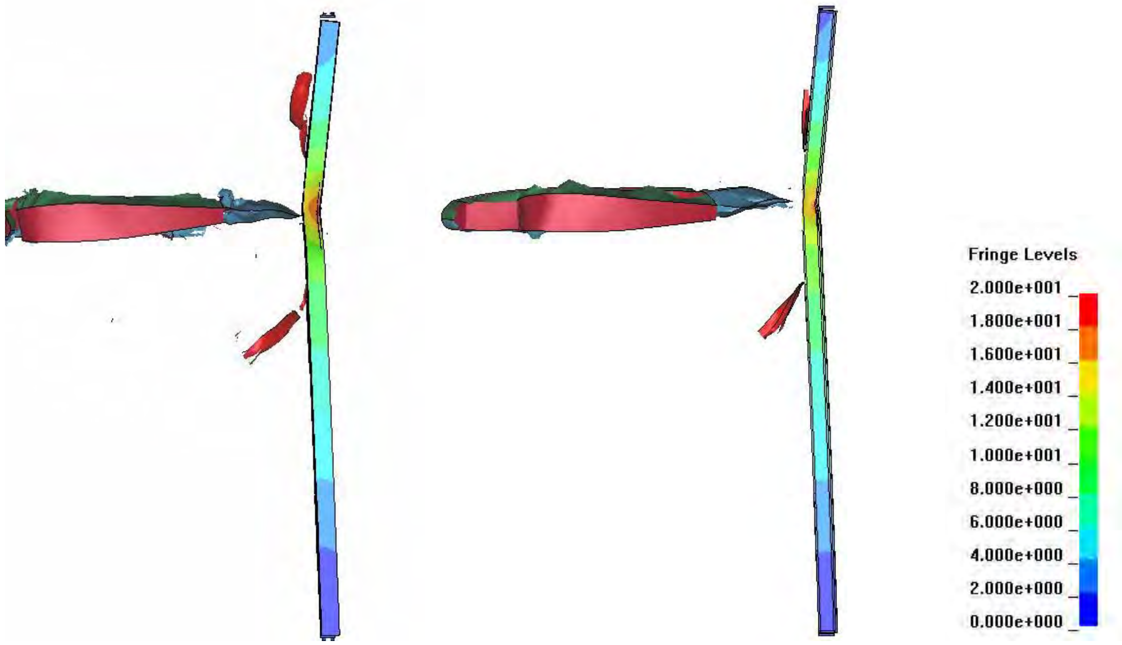
Overall contact in the impact analyses is modeled using the automatic single surface contact algorithm in LS-DYNA. Interacting components are defined by a material list, and contact segments are automatically generated by LS-DYNA. This greatly simplifies the specification of contact between various components in the aircraft and tower structures. The type 1 soft constraint option is used in the contact algorithm that determines the contact stiffness based on stability considerations, time step size, and nodal mass. This soft constraint option was found to be more robust than the default penalty formulation for modeling the complex contact behaviors in large impact and crash simulations.

C.4.2 Exterior Column Impact Analyses

Various exterior column component impact analyses were performed with different objectives. The preliminary exterior column component impact analyses were performed on a single column using a highly refined mesh of brick elements. These analyses were used to investigate the details of the column response and develop analysis techniques that can be applied to the subassembly and global impact analyses. These preliminary exterior column impact analyses used a simplified wing section impactor that was developed prior to gathering detailed structural information of the Boeing 767 wing design.

Subsequent exterior column impact analyses were performed using less refined models of the exterior columns in full panel configurations impacted by detailed wing section component models with and without fuel. The primary objective of these component analyses was to study the impact response of the aircraft wing structures and investigate various modeling techniques for including aircraft fuel in the analyses.

Figure C–59 shows the calculated response of a single exterior column component, impacted by an empty wing section. The figure compares the column damage calculated with two different models of very different resolutions. The column model on the left is a fine mesh made of brick elements and the column on the right is a coarser mesh of shell elements. The failure strains of the coarse model were estimated by calibrating the response of the coarse model against the refined model (see Section C.2.2). Contours of resultant displacements are shown on the column components. It can be seen that the overall response, both in column and wing segment damage, is very similar. These analyses were used to develop the coarse column models used in subsequent analyses.



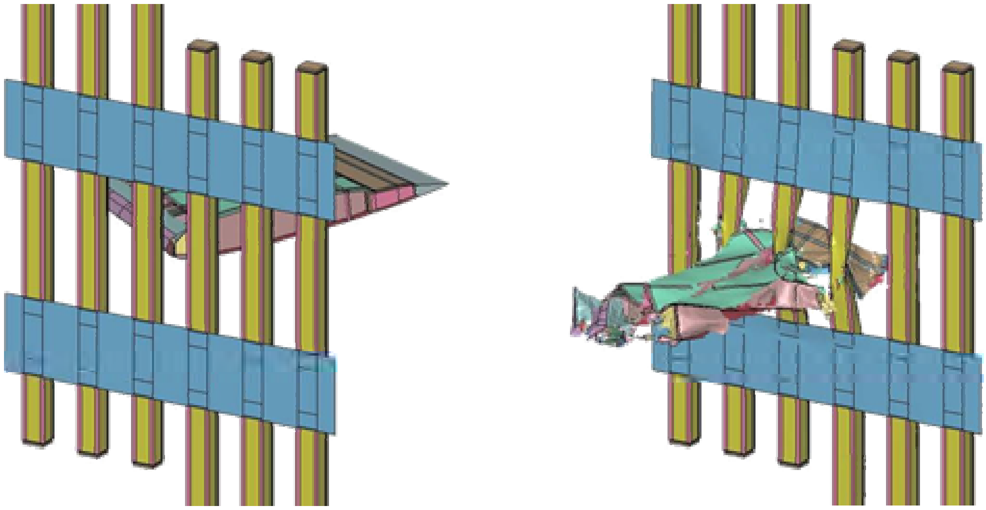
(a) Fine brick element column **(b) Coarse shell element column**

Figure C–59. Exterior column response comparison, showing contours of the displacement magnitude (in.).

Empty Wing Section Impact Analysis

Figure C–60 shows the impact of the small empty wing section into two exterior wall panels at a speed of 500 mph (223 m/s). These analyses use a wing section model with significantly improved structural fidelity over the preliminary wing segment model used in the column component impact analyses described above. The development of the wing section component model used here was described previously in Section C.3.2. The wing section model shown in Fig. C–60 consists of approximately 24,000 shell elements.

The model for the two exterior panels is made primarily of shell elements with a medium mesh resolution. The exceptions are the butt plates, which were made of solid brick elements and the bolted butt plate connections using beam element bolts, as described later. The boundary conditions at the ends of the exterior columns are a bolted connection to an adjacent butt plate with constrained displacements at the edges. In addition, the displacements in the direction of the impact were constrained on the spandrel plates at the location of the floor slab. The resulting model for the exterior panel had 54,096 shell elements (columns and spandrels), 2,112 solid brick elements (24 butt plates), and 48 beam elements for the bolts.



(a) Impact Configuration (b) Calculated Response

Figure C–60. Calculated impact damage with the empty short wing section component model.

The calculated impact response produces large scale damage and fragmentation of the empty wing section and significant damage to the exterior columns. The calculated impact damage to the exterior column panel is shown in Fig. C–61. The damage includes significant distortion of the columns, large plastic strains, and fracture of the plate connections within the columns. However, the columns are not completely severed and still maintain some load carrying capacity.

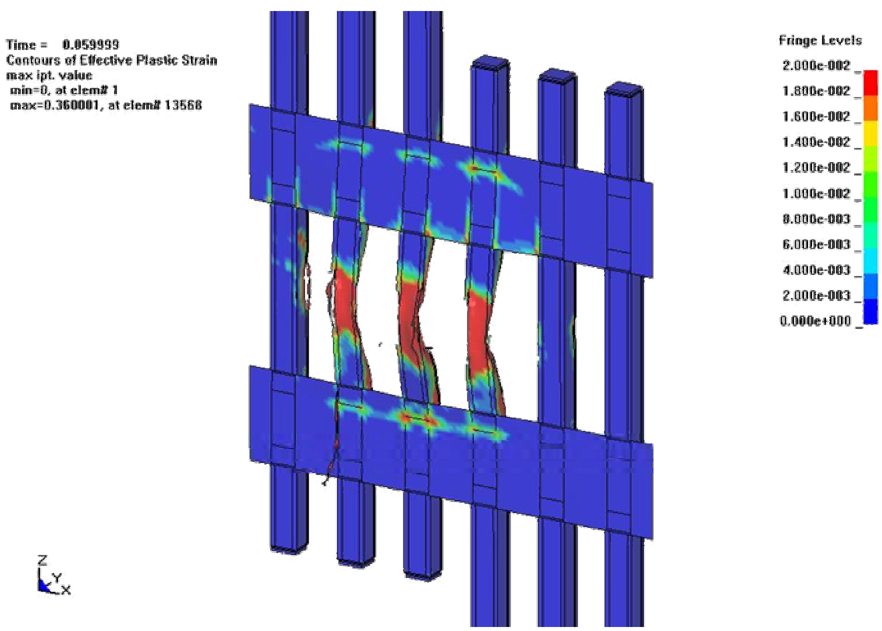


Figure C–61. Damage produced by the empty wing section impact.

Wing Section with Fuel Impact Analysis

A significant portion of the weight of a Boeing 767 wing is from the fuel in its integral fuel tanks. At the time of impact, it is estimated that each aircraft had approximately 10,000 gal of fuel on board. Upon impact, this fuel is responsible for large distributed loads on the exterior columns of the WTC towers and subsequently on interior structures, as it flows into the building. It therefore could have a significant effect on the damage done to the building structure. Accurate modeling of the fluid-structure interaction is needed to accurately predict the extent of this damage and the fuel dispersion within the building to help establish the initial conditions for the fire dynamics modeling.

A number of approaches to solving Fluid-Structure Interaction (FSI) problems are available in LS-DYNA. One approach is the standard Lagrangian finite element analysis with erosion, where the fuel is modeled using a solid mechanics approach. This approach accounts for the inertial effects of the fuel, but does not simulate the fuel flow during impact. The ALE method has been developed as one good approach to solve fluid and solid material interaction. With this methodology, fluids are modeled with a fixed Eulerian mesh, which allows for materials to flow between mesh elements. Solid materials are modeled with a moving Lagrangian mesh. With ALE, both mesh types can interact. An alternative approach is to use mesh-free methods such as SPH. SPH modeling for fuel effects has the advantage of a smaller mesh size and potentially much faster run times than ALE analyses. Both methods will be applied to the analysis of fuel impact and dispersion.

The small wing segment was used for performing component level analyses of the wing with fuel. The small wing segment is from rib 14 to rib 18, the outboard baffle rib. For this location, the segment was considered to be completely full of fuel. Figure C–62 shows the fuel-filled wing section model with an SPH mesh for the fuel, shown in blue. The wing section model is identical to that used in Fig. C–60, but with the addition of 4,400 SPH fuel particles.

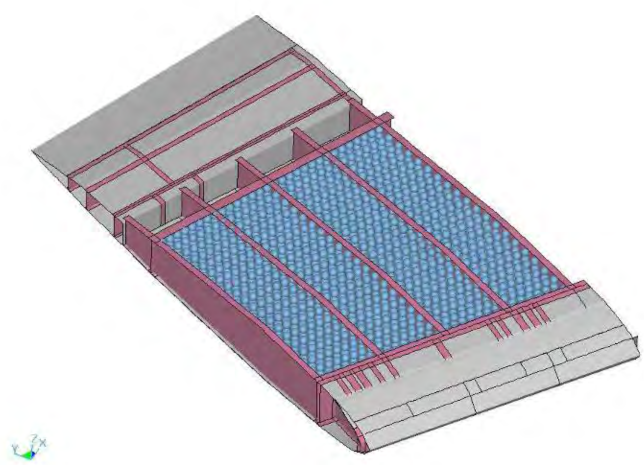
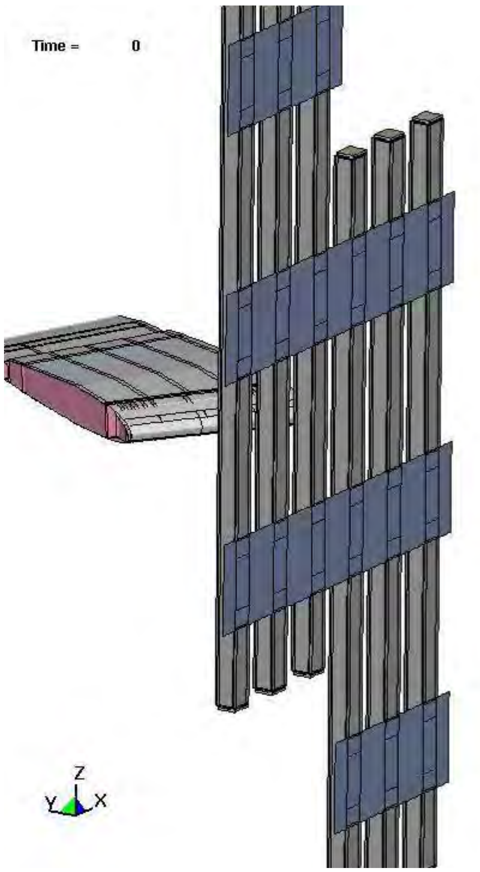
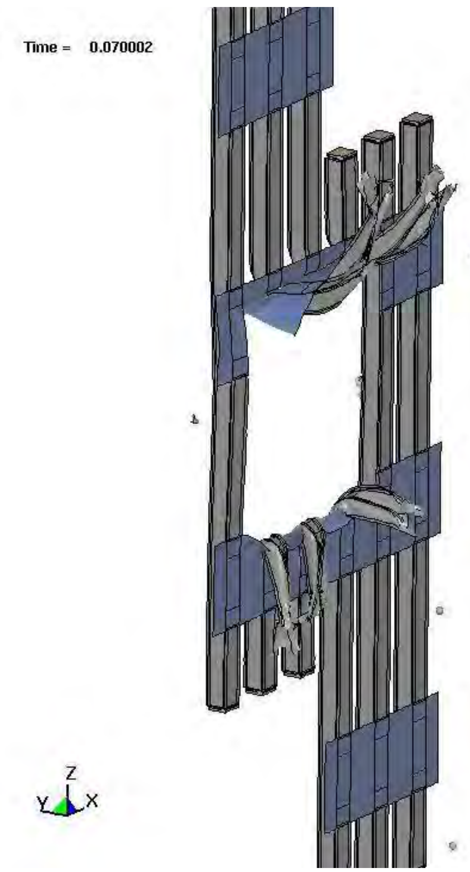


Figure C–62. SPH fuel in the small wing segment.

For the wing section with fuel component analyses, the impacted structures are the same two exterior panels as used in the empty wing section analysis above. As shown in Fig. C–63, the columns of the exterior panels are completely destroyed due to impact. Both the SPH and ALE methods simulate this extensive damage. This was not the case for the empty wing section model, as discussed in the previous section.



(a) Prior to impact



(b) After impact

Figure C–63. Exterior panels impact behavior for a wing segment with fuel (SPH fuel model).

Figures C–64 through C–67 show the fuel dispersion and wing break up predicted by the two fuel modeling methods (ALE and SPH). As most clearly shown in the side view, the current SPH modeling method predicts greater fuel dispersion and wing break up than from using ALE. Current efforts are underway to study the effect of mesh density on these results. Both analysis methods calculate an impact load sufficiently large to easily rupture the exterior columns.

Run-times from these component analyses clearly indicate that the SPH method will be more practical for the global impact analyses. The current SPH model runs significantly faster than the ALE method as it requires a smaller mesh and does not need to rezone after each time step, as is done in the ALE method. In addition, the ALE method requires a mesh for both the fuel region and the void zone into which the fuel can flow. In the example shown, this required the addition of approximately 110,000 solid elements for the ALE analysis. These preliminary calculations indicate that the ALE analysis run-times are as much as 10 times longer than those for the SPH analyses. However, additional work is underway to determine the mesh refinement in each approach required to calculate the fuel impact load effects and dispersion with sufficient fidelity.

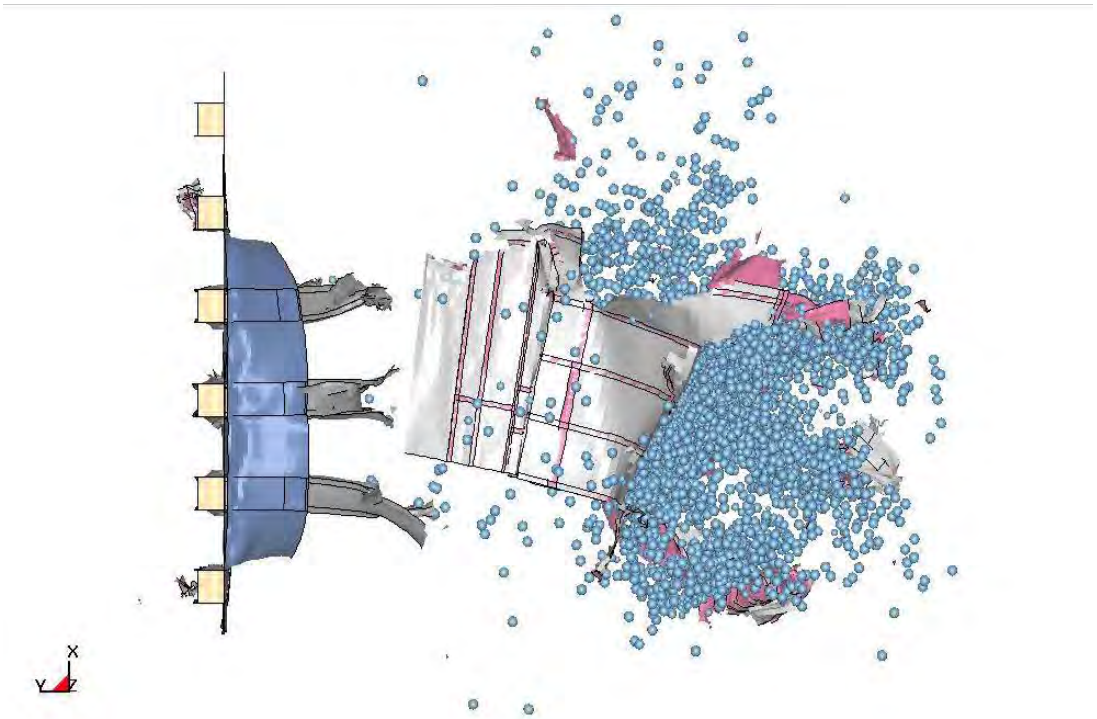


Figure C–64. SPH Analysis of structural damage and fuel dispersion (top view).

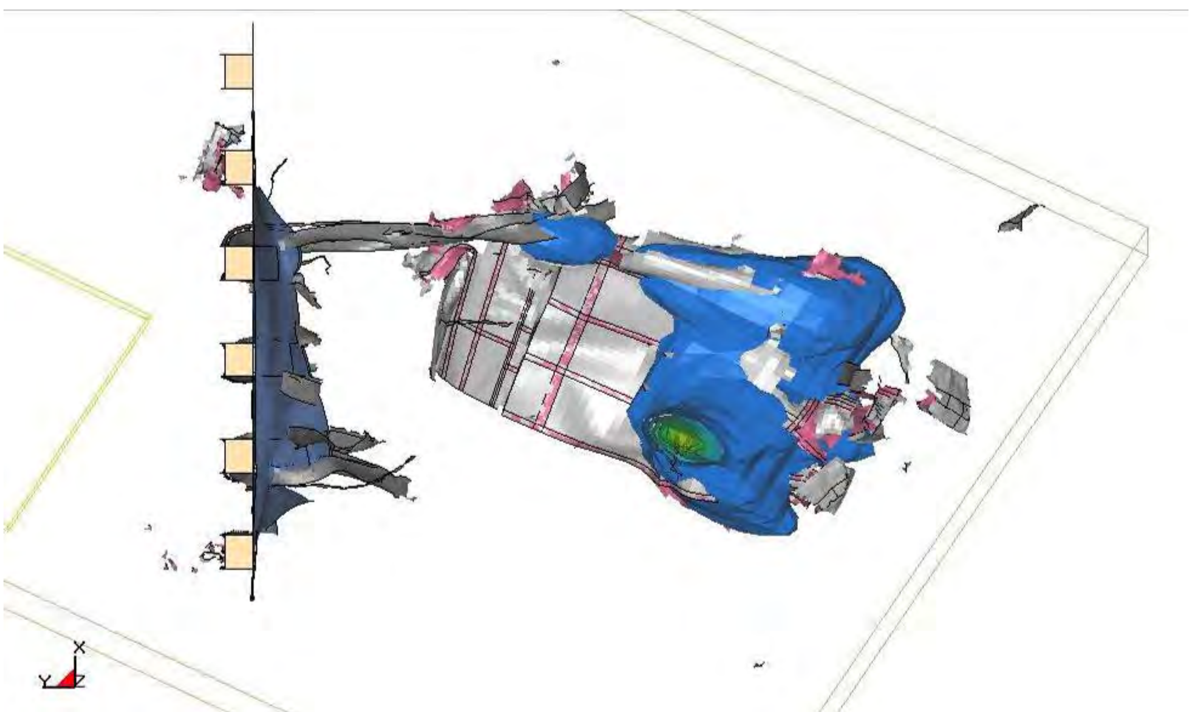


Figure C–65. ALE analysis of structural damage and fuel dispersion (top view).

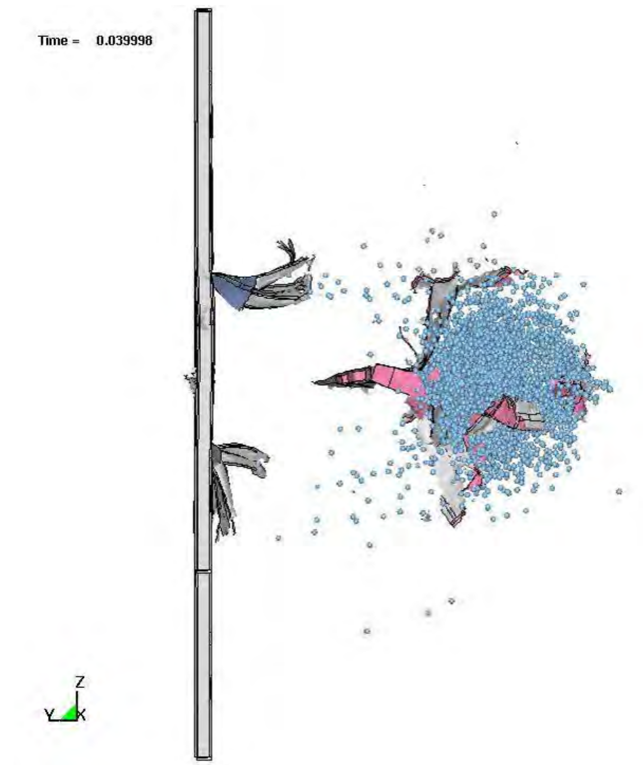


Figure C–66. SPH analysis of structural damage and fuel dispersion (side view).

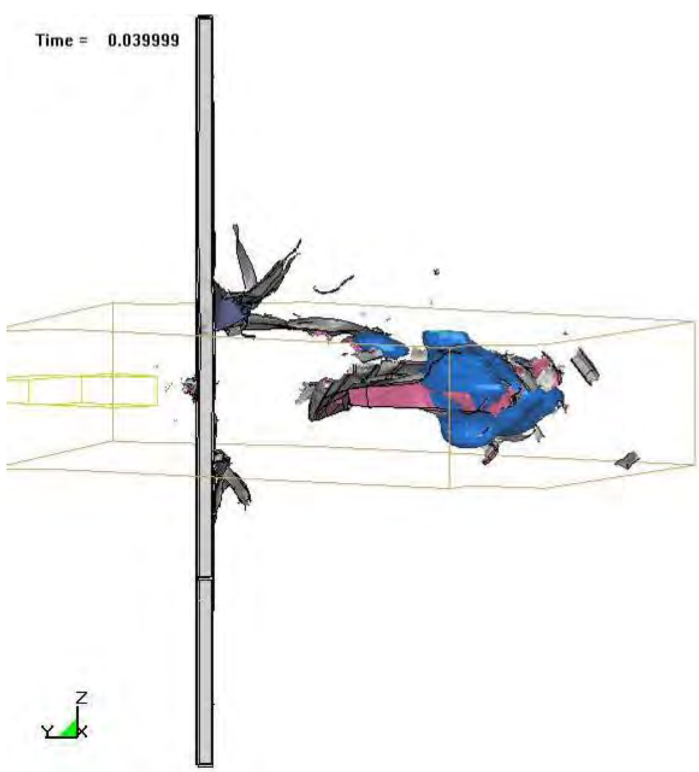


Figure C–67. ALE analysis of structural damage and fuel dispersion (side view).

Bolted Connection Modeling

There are a wide variety of different connections required for the assembly of the towers and aircraft. Wherever possible, evidence gathered from WTC steel will be applied to determine the importance of including connection details in the model and failure modes of those connections. For example, photographic and structural debris evidence clearly demonstrated that the external column connections played a significant role in the mode of column failure and extent of the external damage.

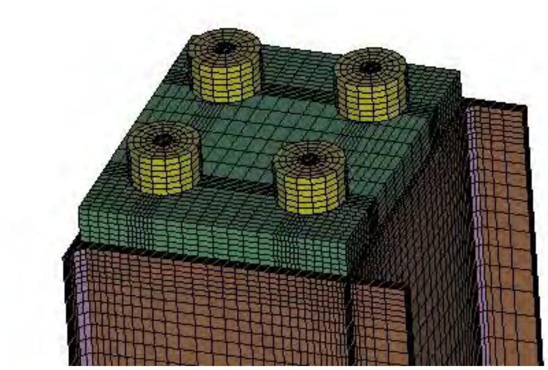
Two connections are of particular interest to the impact analysis of the aircraft into the WTC towers and are modeled in this section. These include the bolted connections at exterior column butt plates and the bolted spandrel connections. The objective of connection component analyses is to develop connection models for the global impact analyses that accurately capture the capacity and failure modes of the connection. Various connection component models developed include both fine models of these complex connection components (e.g., 3-D brick element models of bolts), to simple models such as beam element bolt models and tied constraints with failure. The constitutive behavior of the beam element bolt was described previously in Section C.2.1. A tied constraint with failure requires that two nodes or a node and a surface segment (shell element or solid element face) have tied degrees of freedom until a failure criterion is exceeded.

The coarse component models utilize many of the simplified connection and element types available in LS-DYNA to approximate the behavior observed in the fine models. For example, in the exterior column bolted connections, the bolts were modeled with elastic-plastic beam elements calibrated to match the fine model. Bolted joints, such as in the spandrel connections, were approximated with tied node algorithm that constrains degrees of freedom of adjacent nodes and element faces. Various options are investigated in these component analyses and final selections of the modeling methodology are based on both the fidelity and efficiency of the modeling approaches.

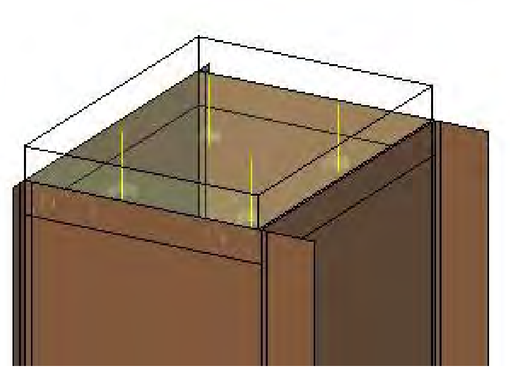
Component modeling of the exterior column butt plate connections has been completed. As shown in Fig. C–68 the detailed model includes individual bolts modeled with solid brick elements. The simplified model uses coarse brick butt plates joined by beam element representations of the bolts. A dynamic analysis was carried out to calibrate the beam element bolt model. The loading condition was a dynamic separation of the two butt plates. The velocity profile used to separate the butt plates was obtained from a preliminary engine impact analysis against the exterior wall similar to those described in the next subsection. The profile is a linearly increasing separation velocity between the butt plates with an initial velocity of zero and a velocity of 43 ft/s (13 m/s) at a time of 5.0 ms.

Failure strain in the beam models was calibrated such that the beam bolts failed at the same time as the brick element bolts. Failure of the bolts occurs at a time of approximately 3.0 ms. These connection models were used in the corresponding brick and shell models of the exterior column component impact analyses shown previously in Fig. C–59. Connection failure at the column ends was quite similar in both cases.

The spandrel connections consist of an overlapping splice plate across a spandrel joint with a row of bolts on either side of the joint. Typical failure of these spandrel joints in the impact zone resulted from bolt bearing shear failures, typically in the spandrel plate, as shown in the photographs in Fig. C–69. A common configuration in the impact zone would have six bolts on either side of the joint. The bolt bearing shear failure mechanism would be difficult to model using a beam element bolt as applied previously to the exterior column bolted connections. In the exterior columns, the bolts fail primarily in tension and details of the contact between bolts and butt plates are not as important for capturing the failure behavior. As a result, an alternate modeling approach was required for the spandrel splice failures.



(a) Brick element bolts

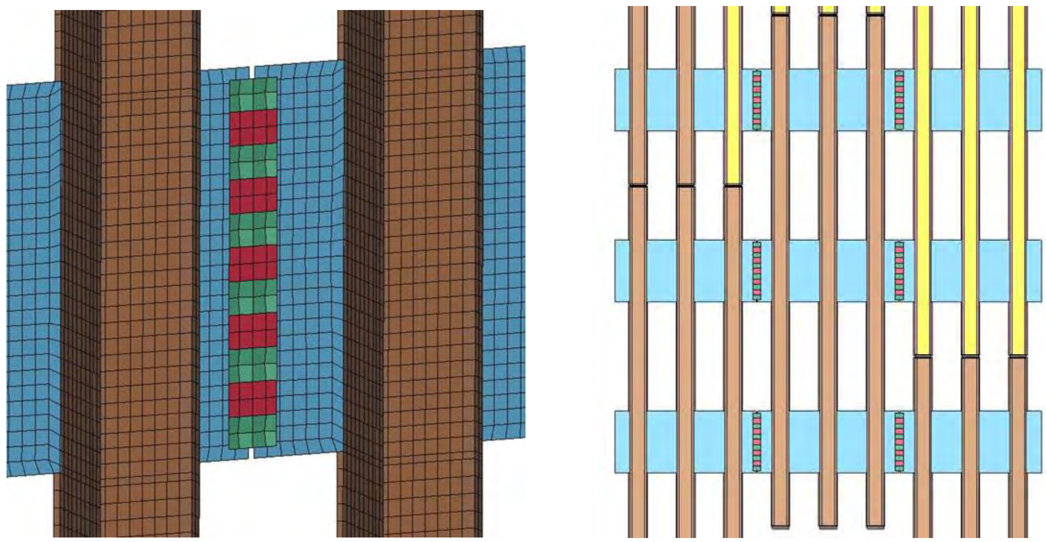


(b) Beam element bolts (butt plates shown as transparent)

Figure C–68. Exterior column end connection modeling.

Figure C–69. Typical bolt bearing shear failures of spandrel connections.

Spandrel connections are modeled using a splice plate made up of shell elements, as shown in Fig. C–70. Connections corresponding to individual bolts are treated by tying single nodes on the splice plates (center of green squares in Fig. C–70) to the spandrels. Two material definitions are used to make up the splice plate to allow for contact between the splice plates and the spandrels as well as having tied contact. The first splice plated material definition (shown in green in Fig. C–70) is used to allow the center node tied constraint (representing the bolt connection) to be aligned with the center of the spandrel plate. Since this material includes the tied constraint, it is not included in the automatic contact definition. The second material definition in the splice plate (shown in red in Fig. C–70) has a standoff distance equal to one half of the combined thickness of the spandrel and splice plate and is included in the automatic contact definition. Both material definitions have the same constitutive properties; however, using a single material definition would sometimes result in a numerical instability due to conflicting constraint and interface algorithms on those segments. The application of the spandrel splice plate connection model is demonstrated in the engine impact component analyses below.



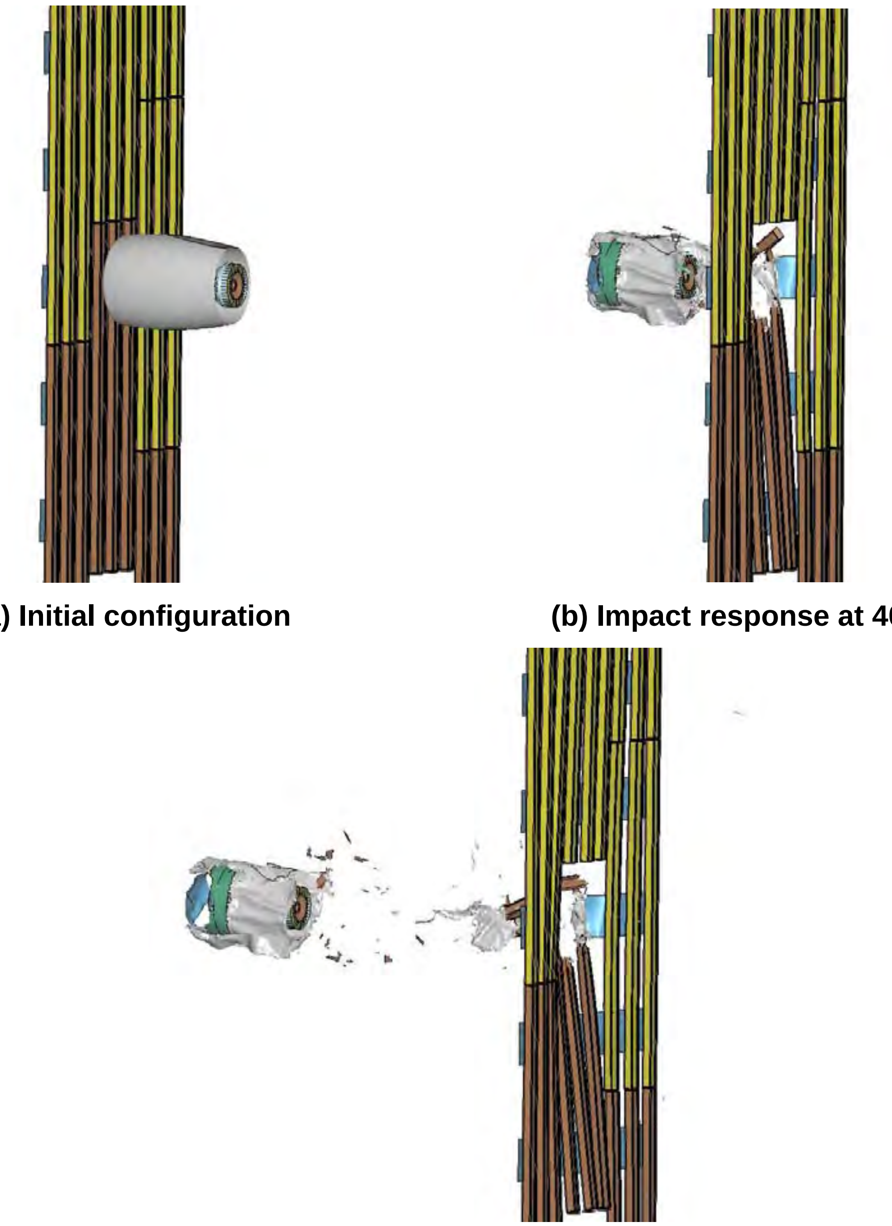
(a) Detail of spandrel connection **(b) Multiple panel connections**

Figure C–70. Bolted spandrel connection modeling.

Engine Impact Analysis

An example of an engine impacting an assembly of exterior panels is shown in Fig. C–71. The analysis includes an engine impacting an exterior panel of WTC 1 and is centered on panel 124 at floor 96 (the impact is centered on the middle spandrel). The exterior wall model does not contain any boundary conditions or components to represent the truss floor behind the panels. The initial engine speed was 500 mph (223 m/s). In this example, a medium mesh density shell element panel was used. Columns were connected with beam element models of individual bolts. Spandrels were merged together in this model—splice plates were not used. Fixed butt plates were bolted at the free column end and no boundary conditions were applied to represent the floors. Velocity time-histories for representative engine materials are shown in Fig. C–72. The plot shows an overall reduction in speed of about 13 percent after impact with the exterior wall.

A second engine impact analysis was performed with similar conditions except the impact location was moved downward by one half floor to create an impact centered between spandrels. A comparison of the two calculated impact behaviors is shown in Fig. C–73. In both analyses the engine breaks through the exterior wall with relatively little breakup of the engine core. The impact centered between spandrels results in a reduction in the velocity of the engine core of 56 mph (25 m/s).



(a) Initial configuration **(b) Impact response at 40 ms**

(c) Impact response at 80 ms

Figure C–71. Example engine impact analysis with exterior columns.

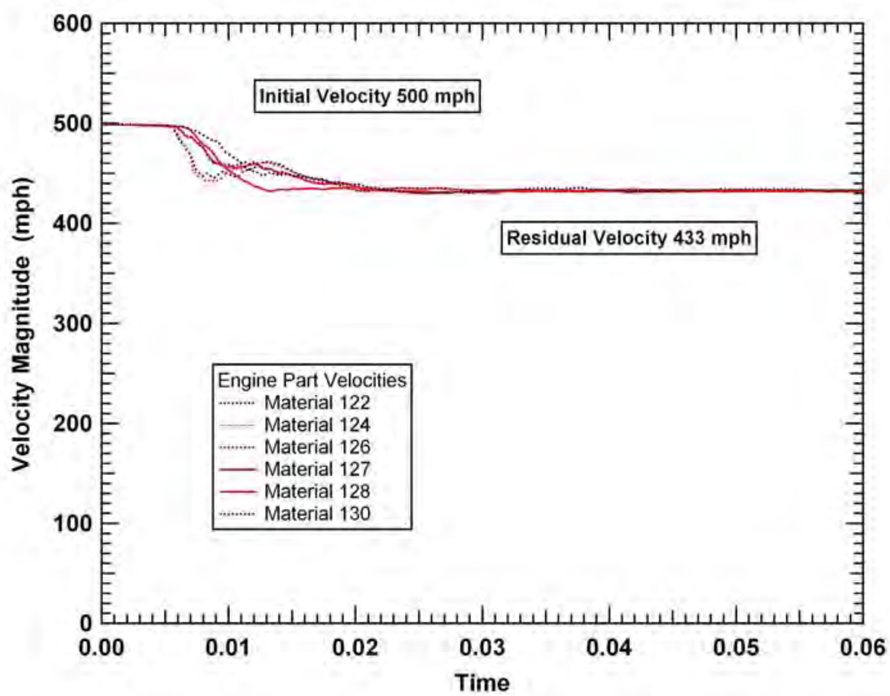
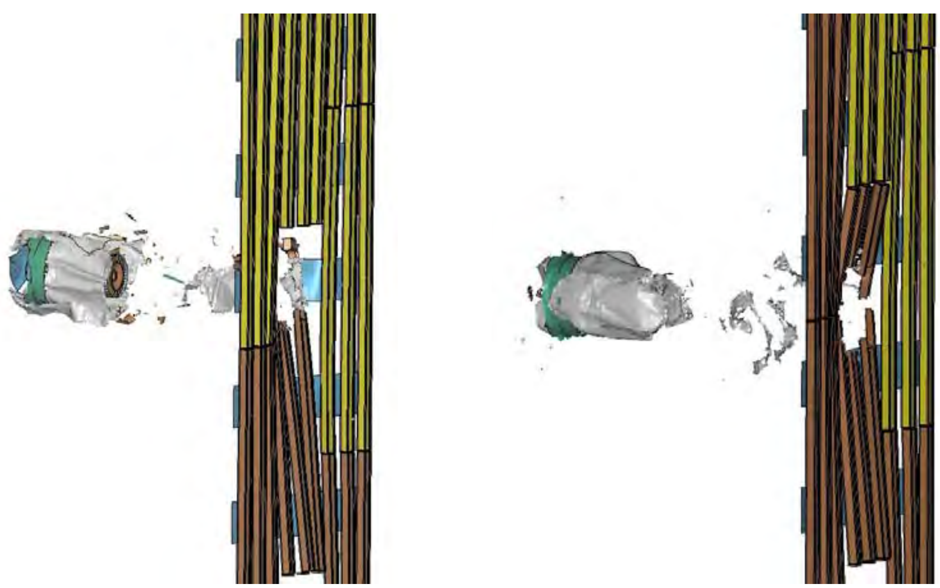


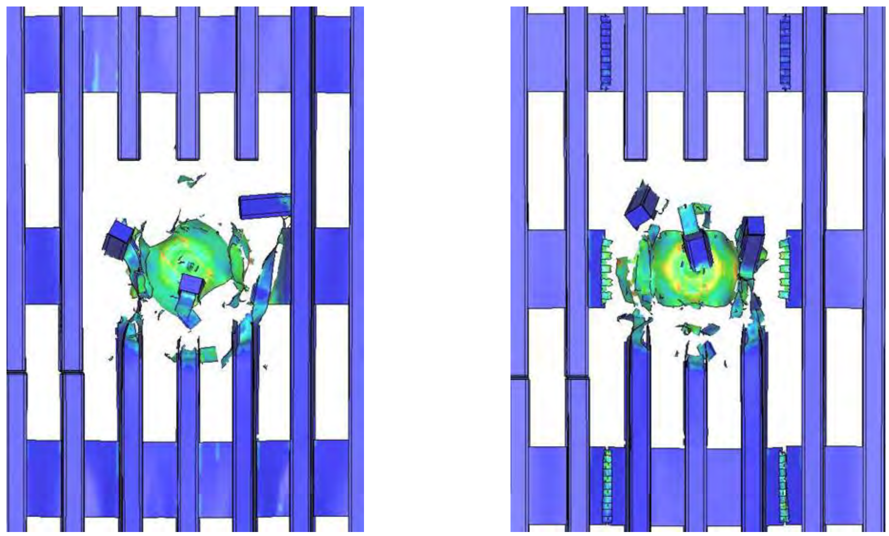
Figure C–72. Engine velocity history for the exterior wall impact.



(a) Spandrel centered impact **(b) Between-spandrel impact**

Figure C–73. Example engine impact analysis with different impact locations.

A final revision of the above analysis included adding splice plates at the spandrel connections as explained in the previous subsection. Figure C–74 presents a comparison of both models. The figure shows exterior wall damage as seen from the outside without engine components. Contours of plastic strain are shown in the plot with blue being zero strain and red being at the failure strain threshold (20 percent in this case). Material exceeding the maximum failure strain is eroded and no longer shown in Fig. C–74. The analyses differ most in spandrel failure modes. Spandrels fail at the column connection in the merged spandrel case, while the connection fails in the splice plate case. In the latter case, a realistic bearing stress type tear-out mode is seen in the splice plate. Engine core velocities for the three engine impact analyses are compared in Fig. C–75 for a single representative engine component. The splice plate model resulted in a 74 mph (33 m/s) velocity reduction of the engine core. The splice model results in the largest reduction in velocity in the engine.



(a) Merged spandrel analysis **(b) Spliced spandrel analysis**

Figure C–74. Example engine impact analysis with different spandrel connection treatments.

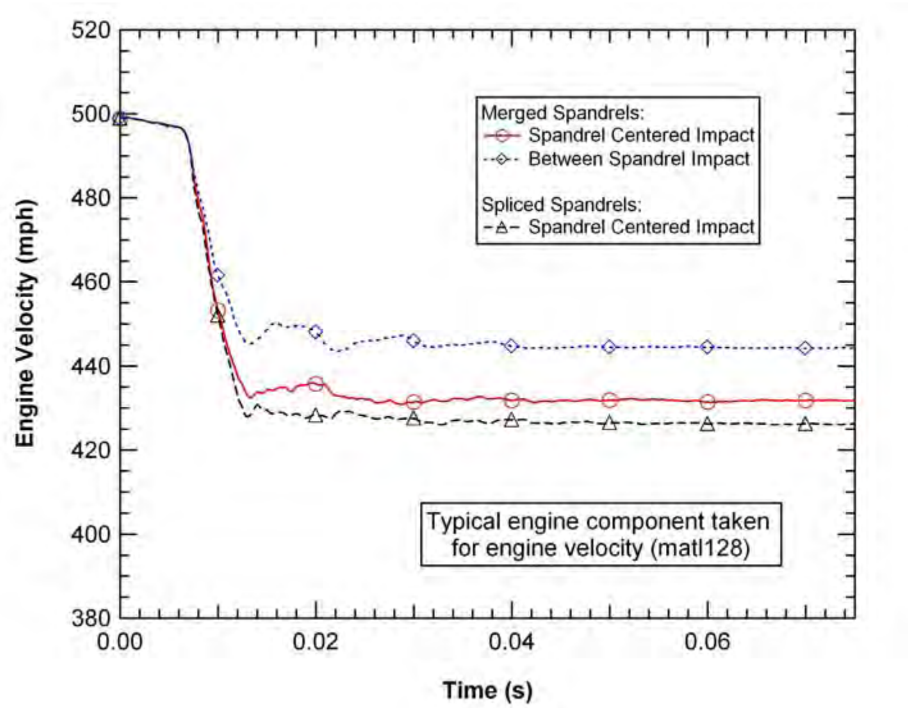


Figure C–75. Engine velocity history for the exterior wall impact.

The above comparison suggests that the splice plate has a relatively small influence on the exterior wall strength. The addition of the splice plate has approximately a 10 percent effect on the change in engine velocity during impact and penetration of the exterior wall. However, the spandrel connection model does not introduce a large computational cost and results in a more appropriate failure mode for the spandrels in the impact zone. Therefore, the spandrel splice connections will be maintained in subsequent impact analyses.

C.4.3 Core Column Impact Analyses

Engine Impact Analysis

Preliminary analyses of an engine impacting a single core column indicated that the impact load was sufficient to penetrate and overwhelm the column. Subsequent analyses of an engine impacting core columns have included multiple impacts and will be reported later along with the subassembly analysis and the uncertainty analyses.

Wing Section with Fuel Impact Analysis

Component analyses for core columns impacted by fuel-filled wing sections were conducted for both wide flange and box type columns (Figs. C–76 through C–79). Similar to exterior column analyses, the primary purpose was to progress from the initial finely meshed brick element models to coarser shell element models. Figure C–76 (wide flange core column) and Fig. C–78 (box section core column) compare the fine brick model and coarser shell model response under the same loading conditions. Figures C–77 and C–79 show corresponding displacement and kinetic energy comparisons for, respectively, the wide flange and box section core column models. The figures indicate that the response of the coarser shell models is very similar to that of the fine brick models. Therefore, the shell element formulation and mesh refinement of the coarse model are sufficient to capture the impact damage mechanisms in this component impact scenario.

C.4.4 Combined Engine Impact Analyses

An example of engine impact analysis is presented in this section to demonstrate the damage response to both exterior and interior columns. The impact configuration is an engine impacting at 560 mph (250 m/s) against a set of exterior columns in a single exterior panel, an interior box column, and an interior wide flange column. The columns are modeled using shell elements. The spacing between exterior and core columns was reduced to shorten the run time necessary for the complete impact scenario. The simulation included three external panels stacked vertically such that the impacted column was bolted to additional panels both above and below. The core columns models were several floors tall to reduce the influence of the clamped boundary conditions at the ends.

(a) Fine brick element column **(b) Coarse shell element column**

Figure C–76. Wide flange core column response comparison, showing contours of the displacement magnitude (in.).

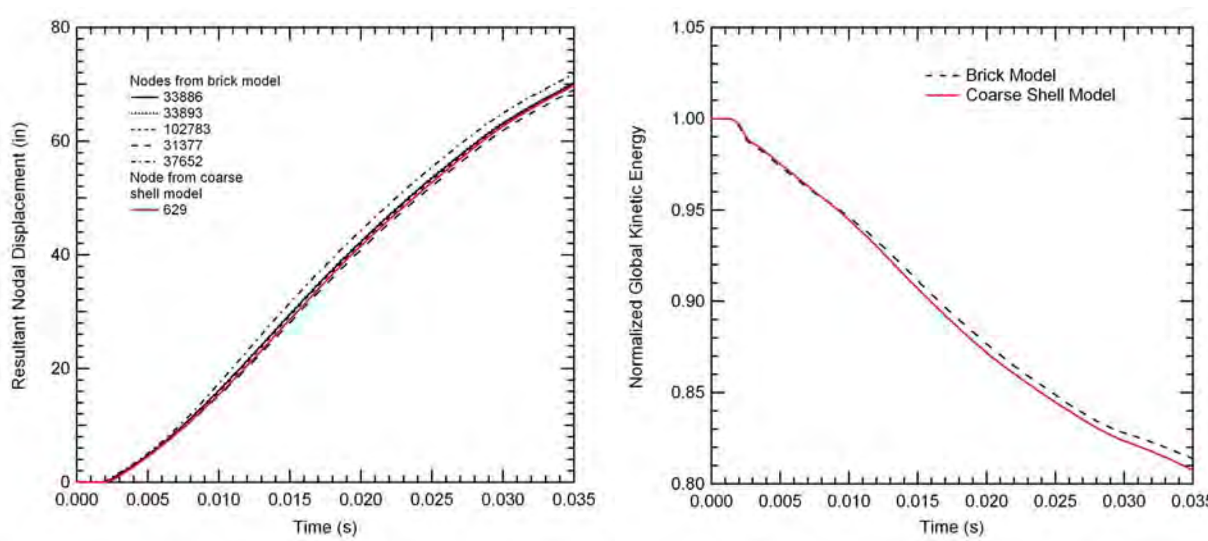
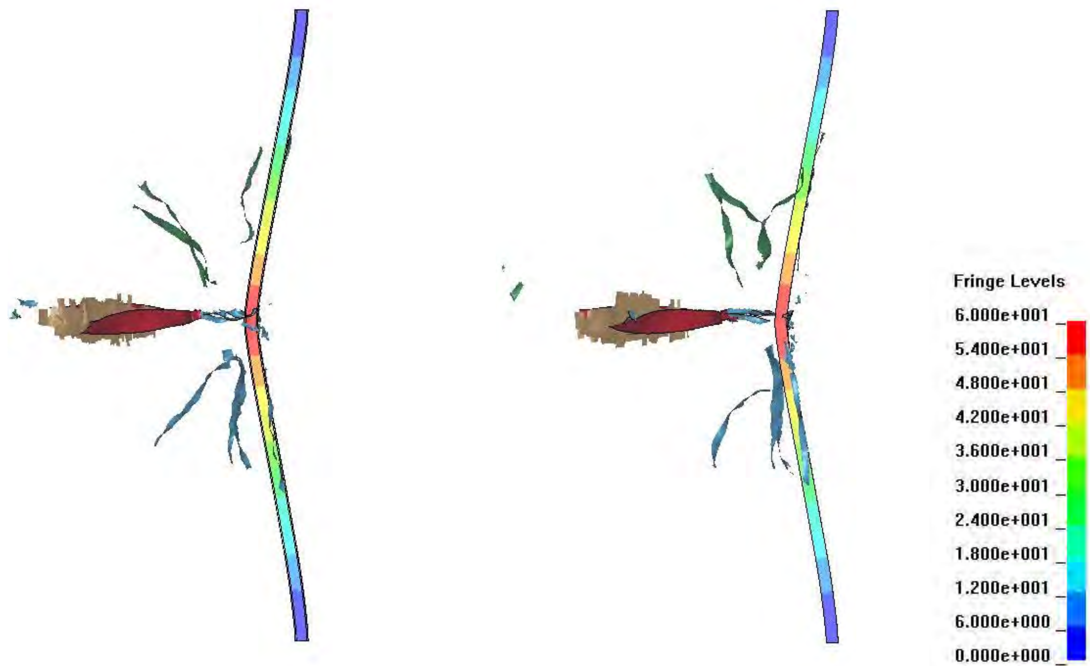


Figure C–77. Displacement and kinetic energy comparison for wide flange core column wing impact analysis.



(a) Fine brick element column **(b) Coarse shell element column**

Figure C–78. Box core column response comparison, showing contours of the displacement magnitude (in.).

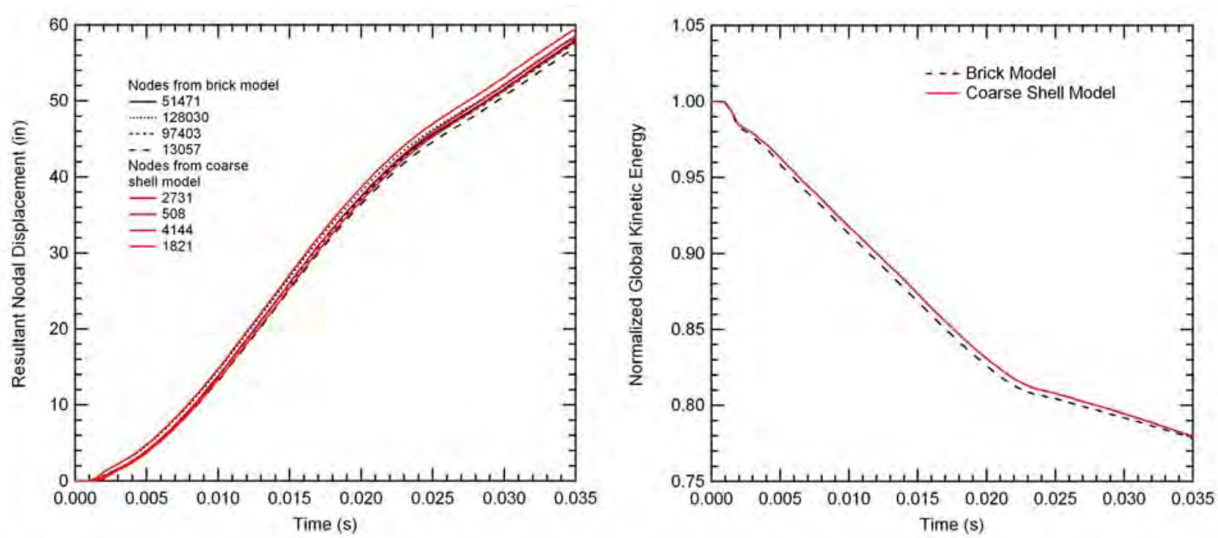
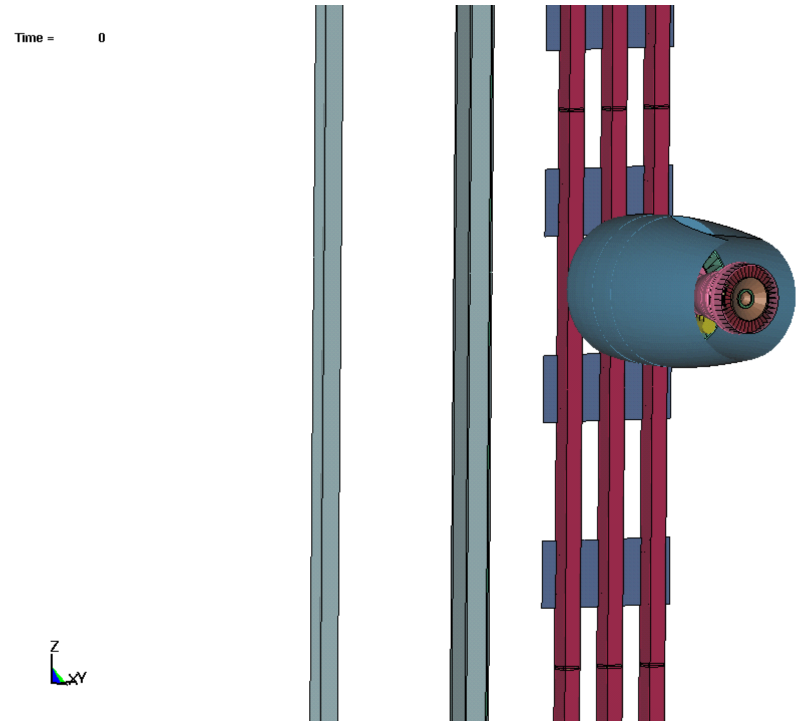
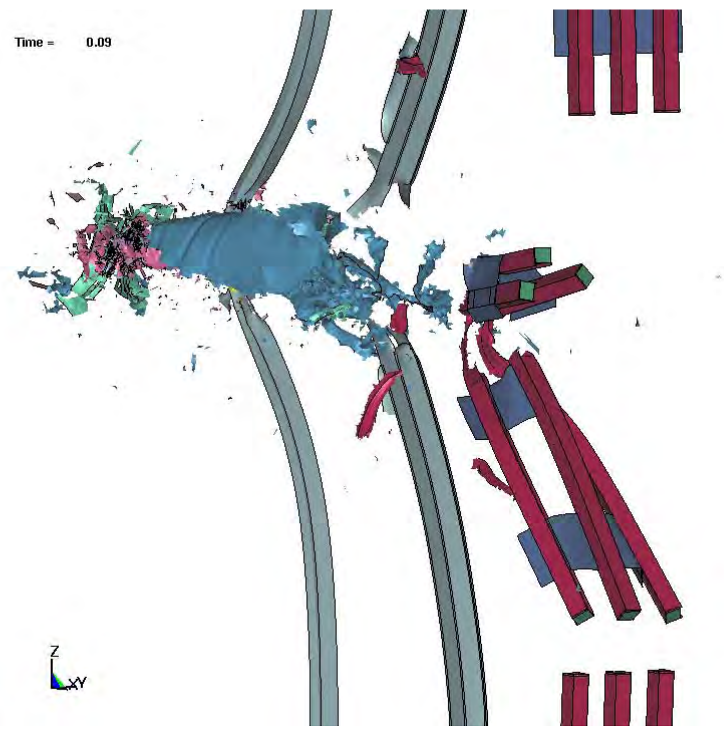


Figure C–79. Displacement and kinetic energy comparison for box core column wing impact analysis.

The impact scenario is illustrated in Fig. C–80 (a). The calculated impact damage is shown at a time of 90 ms in Fig. C–80 (b). The calculated engine impact response completely fails all of the columns with a residual velocity of the engine of approximately 224 mph (100 m/s). The deformations of the column include large lateral displacements which would be significantly reduced if the constraint effects of the concrete floor slab were added. The deceleration profile of the major engine debris fragment is given in



(a) Initial configuration



(b) Impact response at 90 ms

Figure C–80. Example engine impact analysis with interior and exterior columns.

Fig. C–81. The majority of the engine structure has been broken into fragments by the combination of the three impacts. The resulting size, strength, and velocity of the engine debris are not likely to produce severe impact damage or failure to a subsequent core column.

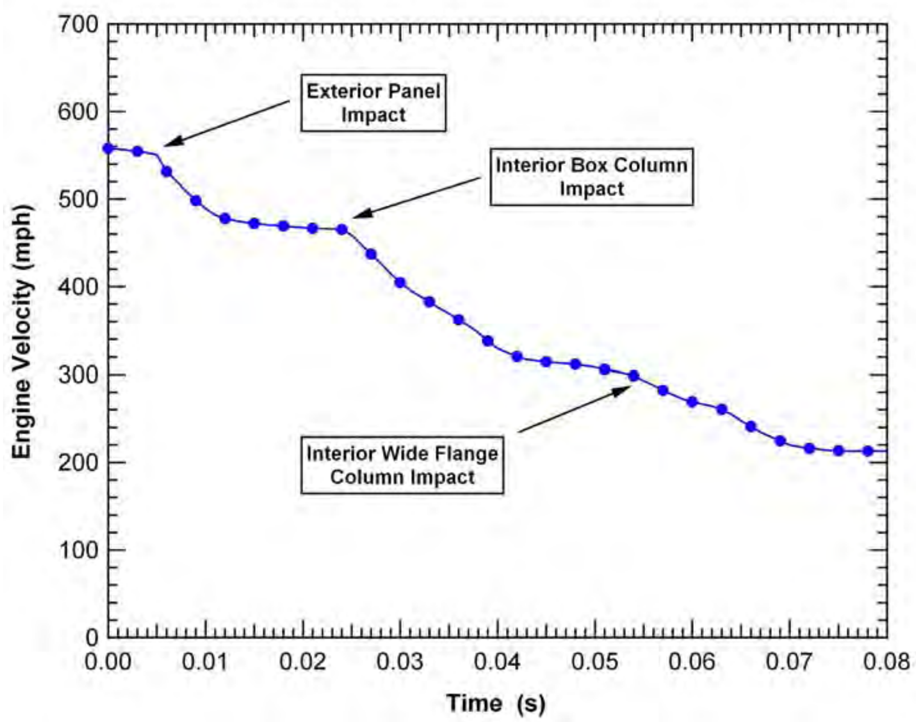


Figure C–81. Example engine impact analysis with interior and exterior columns.

This example illustrates the large level of damage produced by a massive aircraft component, such as an engine, at high impact velocity. The impact energy is sufficient to overwhelm a single core column and, therefore, makes it difficult to determine the effects of column component modeling parameters on the impact response using this impact scenario.

C.4.5 Floor Assembly Component Analyses

An additional component model analyzed is the truss floor assembly. The failure and penetration of the floor structures is important for assessing the extent of damage and the spread of fuel and debris through the structure. The integrity of the floor structures could also be significant in the analysis of the subsequent fire behavior in the towers. Loading of the floor structure was achieved by direct impact by the engine and a simplified impactor.

Component models of a section of the composite floor assembly outside the core were generated and used in various impact analyses. Initial floor component models used the detailed combination of brick, shell, and beam elements. Subsequent floor assembly models were less refined with shell and beam elements only. A comparison of the two floor system models and analyses is given in Table C–7. The modifications have reduced the size of the model by an order of magnitude and the run times by more than 80 percent.

Table C–7. Truss floor assembly component analyses comparison.

Model Type	Fine Brick Model	Coarse Shell Model
Number of beam elements	6,928	3,440
Number of brick elements	230,778	0
Number of shell elements	148,256	39,000
Total nodes	372,084	48,971
CPU time	16,796 s (4.7 h)	2,482 s (0.7 h)
Elapsed time	26,553 s (7.3 h)	4,454 s (1.2 h)

The impactor used in the initial component modeling is a simplified plow type impactor which promotes repeatable damage, not complicated by all the debris and randomness associated with an engine-floor impact. The weight of the plow impactor is comparable to an engine and the impact velocity was 500 mph. An example of a plow impactor analysis with the fine mesh floor model is shown in Fig. C–82. The calculated impact damage with a corresponding shell element floor system model is shown in Fig. C–83. This component impact configuration is useful for comparing the differences in response with changes in the modeling methods or refinement. Additional simulations with an engine impactor are being performed to validate the modeling approach and will be reported at a later date.

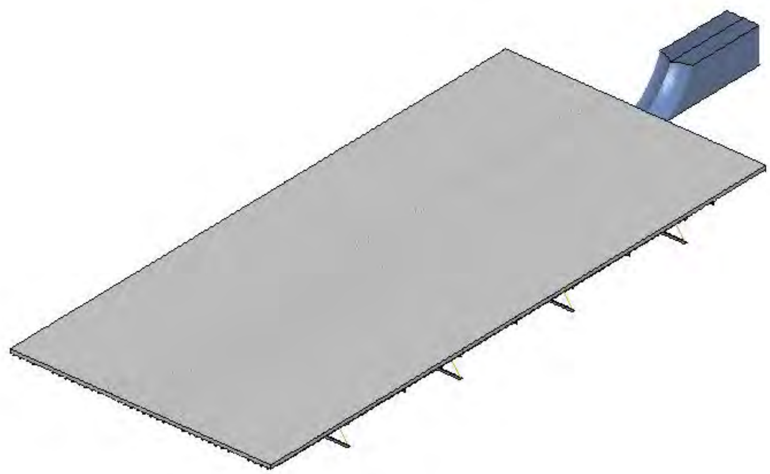
C.5 SUMMARY AND PRELIMINARY FINDINGS

The objectives of this project are to calculate the aircraft impact response of the WTC towers, including damage to WTC tower structural systems, acceleration environment, and fuel and debris dispersion. The impact analyses are conducted at various levels, including: (1) the component level, (2) the subassembly level, and (3) the global level to estimate the probable damage to the towers due to aircraft impact. The analyses also include simplified and approximate methods. Analysis of uncertainties using the component, subassembly, global, and simplified analyses will also be conducted to assess the effect of variability associated with various parameters on the damage estimates.

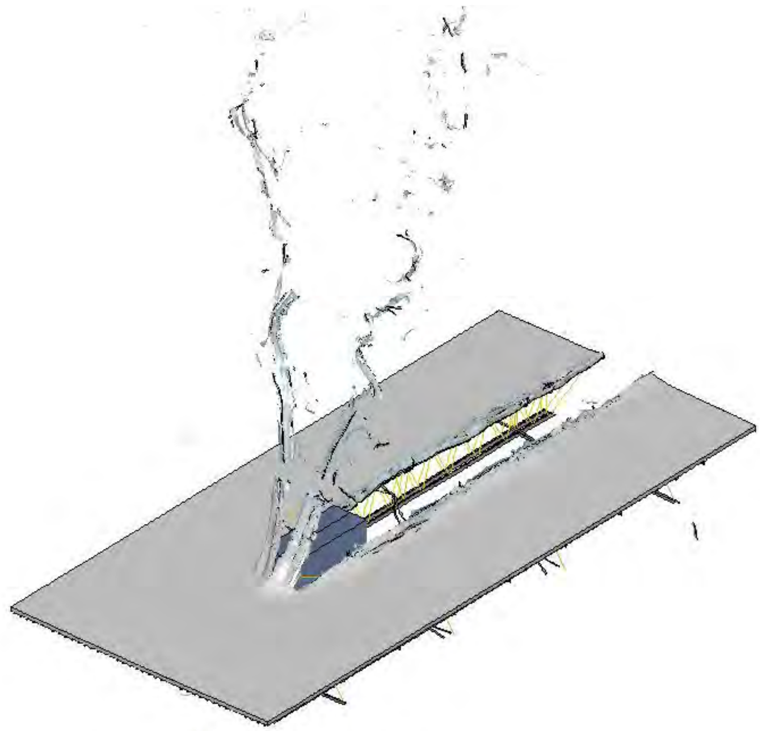
Significant progress has been made to identify the proper constitutive relationships, including high strain-rate effects and failure criteria for the various materials included in the analysis of aircraft impacts into the WTC towers. These materials include steels used in the exterior walls and core columns of the towers, weldment, bolts, reinforced concrete, and aircraft materials.

The development of the Boeing 767 aircraft model for impact analysis is nearing completion. The engine and wing models have been completed and are being used in the component and subassembly analyses. Work is underway to finalize the model of the fuselage, nose, tail, and nonstructural components of the aircraft.

The WTC towers and Boeing 767 aircraft are extremely complex structural systems, and a large database of detailed structural information has been collected on both the towers and the aircraft. In the model development process, the objective was to include all of the structural components and details of both the aircraft and towers that would influence the impact response and damage. This approach results in very large models for the tower and aircraft. The application of the models in the component and subassembly analyses were used to determine model simplifications that can reduce the overall model size while maintaining fidelity in determination of the impact damage.

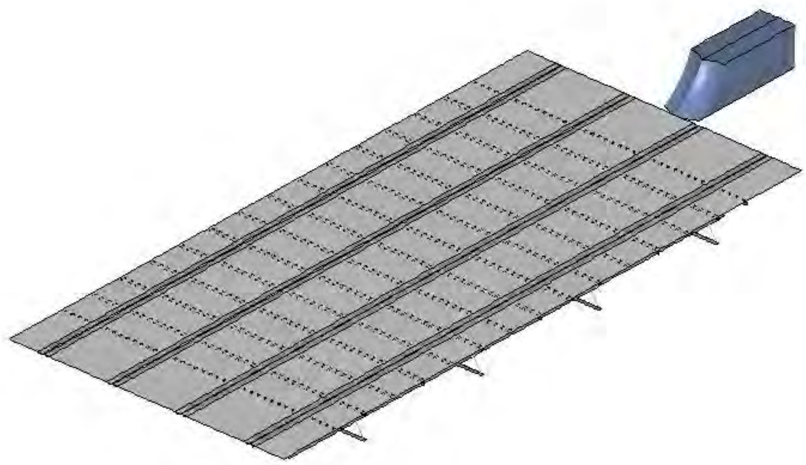


(a) Initial configuration

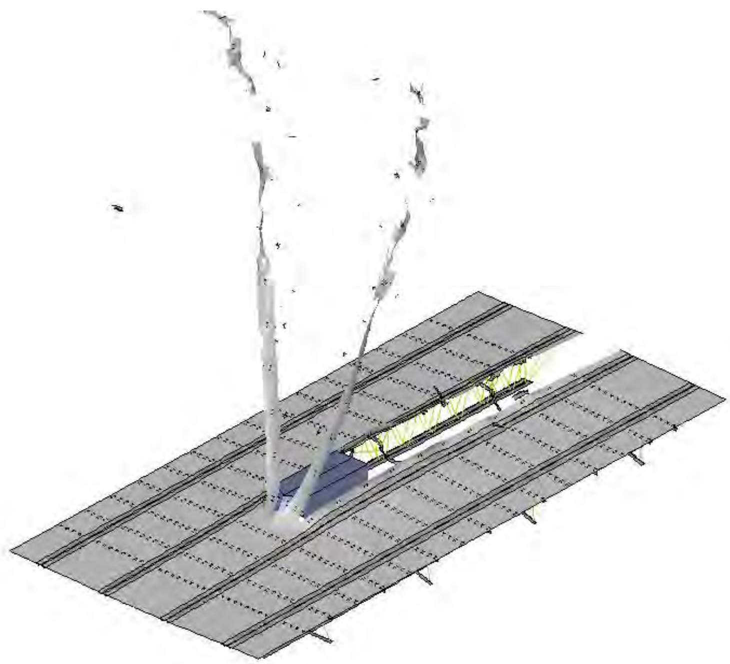


(b) Impact response at 0.10 s

Figure C–82. Floor assembly impact analysis with brick element concrete slab.



(a) Initial configuration



(b) Impact response at 0.10 s

Figure C–83. Floor assembly impact analysis with shell element concrete slab.

A series of component impact analyses were performed using models of tower exterior and core columns with wing section and engine component models as impactors. These models are used to develop the simulation techniques required for the global analysis of the aircraft impacts into the WTC towers. The following results were obtained from the component impact analyses:

- A 500 mph (223 m/s) engine impact against an exterior wall panel results in a penetration of the exterior wall and failure of impacted exterior columns. If the engine does not impact a floor slab, the majority of the engine core will remain intact through the exterior wall penetration with a reduction in velocity between 10 percent and 20 percent. The residual velocity and mass of the engine after penetration of the exterior wall is sufficient to fail a core column in a direct impact condition. Interaction with additional interior building contents prior to impact or a misaligned impact against the core column could change this result.

- A normal impact of the exterior wall by an empty wing segment from the wing tip region will produce significant damage to the exterior columns but not necessarily complete failure. This is consistent with photographs showing the exterior damage to the towers due to impact. Specific details of the damage will depend on details of the impact orientation and locations of internal wing components such as control surface actuators and arms.

- A fuel-filled wing section impact results in extensive damage to the external including complete failure of the exterior columns. This is also consistent with photographs of the exterior damage. The resulting debris propagating into the building maintains the majority of the initial momentum prior to impact.

- Three different numerical techniques were investigated for modeling impact effects and dispersion of fuel: (1) standard Lagrangian finite element analysis with erosion, (2) Smoothed Particle Hydrodynamics (SPH) analysis, and (3) Arbitrary-Lagrangian-Eulerian (ALE) analysis. Of these approaches, SPH analyses appear to offer the greatest potential for modeling fuel in the global impact analysis due to the combination of both computational efficiency and modeling fidelity.

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Appendix D
INTERIM REPORT ON PRELIMINARY STABILITY ANALYSIS OF THE WTC TOWERS

D.1 INTRODUCTION

The objective of this appendix is to present preliminary system stability analyses of the World Trade Center (WTC) towers to: (1) examine the overall stability of the towers when floors are removed; (2) study possible redistribution mechanisms when core columns are destroyed by aircraft impact; and (3) study the response of the tower when columns and spandrels in the exterior walls and columns in the core are destroyed by aircraft impact, and columns in the exterior are damaged due to the subsequent fires, as observed in photographs and videos of WTC 1. The analyses use a reduced and modified version of the global reference structural model of WTC 1 and the model of a typical truss-framed floor (floor 96 of WTC 1), both developed within the framework of Project 2 of the investigation using SAP2000, version 8 (see Appendix B). Although analyses are conducted using models of WTC 1, some of the results and findings apply to WTC 2 as well.

The analyses use a staged construction technique to account for the sequential construction of the towers, especially in the zone of the hat trusses. Linear buckling analysis and nonlinear analysis with plastic hinges are used in the reduced global model of WTC 1 to study the effects of removal of, respectively, floors and damaged exterior and core columns, representing the effects of aircraft impact and subsequent fire effects. In addition, a linear analysis of the typical floor model is used to study the load redistribution mechanisms after losing columns in the core of the tower.

Section D.2 presents a description of the reduced global model of WTC 1 used in this study, including the modifications that were made to the reference model. Section D.3 outlines the analysis procedures, including the staged construction methodology, the eigenvalue buckling analysis, the nonlinear analysis with plastic hinges, and the linear analysis of the typical floor model. The results of these analyses are presented in Section D.4, and Section D.5 presents a summary of the analysis and results.

D.2 MODEL DESCRIPTION AND ASSUMPTIONS

The models considered for the preliminary system stability analyses of the WTC towers were based on the reference structural analysis global model of WTC 1 and the typical truss-framed floor model developed by the firm of Leslie E. Robertson Associates, R.L.L.P. (LERA) under contract from NIST within the framework of Project 2. These reference models, developed using Computers and Structures, Inc.'s SAP2000 Software, Version 8, were reviewed and approved by National Institute of Standards and Technology (NIST) (see Appendix B). The reference global models are linear elastic, three-dimensional structural analysis models and include the 110-story above grade structure and 6-story below grade structure for each of the two towers. The original models use frame elements to represent the exterior columns and spandrels, the core columns, and the hat trusses. Each element in the models is assigned cross-sectional properties and steel strength according to the original design documents, as well as later modifications made to the towers.

A reduced version of the original WTC 1 global model was used in this project to assess typical behavior of the intact structure, as well as the performance of the damaged structure, due to aircraft impact and fire effects. The intent of the reduced model was to minimize the computational effort without a major sacrifice in model performance. The reduced model included the global model of the structure above floor 84 of WTC 1. The structure below was removed and replaced with equivalent springs as summarized in Section D.2.2. The modifications and loads applied to the model are summarized below.

The model of a typical truss-framed floor (floor 96 of WTC 1) was used to study the load redistribution mechanisms inside the core upon losing core columns due to aircraft impact. The floor model contains all primary structural members of the floor system, including primary and bridging trusses, beams in the core, strap anchors, viscoelastic dampers, exterior and core columns above and below floor level, spandrel beams, and concrete slabs. The gravity loads applied to the model, including dead loads, superimposed dead loads, and service live loads are presented in Section D.2.4.

D.2.1 Steel Properties

The values of the yield and ultimate strengths of the structural steel used in the WTC 1 reduced tower model were set to match the room-temperature properties, which were determined by Project 3 of the NIST investigation, by replacing the nominal strength included in the reference models with actual strength values. Project 3, "Mechanical and Metallurgical Analysis of Structural Steel," provided estimates for typical steel properties based on test results from a limited number of steel specimens from the towers, construction documents that indicate the occasional substitution of higher strength steels for lower strength steels, and historical data from steels of that era. These estimates differentiate among steels with the same designation from different manufacturers and from different areas of the buildings; in particular, steel in the exterior columns, core columns, and floor trusses each had slightly different properties for steels with the same designation. In cases where steel with a particular designation had different properties in the exterior and core columns, the properties associated with the steel in the exterior columns were used in the reduced global model. The steel properties used for the work reported herein are listed in Table D–1.

D.2.2 Boundary Conditions: Spring Supports

The reduced model of WTC 1 was supported by vertical springs assigned to each joint (core and exterior) at floor 84. The spring stiffness coefficients were obtained from a separate model of the tower below floor 84. For that purpose, a concentrated gravity load was applied to each column node at floor 84 of the tower model below floor 84, and the spring stiffnesses were estimated by dividing the applied load by the measured vertical displacement of each column at floor 84. At the bottom of the reduced model, each joint with an assigned vertical spring was restrained from horizontal translation and rotation about all three axes. These boundary conditions provided results that most closely matched those obtained from analyzing the whole tower (i.e., all 116 floors).

Table D–1. Steel strength used in the reduced tower model.

Design Yield Strength ksi (MPa)	Model Yield Strength F_y ksi (MPa)	Model Ultimate Strength F_u ksi (MPa)
36 (248.2)	35.6 (245.5)	61.2 (422.0)
42 (289.6)	53.1 (366.1)	74.9 (516.4)
45 (310.3)	53.1 (366.1)	74.9 (516.4)
46 (317.2)	53.1 (366.1)	74.9 (516.4)
50 (344.7)	54.0 (372.3)	75.6 (521.2)
55 (379.2)	60.8 (419.2)	82.6 (569.5)
60 (413.7)	62.0 (427.5)	87.3 (601.9)
65 (448.2)	69.6 (479.9)	90.4 (623.3)
70 (482.6)	76.7 (528.8)	92.0 (634.3)
75 (517.1)	82.5 (568.8)	96.8 (667.4)
80 (551.6)	91.5 (630.9)	99.4 (685.3)
85 (586.1)	104.8 (722.6)[a]	116.0 (799.8)[a]
90 (620.5)	104.8 (722.6)[a]	116.0 (799.8)[a]
100 (689.5)	104.8 (722.6)	116.0 (799.8)

a. The steel fabricator used steel with a nominal strength of 100 ksi in place of steels with specified strengths of 85 ksi and 90 ksi.

D.2.3 Floor Systems

Floor systems distribute gravity loads to the core and exterior columns. Actual member properties of the floor elements have relatively little effect on the towers' stability, but would significantly increase model complexity and decrease its efficiency. To capture their effect, each floor of the tower in the reduced global model was modeled with a rigid diaphragm, except for floor 107 to the roof, which were modeled using flexible diaphragms as described in Appendix B. Rigid diaphragms constrain all nodes at a particular floor to move as a single unit. Flexible diaphragms differ only in the level of constraint. Both types of diaphragms do not affect the relative vertical displacements of the nodes. Modeling floors with diaphragms (rigid or flexible) ignores the floor's capability of redistributing loads from column to column in a damaged case (particularly within the core columns). More detailed models developed within the framework of Project 2 for the aircraft impact analysis and Project 6 for the thermal-structural and collapse initiation analysis will address this issue.

D.2.4 Applied Loads

Gravity loads from the floor systems were applied as joint loads on columns in the model. They are comprised of: dead load (DL), which includes the self-weight of all structural members and the floor systems; superimposed dead load (SDL), which includes additional static dead loads such as partitions, fireproofing, ceiling systems, and floor coverings; and service live load (LL) which includes occupants and furnishings. Each load was applied to the model as a separate load case. Identical loads were used in the reduced global model of WTC 1 on floors 85 through 106. These floors were considered to be typical floors as described below. Loads varied on floors 107 through 110, and on the roof. The antenna load

(724 kips) was distributed among eight points near the center of the roof. These loads should be considered preliminary; NIST is working with LERA to further refine them.

The loads were developed based on a realistic assessment of service loads and their distribution throughout the towers. The service live loads were assumed to be 25 percent of the design live loads. This parametric value can easily be varied in a sensitivity analysis. Actual self-weights were used for the dead loads, and additional loads on the mechanical floors were accounted for explicitly.

The detailed model of a typical truss-framed floor, floor 96 of WTC 1 (see Appendix B), was used to determine the actual loads on floors 85 through 106 of the reduced global model. The detailed floor model contains four distinct areas, each with its own load, as shown in Fig. D–1. The superimposed dead and live loads applied to each of these areas were determined from the design documents and are listed in Table D–2. In the core, the design live loads varied from 40 psf (1.92 kN/m^2) to 100 psf (4.79 kN/m^2) and were scaled to service live loads from 10 psf (0.479 kN/m^2) to 25 psf (1.20 kN/m^2), while the superimposed dead load varied from 29 psf (1.39 kN/m^2) to 49 psf (2.35 kN/m^2). The loads due to the slab, trusses, beams, columns, and spandrels were all calculated based on the actual weights of the members. The reactions at each column due to the loads and self-weight of this typical floor were calculated separately for the dead load and superimposed dead load cases. The weights of the columns and spandrels were then subtracted from the DL results to determine the load on each column from the floor system. These loads were then applied as point loads to the reduced global tower model. The loads due to the self-weight of the columns and spandrels were calculated by the analysis software (SAP2000) for the reduced global tower model.

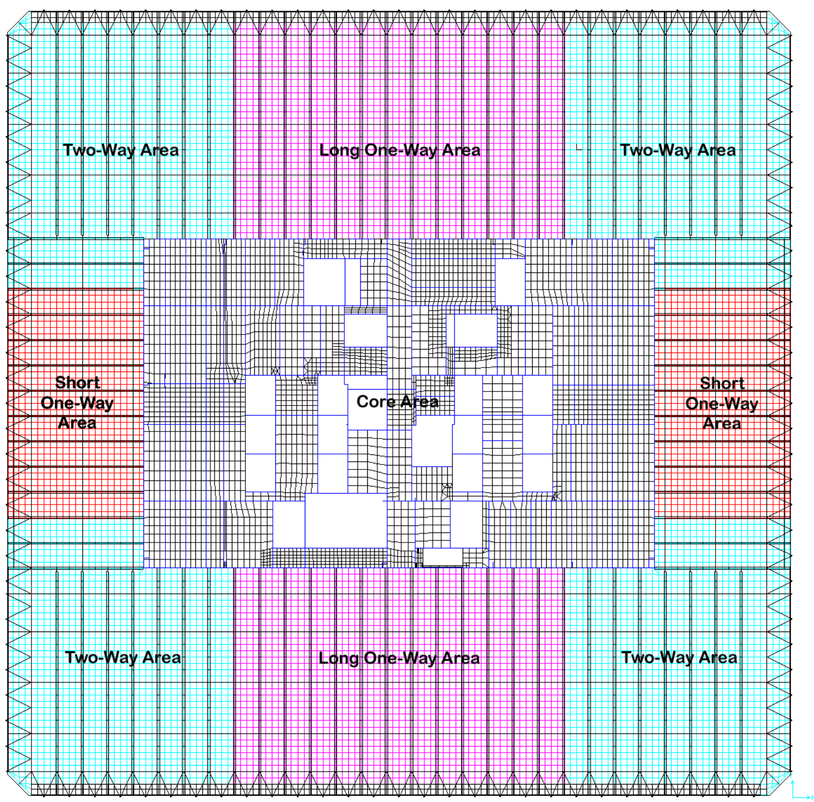


Figure D–1. Plan of typical truss-framed floor with loading areas indicated.

Table D–2. Superimposed dead load and service live load on typical floor.

	Core	Long One-way Slabs	Short One-way Slabs	Two-way Slabs
Area [ft^2 (m^2)]	9274 (861.6)	13186 (1,225.1)	5228 (485.7)	12233 (1,136.5)
Superimposed Dead Loads [psf (kN/m^2)]				
Mechanical & electrical		2.0 (0.096)	2.0 (0.096)	2.0 (0.096)
Ceiling		2.0 (0.096)	2.0 (0.096)	2.0 (0.096)
Floor covering		2.0 (0.096)	2.0 (0.096)	2.0 (0.096)
Fireproofing		2.0 (0.096)	2.0 (0.096)	4.0 (0.192)
Total SDL	**Varies**	**8.0 (0.383)**	**8.0 (0.383)**	**10.0 (0.479)**
Live Loads [psf (kN/m^2)]				
Service LL	Varies	17.5 (0.838)	21.3 (1.020)	13.8 (0.661)

For the mechanical floors (floors 107 through 110) and roof, the design DL, design SDL, and service LL values were determined from the design documents and information provided by LERA. These loads, which are listed in Table D–3, were applied as uniform loads to the typical floor model to estimate the corresponding column reactions. The column reactions were then applied as point loads on the reduced tower model.

Table D–3. Dead, superimposed dead, and live loads on mechanical floors.

Floor	**107**	**108**	**109**	**110**	**Roof**
Dead Loads [psf (kN/m^2)]					
Concrete slab	100.0 (4.788)	69.0 (3.304)	69.0 (3.304)	104.0 (4.980)	48.1 (2.304)
Reinforcing steel	2.0 (0.096)	3.0 (0.144)	3.0 (0.144)	3.0 (0.144)	2.0 (0.096)
Steel deck	2.0 (0.096)	2.0 (0.096)	2.0 (0.096)	2.0 (0.096)	2.0 (0.096)
Structural steel	13.0 (0.622)	20.0 (0.958)	20.0 (0.958)	20.0 (0.958)	(a)
Total DL	117.0 (5.602)	94.0 (4.501)	94.0 (4.501)	129.0 (6.177)	52.1 (2.496)
Superimposed Dead Loads [psf (kN/m2)]					
Partitions	12.0 (0.575)	-	-	-	-
Ceiling	2.0 (0.096)	10.0 (0.479)	10.0 (0.479)	-	-
Mech. & elec.	2.0 (0.096)	3.0 (0.144)	3.0 (0.144)	50.0 (2.394)	50.0 (2.394)
Fireproofing	2.0 (0.096)	5.0 (0.239)	5.0 (0.239)	5.0 (0.239)	5.0 (0.239)
Flooring	57.0 (2.729)	31.0 (1.484)	31.0 (1.484)	-	5.0 (0.239)
Total SDL	75.0 (3.591)	49.0 (2.346)	49.0 (2.346)	55.0 (2.633)	60.0 (2.873)
Live Loads [psf (kN/m2)]					
Service LL	25.0 (1.197)	18.8 (0.898)	37.5 (1.796)	18.8 (0.898)	37.5 (1.796)

a. The roof structural steel is explicitly included in the tower model.

D.3 ANALYSIS PROCEDURES

Both linear and nonlinear analyses were performed on the reduced global model of WTC 1 to examine the tower stability and assess how the tower responded to the representative impact and fire damage. Due to the difference in stiffness between the core and the exterior columns, and the presence of the hat trusses, it was necessary to use nonlinear, staged construction to analyze the intact structure. Subsequently, two different series of analyses were performed independently.

An eigenvalue-based buckling analysis was performed using the reduced global model of WTC 1 to determine the reserve capacity of the columns to buckling, and to determine how much the unsupported column length would need to increase, through floor-constraint removal, before the columns lacked any reserve capacity.

A nonlinear analysis of the tower with damage to exterior walls and core columns was performed on the reduced global model of WTC 1 to determine if the tower could withstand that level of structural damage, and to assess the response of the tower when columns are lost due to aircraft impact and fire effects.

In addition, an analysis was conducted of the typical truss-framed floor model to study the mechanism by which the floor loads were redistributed when the core columns were severed by aircraft impact. In this analysis, the core columns that were assumed to be missing were replaced by equivalent vertical springs, representing the stiffness of the hat trusses and columns between the affected floors and hat trusses. The following describes the details of the various analyses.

D.3.1 Staged Construction

From a linear analysis of the response of the intact WTC towers to gravity loads, it was determined that a simple linear analysis does not produce realistic stress distributions in the core and exterior columns. All loads in a linear model are applied instantaneously, which is not unreasonable for most structural models. Tall buildings sustain loads gradually, as the structure is built from the ground up, and any differential deformation is accounted for during construction. In addition, the hat trusses atop the tower were applied stress-free to the existing structure subjected to dead loads, but prior to the application of live loads. The linear analysis (without staged construction) of the tower models resulted in unrealistic, large forces and stresses in some hat truss members, connecting spandrels, and core columns within the hat trusses, due to differential settlement between the core and exterior columns in the model.

A staged construction analysis of the towers eliminates these nonexistent, large stresses. This method more closely approximates the way in which the towers were constructed and the loads applied. The staged construction analysis had three stages. First, the floors below the hat truss (up to floor 106) constituted the initial model. The dead and superimposed dead loads were applied to these elements, and the model was analyzed. Second, the upper, remaining stories including the hat truss were added to the model. These newly added components did not initially have internal forces or stresses, even though the components added in the first stage were loaded and stressed. The remaining portions of the dead and superimposed dead loads were then added to these top floors, and the model was again analyzed; this analysis continued from the stress and strain state at the end of the first stage. In the third construction stage, the live and antenna loads were added to the entire model, and the analysis continued from the end of the second stage. This analysis method produced reasonable stresses in the hat truss region of the undamaged towers.

D.3.2 Eigenvalue Buckling Analysis

The objective of this analysis was to assess the overall stability of one of the towers, namely WTC 1, under service loads, without any aircraft impact damage and subject to a progression of floor removal. Stability was measured through column buckling strength, which was reduced in each column as floors were removed in the model and column unbraced lengths were increased. Floor removal was modeled by removing the rigid diaphragm constraint at all columns, discussed in Section D.2.3, for that particular floor. Each node within that floor was then free to translate (e.g., buckle) in either lateral direction. The four columns above and below each removed floor were subdivided into sixteen segments per floor to achieve sufficient resolution for estimation of buckling loads. If instability was identified using a linear stability (eigenvalue buckling) analysis, the analysis was rerun after buckled columns were removed and their loads were redistributed to neighboring columns. This process continued until either the structure was stable or the progression of local instabilities indicated overall system instability.

The buckling analysis began with the “removal” of floor 96. The analysis calculated the load factor (eigenvalue), λ, for the first buckled column. If λ was greater than one, all columns were stable under the given loading condition, which signifies system stability. If λ was less than one, a column had buckled under the applied loads. This column was identified by visually examining the buckled mode shape of the structure at the end of the analysis. Only the first buckling mode was considered in the analysis.

Buckling of a single column might not result in a collapse of the tower due to the load-redistribution capability of the structure. To investigate overall stability, the buckled column was removed from the model above and below the removed floor(s). Any joint loads applied to a removed column were distributed to neighboring nodes. This eliminated any load carrying capacity of the failed column without eliminating its applied load, but rather redistributing it. The analysis was rerun, and the next buckled column was identified until λ was greater than one or until the progression indicated that a global instability had likely been attained. If the structure attained stability, floor “removal” progressed sequentially to floors 95, 97, 94, 98, etc.

The linear bucking analysis in SAP2000 only provided the load factor, λ_L, for the linear combination of DL, SDL, service LL, and antenna loads, but without staged construction. Since staged construction was employed to best represent the application of these loads, λ must be obtained from a relationship with λ_L. The buckling load in the linear case (staged construction not used) is equal to the axial force in the critical (i.e., buckled) column, F_L, times λ_L. Since the buckling load is the same in either the linear case or the staged construction case, the load factor is defined as:

$$\lambda = \frac{\lambda_L F_L}{F_{SC}} \tag{1}$$

where F_{SC} is the axial load in the same column from the staged construction analysis.

The procedure described above was also performed on the undamaged WTC 1 reduced global model with a reduced modulus of elasticity (E’) applied to all core and exterior columns directly above and below removed floors. A value for E’ equal to 21,460 ksi (E’ = 0.74E), corresponding to a uniform column temperature of 600 ºC, was used in the analysis.

D.3.3 Redistribution of Forces within the Core Areas

Analyses of the global models of the towers indicated that when columns are severed in the exterior walls, the walls can redistribute their load through the vierendeel action of the wall above the severed columns. However, when columns are severed in the core, the possible load redistribution mechanisms include: (1) load redistribution to neighboring core columns through the nonlinear, large deflection, tensile membrane action of the floor, (2) load redistribution to the hat truss through tensile loads on the columns between the affected floors and the hat truss, or (3) a combination of both. The objective of this analysis is to determine the actual mechanism that occurs for a given damage pattern in the core columns of WTC 1.

The reduced global model of the tower lacks a complete floor system. As described in Section D.2.3, the floor systems were modeled as rigid or flexible diaphragms, which do not provide a path for vertical loads to be redistributed within the floors. Instead, when a core column is assumed to be damaged, all loads on that column from floors above the damage zone are redistributed through the hat truss in the model. This causes large tension forces in the damaged core columns.

A two-step approach was used to examine how the loads might redistribute. First, the typical floor model was analyzed with assumed damage to core columns. The severed columns were replaced by equivalent vertical springs, representing the combined stiffness of the hat truss and the axial stiffness of the columns between the floor and hat truss. In the analysis of the floor system, damage to the exterior walls of the tower was ignored, since it is assumed that the walls are capable of redistributing their loads. This analysis estimated what portion of the load would be redistributed as forces in the springs that will be transmitted to the hat truss, and what portion would be redistributed to neighboring columns through the floor system. Second, the tensile capacities of the core column splices between the affected floors and the hat trusses were estimated to determine if the columns could carry the calculated tensile loads.

To determine the equivalent stiffness of the hat truss, a separate model of the hat truss was first analyzed. For that purpose, a concentrated gravity load was applied at the node corresponding to the severed column, and the spring stiffness was estimated by dividing the applied load by the measured vertical displacement at that node. Then the axial stiffness of each of the columns above the damaged area was calculated. Finally, the model of the typical floor (Floor 96 of WTC 1) was modified to simulate the case of a floor above the damaged zone of the tower. The vertical support was removed from the base of the severed core columns, and spring restraints equivalent to the combined stiffness of the columns above and the hat truss were added to the tops of these columns. The floor model was then analyzed to determine how the loads would redistribute.

Two damage patterns in the core of WTC 1 were considered for this analysis: the first assumes destruction of fifteen columns (Core Damage Case 1), and the second assumes that only eight columns were severed (Core Damage Case 2). Table D–4 contains a list of core columns that were assumed to be destroyed for both damage cases (see Fig. D–2 for column locations).

Table D–4. Core columns removed from WTC 1, assumed destroyed by aircraft impact.

Core Column	Core Damage Case 1		Core Damage Case 2	
	Lowest Floor	Highest Floor	Lowest Floor	Highest Floor
503	96			
504	95	96	94	97
505	95	96	94	96
506	95			
603	96			
604	95	96	94	97
605	95	96	94	95
606	94			
703	96		94	95
704	96		94	97
705	96			
706	96		94	
803	96			
804	96			
805	96			
903			95	96

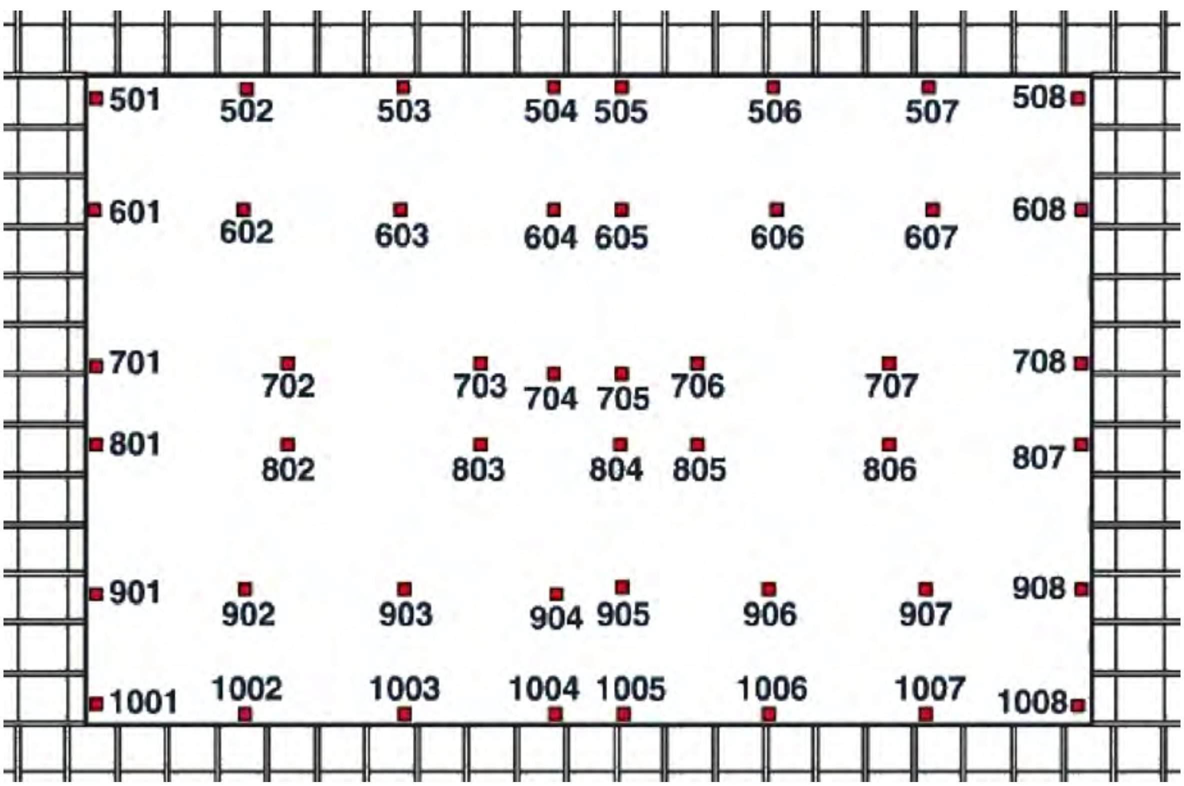


Figure D–2. Plan of typical core column layout (courtesy FEMA).

D.3.4 Nonlinear Analysis with Plastic Hinges

This analysis considered the nonlinear response of WTC 1 when an estimated pattern of damage had occurred. The damage scenario considered for this analysis included the following:

- Representative aircraft impact damage: based on photographic evidence, members in the north exterior wall of WTC 1 that were visibly severed or missing were assumed to be incapable of carrying load and were removed from the model, while members that appeared to be mostly intact were assumed to be capable of still carrying full load. This damage case also includes an exterior panel in the south face of the tower (columns 329 through 331 between floors 94 and 96) that was destroyed by the aircraft impact. In addition, eight columns in the core were assumed severed (see Core Damage Case 2 in Table D–4).

- Representative fire damage: 24 columns on the south face of WTC 1 between floors 96 and 98 were assumed to have buckled and lost all load carrying capacity. This assumption is based on video evidence that indicates that columns in this area were visibly deformed inward a few minutes before the tower collapsed.

The exterior members that were removed in this damage scenario are indicated in Fig. D–3.

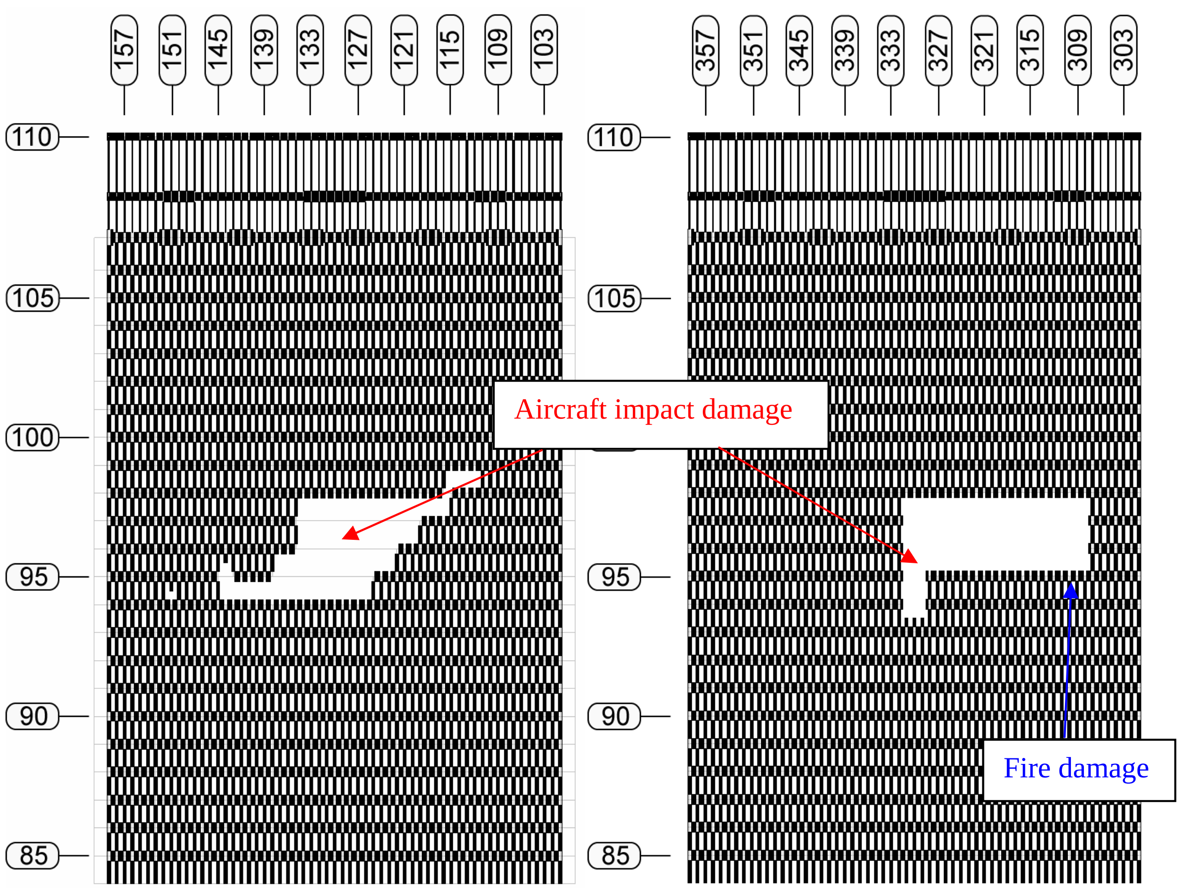


Figure D–3. North and south elevations of WTC 1 indicating columns and spandrels removed due to aircraft impact and fire effects.

To estimate how the damaged structure responded, the analysis considered geometric nonlinearities (large deflections and P-Δ effects) and material nonlinearities through a series of nonlinear, plastic hinges that were added to capture the post-yield behavior of structural members. The plastic hinges were placed in the reduced global model of WTC 1 based on a linear analysis of the damaged structures to determine the most stressed zones using a demand/capacity analysis. These hinges allow the members to act as nonlinear components, yielding once the stress on the member exceeds the material yield stress, continuing to accept some load at a reduced stiffness, and finally failing once an ultimate strain has been reached at an assumed ductility of 6. Hinges that considered both axial and bending forces (PMM hinges) were used in columns and hat truss members. Hinges that considered bending about the primary axis of the member, and shears in both the primary and secondary directions (MVV hinges) were used for most of the spandrels. Hinges that considered only bending about the primary axis (M3 hinges) were used for a small number of spandrels at the tower corners.

This analysis does not account for local bucking of columns; neither does it consider the failure or the role of the floor system in redistributing the loads. More detailed models, currently being developed within the framework of Projects 2 and 6, will account for these factors.

The damage due to aircraft impact analysis started from the end of the staged construction, described in Section D.3.1. At this stage, the set of damaged structural members that represent members destroyed by aircraft impact were removed from the reduced global model of WTC 1. This was followed by another stage, where the set of damaged structural members that represent members severely weakened by fire were removed from the model. For all analysis stages, room-temperature mechanical properties were used for all steels.

D.4 RESULTS

D.4.1 Results of Linear Stability Analysis

An initial analysis of the reduced undamaged model of WTC 1 under service loads with the 96th floor removed (i.e., the diaphragm constraint removed for all nodes at floor 96) produced a load factor for staged construction, λ, of 1.91. This indicated that no columns buckled under the application of DL, SDL, service LL, and antenna loads, and that the structure was stable. The structure was still stable with the additional removal of floor 95 ($\lambda = 1.03$). The analysis with floors 95, 96, and 97 removed yielded $\lambda = 0.65$ and the buckled column (core column 705, see Fig. D–2) was identified through visual observation of the first buckled mode shape.

Column 705 was removed from the model between floors 94 and 98, and the column's joint loads at floors 95, 96, and 97 were evenly distributed to joints of columns 704, 706, and 804. The analysis produced $\lambda = 0.78$ and indicated that column 704 had buckled. Column 704 was then removed from the model in a similar fashion to column 705. The combined joint loads of columns 704 and 705 were then distributed to neighboring joints at columns 703, 706, 803, 804, and 805. The load redistribution was proportional to the distance of each joint from the point halfway between joints 704 and 705. This analysis produced $\lambda = 1.38$, which indicated a stable tower with three floors removed.

The rigid diaphragm constraint at floor 94 was then removed from this latest model, i.e., floors 94 through 97 were unconstrained, columns 704 and 705 were omitted between floors 93 and 98, and the joint loads at each removed floor from these two columns were redistributed as above. The eigenvalue buckling analysis produced a load factor of 0.92 for this model and indicated that column 601 buckled. Column 601 was removed between floors 93 and 98, and its joint load was distributed to columns 501, 502, 602, and 701. This analysis produced $\lambda = 0.97$ and indicated column 608, similarly located along the perimeter of the core columns like column 601, buckled. The model with column 608 removed and its load distributed to columns 508, 507, 607, and 708 yielded $\lambda = 1.25$. Thus, the tower was stable with four floors removed and with the redistribution of loads from removed columns 705, 704, 601, and 608. Subsequent models with floor 98 removed suggested that the structure did not attain λ greater than one, and that the structure had likely reached a failed state.

When the modulus of elasticity was reduced from E = 29,000 ksi to E' = 21,460 ksi in columns above and below removed floors, the conclusions changed slightly but the progression of column failures remained the same. The tower maintained its overall stability with floor 96 removed ($\lambda = 1.38$). With floors 96 and 95 removed, the model of the intact structure indicated that column 705 buckled ($\lambda = 0.83$), but that stability was achieved through the removal of column 705 ($\lambda = 1.02$). With the removal of a third floor (97th), column 704 was also removed and its load redistributed in the model to maintain overall stability ($\lambda = 1.11$). The additional removal of a fourth floor (94th) produced a series of buckled columns (601, 608, 904, and 604) that indicated the structure would likely not achieve overall stability.

To summarize, the eigenvalue analysis examined the stability of the undamaged tower under service loads through increased unbraced column lengths in the absence of material nonlinearities. For the case with columns at room temperature, the tower was stable when two floors were removed. Two core columns buckled when three floors were removed, but the tower maintained its overall stability. Similarly, the tower maintained its stability when four columns buckled with four floors removed. This analysis suggested that global instability of the tower occurred when five floors were removed from the model. The case with columns at the region of removed floors at temperature of 600 ºC showed the tower maintained overall stability with one floor removed, with two floors removed and one buckled column, and with three floors removed and two buckled columns. This case produced tower instability with four floors removed from the model.

D.4.2 Results of Redistribution of Forces within the Core Analysis

The typical floor model was analyzed with 15 severed core columns (Core Damage Case 1) replaced with springs representative of the combined stiffness of the columns and hat truss. The analysis indicated that most of the load redistribution would take place initially through the hat truss, with the columns above the damaged zone carrying large tensile forces. A small portion of the load would be redistributed within the floor system. This is due to the greater stiffness of the hat truss-column assembly relative to the flexural stiffness of the floor system with fifteen severed columns.

When eight columns were assumed severed in the core (Core Damage Case 2), the floor system had a larger stiffness than that with fifteen columns severed (Core Damage Case 1). Consequently, the contribution of the floor in redistributing the gravity loads was larger, and the tensile forces in the columns above the damaged zone were reduced relative to the tensile forces for the case of fifteen severed core columns.

Above the impact levels, the core columns were typically two or three stories high, wide flange segments that were connected together with bolted splices. In the upper floors of WTC 1, where the column tensile capacities were analyzed, the columns were spliced at floors 98, 101, 104, and 106. Only the splices for columns that were assumed to be damaged were analyzed. The splice connections on these columns typically consisted of two splice plates, one bolted to each flange of the column, connected to the columns by eight or twelve, 3/4 in. (19 mm) A325 bolts. At floor 106, where the columns connected to the hat truss, 7/8 in. (22.2 mm) bolts were used. The splice plates were made with A36 steel, which was assumed to have an ultimate tensile capacity of 61 ksi (422 MPa) (see Table D–1). Since the connections were bearing connections with the bolts in single shear, the ultimate shear capacity of each 3/4 in. (19 mm) bolt was estimated to be 31.8 kip (219 MPa). When the strengths of both the splice plates and bolts were estimated, the splice plates were found to be consistently stronger than the bolts, so the columns would fail in tension through shearing of the bolts. Table D–5 lists the ultimate capacities of the splices on each of the columns assumed to be damaged in this analysis.

Table D–5. Ultimate tensile capacities of core column splices.

Column Number	Floor 98		Floor 101		Floor 104		Floor 106	
	kip	(kN)	kip	(kN)	kip	(kN)	kip	(kN)
503	519.6	(2,311)	381.6	(1,697)	381.6	(1,697)	346.4	(1,541)
504	381.6	(1,697)	381.6	(1,697)	254.5	(1,132)	346.4	(1,541)
505	381.6	(1,697)	381.6	(1,697)	254.5	(1,132)	346.4	(1,541)
506	519.6	(2,311)	381.6	(1,697)	381.6	(1,697)	346.4	(1,541)
603	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
604	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
605	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
606	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
703	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
704	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
705	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
706	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
803	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
804	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
805	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
903	381.6	(1,697)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)

A comparison of the columns' tensile forces with the capacities of column splice connections is presented in Table D–6 for Core Damage Case 1. The table indicates that, for the assumed service loads and damage pattern, the splice connections for interior core columns at the 700 and 800 lines are capable of resisting the tensile forces imposed on them. Splice connections on the 600 line are at the critical stage, but splices at the north perimeter core columns (500 line) are likely to fail. Failure of the column connections at the 500 line will require the floor to redistribute its loads to neighboring intact columns. This will result in overloading the columns at the 600 line, and consequently, the floor has to redistribute its loads through nonlinear, large deflection, tensile membrane action.

Table D–6. Tensile loads on columns above damaged area, and column load to ultimate capacity ratios for Core Damage Case 1.

Column Number	Floor 98		Floor 101		Floor 104		Floor 106	
	Column Load kip (kN)	Load to Capacity Ratio	Column Load kip (kN)	Load to Capacity Ratio	Column Load kip (kN)	Load to Capacity Ratio	Column Load kip (kN)	Load to Capacity Ratio
503	109.3 (486)	0.21	273.2 (1215)	0.72	437.1 (1944)	***1.15***	546.3 (2430)	***1.58***
504	82.8 (368)	0.22	207.0 (921)	0.54	331.2 (1473)	***1.30***	414.0 (1841)	***1.20***
505	92.7 (412)	0.24	231.8 (1031)	0.61	370.9 (1650)	***1.46***	463.6 (2062)	***1.34***
506	214.7 (955)	0.41	375.7 (1671)	0.98	536.7 (2387)	***1.41***	644.0 (2865)	***1.86***
603	64.6 (287)	0.25	161.5 (718)	0.63	258.4 (1149)	***1.02***	323.0 (1437)	0.93
604	57.5 (256)	0.23	143.8 (640)	0.57	230.1 (1024)	0.90	287.7 (1280)	0.83
605	72.3 (321)	0.28	180.7 (804)	0.71	289.1 (1286)	***1.14***	361.3 (1607)	***1.04***
606	138.7 (617)	0.55	242.8 (1080)	0.95	346.9 (1543)	***1.36***	416.2 (1851)	***1.20***
703	39.1 (174)	0.15	97.8 (435)	0.38	156.5 (696)	0.61	195.6 (870)	0.56
704	20.1 (90)	0.08	50.4 (224)	0.20	80.6 (358)	0.32	100.7 (448)	0.29
705	27.3 (121)	0.11	68.2 (303)	0.27	109.1 (485)	0.43	136.3 (606)	0.39
706	27.2 (121)	0.11	68.0 (303)	0.27	108.9 (484)	0.43	136.1 (605)	0.39
803	24.8 (110)	0.10	62.0 (276)	0.24	99.2 (441)	0.39	124.0 (552)	0.36
804	36.2 (161)	0.14	90.4 (402)	0.36	144.7 (644)	0.57	180.9 (805)	0.52
805	18.9 (84)	0.07	47.1 (210)	0.19	75.4 (335)	0.30	94.3 (419)	0.27

For the case where eight columns were assumed severed in the core (Core Damage Case 2), the results are presented in Table D–7. The Table indicates that the columns splice connections are capable of resisting the tensile loads except for column 505, where the load to capacity ratio is approximately 1.25 at floor 104 and 1.15 at floor 106. Successive removal of columns that were assumed to lose their splice connections indicated that the failure of connections would propagate to the 500 and 600 column lines in the core. Splice connections at columns 704, 705, and 903 should remain intact. Table D–7 indicates, however, that the load to capacity ratio at the splices did not exceed 1.25 for all cases considered. These values might not be conclusive to determine connection failure or survival due to the uncertainties in the loads on the floors and the capacities of the splice connections.

D.4.3 Results of Nonlinear Analysis

Two cases were considered for this analysis based on the results presented in Section D.4.2. The first assumed that core column splices above the eight severed columns (Core Damage Case 2) had failed, and the load was being distributed through the floor system to neighboring columns (Case A). The second case assumed that splices were intact, and the load was being transmitted to the hat truss via tensile forces in the columns (Case B). For both cases, the results of the nonlinear analysis show that WTC 1 had significant reserve structural capacity after aircraft impact. Moreover, the loads and deformations in critical members varied little between the two cases. The results are described in detail for Case A only.

Table D–7. Tensile loads on columns above damaged area, and L/C ratios indicating likely progression of splice failures for Core Damage Case 2.

Column Number	Floor 98		Floor 101		Floor 104		Floor 106	
	Column Load kip (kN)	L/C Ratio	Column Load kip (kN)	L/C Ratio	Column Load kip (kN)	L/C Ratio	Column Load kip (kN)	L/C Ratio
Loads with all column splices intact								
504	34.7 (154)	0.09	138.9 (618)	0.36	243.1 (1081)	0.96	312.5 (1390)	0.90
505	79.5 (354)	0.21	198.8 (884)	0.52	***318.1 (1415)***	***1.25***	***397.6 (1769)***	***1.15***
604	21.3 (95)	0.08	85.2 (379)	0.33	149.1 (663)	0.59	191.7 (853)	0.55
605	77.0 (343)	0.30	154.0 (685)	0.61	231.0 (1028)	0.91	282.4 (1256)	0.82
703	45.0 (200)	0.18	90.0 (400)	0.35	134.9 (600)	0.53	164.9 (734)	0.48
704	3.6 (16)	0.01	14.3 (64)	0.06	25.1 (112)	0.10	32.2 (143)	0.09
706	21.0 (93)	0.08	36.7 (163)	0.14	52.4 (233)	0.21	62.9 (280)	0.18
903	36.2 (161)	0.09	90.6 (403)	0.36	145.0 (645)	0.57	181.2 (806)	0.52
Loads with splices at columns 504 and 505 failed								
504	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00
505	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00
604	29.0 (129)	0.11	115.9 (515)	0.46	202.8 (902)	0.80	260.7 (1160)	0.75
605	103.6 (461)	0.41	207.3 (922)	0.81	***310.9 (1383)***	***1.22***	***380.0 (1690)***	***1.10***
703	43.1 (192)	0.17	86.2 (383)	0.34	129.3 (575)	0.51	158.0 (703)	0.46
704	3.0 (13)	0.01	12.0 (53)	0.05	20.9 (93)	0.08	26.9 (120)	0.08
706	20.1 (89)	0.08	35.1 (156)	0.14	50.2 (223)	0.20	60.2 (268)	0.17
903	36.2 (161)	0.09	90.5 (403)	0.36	144.8 (644)	0.57	181.0 (805)	0.52
Loads with splices at columns 504, 505, and 605 failed								
504	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00
505	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00
604	42.8 (191)	0.17	171.4 (762)	0.67	***299.9 (1334)***	***1.18***	***385.6 (1715)***	***1.11***
605	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00
703	45.8 (204)	0.18	91.5 (407)	0.36	137.3 (611)	0.54	167.8 (746)	0.48
704	3.5 (16)	0.01	14.0 (62)	0.05	24.5 (109)	0.10	31.5 (140)	0.09
706	22.6 (101)	0.09	39.6 (176)	0.16	56.5 (251)	0.22	67.8 (302)	0.20
903	36.2 (161)	0.09	90.5 (403)	0.36	144.8 (644)	0.57	181.0 (805)	0.52
Loads with splices at columns 504, 505, 604, and 605 failed								
504	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00
505	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00
604	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00
605	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00
703	57.6 (256)	0.23	115.2 (513)	0.45	172.9 (769)	0.68	211.3 (940)	0.61
704	5.1 (23)	0.02	20.6 (91)	0.08	36.0 (160)	0.14	46.3 (206)	0.13
706	25.5 (113)	0.10	44.6 (198)	0.18	63.7 (284)	0.25	76.5 (340)	0.22
903	36.1 (161)	0.09	90.3 (402)	0.35	144.4 (642)	0.57	180.5 (803)	0.52

The analyses show that the most stressed members were the columns next to the damaged area on the north wall of the tower. The analyses show that the tower also remained standing after losing columns in the south wall due to fire effects with some reserve capacity left. This indicates that additional loss or weakening of columns in the core, weakening of additional columns in the exterior, or additional loss of floors is needed to collapse the tower.

Figures D–4 and D–5 show the load deformation curves for columns 111 and 145 (see Fig. D–3) on either side of the damage on the north face of the tower. Both of these columns yielded during the impact damage analysis, but had sufficient strength and ductility to resist the peak loads. Note that the loads on these two columns were slightly reduced during the fire damage analysis. This occurred because the south face of the tower lost stiffness when members were lost to the fire, which caused the upper portion of the tower to rotate slightly toward the south. This redistributed a small portion of the load on the north face through the hat trusses to the core. The columns at the edge of the damage on the south face experienced the opposite effect. As can be seen in Fig. D–6, column 332 on the west side of the damage on the south face (see Fig. D–3) did not have a significant change in its load during the impact damage stage, but received a large additional load during the fire damage analysis. This column remained within the linear response range throughout the analysis.

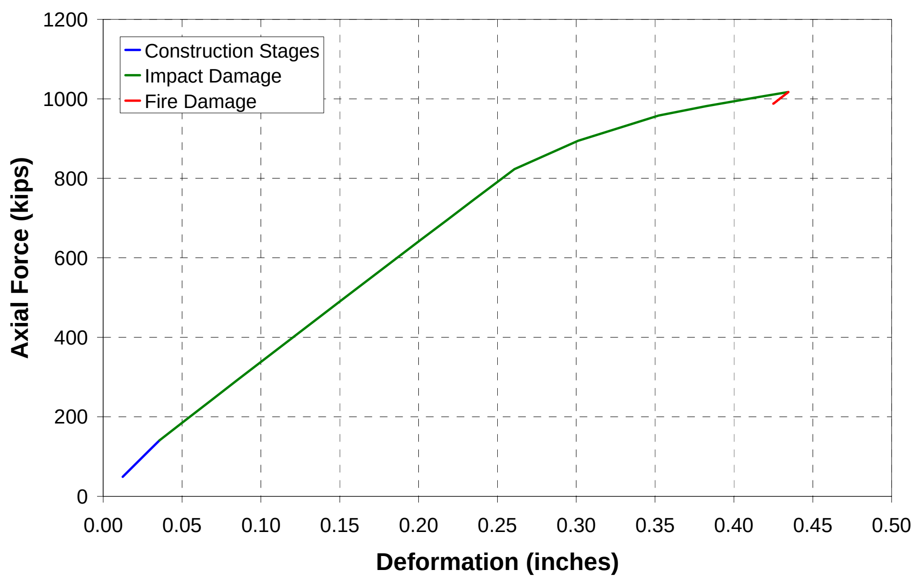


Figure D–4. Load vs. deformation in column 111 at floor 98 (north face, west side of damage).

Table D–8 shows the distribution of axial loads in columns at floor 99, immediately above the damaged zone for the various loading stages. For the floors below the hat trusses (construction stage 1) about 57 percent of the dead load was carried by the core columns, with the rest distributed among the four exterior walls. The mechanical floors and roof tended to have a large percentage of their loads carried by the exterior, so at the end of the final construction stage, the load was nearly evenly distributed between the core and exterior. Significant load redistribution occurred during the damage cases; however most of the redistribution was from the north and south walls to the east and west walls. Only 1.7 percent of the total load was redistributed from the exterior to the core.

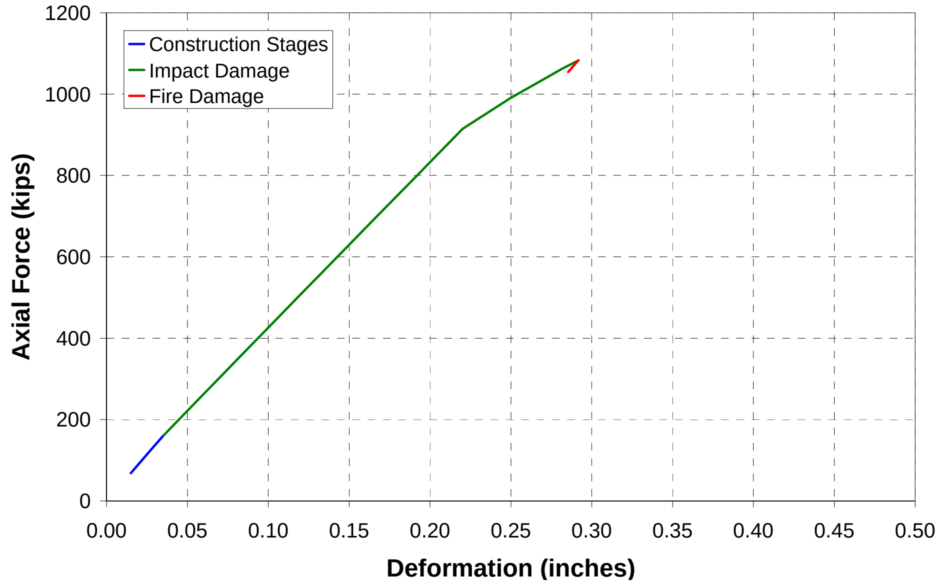


Figure D–5. Load vs. deformation in column 145 at floor 95 (north face, east side of damage).

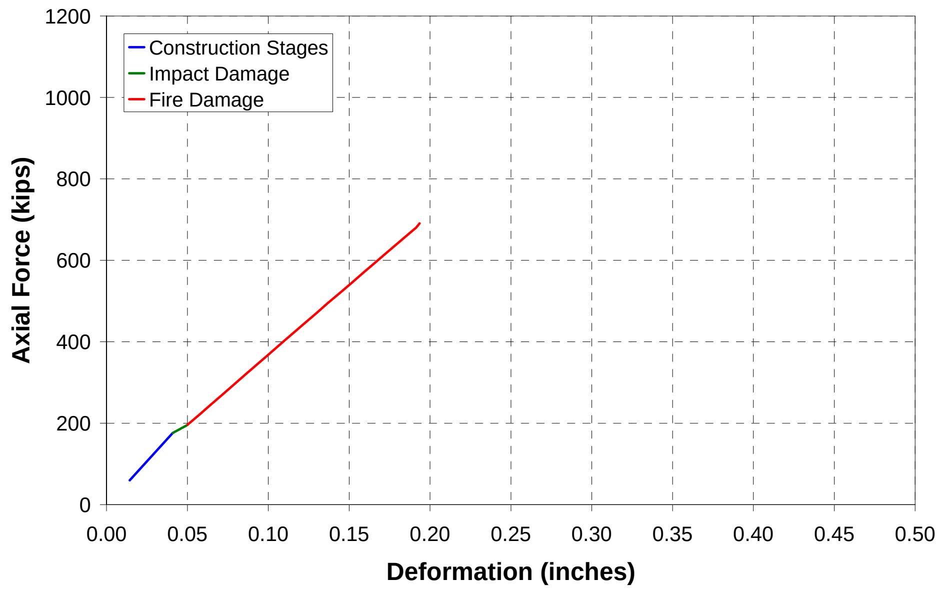


Figure D–6. Load vs. deformation in column 332 at floor 97 (south face, west side of damage).

Table D–8. Distribution of loads on exterior walls and core columns at floor 99 for Core Damage Case 2.

Case	Const. Stage 1 (kip)	Const. Stage 1 (%)	Const. Stage 2 (kip)	Const. Stage 2 (%)	Const. Stage 3 (kip)	Const. Stage 3 (%)	Impact Damage (kip)	Impact Damage (%)	Fire Damage (kip)	Fire Damage (%)
Total axial force	27,719		45,694		54,639		54,641		54,641	
Force Distribution between Core and Exterior										
Core columns	15,828	57.1	24,466	53.5	27,397	50.1	27,791	50.9	28,318	51.8
Exterior columns	11,891	42.9	21,228	46.5	27,242	49.9	26,850	49.1	26,323	48.2
Force Distribution between Exterior Faces										
100 face	3,562	12.9	6,104	13.4	7,610	13.9	6,732	12.3	6,519	11.9
200 face	2,389	8.6	4,551	10.0	6,084	11.1	6,539	12.0	6,765	12.4
300 face	3,562	12.9	6,064	13.3	7,548	13.8	7,000	12.8	6,445	11.8
400 face	2,378	8.6	4,509	9.9	6,000	11.0	6,579	12.0	6,594	12.1

Even with the loss of 34 columns on the north face and 24 columns on the south face, relatively few structural members were overstressed. As listed in Table D–9, plastic hinges had formed in only nine members, and only three members had more than a small amount of plastic deformation. Of the members with plastic hinges, six were exterior columns, and three were exterior spandrels. None of the core columns had hinges form. Three beams in the hat truss at floor 107 experienced some yielding. These members were all light, short connecting members, and the yielding was most likely due to modeling idealization rather than overloading that would have occurred in the real structure. The hinges in the columns and spandrels all formed near the sides of the openings created by the aircraft impact and at the edges of the region that appeared to fail inward due to the fires. Figure D–7 shows the displacements and plastic hinges that occurred in the north and south faces of the tower during the impact damage analysis stage, while Fig. D–8 shows the displacements and plastic hinges from the fire damage analysis stage.

Table D–9. Hinge states of members where plastic hinges formed during nonlinear analysis.

Columns with Plastic Hinges				
Column Line	**Lower Floor**	**Upper Floor**	**Impact Damage**	**Fire Damage**
111	98	99	Some strain hardening	Some strain hardening
111	99	99 mid floor	Yielded	Yielded
111	99 mid floor	100	Some strain hardening	Some strain hardening
145	94	95	Yielded	Yielded
145	95	95 mid floor	Yielded	Yielded
145	95 mid floor	96	Some strain hardening	Some strain hardening
Spandrels with Plastic Hinges				
Floor Level	**Start Column**	**End Column**	**Impact Damage**	**Fire Damage**
99	110	111	Yielded	Yielded
96	144	145	Yielded	Yielded
98	331	332	-	Yielded

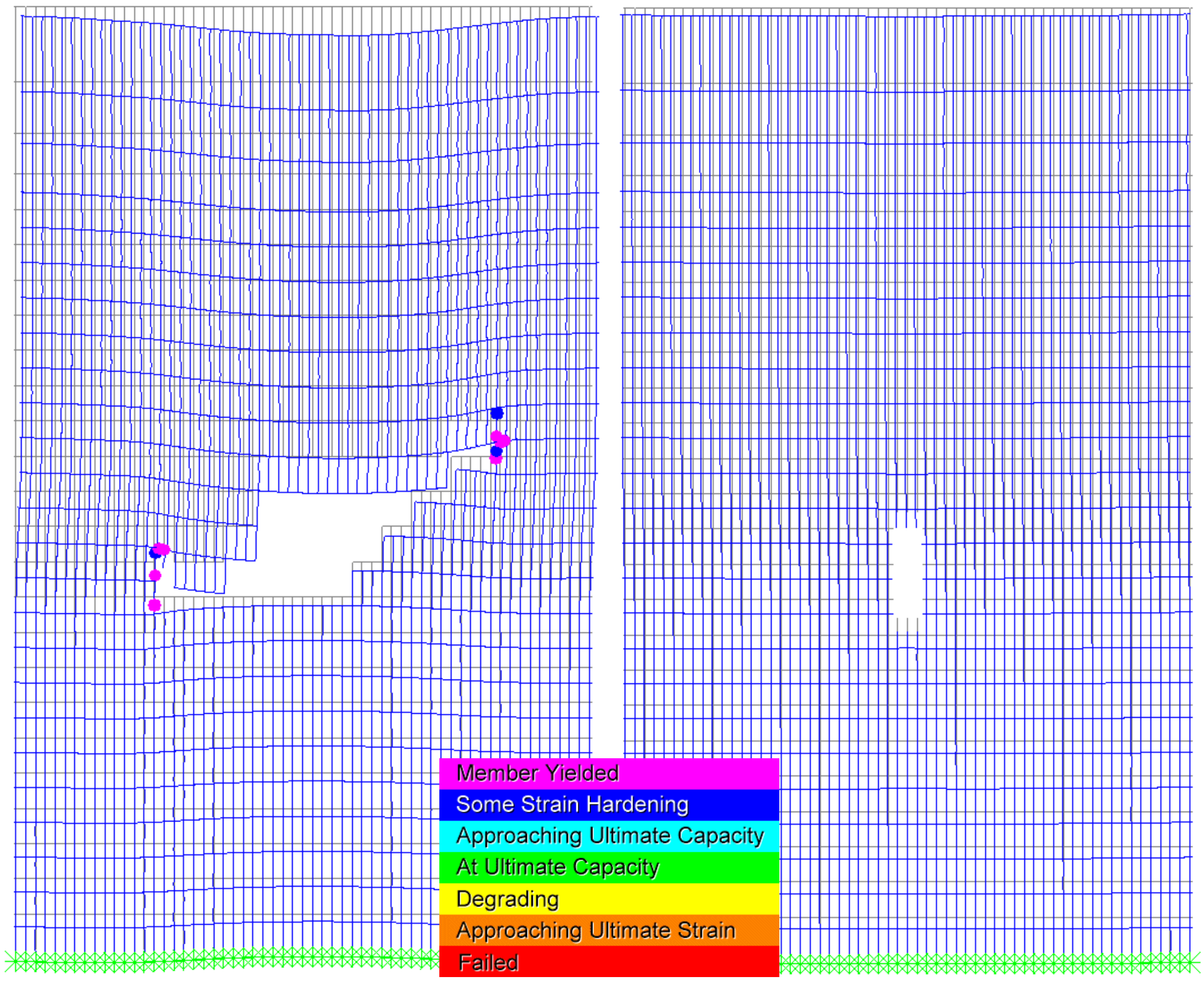
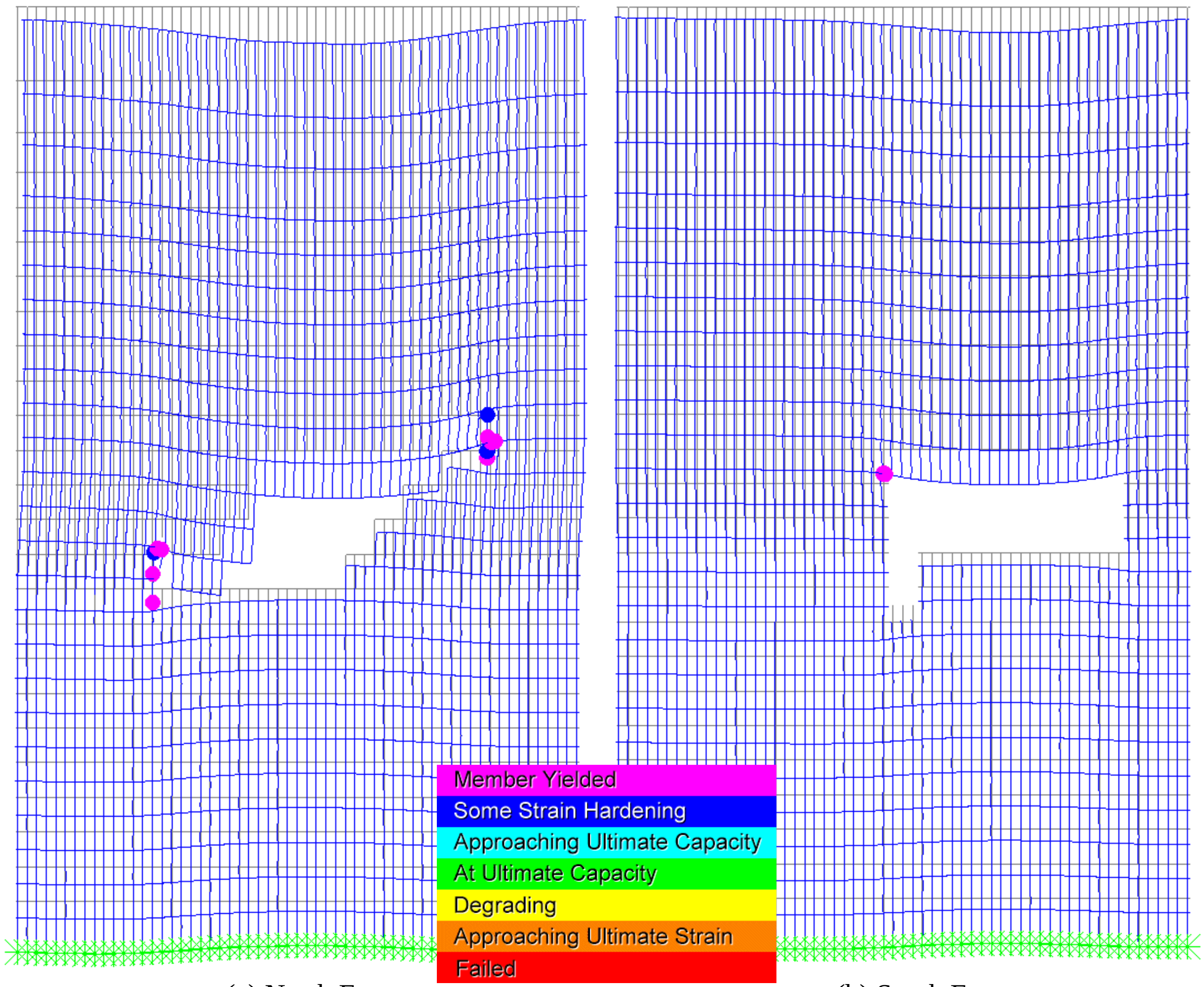


Figure D–7. Displacements and locations of plastic hinges in the north and south exterior walls of WTC 1 after impact.

D.5 SUMMARY AND PRELIMINARY FINDINGS

Preliminary system stability analyses of the WTC towers have been performed to: (1) examine the overall stability of the undamaged tower upon removal of floors, (2) study possible load redistribution mechanisms upon losing columns in the core due to aircraft impact, and (3) study the response when columns in both the exterior walls and the core are assumed destroyed due to aircraft impact, and columns in the exterior are damaged due to the subsequent fires, as observed in photographs and videos of WTC 1.

The analyses used the typical truss-framed floor model and a reduced version of the global reference model of WTC 1 with proper modifications. Modifications included adding vertical springs at the bottom of the reduced models to account for the removed lower portion of the towers, and using actual (vs specified) steel properties and service loads on the towers. The analyses used the staged construction technique to account for the sequential construction of the towers, especially in the zone of the hat trusses.



(a) North Face (b) South Face

Figure D–8. Displacements and locations of plastic hinges in the north and south exterior walls of WTC 1 after impact and fire.

Linear buckling analysis and nonlinear analysis with plastic hinges were used to study the effects of removal of floors and loss of exterior and core columns, respectively. In addition, analysis of the floor system, where severed core columns were replaced by equivalent springs representative of the combined stiffness of the hat trusses and columns between the floors and hat trusses, was conducted to study the mechanism by which the floor loads were redistributed when the core columns were destroyed by aircraft impact.

The following presents some preliminary findings based on the analyses under service loading conditions:

- Linear stability analysis was used to examine the stability of the undamaged WTC 1 under service loads through increased unbraced column lengths (floor removal). The tower was stable when two floors were removed. Two core columns buckled when three floors were removed, but the tower maintained its overall stability. The tower also maintained its stability when four columns buckled with four floors removed. The analysis suggested that global instability of the tower occurred when five floors were removed from the model. Assuming that all columns at the region of removed floors reached a temperature of 600 ºC (reduced modulus of elasticity), the analysis indicated that removal of four floors would induce global instability.

- Analysis of the typical truss-framed floor model with fifteen core columns assumed severed indicated that, under service loads, the floors first attempted to redistribute their loads to the hat trusses through tension in the columns above the damage. The load followed this path due to the relatively large stiffness of the hat trusses-column system compared to the flexural stiffness of the floors. This resulted, however, in the ultimate tensile capacity of some column splices below the hat trusses to be exceeded, and ultimately, the floors would have redistributed their loads directly to neighboring core columns. When only eight core columns were assumed severed, the analysis indicated that the tensile forces in the columns were smaller, due to the relatively larger stiffness of the floor. These forces may still have failed the columns at the splices. Since the load to capacity ratio at the splices did not exceed 1.25 when eight columns were severed, and due to the uncertainties in the loads on the floors and the capacities of the splice connections, the results are not conclusive as to whether splice failure would occur or not.

- Nonlinear analysis that included geometric nonlinearities and material nonlinearities using plastic hinges was conducted on the reduced global model of WTC 1. The model assumed the following damage to the tower: (1) due to aircraft impact, loss of columns and spandrels in the north face, and an exterior panel in the south face of the tower (both based on photographic evidence), as well as eight columns in the core; and (2) due to fire, loss of columns in the south face, which were shown in videos to be bowing inward a few minutes prior to collapse. The analysis indicated that after aircraft impact, the tower maintained its stability, where the highest stressed elements were the exterior columns next to the damaged area on the north face of the tower. The tower also maintained its stability after losing columns in the south wall due to fire effects with some reserve capacity left, indicating that additional loss or weakening of columns in the core, weakening of additional columns in the exterior, or additional loss of floors is needed to collapse the tower. More detailed models will account for local bucking of columns, and the failure and role of the floor system in redistributing the loads, factors that are not considered in this analysis.

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Appendix E
INTERIM REPORT ON CONTEMPORANEOUS STRUCTURAL STEEL SPECIFICATIONS

The purpose of Project 3, Mechanical and Metallurgical Analysis of Structural Steel, of the National Institute of Standards and Technology (NIST) World Trade Center (WTC) Investigation is to analyze structural steel available from WTC 1, 2, and 7 for determining the metallurgical and mechanical properties and quality of the metal, weldments, and connections, and providing these data to other investigation projects. (For test plan details, see <http://wtc.nist.gov/media/WTCplan_new.htm#proj3>.) The properties determined under this project will be used in two ways:

1. Properties will be correlated with the design requirements of the buildings to determine if the specified steel was in place in the towers.

2. Properties will be supplied for other projects in the Investigation as input for models of building performance.

E.1 SCOPE OF REPORT

This appendix describes the WTC tower structure and critical structural elements to be characterized in Project 3. This includes the structural design and properties specified by the structural engineers for columns, floor systems, and connections.

Contemporaneous (late 1960s era) specifications are described for various types and grades of steel designated by the ASTM International, the American Institute of Steel Construction (AISC), and other national and international organizations. It also includes information from numerous suppliers of the steel for the structure. The structural steel for the towers was supplied through at least a dozen contracts to suppliers and fabricators. Substantial understanding of the consistency, quality, and actual strength of the steel (as opposed to specified minimum values) can be gained if the production practices and quality control procedures used by the various steel suppliers are understood. Practices and data from the numerous WTC steel suppliers have been investigated and are reported for both structural steel and construction practices. In addition, this information has been used to estimate typical mechanical property values for the many of the grades of steel. These typical values can serve as a guide for the properties to be inserted into the finite element models of building performance and as a point of comparison for actual properties measured on the recovered steel.

The appendix also includes a review of the standards and specifications used in welding the built-up columns, and those used in the erection of the towers.

E.2 UNITS AND ABBREVIATIONS

Yield strengths of the steels and the dimensions of the building are expressed in English units with metric (SI) equivalents. The steels were specified to English unit-based ASTM standards, and the building was built to foot and inch dimensions. ASTM standards differentiate between English and metric units by denoting them with completely different designations and frequently by publishing them as separate documents. This appendix uses English units for values that were contractually specified during the construction (primarily component dimensions and steel strengths). Table E–1 shows the SI equivalents of the common yield strength grades of steel.

Table E–1. Metric equivalents of common yield strengths.

ksi	MPa
36	248
42	290
45	310
46	317
50	345
55	379
60	414
65	448
70	483
75	517
80	552
85	586
90	621
100	689

In reviewing some of the historical documents, NIST found ambiguities in the use of the measure "ton." NIST has assumed that in any source originating in the United States, a "ton" refers to 2,000 lb (i.e., a short ton). For sources originating in Japan, NIST assumes that a "ton" refers to 1,000 kg (= 2,204.6 lb, i.e., a metric ton). For any source originating in Great Britain, NIST assumes that a "ton" is 2,240 lb (a "long" or U.K. ton) and that a "tonne" is 1,000 kg. In this appendix, all weights in tons are converted to short tons (= 2,000 lb).

This appendix follows the American Iron and Steel Institute (AISI) convention and denotes yield strength with the symbol F_y. The ASTM International uses the symbols YS (or YP) and S_y.

E.3 SOURCES OF INFORMATION

This appendix is based on three different types of sources. Open literature sources like journal and trade magazine articles, books, historical standards, and publicly searchable databases comprise the first type. The second type comprises personal interviews by NIST investigators with individuals and company representatives, and information they provided voluntarily. Sources of information where NIST has entered into material release agreements with organizations or individuals comprise the third type. Documents provided by Leslie E. Robertson Associates (LERA), which is the source of most of the contemporaneous information on the construction of the buildings, is an example of the third type. This archive has been useful in identifying the specific steels and standards used in the construction. Although it is voluminous, the LERA archive does not include every document generated during the construction of the towers. Section E.5.4 summarizes the search strategies for open literature information and provides details on the companies and individuals contacted and the information they provided

This report identifies the type of source in the reference. For example, a reference to a book or other publicly available document appears as (Smith 1968). The symbol † denotes a personal communication to a NIST investigator, for example (Jones 2003 †). In the case of a source bound by a Material Transfer Agreement, the symbol § appears in the reference, for example (Monti 1969 §). The reference lists appear as Section E.7.

E.4 TOWER DESIGN – STRUCTURAL STEEL DOCUMENTS

E.4.1 Specification of Steel Grades (Minimum Yield Strength)

Specifications (ASTM, AISI, etc.) typically place limits on chemical composition or mechanical properties or, most commonly, both. Various mechanical properties may be specified, such as tensile strength, minimum yield strength, ductility, and toughness. Other material properties may not appear in a specification, yet are critical in building design; the most important such property is perhaps the elastic modulus, or stiffness, which does not appear in specifications because there is little variability among the various steels.

In practice, the material property of greatest importance for characterizing a particular steel is the yield strength (F_y). In the United States, steel is often referred to according to its yield strength; for example, a "50 ksi steel" is steel with a minimum yield strength of 50,000 lb/in^2. Skilling, Helle, Christiansen, & Robertson (SHCR), structural engineers for the WTC towers, followed this convention, and the structural engineering plans are marked with the minimum yield strength for each piece of structural steel.

E.4.2 Structural Overview

The WTC tower buildings had a frame-tube construction consisting of closely spaced perimeter columns coupled to a rectangular service core (Fig. E–1). The buildings had a square footprint, 207 ft 2 in.

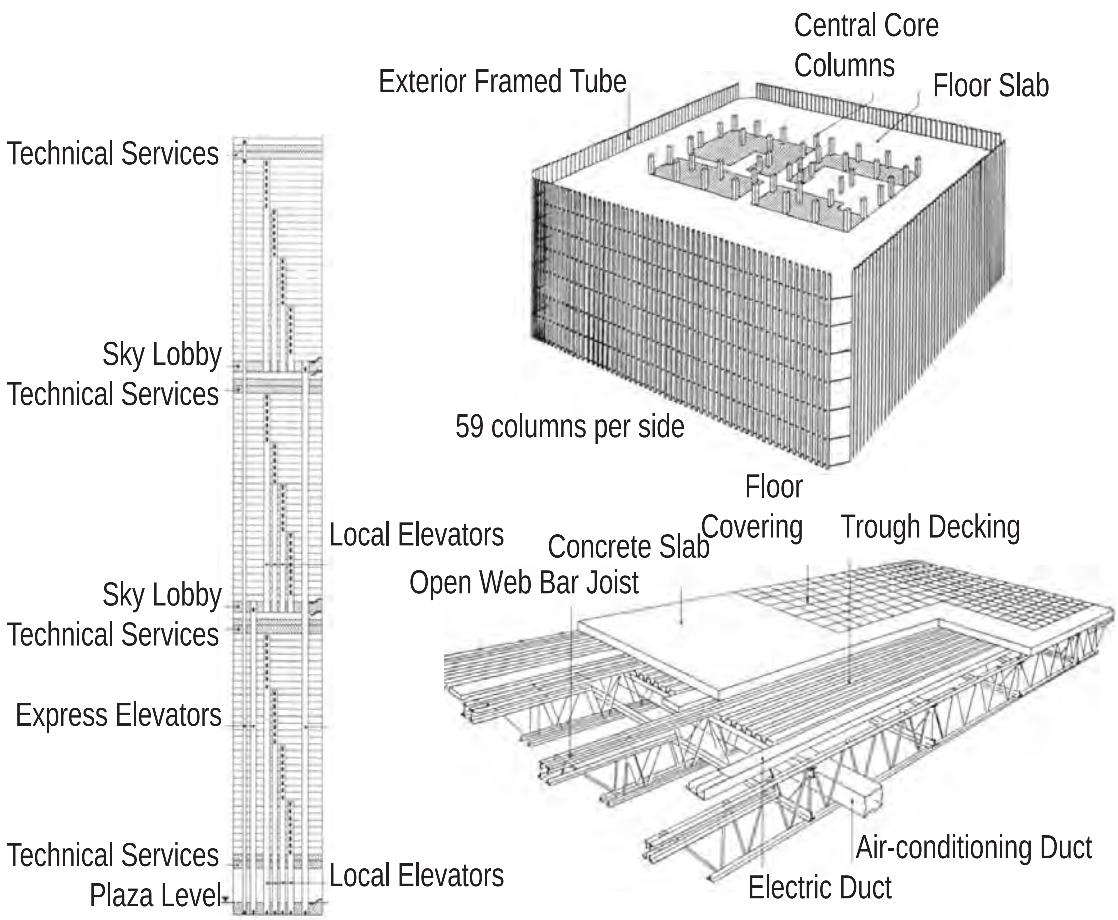
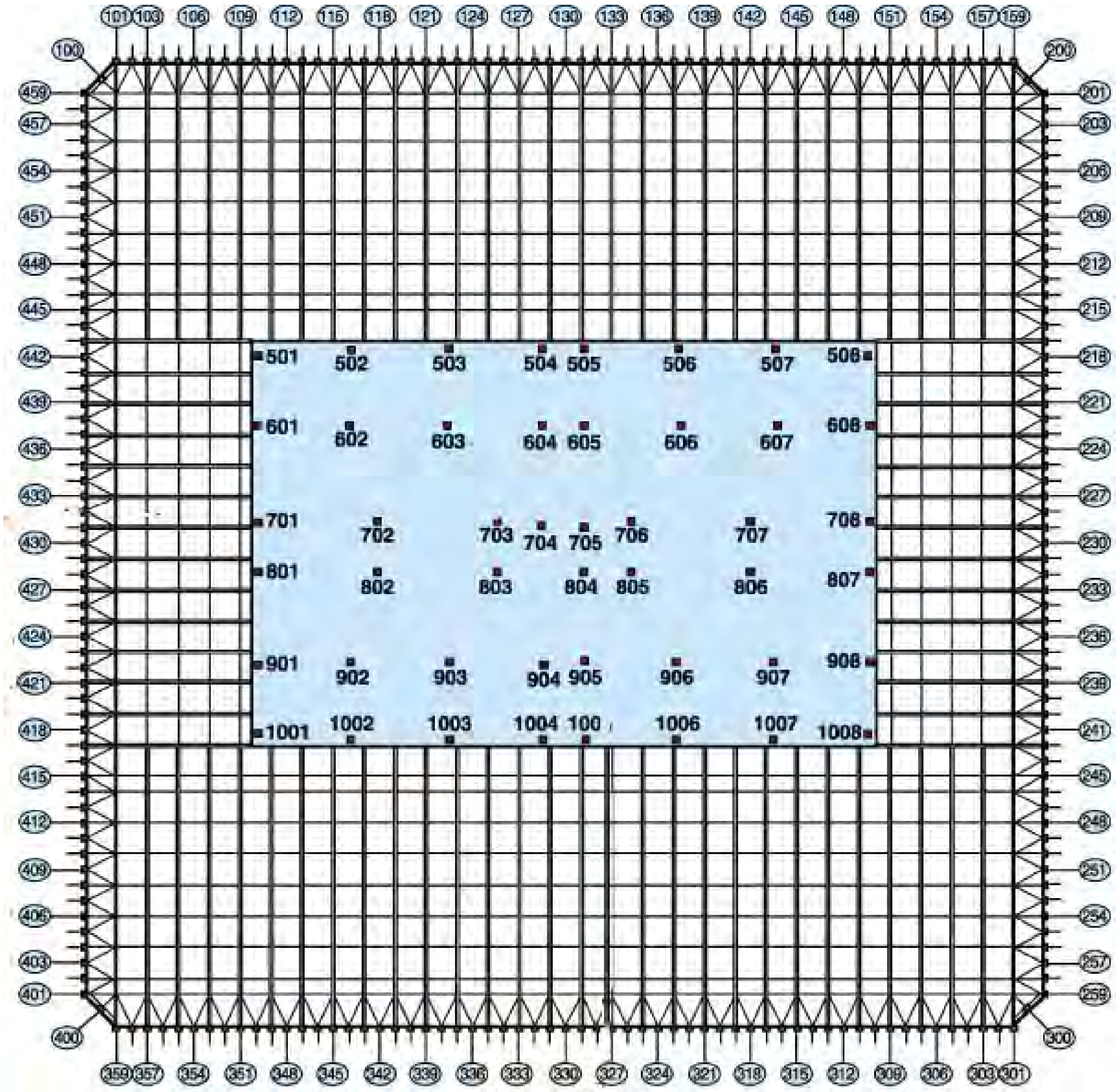


Figure E–1. Schematic diagram of the tower structure.

(63.14 m) on a side with chamfered corners. From floor 9 to floor 107, the perimeter columns consisted of closely spaced built-up box columns. The service core at the building center was approximately 87 ft by 137 ft (26.5 m by 41.8 m) and connected to the perimeter columns by a floor panel system that provided an essentially column-free office space, see Fig. E–2. In addition to showing the location of perimeter and core columns, Fig. E–2 describes the column numbering scheme used to identify each column on a given floor.

The WTC tower structural steel plans (SHCR 1967 §) point out the major structural elements of interest. The main features of structural interest are the perimeter columns, the core columns and associated framing, the trusses that supported the floors, and the connections between and within these elements. In addition, a hat truss located within floors 107 to 110 tied the core to the perimeter columns and provided a base for the television mast atop WTC 1 and support for a proposed mast atop WTC 2.



Source: McAllister 2002.

Figure E–2. WTC tower floor plan and column numbers.

The structural engineering plans provide the location, cross-sections, and grade of steel (i.e., required minimum yield strength) for each of the thousands of structural elements in the buildings. In all, 14 different grades of steel were specified, ranging in yield strength from 36 ksi to 100 ksi. In addition to yield strength requirements, Port of New York Authority (PONYA) documents provided by LERA specified allowable steels using ASTM or other standards (details in Section E.5 in this report). Requirements for bolts and welds are also given.

Perimeter Columns and Spandrels

Between floors 9 and 107, the perimeter structure consisted of closely spaced, built-up box columns. Each building face consisted of 59 columns spaced at 40 in. (1.02 m). The columns were fabricated by welding plates of steel to form an approximately 14 in. (0.36 m) square section (Fig. E–3). Adjacent columns were interconnected at each floor level by deep spandrel plates, typically 52 in. (1.32 m) deep (Fig. E–4).

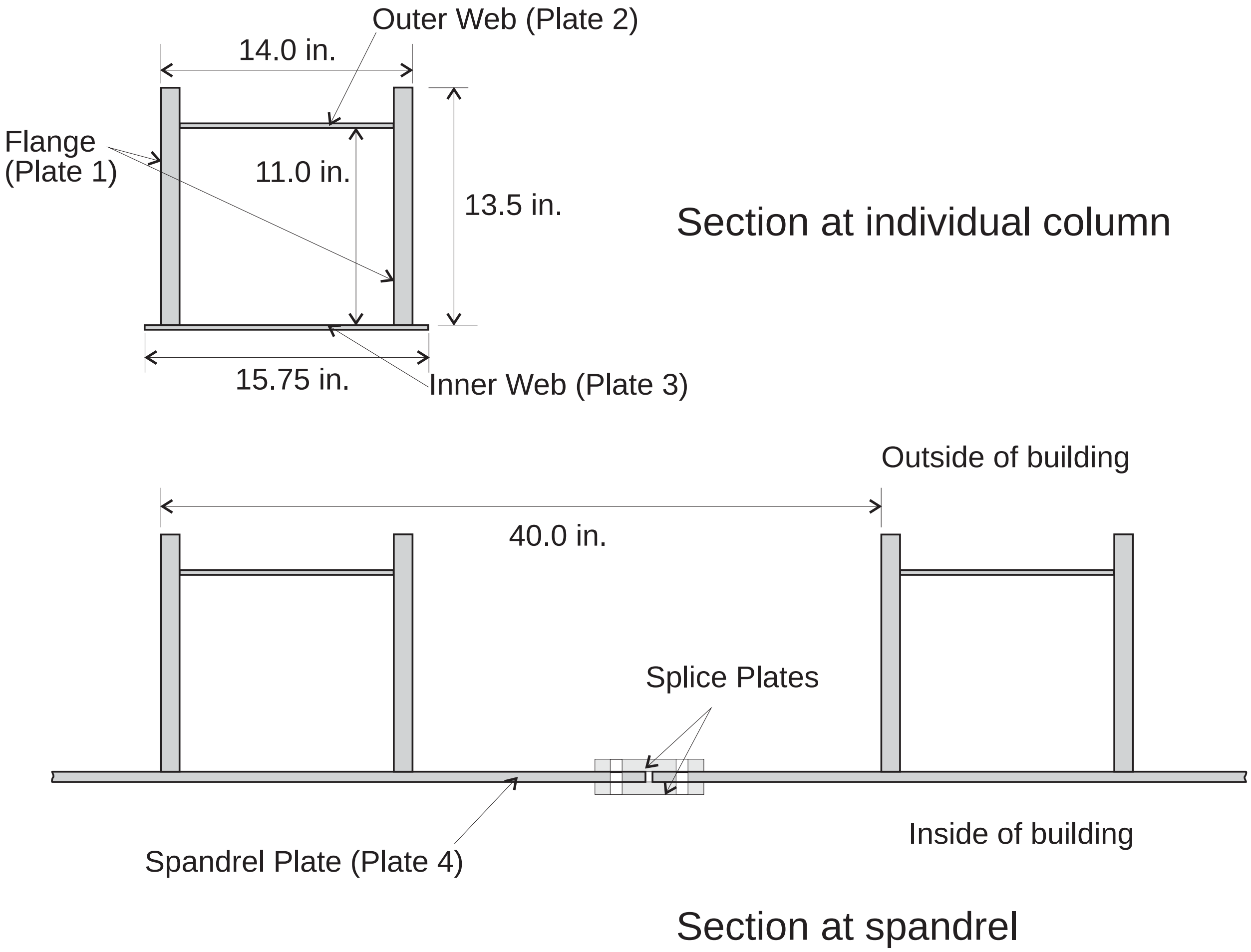


Figure E–3. Cross-section of perimeter columns; sections with and without spandrels.

The perimeter columns were prefabricated into panels, typically three stories tall and three columns wide (Fig. E–4). Heavy end, or "butt" plates with F_y = 50 ksi and 1.375 in. to 3 in. (3.5 mm to 7.6 mm) thick were welded to the top and bottom of each column. Fillet welds were used inside the columns along three edges, with a groove weld on the fourth, outside edge. During erection, abutting spandrels were bolted together, and columns were bolted to the adjacent columns, all using ASTM A 325 bolts except for the heaviest butt plates, which used ASTM A 490 bolts. Other than at the mechanical floors, panels were staggered (Fig. E–5) so that only one third of the units were spliced (i.e., connected) in any one story. At the mechanical floors (75 and 76 in the upper level of the buildings), however, every perimeter column was spliced at the same level, floors 74 and 77. These splices were both welded and bolted.

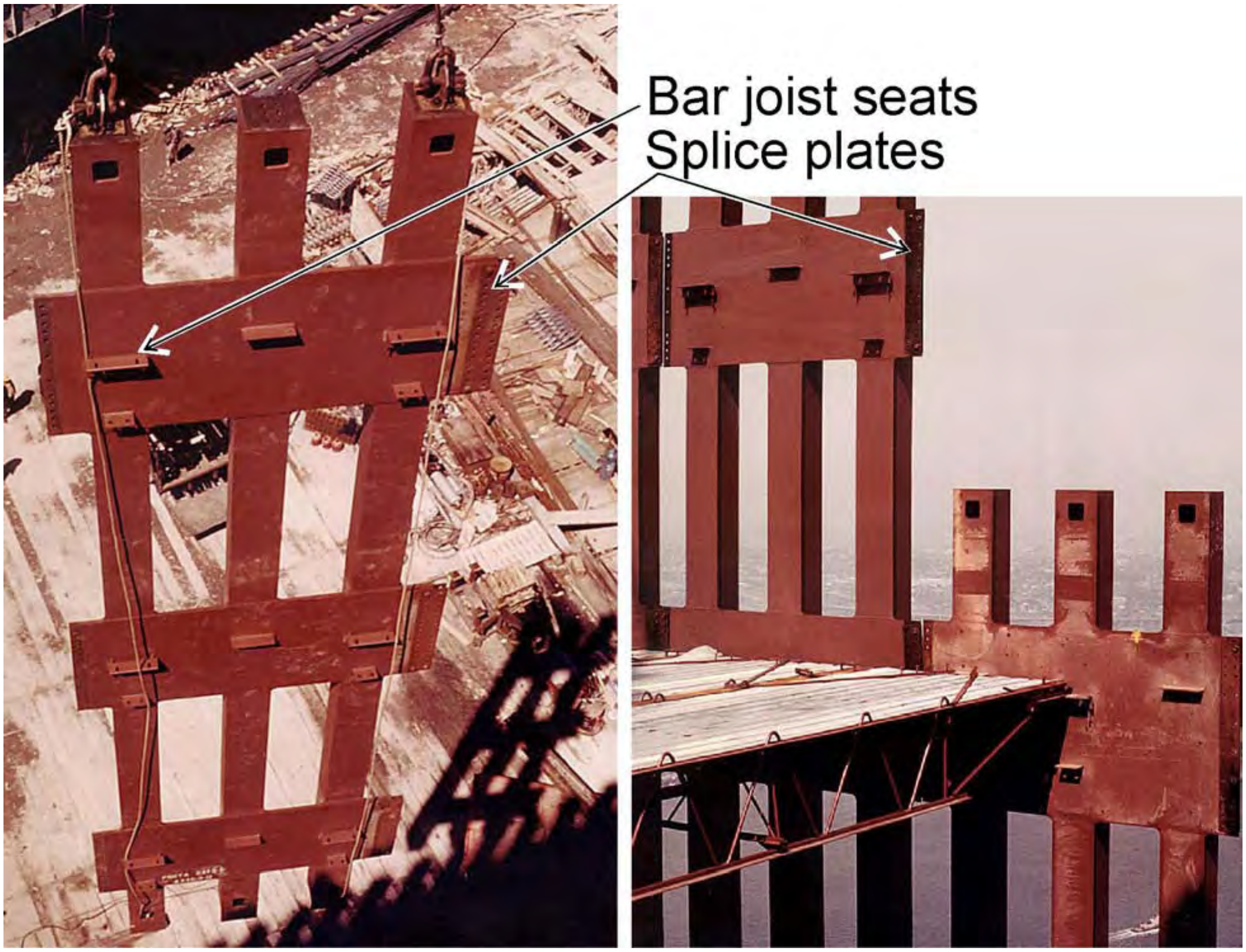


Figure E–4. Characteristic perimeter column panel consisting of three full columns connected by three spandrels.

Fourteen grades of steel were specified in the design documents for the perimeter columns, with minimum yield strengths of (36, 42, 45, 46, 50, 55, 60, 65, 70, 75, 80, 85, 90, and 100) ksi. Twelve grades of steel were specified for the spandrels, with the same strength levels as the columns but without the two highest strength steels. The structural engineering plans indicate that the flanges and webs of a given column section consist of a single grade (i.e., minimum yield strength) of steel, but each column and spandrel within a single prefabricated panel could be fabricated from different grades of steel.

Columns in the upper stories were typically fabricated of lighter gage steel, as thin as 0.25 in. (6.35 mm), with the grade of steel dictated by the calculated gravity and wind loads. In this manner the gravity load on the lower stories was minimized. In the lower stories the perimeter column flanges were often more than 2 in. (51 mm) thick.

The spandrels formed an integral part of the columns: there was no inner web plate at spandrel locations. (Fig. E–3). Spandrels were generally specified with a yield strength lower than that of the column webs and flanges, as well as a heavier gage than the adjacent inner webs.

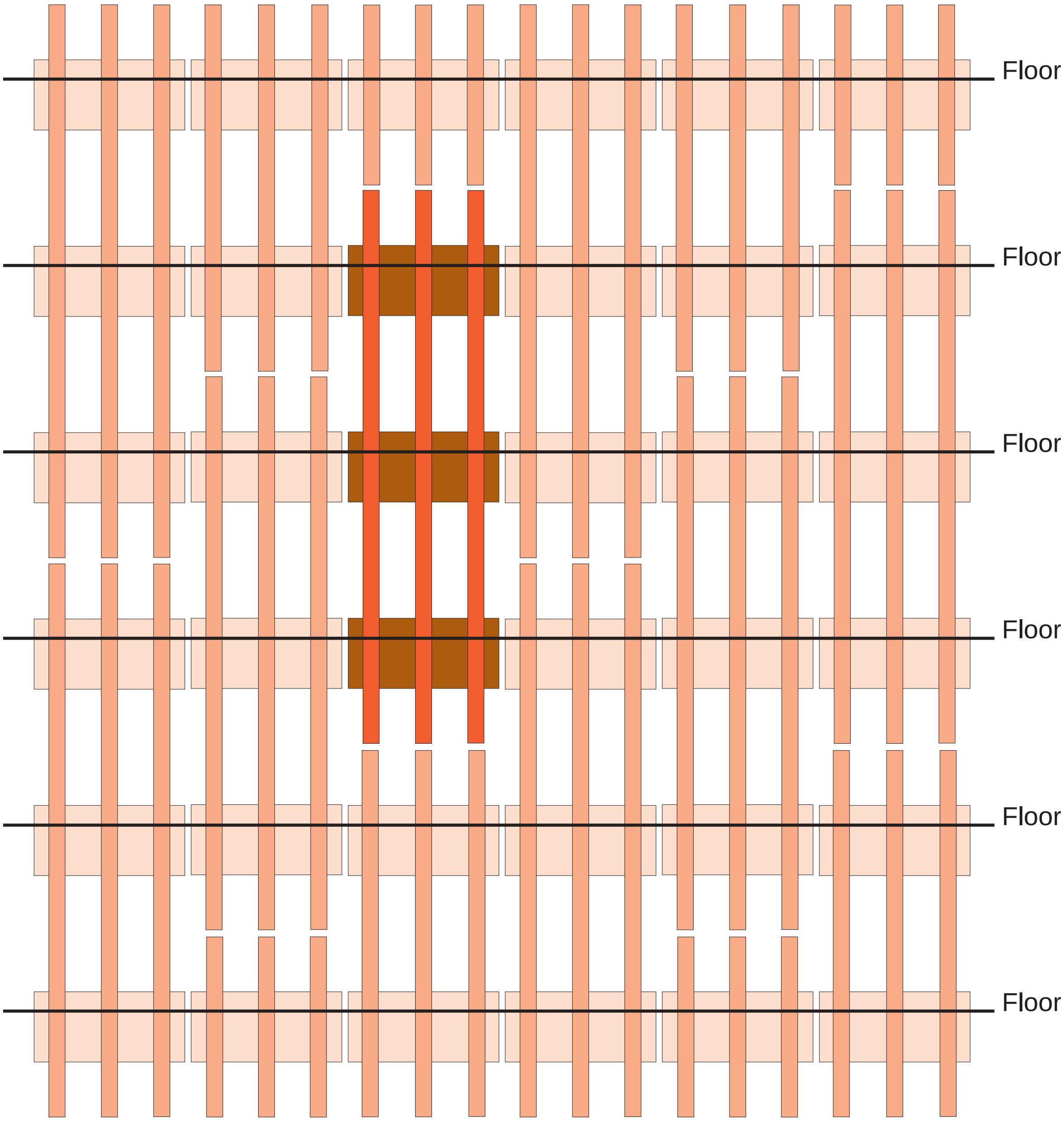


Figure E–5. Partial elevation of exterior bearing-wall frame showing perimeter column panel construction. Highlighted panel is three stories tall (36 ft) and spans four floors. Distance between panels has been exaggerated.

Core Columns

Core columns were of two types: welded box columns and rolled wide flange (WF) shapes (Fig. E–6). The columns in the lower floors were primarily very large box columns as large as 12 in. by 52 in. (0.30 m by 1.32 m) composed of welded plates up to 7 in. (178 mm) thick. In the upper floors the columns shifted to the rolled WF shapes. The transition floors are indicated in Fig. E–7 for each of the core columns. Core columns were typically spliced at three-story intervals. The splices in the impact and fire zones were at floors 75, 77, 80, 83, 86, 89, 92, 95, 98, and 101. Diagonal bracing was used at the mechanical floors and in the area of the hat truss. Core box columns were 36 ksi or 42 ksi. Core wide flange columns were specified to be one of four grades, but were primarily 36 ksi and 42 ksi steel; only about 1 percent of all the core columns were made of 45 ksi or 50 ksi steel.

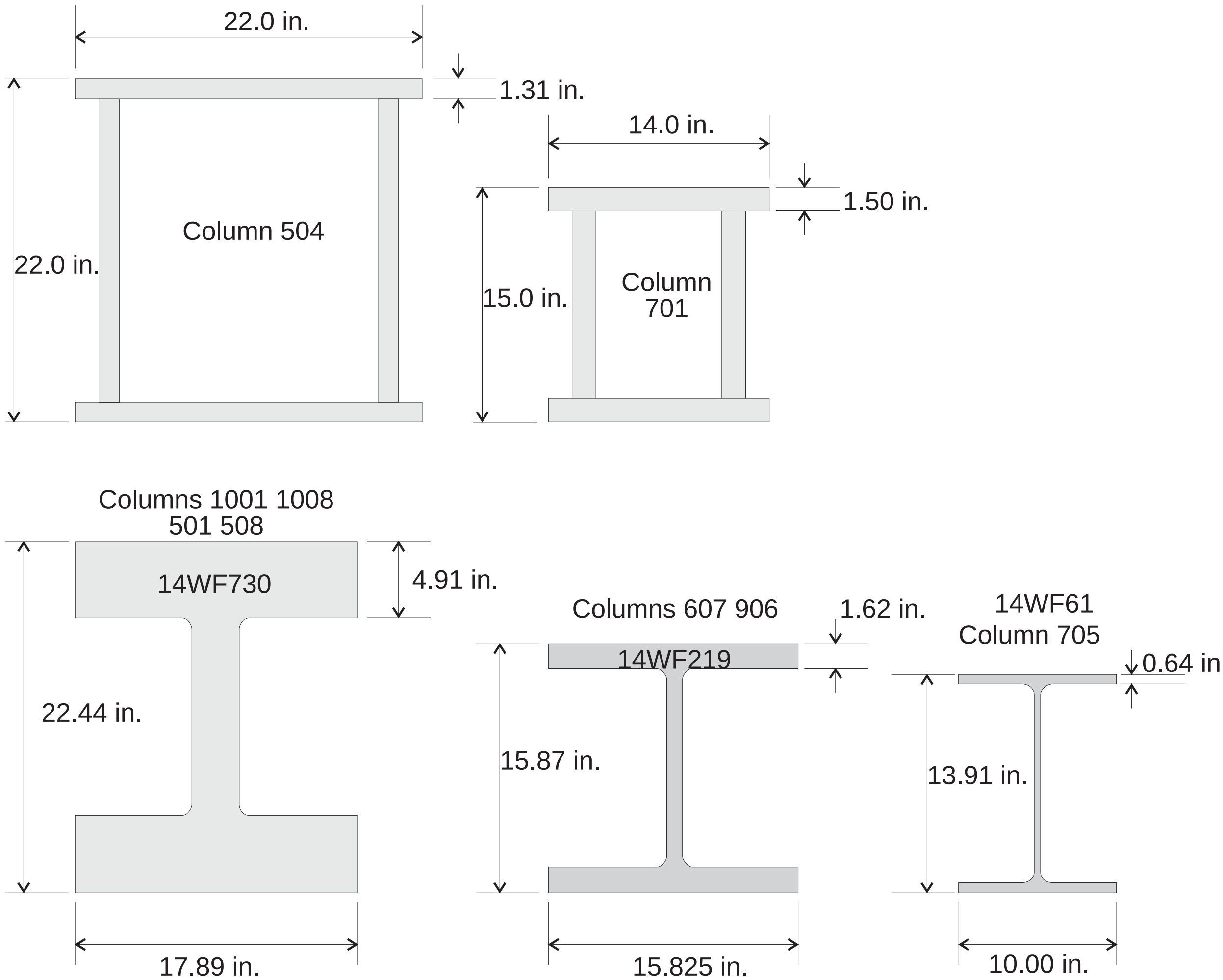


Figure E–6. Typical welded box members and rolled wide flange shapes used for core columns between floors 83 and 86 (to scale).

The core area was framed conventionally with beams. There were numerous openings in the core area floor for elevators and stairwells. Since fewer elevators were needed at the upper floors, part of the core area was not needed for services. In Fig. E–7, the dashed line shows the perimeter of the core, and shaded areas indicate typical enclosed areas for elevators and other services.

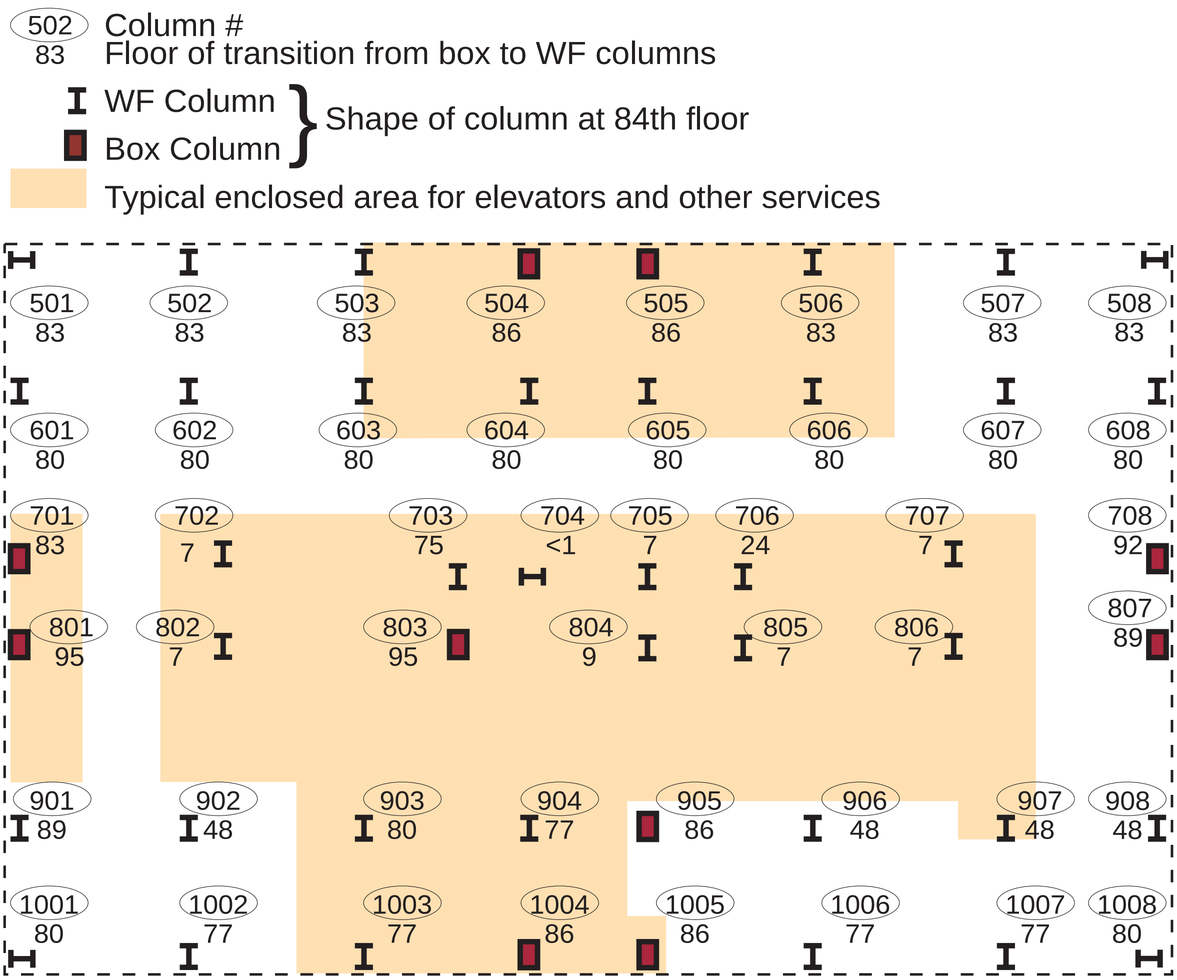
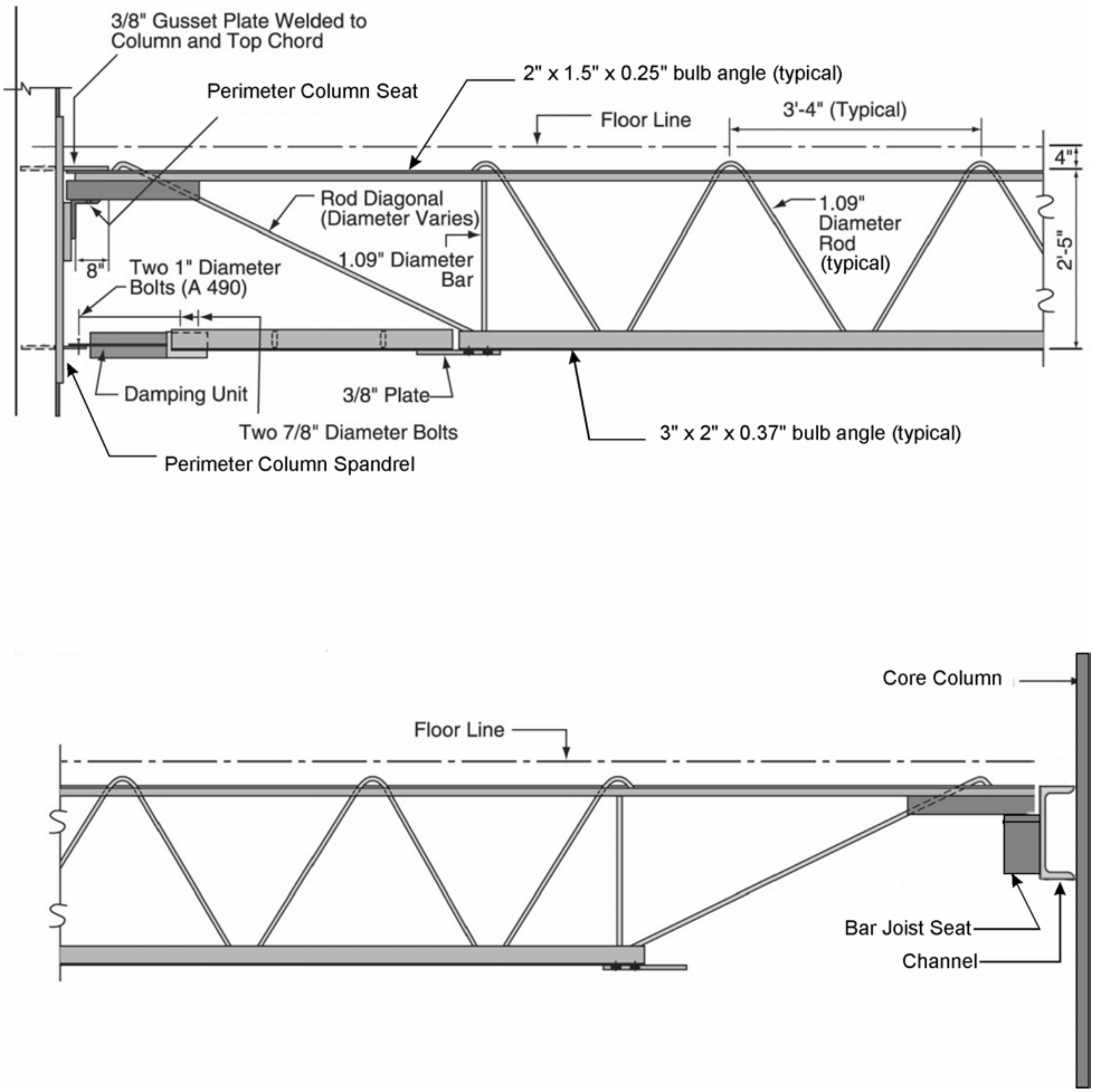


Figure E–7. Core column layout in WTC towers.

Flooring System

In the great majority of floors, the floor area outside the central core was supported by a series of 29 in. (0.74 m) deep, composite open web bar joists ("floor trusses") that spanned between the core and perimeter wall (see Fig. E–8). At the core, the floor trusses were bolted to seats generally attached to channels that ran continuously along the core columns. At the perimeter columns, the floor trusses were bolted and then welded to seats, mounted on spandrels at every other column. The floor trusses were approximately 60 ft (18.3 m) or 35 ft (10.7 m) long (depending upon the relative orientation of the building core), spaced at 6 ft 8 in. (2.0 m). There were of dozens of variants.



Source: McAllister 2002.

Figure E–8. Schematic diagram of a floor truss.

The prefabricated floor modules were typically 20 ft (6.1 m) wide, containing two sets of doubled trusses in the interior and a single truss along each edge. Thus, each seat supported either a double truss within a floor panel, or two single trusses from adjacent floor panels. In addition, the bottom chord of each pair of trusses was attached to perimeter spandrels with visco-elastic dampers. Bridging trusses ran perpendicular to the main bar trusses and were spaced at 13 ft 4 in. (4.06 m). The floor panels were covered with a corrugated steel floor deck that rested on the bridging trusses. Flutes in the deck ran parallel to the main trusses. Once in place, 4 in. (100 mm) of lightweight concrete was poured for the floor. Figure E–4 shows an assembled floor panel before the concrete floor was poured.

The minimum yield strength of the steel for the floor trusses was specified to be 50 ksi "unless otherwise noted." In practice, several of the designs specified 36 ksi steel as well as 50 ksi steel (see Section E.5.2 for complete details).

All seats were specified to be of 36 ksi minimum yield strength. There were over 30 varieties of perimeter seats, with various thicknesses from 3/8 in. to 7/8 in. in 1/8 in. increments (9.5 mm to 22.2 mm in 3.2 mm increments). Core seats were 7/16 in., 1/2 in., 5/8 in., or 3/4 in. thick (11.1 mm, 12.7 mm, 15.9 mm, or 19 mm).

The floor in the core area was typically framed with rolled structural steel shapes acting compositely with formed concrete slabs. Certain floors outside the core were also supported by rolled structural steel shapes rather than trusses. These included the mechanical floors and the floors just above the mechanical floors (e.g., floors 75, 76, and 77). Beam framing was typically W27[1] beams in the long span region and W16 beams in the short direction with beams spaced at 40 in. The floor was 5.75 in. thick, normal-weight concrete poured on a 1.5 in. fluted steel deck, acting compositely with the steel beams. The concrete on the beam-framed floors above the mechanical floors was 8 in. thick, normal-weight concrete in the core area and 7.75 in. thick normal-weight concrete outside the core.

Floors 107 to 110

At the top of each tower (floor 107 to the roof), a hat truss interconnected the core columns (Fig. E–9). Diagonals of the hat truss were typically W12 or W14 wide flange members. In addition, four diagonal braces (18 in. by 26 in. box beams spanning the 35 ft gap, and 18 in. by 30 in. box beams spanning the 60 ft gap) and four horizontal floor beams connected the hat truss to each perimeter wall at floor 108 spandrel. The hat truss was designed to provide a base for antennae atop each tower, although only the WTC 1 antenna was actually built.

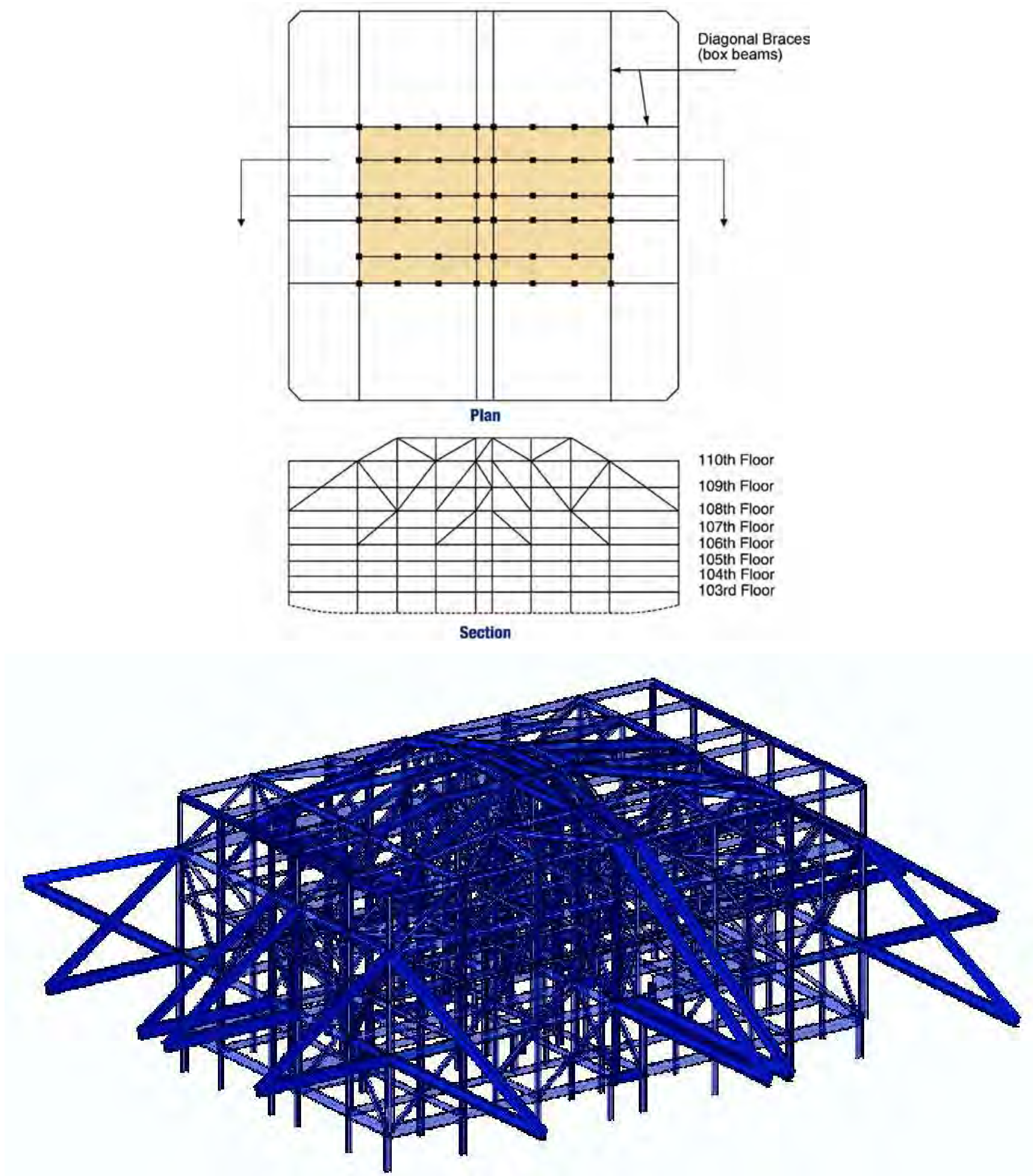
Perimeter columns for floors 107 to 110 also differed from the lower floors, and were alternating small tube columns or wide flange columns, with the wide flange columns supporting the floor system.

Impact Zone

The impact zones of the two towers are of particular interest, and special testing of the steels in this region will be conducted. High strain-rate mechanical tests and high-temperature mechanical property tests will focus on those steels most prominent in the impact zones, as indicated below.

In WTC 1, the perimeter columns torn out or otherwise damaged by the airplane impact (as judged from photographs of the building) were predominantly specified as 55 ksi and 60 ksi steel. In WTC 2, most damaged columns were specified in the 55 ksi to 65 ksi range, though there was a wide range of steel

[1] The "W" in W27 beam denotes the shape of the beam, which is like the letter "H" (see Figure E–6). The number following the "W" is the nominal depth of the beam in inches. The second number denotes the weight of the beam in pounds per foot. A W27 by 114 beam is 27.28 in. high and weighs 114 lb/ft. W shapes should not be confused with HP (sometimes called H) shapes. Like W shapes, HP shapes have flanges with parallel faces, but unlike W shapes, the webs and flanges of HP shapes have equal thickness. The common I-beam is denoted as an S shape, which differs from a W shape in that the flange faces are not parallel. Instead, the inside flange surface has a slope of 1/6.



Source: McAllister 2002:Fig. 2-10; Leslie Robertson Associates.

Figure E–9. Hat truss in upper floors.

grades involved. Table E–2 summarizes the steel grades in the perimeter columns damaged by the impact. In the table, the impact zone is defined as floors 94–98 in WTC 1 and floors 78–83 in WTC 2.

Table E–2. Number of WTC 1 and WTC 2 perimeter columns damaged by aircraft impact.

	Column Design Minimum Yield Strength F_y (ksi)									
	50	55	60	65	70	75	80	85	90	100
WTC 1	3	27	17	5	-	-	-	-	-	-
WTC 2	1	6	13	16	2	1	1	-	2	1

Although the extremities of the airplanes extended onto surrounding floors, these are the floors over which the airplanes penetrated into the buildings.

The number of core columns damaged by the impact is not known. In the WTC 1 impact zone, the core columns were almost entirely wide flange shapes. In the WTC 2 impact zone, the core columns were a mix of box and wide flange shapes. As is typical of all core columns, the steel was predominantly specified as 36 ksi and 42 ksi minimum yield strength. Table E–3 describes the distribution of core column types in the impact zones.

Table E–3. Number of core columns with a given minimum yield strength within the floors penetrated by the aircraft.

Column Type	Yield Strength F_y (ksi)							
	WTC 1 (floors 94 to 98)				WTC 2 (floors 78 to 83)			
	36	42	45	50	36	42	45	50
Box	0	3	-	-	38	15	-	-
Wide flange	88	44	3	3	81	6	0	1

Note: Core columns were three stories tall and were spliced at floors 77, 80, 83, 86, 89, 92, 95, and 98. The splice is several feet above the floor at the story indicated. Therefore, in the WTC 1 impact zone there were three sets comprising 141 individual columns.

Floors Involved in Post-Impact Fires

Special attention will be given to characterizing the performance of the structural steel found in floors engaged in the post-impact fires. The steels most vulnerable to heat from the fires were located in the zone damaged by the impact since those members were already under additional loads. Table E–4 lists the perimeter column types and grades of steel within these floors, defined here as floors 92 to 100 for WTC 1, and floors 77 to 83 for WTC 2. Table E–5 lists this information for the core columns.

Table E–4. Number of perimeter columns of specified grades in floors with significant fire.

	Floors	Perimeter Column Design Minimum Yield Strength F_y (ksi)											
		45	46	50	55	60	65	70	75	80	85	90	100
WTC 1	92 to 100	0	1	26	225	246	196	122	83	40	16	7	16
WTC 2	77 to 83	1	3	34	217	255	88	29	25	26	40	91	105

Table E–5. Number of core columns of specified grades in floors with significant fire.

Column Type	Yield Strength F_y (ksi)							
	WTC 1 (floors 92 to 100)				WTC 2 (floors 77 to 83)			
	36	42	45	50	36	42	45	50
Box	0	7	-	-	69	16	-	-
Wide flange	115	58	3	5	86	13	1	3

E.5 CONTEMPORANEOUS STEEL SPECIFICATIONS

This section integrates information from many sources on the steels used in the WTC and has three primary goals. First, contemporaneous (1960s era) American and Japanese steel specifications are summarized. Second, relevant information on steel properties from the construction documents and open literature sources is presented. Finally, estimated values for typical yield and tensile strengths and elongations for the numerous steels in the buildings are given (as opposed to the specified minimum values).

The first and second goals are approached from several directions. As is common practice, the structural engineering plans (obtained from LERA) only specify the minimum yield strengths and dimensions of the beams and columns. The steel contracts that the Port Authority (PONYA 1967, Ch. 2 §) awarded for the fabrication provided the specifications for the allowable steels to meet those minimum yield strengths. Those contracts allowed the fabricators to use steels that conformed to certain ASTM Standard Specifications. In addition, the contracts also permitted the fabricators to use certain proprietary steels from U.S. steel mills. These were required to conform to specific, dated and published data sheets that the steel mills provided. Finally, the contracts also allowed other proprietary steels not listed in the contract, provided that the Port Authority chief engineer of the project reviewed and formally approved their specifications (PONYA 1967, Clause 1). In all cases, the steels required extensive documentation to be acceptable for use.

Regarding the third goal, the best documentation of typical steel properties is contained in the mill test reports that detail the properties (F_y, tensile strength, elongation, chemistry, etc.) of the individual steel plates and shapes for the steels supplied. During late 2002 and early 2003, NIST investigators contacted the fabricating companies still in existence, their successors where possible, or in many cases their former employees, in a search for these mill test reports for steels used in the fire and impact zone, as well as other documents. None of the individuals or corporations retained these records. Section E.5.4 summarizes these contacts. The sources for steel properties NIST has obtained to date sometimes supply inconsistent values for the properties, so this report is a best effort to supply the steel properties.

This section focuses on the steels used in the area of the impact and fire: the floor panels, the perimeter columns, the welded core box columns, and the rolled core columns, fabricated by Laclede, Pacific Car & Foundry, Stanray Pacific, and Montague-Betts, respectively. It does not consider any of the sections of the buildings remote from the impact and fire sites, so fabricators of sections below the 9th floor (Mosher, Drier, Levinson, Pittsburgh-Des Moines, and Atlas) are not addressed, although Attachment 1 provides some background information on these companies.

In this document, "contemporaneous" refers to the standards in effect at the time of construction, in contrast to contemporary (or present-day) standards. ASTM standards are modified and renewed at regular intervals, so the current requirements of a standard may not have been in force during the WTC era. This distinction is also important because historical versions of standards can be difficult to locate. Attachment 2 summarizes the generally minor differences between the contemporaneous and contemporary versions of the relevant standards.

E.5.1 Standards Called for in the Steel Contracts

The Port Authority had a generic contract that listed allowable steel standard specifications, which went to all the fabricators. Generally, it specified that a given steel was acceptable for use if it conformed to one of a list of ASTM standards that were in force during September 1967. It also allowed several steels that were modifications of these ASTM standards. In addition, it allowed a number of proprietary steels made by U.S. steel mills. Finally, it allowed the use of other proprietary steels after formal approval by the Project Engineer, an employee of PONYA. It was by this last method that Pacific Car and Foundry (PC&F) received approval to use the Japanese steels in the perimeter columns.

It is important to remember that an ASTM standard can admit a wide variety of steel compositions and strengths. A specific steel might be capable of meeting several distinct ASTM steel standards. For instance, in the WTC construction era, ASTM A 36 only specified a minimum 36 ksi yield strength, an upper and lower tensile strength and carbon, manganese, silicon, phosphorus, and sulfur contents. Many high-strength low-alloy steels designed to meet other ASTM structural steel standards (e.g., A 572, A 242) will also meet A 36. Simply identifying a specific steel as meeting a given ASTM standard will not uniquely identify its composition or mechanical properties.

In terms of shapes and tolerances, all the steel was required to meet ASTM A 6, "General Requirements for Delivery of Rolled Steel Plates…"

Steels

Table E–6 summarizes the allowable steels listed in the contract (in "Chapter 2 (Materials)") between the Port Authority and all the fabricators. Note that it does not list ASTM A 572, a common, current standard for niobium-vanadium structural steels, which was established only in 1966. The proprietary steels allowed by the contract do include U.S.S. EX-TEN and Bethlehem V-series, however. These steels would conform to ASTM A 572, which was under development in that era. Tables E–7 and E–8 summarize the relevant structural steel specifications from the WTC construction era, including data on the various "modified" standards allowed in the Materials chapter of the fabricators' contracts.

Although Japanese steel mills supplied much of the steel, NIST has found no evidence that PONYA or the fabricators ever referred to any Japanese standards. Table E–9 summarizes the relevant Japanese Industrial Standard (JIS) from the era. They not as detailed as the corresponding ASTM steel standards, and mostly just specify minimum yield strength and maximum carbon content.

Table E–6. Steels specified as acceptable by PONYA in its contract with steel fabricators.

Standard	F_y (ksi)	Description of Standard
Structural Steels		
A 36	36	Structural steel
A 242	50	High-strength structural steel
A 440	50	High-strength structural steel
A 441	50	High-strength manganese vanadium steel
A 441 modified[a]	50	As A 441 with Cr and increased Cu
A 514	100	Quenched and tempered alloy steel plate for welding
A 514 Modified	100	As A 514, but TS requirements waived
USS CON PAC		Grades 70 and 80
Bethlehem V series		Grades 42, 45, 50, 55, 60, 65
Lukens		Grades 45, 50, 55, 60, 80
USS EX-TEN		Grades 42,45,50,55, 60, 65, 70
USS COR-TEN		“considered to conform to A 441 modified”
Lukens COR-TEN		“considered to conform to A 441 modified”
Pressure Vessel Steels		
A 302		Manganese molybdenum steel for pressure vessels
A 302 modified		
A 533		Mn-Mo and Mn-Mo-Ni steels for pressure vessels
A 533 modified		
A 542		Cr-Mo steel for pressure vessels

a. Apparently (Irving 1968) “A 441 modified” was a catch-all term for a group of steels that were codified in 1968 under ASTM A 588 “High-Strength Low-Alloy Structural Steel with 50,000 psi Minimum Yield Point to 4 in. Thick.”

Key: Cr, chromium; Cu, copper; Mn, manganese; Mo, molybdenum; Ni, nickel; TS, tensile strength.

Table E–7. Summary of mechanical properties from relevant ASTM International structural steel standards from WTC era.

Standard	Title	F_y Min. (ksi)	TS Min. (ksi)	TS Max. (ksi)	Elong Min. (%)	Notes
A 36-66[a]	Structural steel	36	58	80	20	For shapes; plates have higher C, Mn, and Si requirements
A 242-66[a]	High-strength low-alloy structural steel	50	70		18	Plates and bars t <= 0.75 in.; Group 1&2 shapes
A 440-67[a]	High-strength structural steel	50	70		18	Plates and bars t <= 0.75 in.; Group 1&2 shapes
A 440-67[a]	High-strength structural steel	46	67		19	Plates and bars 0.75 in. < t <=1.5 in.; Group 3 shapes; elongation reductions based for t > 0.75 in.
A 440-67[a]	High-strength structural steel	42	63		16	Plates and bars 1.5 in. < t <=4 in.; Group 4&5 shapes.; elongation reductions for t > 3.5 in.
A 441-66[a]	High-strength low-alloy structural manganese vanadium steel	50	70		18	Plates and bars t <= 0.75 in.; Group 1&2 shapes
A 441-modified[a]	As A 441, but modified by PONYA	50	70		19	Plates & bars 0.75 in.<= t<= 4 in.; Group 1,2,3 shapes
A 441-66[a]	High-strength low-alloy structural manganese vanadium steel	46	67		19	Plates and bars 0.75 in. < t <=1.5 in.; Group 3 shapes.; elongation minimums relaxed for t > 0.75 in.
A 441-66[a]	High-strength low-alloy structural manganese vanadium steel	42	63		16	Plates and bars 1.5 in. < t <= 4 in.; Group 4&5 shapes
A 441-66[a]	High-strength low-alloy structural manganese vanadium steel	40	60			Plates and bars 4 in. < t <= 8 in.; elongations on 2 in. GL
A 514-65[a]	High-yield-strength, quenched and tempered alloy steel plate, suitable for welding	100	115	135	18	t <= 0.75 in.
A 514-65[a]	High-yield-strength, quenched and tempered alloy steel plate, suitable for welding	100	115	135	18	0.75 in. < t <= 2.5 in.
A 514-65[a]	High-yield-strength, quenched and tempered alloy steel plate, suitable for welding	90	105	135	17	2.5 in. < t <= 4 in.
A 514-modified[a]		100	x	x	See std.	As A 514, but TS waived in PONYA steel contract
A 529-64	Structural steel with 42 ksi minimum yield point	42	60	85	19	
A 572-70	High strength low-alloy columbium vanadium steels of structural quality	50	65		18	6 grades: F_y = (42 45 50 55 60 65) ksi; different C contents
A 573-70	Structural carbon steel plates of improved toughness	35	65	77	20	2 grades 65 ksi or 70 ksi TS
A 588-70	High-strength low-alloy structural steel with 50 ksi minimum yield point to 4 in. thick	50	70		18	9 chemistries

a. Allowed by PONYA Steel contract, Chapter 2 “Materials.”

Key: C, carbon; Elong, elongation to failure; F_y, specified minimum yield strength; Mn, manganese; Si, silicon; TS, tensile strength.

Table E–8. Summary of chemistry data from relevant ASTM International structural steel standards from WTC era.

Standard	Chemistry (%)									Other/Notes
	C Max.	Mn Max.	Si Max.	Ni	Cr	V Min.	Cu Min.	P Max.	S Max.	
A 36-66 shapes	0.26	NR	NR				0.2	0.04	0.05	Cu where specified
A36-66 plates with $t \leq 0.75$ in.	0.25	NR	NR					0.04	0.05	Cu where specified
A36-66 plates with 0.75 in. $< t \leq 1.5$ in	0.25	0.8-1.2	NR					0.04	0.05	Cu where specified
A36-66 plates with 1.5 in. $< t \leq 2.5$ in	0.26	0.8-1.2	0.15-0.3					0.04	0.05	Cu where specified[a]
A 242-66	0.22	1.25							0.05	Type 1
A 242-66	0.15	1.40							0.05	Type 2
A 440-67	0.28	1.1-1.6	0.3				0.2	0.06	0.05	
A 441-66	0.22	0.85-1.25	0.3			0.02	0.2	0.04	0.05	
A 441-modified	0.19	0.85-1.25	0.15-0.3		0.4-0.65	0.02	0.25-0.4	0.04	0.05	
A 441-66	0.22	0.85-1.25	0.3			0.02	0.2	0.04	0.05	
A 514-65										8 individual chemistries, with Cr, Mo, B
A 529-64	0.27	1.2					0.2	0.04	0.05	
A 572-70	0.22	1.35	0.3					0.04	0.05	4 variants with Nb or Va or Nb+Va, or V+N
A 573-70	0.24	0.85-1.25	0.15-0.30					0.04	0.05	
A 588-70										9 individual chemistries, generally with Cr, Ni, V, Nb

a. A 36 plates have different requirements for thicker sections that include higher carbon allowables and slightly different manganese requirements.

Key: B, boron; C, carbon; Cr, chromium; Cu, copper; Mn, manganese; Mo, molybdenum; Nb, niobium; Ni, nickel; NR, no requirement; P, phosphorus; Si, silicon; S, sulfur; V, vanadium.

Table E–9. Summary of Japan Industrial Standard structural steel standards from 1974.

Standard	Grade	F_y Min. (ksi)	TS Min. (ksi)	TS Max. (ksi)	C Max. (%)	Mn Max. (%)	Si Max. (%)	Cr (%)	Cu Min. (%)	P Max. (%)	S Max. (%)	Other
JISG3106-73 Rolled Steel for Welded Structure	SM50a	45	71	88	0.20	1.5	0.55			0.04	0.04	Add any element "if necessary"
	SM50b SM50c	45	71	88	0.18					0.04	0.04	Add any element "if necessary"
	SM50Ya SM50Yb	51	71	88	0.20	1.5	0.55			0.04	0.04	Add any element "if necessary"
	SM53b SM53c	51	75	92								Add any element "if necessary"
	SM58	65	82	104	0.18	1.5	0.55			0.04	0.04	
JIS G3114-73 Hot Rolled Atmospheric Corrosion Resistant Steel for Welded Structure	SMA50a SMA50b SMA50c	51	71	88	0.19	1.4	0.75	0.3–1.2	0.2–0.7	0.04	0.04	+ Mo or Nb or Ni or Ti or V or Zr
	SMA58b	65	82	104	0.19	1.4	0.75	0.3–1.2	0.2–0.7	0.04	0.04	+Mo, Ni, Nb, Ti, Va and or Zr
JIS G3101-73 Rolled Steel for General Structure	SS55	57	78		0.30	1.6				0.04	0.04	Add any element "if necessary"
	SS50	40	71	88						0.05	0.05	

Key: C, carbon; Cr, chromium; Cu, copper; F_y, yield strength; JIS, Japan Industrial Standard; Mo, molybdenum; Nb, niobium; Ni, nickel; P, phosphorus; Si, silicon; S, sulfur; Ti, titanium; TS, tensile strength; V, vanadium; Zr, zirconium.
Note: Compositions are given as mass fractions. Thickness range for all standards is 16 mm< t < 40 mm.
Source: World Steel Standards, Handbook of Comparative (1974).

Fasteners

Section E.6 covers fastener standards in the section on connections (bolts and welds).

E.5.2 Steels Used in Construction

Information from the suppliers and fabricators was used to identify the specific steels supplied to meet those contractual requirements. Table E–10 and Attachment 1 provide background information on the various fabricators of WTC steel, including tons of steel reported in their contracts. The rest of this section summarizes information on the steels used in the impact and fire zones of the towers.

Floor Trusses

Laclede Steel manufactured the trusses for the composite floor panels for both WTC 1 and WTC 2 from steel they made and rolled at their mill in Alton, Illinois. The chords were fabricated from hot-rolled angles, while the web was fabricated from hot-rolled round bar, Fig. E–10.

Table E–10. Steel companies involved in WTC construction and their contracts.

Fabricator	Current Status	Component	Tons
Pacific Car & Foundry, Co.	Sold in 1974	Exterior columns and spandrels	55,800
Montague Betts, Co. Inc	No longer a steel fabricator	Rolled columns and beams above 9th floor	25,900
Pittsburgh-Des Moines Steel Co.		Bifurcation columns (“trees”) 4th to 9th floor	6,800
Atlas Machine & Iron Works	No longer in business	Box columns below the bifurcation columns to 4th floor	13,600
Mosher Steel Co.	Currently active	Core box columns below the 9th floor	13,000
Stanray Pacific Corp.	Closed in 1971	Core box columns above the 9th floor	31,100
Levinson Steel Co.	Sold in 1997, parent company in bankruptcy	Supports for slabs below grade	12,000
Laclede Steel Co.	Bankrupt in 2001, new owners of rolling mill	Floor trusses	Unknown
Drier Structural Steel Co, Inc.	Unknown	Grillages	Unknown
		Total	141,170

Source: Feld 1971.

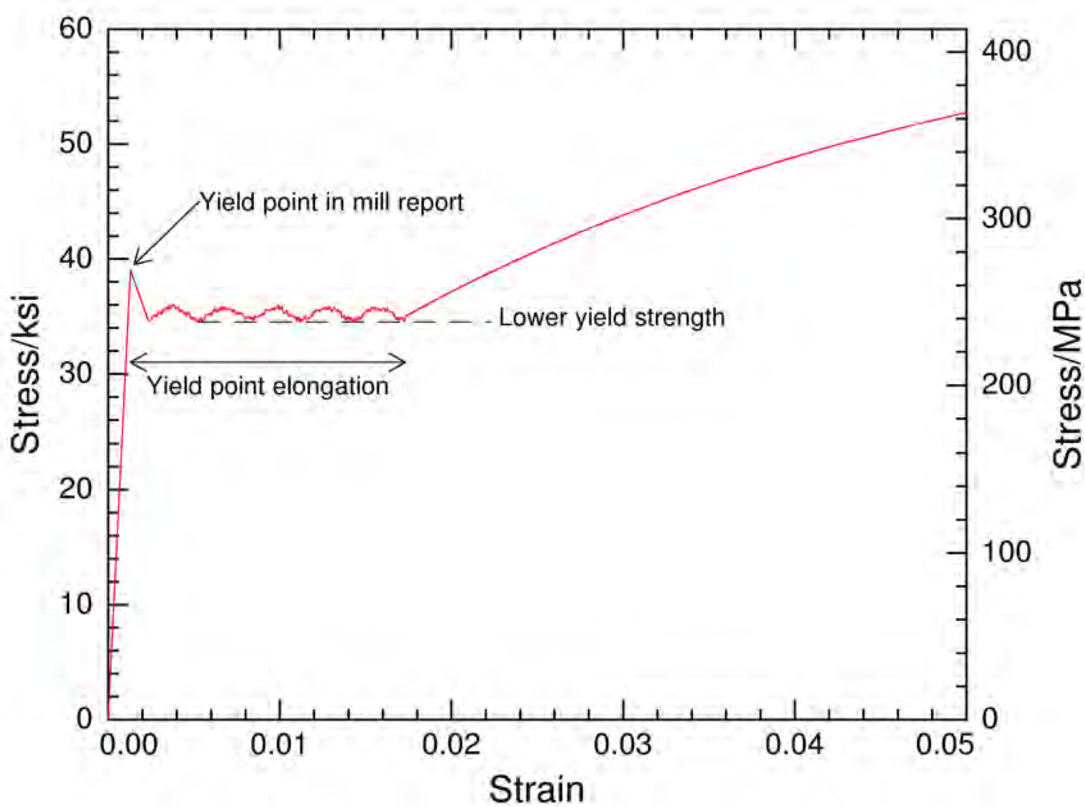


Figure E–10. Schematic diagram of the stress-strain behavior of a structural steel.

According to internal Laclede documents (Bay 1968 †), the top chord angles, as well as most round bars, were fabricated to meet ASTM A 242 (F_y = 50 ksi). Only 1.09 in. (27.7 mm) and 1 13/16 in. (46.0 mm) round bars and the bottom chord angles were specified as ASTM A 36. Conversations with Laclede metallurgists (Brown 2002 †) active during the WTC construction revealed that even for components specified as ASTM A 36, Laclede would have supplied a vanadium, micro-alloyed steel with a typical F_y = 50 ksi, similar to a contemporary A 572 steel. In all the Laclede documents NIST examined, there were only two different mill test reports on A 242 steel, both from mid-1969; see Table E–11. These mill test reports indicate that the A 242 steel supplied is a niobium-containing steel similar to modern ASTM A 572 steels with yield points that exceed the specified minimum by about 10 ksi.

Table E–11. Properties of Laclede ASTM A 242 steels obtained from Laclede mill test reports.

Component	F_y (ksi)	Element Composition (%)						Source
		C	Mn	P	S	V	Nb	
2 in. by 1.5 in. by 0.25 in. bulb angle heat 83033	62.8	0.20	0.86	0.014	0.044	NR	0.020	(Kamper 1968 †)
3 in. by 2 in. by 0.25 in. bulb angle heat 83162	60.1	0.19	0.77	0.013	0.043	NR	0.015	(Kamper 1968 †)
1.14 in. rod heat 76056	54	0.19	0.80	0.005	0.024	NR	NR	(White1969b †) 2 tests

Key: C, carbon; Mn, manganese; Nb, niobium; NR, not reported; P, phosphorous; S, sulfur; V, vanadium.
Note: Compositions are reported % mass fractions.

Perimeter Columns and Spandrels

The perimeter wall columns, fabricated by PC&F, comprise three important sub assemblies: the columns, the spandrels, and the truss seats. The structural engineering plans called for the columns to be fabricated from 14 grades of steel with F_y = (36, 42, 45, 46, 50, 55, 60, 65, 70, 75, 80, 85, 90, and 100) ksi. Above floor 75, more than half of the columns have yield strengths greater than or equal to 55 ksi and less than or equal to 70 ksi. The spandrels were fabricated from 12 grades of steel with F_y = (36, 42, 45, 46, 50, 55, 60, 65, 70, 75, 80, and 85) ksi. The truss seats were specified to be fabricated from steel with F_y = 36 ksi minimum.

Yawata Iron and Steel Co. supplied most of the steel to PC&F for the perimeter columns and spandrels. In general, the exterior (or web) and side (or flange) plates of each column and the spandrels were fabricated from Japanese steel, and the inner web plate (plate 3, see Fig. E–3) was fabricated from domestic steel (Symes 1969a §; White 1969a §). Searches of archival material yielded no information on the steels for the truss seats beyond the fact that they were specified as F_y = 36 ksi.

A contemporaneous Yawata document (Yawata 1969 †) indicates that Yawata shipped 46,000 metric tons of WEL-TEN 60, 60R, 62, 70, and 80 to PC&F. That document refers to WEL-TEN 80, rather than WEL-TEN 80C, which is a Yawata steel with a different chemistry, but identical yield strength. The document certainly must actually mean WEL-TEN 80C, because all other sources, including other Yawata sources, that mention WEL-TEN steels refer to WEL-TEN 80C. Most sources, for instance, Feld (1971), put the PC&F contract at 55,800 tons. Assuming the Yawata document (1969 †) refers to metric tons, that would still leave a minimum of 5,100 tons from other sources. The inner web plate (plate 3, Fig. E–3) represents about 12 percent of the total area of a perimeter column panel. The 5,100 tons unaccounted for in the Yawata contract is not inconsistent with the assertion that the inner web was usually fabricated from domestic steel, while the remaining plates were fabricated from Yawata steel.

Several sources (ENR 1967; Monti 1967a §; White 1967a §; Feld 1967a §) indicate that Kawasaki Steel also supplied PC&F, but apparently only 36 ksi grade (Feld 1967a §). Ronald Symes (2002 †), PC&F chief engineer, could not remember any other foreign steel suppliers other than Kawasaki. However, the fabricators only interacted with the Japanese import companies rather than with the steel mills directly. Mitsui (now Mitsui USA) imported the Japanese steel for PC&F. Because the flanges and spandrels are the primary structural components of the perimeter columns, and they were all fabricated from Yawata steel, the properties of the perimeter columns can be based on the mechanical properties of the Yawata steels.

During the 1960s Yawata produced a number of named, proprietary grades (such as WEL-TEN and YAW-TEN series) of weldable steels with specified minimum properties. Several of these named grades supplied to PC&F (WEL-TEN 60, WEL-TEN 62, WEL-TEN 80C) are common in the contemporaneous literature, and open literature publications (Ito 1965a, Ito 1965b; Goda 1964) describe many of their physical and mechanical properties other than specified minimum strength quite extensively. For two of the named, proprietary grades that Yawata supplied to PC&F (WEL-TEN 60R and WEL-TEN 70), NIST has been unable to find corroborating specifications or mechanical property data, even in consultation with Nippon Steel. It is possible that these names were assigned simply for convenience for the WTC construction. Chemically, WEL-TEN 60, 60R and 62 are similar to contemporary ASTM A 588, with their Cr additions and high silicon contents, though none would meet that specification exactly. WEL-TEN 60, 62, and 70 are heat-treated steels, while WEL-TEN 60R is a hot-rolled steel. WEL-TEN 80C is a Cr-Mo steel that is very similar to contemporary A 514 steels, and possibly could have been manufactured to meet that contemporary specification. According to PC&F documents (Symes 1967c §), Yawata intended to supply grades that would meet the “ASTM A 441-modified” specification (see Table E–7) of PONYA for the lower strength column plates. From the proposed specification, these “A 441-modified” compositions were similar to contemporary A 588 steels, with their added Cr and use of Nb for strengthening. Their chemistries do not correspond to any other named grade of Yawata steel, for example WEL-TEN 50, WEL-TEN 55, YES 36, YES 40 or YAW-TEN 50. For the intermediate strength plates (55 ksi, 60 ksi, and 65 ksi), Yawata intended to furnish heat-treated WEL-TEN grades for the thicker sections and the hot-rolled “A 441 modified” grades for the thinner sections. Tables E–12 and E–13 summarize these specifications and representative properties, obtained from a variety of documents. Note that not all the sources agree on yield strength or chemistries, probably because Yawata could tailor the steels for specific applications. The entries at the top of the table are for the steels that a PC&F memo (Symes 1967c §) mentions, while the bottom entries detail representative data culled from many literature sources for all grades of Yawata weldable steels.

NIST has located a total of six mill test reports (tests performed at the Yawata rolling mill) describing 135 plates (Symes 1969b §; Barkshire 1969a §; White 1969c §) of Yawata steels: two for F_y = 75 ksi, one for F_y = 70 ksi, two for F_y = 50 ksi, and one for F_y = 45 ksi. When the originals were microfilmed after the construction was completed, the technician did not rotate the landscape pages into portrait orientation, so the sheets only show the measured yield point, tensile strength and elongation, but not chemistry. For each steel, the measured yield strength of the plates increases with decreasing thickness. The thickest WEL-TEN 62 plates (t = 1.5 in.) plates typically have yield strengths 5 ksi higher than the specified yield strength. The thinnest plates (t = 0.375 in.) have yield strengths 15 ksi to 20 ksi higher than the stated

Table E–12. Specified properties for Yawata contemporaneous steel grades.

Name	Notes	Fy	TS	TS	Elong	C	Mn	Si	Ni	Cr	V	Cu	P	S	Other
		min	min	max	min	max	max	max		max	max	min	max	max	
		(ksi)	(ksi)	(ksi)	%	%	%	%	%	%	%	%	%	%	
Yawata Steels noted in PC&F documents (1967)															
A 36															to meet ASTM A 36
A 441 modified	class 42	42				0.22	0.85-1.25	0.3				0.2	0.040	0.050	0.2 Nb+V max
A 441 modified	classes 45	45				0.22	1.1-1.6	0.55				0.2	0.040	0.050	0.15 Nb+V max
A 441 modified	classes 50-60	50,55,60				0.22	1.1-1.6	0.55		0.3		0.2	0.040	0.050	0.15 Nb+V max
WEL-TEN 60		55,60,65				0.16	0.9-1.4	0.15-0.55		0.3	0.12		0.035	0.035	
WEL-TEN 60R		65				0.18	1.5	0.55		0.3			0.035	0.040	0.15 Nb+V max
WEL-TEN 62		70,75				0.18	1.4	0.55		0.3	0.12		0.035	0.035	
WEL-TEN 70		80,90				0.18	1.2	0.55		1.3		0.15-0.5	0.030	0.030	0.6Mo
WEL-TEN 80C		100				0.18	0.6-1.2	0.15-0.35		0.7-1.3		0.15-0.5	0.03	0.03	Mo 0.6; B 0.006; max
Yawata/Nippon high-strength structural steels: specified values															
WEL-TEN 50	rolled	47	71	82	20	0.18	0.9-1.5	0.25-0.45							
WEL-TEN 50	rolled or normalized	47	71	83	20	0.18	0.9-1.5	0.25-0.45					0.035	0.040	listed as WEL-TEN 50(A,B)
WEL-TEN 50	rolled or normalized	47	71	82	20	0.18	0.9-1.5	0.25-0.45					0.035	0.040	1.1-1.5Mn for t>30mm
WEL-TEN 55	normalized	51	78	90	20	0.18	1.2-1.5	0.35-0.55							
WEL-TEN 55	as rolled or normalized	51	78	90	18	0.18	1.2-1.5	0.35-0.55							
WEL-TEN 55	as rolled or normalized	51	78	90	18	0.18	1.2-1.5	0.35-0.55					0.035	0.040	
WEL-TEN 60	Heat-treated	65	85		16	0.16	1.3	0-0.55	0.6	0-0.4	0-0.15				
WEL-TEN 60	Q&T	65	85	102	20	0.16	0.9-1.4	0.15-0.55	0.6	0.3	0.12		0.035	0.035	can be supplied as Fy=71ksi "Ni can be added if necessary"
WEL-TEN 60		65	85	100	16	0.16	1.3	0.55	0.6	0.4	0.15		0.040	0.040	
WEL-TEN 60-LT		65	85			0.16	0.9-1.4		0.6	0.3	0.12				
WEL-TEN 60-LT	Q&T	65	85	102		0.16	0.9-1.4	0.15-0.55	0.6	0.3	0.12		0.035	0.035	
WEL-TEN 60H	normalized	64	85	102	20	0.18	1.0-1.5	0.15-0.55	1						Nb+V 0.15(max)
WEL-TEN 60H	normalized	64	85	102	20	0.18	1.0-1.5	0.15-0.75	1				0.035	0.035	Nb+V 0.15(max)
WEL-TEN 60H	as rolled or normalized	64	85	102	20	0.18	0.8-1.5	0.15-0.75	0.3-1.0				0.035	0.040	Fy = 60ksi t < 38mm all grades: Nb+V 0.15 (max)
WEL-TEN 60R		65				0.18	1.50	0.55		0.3			0.035	0.040	0.15 Nb+V max
WEL-TEN 62	Q&T	71	88	107	19	0.18	0.9-1.4	0.15-0.55	0.6	0-0.3	0-0.12				
WEL-TEN 62	Q&T	71	88	107	19	0.18	0.9-1.4	0.15-0.55	0.6	0.3	0.12		0.035	0.035	"Ni can be added if necessary"
WEL-TEN 68	Q&T	80	97	117		?									
WEL-TEN 70		103	113		44	0.11	1.0	0.45	0.9	0.3	0.04	0.02	0.010	0.003	0.4Mo
WEL-TEN 74	Q&T	90	105	121											
WEL-TEN 80	Q&T	112	119		23	0.11	0.85	0.21	0.97	0.53	0.05	0.22	0.015	0.006	Mo 0.43 B 0.0008
WEL-TEN 80		100	114	135	22	0.18	0.6-1.2	0.15-0.35	1.5	0.4-0.8	0.1	0.15-0.5	0.030	0.030	Mo 0.6; B 0.006; max
WEL-TEN 80	Q&T	100	114	135	18-20	0.18	0.6-1.2	0.15-0.35	1.5	0.4-0.8	0.1	0.15-0.5			Mo 0.6; B 0.006; max
WEL-TEN 80		100	114	135	18-20	0.18	0.6-1.2	0.15-0.35	1.5	0.4-0.8	0.1	0.15-0.5	0.035	0.040	Mo 0.6; B 0.006; max
WEL-TEN 80	Q&T	100	114	135	16	0.18	0.6-1.2	0.15-0.35	1.5	0.4-0.8	0.1	0.15-0.5	0.030	0.030	Mo 0.6; B 0.006; max
WEL-TEN 80	Q&T	100	114	135	18	0.18	0.6-1.2	0.15-0.35	1.5	0.4-0.8	0.1	0.15-0.5	0.035	0.040	Mo 0.6; B 0.006; max
WEL-TEN 80C-LT		100	114	136	x	0.18	0.6-1.2			0.7-1.3		0.15-0.5			Mo 0.6; B 0.006; max
WEL-TEN 80C-LT	Q&T	100	107	114	23	0.18	0.6-1.2	0.15-0.35		0.7-1.3		0.15-0.5	0.030	0.030	Mo 0.6; B 0.006; max
WEL-TEN 80C	Q&T	100	114	135	23	0.18	0.6-1.2	0.15-0.35		0.7-1.3		0.15-0.5			Mo 0.6; B 0.006; max
WEL-TEN 80C	Q&T	100	114	135	16	0.18	0.6-1.2	0.15-0.35		0.7-1.3		0.15-0.5	0.030	0.030	Mo 0.6; B 0.006; max
WEL-TEN 80C		100	114	135	13 ?	0.18	0.6-1.2	0.15-0.35		1.3		0.15-0.5	0.035	0.040	Mo 0.6; B 0.006; max
WEL-TEN 100N	Q&T	130	140	163	15	0.18	0.6-1.2	x	1.5	0.4-0.8	0.1	0.15-0.5			Mo 0.6;
WEL-TEN 100N	Q&T	130	140	163	15	0.18	0.8		1.5	0.6	0.1	0.15-0.5			Mo 0.6
WEL-TEN 100N	Q&T	128	138	164	13	0.18	0.6-1.2	0.15-0.35	1.5	0.4-0.8	0.1	0.15-0.5	0.030	0.030	Mo 0.6;
WEL-TEN 100N		128	138	164	15	0.18	0.6-1.2	0.15-0.35	1.5	0.4-0.8	0.1	0.15-0.5	0.350	0.040	Mo 0.6;
YES36A/B	?	51	68		17	0.23	1.4	0.15					0.035	0.040	Nb+V 0.15(max)
YES40A	?	57	75		15	0.28	1.4	0.15					0.035	0.040	Nb+V 0.15(max)
YAW-TEN 50	?	57	71		22	0.12	0.9	0.35				0.25-0.5	0.06-0.12	0.040	Ti 0.15;

Sources

1 Iron age, 1966 Yawata advertisement.
2 Alloy Digest Dec. 1968
3 Alloy Digest 1969
4 Alloy Digest July 1968
5 Alloy Digest 1965
6 Alloy Digest 1967
7 Goda 1964
8 Ito 1965
9 Japan's Iron and Steel Industry 1968
10 Yawata 1969b
11 Otani 1966
12 Symes 1967c
14 Woldman's 1990
16 Yawata 1969b
17 Zaizen 1968
18 Symes 1967c

Table E–13. Reported properties for Yawata contemporaneous steel grades.

Name	Notes	Fy min (ksi)	TS min (ksi)	TS max (ksi)	Elong min %	C %	Mn %	Si %	Ni %	Cr %	V %	Cu %	P %	S %	Other	Source
Yawata/Nippon high-strength structural steels: actual values taken from reports																
WEL-TEN 50		46	73		23.5											11
WEL-TEN 50		51	75		27	0.14	0.125	0.37					0.021	0.022		11
WEL-TEN 50		51	80		34.8	0.19	1.36	0.37		0.05			0.027	0.027		11
WEL-TEN 60		76	90		13.6											11
WEL-TEN 60		75	91		29.7	0.13	1.24	0.46		0.22			0.015	0.006		11
WEL-TEN 60	12 mm plate	79	92		31.5	0.12	1.22	0.46		0.25	0.09		0.012	0.010		7
WEL-TEN 60H		73	94		33.8	0.15	1.41	0.47	0.51		0.06		0.017	0.006	Nb.04	
WEL-TEN 80		111	119		11.8											11
WEL-TEN 80		113	119		22.5	0.15	0.88	0.28	0.96	0.56	Tr	0.25	0.012	0.006	Mo 0.48	11
WEL-TEN 80	heat a	115	123		32	0.12	0.76	0.24	0.92	0.53	0.06	0.25	0.012	0.009	Mo 0.38; B 0 .0021	16
WEL-TEN 80	heat b	116	122		30	0.13	0.67	0.22	1.02	0.49	0.07	0.22	0.013	0.011	Mo 0.42; B 0.0027	16
WEL-TEN 80	heat c	115	121		25	0.15	0.77	0.23	1.02	0.48	0.07	0.25	0.012	0.008	Mo 0.43; B 0.0018	16
WEL-TEN 80	25mm plate	118	122		24	0.11	0.84	0.26	0.95	0.54	0.05	0.27	0.016	0.007	Mo 0.43; B 0.001	10
WEL-TEN80C	25mm plate	102	114		25	0.15	0.90	0.27		1.06		0.30	0.016	0.007	Mo 0.52	8
WEL-TEN80C	40 mm plate	108	118		23	0.15	0.90	0.27		1.06		0.30	0.016	0.007	Mo 0.52	8
YES 36 A	15 mm plate	55	77		24	0.20	0.72	0.08					0.014	0.019	Nb 0.04;	7
YES 40 A	25 mm plate	62	81		24	0.22	0.92	0.07					0.018	0.018	Nb 0.04;	7
YAW-TEN 50	12 mm plate	62	76		36	0.11	0.76	0.2				0.35	0.082	0.013	Ti 0.06;	7

Sources

1	Iron age, 1966 Yawata advertisement.	7	Goda 1964	14	Woldman's 1990
2	Alloy Digest Dec. 1968	8	Ito 1965	16	Yawata 1969b
3	Alloy Digest 1969	9	Japan's Iron and Steel Industry 1968	17	Zaizen 1968
4	Alloy Digest July 1968	10	Yawata 1969b	18	Symes 1967c
5	Alloy Digest 1965	11	Otani 1966		
6	Alloy Digest 1967	12	Symes 1967c		

yield strength. For the lower strength plates (F_y = 45 ksi and F_y = 50 ksi) the measured yield strength increases less rapidly with decreasing thickness: to a first approximation, their strength is independent of thickness. They average 7.4 ksi and 11.8 ksi higher than the specified yield strength, respectively.

Contemporaneous documents indicate that PC&F also purchased V-series (White 1968a §; 2003 †) and modified V-series plate from Bethlehem Steel (Symes 1967a §), EX-TEN and modified EX-TEN from U.S. Steel (Symes 1967a §; White 2003 †; Barkshire 1968a §), and various Kaisaloy grades (Barkshire 1968b §) from Kaiser steel, for use in the interior plates. The inner plate (plate 3, see Figure E–3) is usually half the thickness of the flange plates, and never exceeds 15/16 in. thick, and so represents at most 5 percent of the mass of steel in the entire contract. Status reports from mid-1968 indicate that PC&F phased out U.S. Steel and Kaiser and replaced them with Bethlehem as the only domestic supplier (Barkshire 1968c §). Presumably most of the inner web plates (plate 3) in the columns (see Fig. E–3) near the impact floors were made from hot-rolled Bethlehem V-series steels. Table E–14 summarizes the properties of the V-series (Alloy Digest 1970) and modified V-series steels (Symes 1967b §).

In summary, NIST has extensive data from open literature sources for properties other than chemistry and yield strength for the 65 ksi WEL-TEN 60, the 70 ksi WEL-TEN 62, and the 100 ksi WEL-TEN 80C. Properties for the “A 441-modified” grades and for WEL-TEN 70 and WEL-TEN 60R must be estimated theoretically, from accepted literature values for plate steels, or experimentally from tests on recovered steels.

Core (Welded Box Columns)

Stanray Pacific Corp. fabricated the welded core columns in both buildings above floor 9. The plans called for two grades of steel with 36 ksi and 42 ksi minimum yield strengths. Contemporaneous Stanray Pacific documents (Morris 1967 §; Warner 1967 §) indicate that Stanray Pacific purchased nearly all the steel plate for the core columns from two sources. The 10,240 tons from Colvilles Ltd. (Dalzell Works, Motherwell, Scotland) and 21,760 tons from Fuji Iron and Steel (Hirohata Works). The total of 32,000 tons is close to the 31,100 tons that Feld (1971) reported in his *Civil Engineering* article that summarizes the construction of the WTC towers. It is likely, then, that these two mills supplied nearly all the steel for the welded core columns. Telephone conversations with M. McKnight (2003 †), formerly with the British Steel Export Association, which imported the steel to the United States, confirmed Colvilles as a supplier to Stanray Pacific.

The same document (Warner 1967 §) that details the major suppliers, indicates that Fuji Steel supplied all plates thinner than 1.75 in. Both mills supplied plates with $t \geq 1.75$ in., but even there, Fuji supplied about 60 percent of the total mass of steel used. In the fire and impact floors of WTC 1 (floor 94 to floor 98), only three of the columns are welded, box columns, and all three are made from plate thinner than 1.75 in. In the fire and impact floors of WTC 2 (floor 78 to floor 84) only 9 of 52 welded box columns are made from plate 1.75 in. or thicker. In terms of steel properties for modeling, then, the columns can be modeled with the properties of the Fuji-supplied plates alone. NIST has located a mill test report for a single Fuji Steel A 36 plate (Morris 1969 §), and a third-party chemical analysis of a Colvilles A 36 plate (Walton 1968 §) (Table E–15). Other than these, NIST has located no other mill records. See Table E–16 for the search details.

Table E–14. Mechanical properties of U.S. Steel and Bethlehem V-series steels.

Name	Notes	FY min (ksi)	TS min (ksi)	Elong min %	Source	C max %	Mn max %	Si max %	Ni min %	Cr min %	V min %	Cu min %	P max %	S max %	Other
Bethlehem Steel															
V42	t<=1.5 in.	42	63	20	(1)	0.22	1.25	0.3			0.02		0.04	0.05	N .015 max
V42	1.5<t<=4 in.	42	63	20	(1)	0.22	1.25	0.25-0.3			0.02		0.04	0.05	N .015 max
V45		45	65	19	(1)	0.22	1.25	0.3			0.02		0.04	0.05	N .015 max
V50		50	70	18	(1)	0.22	1.25	0.3			0.02		0.04	0.05	N .015 max
V50	t>0.75 in.	50	70	17	(1)	0.22	1.25	0.3			0.02		0.04	0.05	N .015 max
V55	t<=0.375 in.	55	70	17	(1)	0.22	1.25	0.3			0.02		0.04	0.05	N .015 max
V55	0.375 in. <t<=0.75 in.	55	70	16	(1)	0.22	1.25	0.3			0.02		0.04	0.05	N .015 max
V55	t>0.75 in.	55	70	15	(1)	0.25	1.35	0.3			0.02		0.04	0.05	N .015 max
V60	t<=0.375 in.	60	75	16	(1)	0.22	1.25	0.3			0.02		0.04	0.05	N .015 max
V60	0.375 in.<t<=0.75 in.	60	75	15	(1)	0.25	1.35	0.3			0.02		0.04	0.05	N .015 max
V60-modified	0.75 in.<t<=1.5 in.	60	75	22	(2)(5)	0.25	1.35				0.02		0.04	0.05	
V60-modified	1.5 in.<t<=2.5 in.	60	75	22	(2)(5)	0.25	1.35	0.15-0.30			0.02		0.04	0.05	
V65	t<=0.375 in.	65	80	15	(1)	0.22	1.25	0.3			0.02		0.04	0.05	N .015 max
V65-modified	0.375 in.<t<=1.5 in.	65	80	21	(2)(5)	0.25	1.35				0.02		0.04	0.05	
V65-modified	0.75 in.<t<=1.5 in.	65	80	21	(2)(5)	0.25	1.35	0.15-0.30			0.02		0.04	0.05	
V75-modified	t<=1.0 in.	75	90	19	(2)(5)	0.25	1.50	0.15-0.30			0.06-0.11		0.04	0.05	
United States Steel															
EX-TEN	Wide flange shapes	42	63	24	(3)	0.22	1.35	0.3			0.02				0.01Nb
		45	60	25	(3)	0.22	1.35	0.1			0.02				0.02 Nb
		50	65	22	(3)	0.26	1.35	0.1			0.02				0.01Nb
		55	70	20	(3)	0.25	1.35	0.1			0.02				0.02 Nb
		60	75	18		0.26	1.35				0.02				0.01Nb
		60	75		(5)(6)	0.25	1.50	0.5			0.02		0.04	0.05	
		65	80	16		0.26	1.35				0.02				0.01Nb ,
		70	85	14	(3)	0.26	1.35	0.4							
		42	60	24	(4)	0.21	1.35	0.3			0.02				0.01Nb &
CON-PAC 80		80	100	20	(3)	0.18	1.25	0.3	0.15	0.15					Mo 0.04 B 0.001
CON-PAC 90		90	110	20	(3)	0.18	1.25	0.3	0.15	0.15					Mo 0.04 B
CON-PAC M		75	90	22	(3)	0.18	1.30	0.4							
COR-TEN A		50	70	19	(3)	0.10	0.40	0.5	0-0.65	1		0.4	0.12	0.05	Ti 0.02-0
COR-TEN B		50	70	19	(3)	0.1-0.19	0.9-1.25		0-0.65	0.4-0.65	0.02-0.1	0.25-0.4			
COR-TEN C		60	80	16	(3)	0.12-0.19	0.9-1.35		0-0.65	0.4-0.7	0.04-0.1	0.25-0.4			

Notes
(1) from Alloy Digest #267
(2) Correspondence from R. Symes (PC&F) to James White (SHCR)
(3) Woldman's 7th edition
(4) ADUSS 02-2408 Sept 1967
(5) Specific to PONYA steel contract with PC&F
(6) PONYA Allowed variation 0.75" < t <= 1.5"

Table E–15. Chemistry and mechanical property data for a Fuji Steel plate and a Colvilles plate used for core columns.

Description	F_y (ksi)	TS (ksi)	Elong (%)	C (%)	Mn (%)	Si (%)	Ni (%)	Cr (%)	V (%)	Cu (%)	P (%)	S (%)	Other	Source
12.6 ton A 36 plate 3 in. by 65.5 in. by 453.75 in. Rolled at Hirohata works, Fuji Steel; tested August 5, 1969.	38.4	64.9	32	0.2	0.96	0.2	ND	ND	ND	ND	0.013	0.008		(a)
Chemical analysis of a 6 in. by 52 in. by 18 ft 0.75 in. Colvilles A 36 plate Heat H218 Slab 1804H by Materials Testing Laboratory, Los Angeles, CA, February 2, 1968.	ND	ND	ND	0.2	0.99	0.3	0.2	<0.01	0.005	0.2	0.017	0.035	0.01 Mo, 0.02 Co	(b)

a. Morris 1969.
b. Walton 1969.
Key: C, carbon; Cr, chromium; Cu, copper; Elong, elongation to failure; F_y, specified minimum yield strength; ND, not determined; Mn, manganese; Ni, nickel; P, phosphorus; S, sulfur; Si, silicon; TS, tensile strength; V, vanadium.
Note: Compositions expressed as mass fractions.

Table E–16. Databases searched for WTC information.

Database	Query	Earliest year covered
Cambridge Scientific Databases: Metadex, Weldasearch	Yawata World Trade Center	1966 (Metadex) 1967 (Weldasearch)
OCLC FirstSearch Database: WorldCat	Search for steel periodicals—used to identify possible additional sources of information Search for library holdings of contemporaneous steel catalogs. Also searched on Alibris for used steel data sheets.	19th century
American Society of Civil Engineers Database www.pubs.asce.org	World Trade Center, Yawata, Stanray, Pacific Car and Foundry: no useful information	1973

The LERA archives contain several examples of steel substitutions from other mills. Because the sum of the Colvilles and Fuji contracts, 32,000 tons (Warner 1967 §), is larger than the PONYA value of the contract, 31,100 tons (Feld 1971), these were probably isolated, uncommon occurrences. They are not relevant for estimating steel properties. The documents were probably retained because they documented substitutions that required authorization by PONYA. These records include mill test reports for a single A 7 plate purchased from Crest Steel and rolled by Nippon Kokan Steel (Fukuyama, Japan) (Tarkan 1969 §) and eight A 36 plates with 3.25 in.>*t*>7.25 purchased from Lukens Steel (Morris 1969b §). A report (Yamada 1967 §) of the first shipment of plates from Japan lists the plates as being A 36 and A 572 grade 42. The appearance of A 572 is notable, because it is not listed in the PONYA steel contract list of steels that could be used without requesting formal approval. The document is not a mill test report, however, so it not completely certain that the 42 ksi plates were actually supplied to A 572. It is also possible that documents authorizing the use of steels meeting A 572 have not survived.

Core (Rolled Wide flange Shapes)

Montague-Betts Steel fabricated all the rolled WF shapes for the core columns as well as all the beams in both towers above floor 9. These rolled shapes represent a significant fraction of the total core columns in the fire and impact zone. Above floor 80 in WTC 2, more than half of the core columns were WF shapes, and above floor 94 in WTC 1, 43 of the 46 columns are WF shapes. The plans called for steels with 36 ksi, 42 ksi, 45 ksi, and 50 ksi minimum yield strengths, but very few of the rolled shapes used the 45 ksi or the 50 ksi material. Various sources (Davis 2002 †; Yawata 1969 †) confirm that Montague-Betts purchased about 12,000 tons (of a total contract of 25,900 tons) of A 36 and A 441 wide flange shapes from Yawata Iron and Steel, Sakai Works. An additional 1,200 tons came from Dorman-Long, Lackenby Works, Middlesborough, England (Gallagher 1968 §; Goode 1967 §). Given the size of the Yawata contract, it is likely that it represents the majority, if not all, of the WF core columns. Because Yawata engineers felt that the “A 441-modified” composition was protected by a U.S. Steel patent (Clarkson 1967 §), they also obtained permission to supply high-strength steel to different “A 441-modified” composition (see Table E–3) with 0.2 percent to 0.4 percent mass fraction added Ni (White 1967b §). Whether this approval represents a complete substitution of a Yawata specific alternate “A 441 modified” for the original A 441 modified, or simply an alternate specification for use in limited instances, is unknown. Montague-Betts CEO William Davis (2002 †), who worked on the project, confirmed that Montague-Betts also purchased steel from Bethlehem and U.S. Steel, the only two domestic mills that produced 14WF rolled sections heavier than 87 lb/ft (AISI, 1973). To date, NIST has found no mill records for chemistry or mechanical properties for any of the column steels used in the Montague-Betts contract. See Table E–15 for the search details.

E.5.3 Recommended Values for Mechanical Properties

From the data NIST has recovered from various sources, it is possible to recommend values for estimated yield strength for the various steels for use in analyzing the performance of the buildings. These sources include mill test reports of WTC steels from corporate archives and contemporaneous studies of the properties of plates and shapes of structural steels. These estimates should be confirmed with results of mechanical tests on recovered steel, because they include assumptions about general steelmaking practices that may not have been employed for the specific steels in the WTC.

Central to the estimation of properties is the data from the mill test report that accompanies every piece of steel sold to ASTM International structural steel specifications. In this report, the steel mill attests to certain measures of the quality and properties of the steel supplied. To understand the steel mechanical properties, it is important to recognize the limitations on the information contained in the mill test reports.

Most of the characterization for structural steel for buildings is conducted on a per heat (or ladle) basis. A heat of steel weighs up to several hundred tons and represents the unit at which the steel mill modifies the chemistry for the intended application. Heats of molten steel are poured into ingots to solidify. After solidification and homogenization, the steel mill rolls the ingots into plates and structural shapes. A single heat may supply the steel for many ingots, and a single ingot can supply the steel for many plates or shapes.

For structural steel intended for buildings, both in the WTC era and now, the mill test report contains the results of a single chemical analysis of the steel, taken before the ladle of molten steel is poured into the ingot to solidify. It also contains the results of one or two tension tests, depending on the size of the heat, to evaluate the yield and tensile strengths of the steel. Yield strength is the stress at which the steel first begins to deform permanently, rather than elastically. Buildings are designed so that the stresses do not exceed the yield strength of the steel. Tensile strength represents the maximum stress the steel can carry. The test specimen does not necessarily come from the plate purchased, nor is it likely that it originates from a plate of identical thickness to the one purchased. Therefore, the properties of the plate may differ slightly from the properties that the mill test reports. In essence, the mill test report is a quality control tool. It represents a check that the properties of that production of plates or shapes are in the range that they should be. The mill test report is not the average of a collection of tests, nor is it a guarantee that the entire plate or shape has a yield strength or chemistry that would meet the specification.

During the 1960s and 1970s, several studies (AISI 1973; Galambos 1976; Alpsten 1972) characterized the variability in properties of steels supplied to various standards. These studies asked the question, "If I buy a 36 ksi steel, what is the mean value of the yield strength of the plates that the steel mill supplies to me?" They answered this question by examining thousands of mill test reports, but not by doing independent product testing. Because the tension test to certify the mechanical properties is conducted near the end of the production process, scrapping a heat of steel because it did not meet the intended specification is undesirable. Thus, steel mills generally strive to make steels in which the strength exceeds the intended specification. Typically, the yield strengths in the mill test reports exceeded the specified minimum values. The exact value depended on the value of the yield strength specified in the standard, and whether the steel was supplied as plates or shapes. Of course, the yield strength in the mill test report will never be less than the standard calls for, because the steel could not have been sold as meeting the standard. The results of these studies are useful in estimating the properties of WTC steels when no other corroborating evidence is available.

A second question that some studies attempted to answer was, "If I buy an A 36 plate (a steel with F_y = 36 ksi), what is the probability that a tensile test that I do on that plate will yield a value less than 36 ksi?" Here, the question is about tests that the user conducts, and the studies attempted to characterize the distribution of strengths, rather than the mean value. The American Iron and Steel Institute commissioned the most complete and relevant of these studies in the early 1970s. It compared the results of subsequent tensile tests to the value listed on the mill test report. The most important conclusion from this study is that it is not uncommon for a product tension test to produce a yield strength that is less than the standard allows. The executive summary of the AISI report shows an example for an A 36 steel (specified minimum yield strength of 36 ksi) where measured yield strength on the mill test report is 38 ksi (i.e., 6 percent over the minimum). Even in this plate, there is a 22 percent probability that a second test will produce a yield strength less than 36 ksi, i.e., below the specified minimum yield strength. Because the distribution of yield strengths is reasonably narrow, there is only a 0.1 percent probability that the test will have a yield strength less than 30 ksi, however. It is likely, therefore, that some tension tests done on recovered steel as part of the WTC investigation will produce yield strengths that are less than the relevant standard called for. The occasional appearance of a low yield strength in tests of recovered steel cannot be interpreted as meaning that the steel was defective, or even that it did not meet the standard to which it was supplied.

Several further corrections must be made in estimating the deformation properties of structural steel for modeling. The three most important of these arise because the tensile test method specified in the standard does not perfectly match real deformation conditions. In the mill tension test, a test specimen is cut from the plate and machined into the proper shape. It is then pulled in tension in a testing machine at a constant, prescribed elongation rate, while measuring the resulting load on the specimen. In contrast, the steel in a building supports loads that can be considered to be quasi-static.

The elongation rates used in the mill tension tests are relatively high, to maximize throughput. ASTM A 370 allows a maximum strain rate of 0.001 s^{-1}, which causes yielding within 5 s for most structural steels. The yield strength of structural steel increases slightly with increasing testing rate. For modeling the static behavior of the building, the relevant strength is not the one measured in the mill test, but instead is the so-called “static yield strength.” This is the strength that would be measured at infinitesimally slow deformation rate, which is naturally the relevant rate for the gravity loads in a statically loaded building. Typically, the static yield strength is 1 ksi to 4 ksi less than the value on the mill test report, as established in extensive studies from the 1960s (Rao 1966; Johnson 1967; Galambos 1976), and methods exist to calculate the expected static yield strength from tests conducted dynamically. These corrections are necessary to estimate properties relevant to the airplane impact conditions.

A second correction that must be made arises from the microstructural behavior of low alloy steels (such as steels specified to A 36 and A 440) near the yield strength. During tensile testing, these steels often exhibit what is known as a yield drop (Fig. E–10). The stress necessary to initiate the first bit of permanent deformation (yield) is larger than the stress necessary to continue the deformation. During the elongation in the test, the load rises linearly until permanent deformation initiates at a single location in the test specimen. Frequently the stress can drop 3 ksi to 5 ksi upon yielding. A localized band of deformation passes through the test specimen, and the load drops to a lower constant value. Because the specimen is tested at constant extension rate, rather than at constant load, the deformation band propagates through the test specimen until the entire specimen has begun to deform permanently. This behavior manifests itself as a region of constant stress deformation known as yield point elongation. For all structural steels with specified yield strengths less than 100 ksi, ASTM standards allow the mill test to report the maximum value of the stress reached before the load drop, called the yield point, rather than the lower, constant value, called the lower yield stress. The yield point phenomena occur only in tests that have uniform stress states. Beams loaded in bending, for example, will not show this sort of stress-strain behavior. For modeling purposes the lower yield stress is the relevant parameter for modeling yield behavior.

For estimating the properties of rolled wide flange shapes, one must correct for variation in yield strength with location from which the test specimen is taken. During the WTC era, but not currently, ASTM standards specified that the test specimen for the mill test report be taken from the web section (in the “cross bar” of an “H” shaped specimen) of the rolled shape, rather than the flange. In typical rolled shapes, however, the flange is the thicker section, and accounts for most of the load-carrying capacity of the column. Because it is thicker, it cools more slowly from the rolling temperature, and generally has a lower yield strength than the flange. Many studies, summarized by Alpsten (1972), have characterized this difference as being 2 ksi to 4 ksi for nominally 36 ksi shapes. It was not uncommon for the yield strength of a flange to be 1 ksi to 2 ksi below the specified nominal value (from the web) for the standard.

There is a similar problem for estimating the yield strength of plates. The steel community has recognized that for a given specified minimum yield strength, thinner plates often exhibit a higher yield strength than thicker plates. Thinner plates have had more hot working and cool faster than thicker plates rolled from the same heat (Alpsten 1973; Galambos 1978). Indeed, the high-strength plates that the surviving Yawata mill test reports describe (Section 0) show this effect, but the low-strength plates do not. The effect is difficult to model, however, because mills can adjust the composition of heats intended for specific thicknesses, while still meeting the chemistry requirements of the standard specification, to keep the actual yield strength close to the specified minimum. In the absence of a well-defined method for estimating the thickness effect, it must be regarded as a source of uncorrectable uncertainty.

Table E–17 lists estimated yield strengths for the relevant steels from the impact and fire zone. It contains two columns. The first, labeled "Estimated mill F_y" is an estimate of the average value of the yield strength that would have been reported on the mill test reports. It is based on surviving mill test reports where they are available, and on literature estimates where no mill test reports have survived. The values were estimated by multiplying the specified minimum value by a constant, k, where $k = 1.12$ for plates (Baker 1969) and $k = 1.2$ for shapes (Alpsten 1972). The second column corrects the estimated average mill test report F_y to the value for the static yield strength, using the correction factor of Rao (1966) of –3.6 ksi. The value for the rolled core shapes is further corrected to the expected yield strength for the flanges (since the mill test reports are for specimens taken from the webs) using the value from the AISI report (1973) of –2.4 ksi.

Floor Trusses

Based on the mill test reports summarized in Table E–11, NIST recommends using $F_y = 58.4$ ksi for angles specified as either A 242 or A 36, and 50.4 ksi for rounds specified as A 242. Based on the conversations with Laclede personnel (Brown 2002), A 36 rounds are estimated to have $F_y = 43.4$ ksi. Table E–17 summarizes these recommendations.

Perimeter Columns and Spandrels

Table E–17 provides the current best estimate of the properties of the Yawata grades for each indicated minimum yield strength. Most important in Table E–17 is the entry that shows that PC&F obtained permission (White 1968b) to substitute $F_y = 100$ ksi (WEL-TEN 80C) material for $F_y = 90$ ksi applications, but not to upgrade any other yield strengths by 10 ksi or larger anywhere else. Documents (Symes 1969a §) from early 1969 indicate that PC&F did not use any $F_y = 85$ ksi steel in the building, so any steel specfied as $F_y = 85$ ksi would have to have been supplied at $F_y = 100$ ksi as well. Nicholas Soldano, PC&F General Manager in 1969 (2002 †), confirmed that they had also been granted permission to substitute 45 ksi steel for 42 ksi. Ronald Symes (2002 †), project engineer for PC&F, confirmed that they followed the 5 ksi yield increments, so with the exception of the $F_y = 85+$ ksi and $F_y = 42$ ksi steels, there would be grades for each yield strength. Estimates of the yield stress use the average values of the plates in the mill test reports NIST has located (for "A 441-modified" with $F_y = 45$ ksi and $F_y = 50$ ksi and WEL-TEN 62 with $F_y = 70$ ksi and $F_y = 75$ ksi). Where no data from mill test reports exist, NIST recommends using the literature value as described above. Estimates for WEL-TEN 80C use the average values for the data found in a literature report (Ito 1965b).

Table E–17. Estimated yield strengths (F_y) for grades of steel above Floor 9.

Grade F_y (ksi)	Estimated Mill test report F_y (ksi) (1)	Estimated static F_y (ksi) (2)	Thickness Range	Steel Source	Notes
Perimeter Column Plates 1, 2, 4 (flanges, exterior of building, and spandrels) –see Fig. E–3)					
36	40.3	35.6		Yawata A 36	(3)
42	56.8	53.2		Yawata "A 441 modified"	(4,5)
45	56.8	53.2		Yawata "A 441 modified"	(5)
50	57.7	54.1		Yawata "A 441 modified"	(5)
55	61.6	58.0	For plates with t<=1.5 in.	Yawata "A 441 modified"	(6) (7)
55	61.6	58.8	For plates with t> 1.5 in.	Yawata WEL-TEN 60	(3) (7)
60	67.2	63.6	For plates with t<=1.25 in.	Yawata "A 441 modified"	(6) (7)
60	67.2	63.6	For plates with t>1.25 in.	Yawata WEL-TEN 60	(3) (7)
65	72.8	69.2	For plates with t>0.5 in.	Yawata WEL-TEN 60	(3) (7)
65	72.8	69.2	For plates with t<=0.5 in.	Yawata WEL-TEN 60R	(3) (7)
70	78.4	74.8		Yawata WEL-TEN 62	(5)
75	84.0	80.4		Yawata WEL-TEN 62	(5)
80	89.6	86.0		Yawata WEL-TEN 70	(3)
85	105.0	101.4		Yawata WEL-TEN 80C	(8) (9)
90	105.0	101.4		Yawata WEL-TEN 80C	(8) (9)
100	105.0	101.4		Yawata WEL-TEN 80C	(9)
Perimeter Column Plate 3 (faces interior of building –see Fig. E–3)					
42	47.0	43.4		Bethlehem V42	(3)
45	50.4	46.8		Bethlehem V45	(3)
50	56.0	52.4		Bethlehem V50	(3)
55	61.6	58.0		Bethlehem V55	(3)
60	67.2	63.6		Bethlehem V60	(3)
60	67.2	63.6	0.75in.<t<=1.5 in.	Bethlehem V60-modified	(3)
65	72.8	69.2	t<=0.375 in.	Bethlehem V65	(3)
65	72.8	69.2	0.375in.<t<=1.5 in.	Bethlehem V65-modified	(3)
75	84.0	80.4	t<=1.0 in.	Bethlehem V75-modified	(3)
Core Box Columns					
36	40.3	36.7		Fuji Steel, Colvilles	(3)
42	47.0	43.4		Fuji Steel, Colvilles	(3)
Core Rolled Columns					
36	43.2	37.3		Yawata + others	(3)
42	50.4	44.5		Yawata + others	(3)
45	54.0	48.1		Yawata + others	(3)
50	60.0	54.1		Yawata + others	(3)
Floor Trusses					
50	62.0	58.4		A 242 and A 36 angles	(6)
36	41.6	38.1	d = 1.09 in. and 1 13/16 in.	Laclede A 36 rounds	(6)
50	54.0	50.4	All other rounds	Laclede A 242 rounds	(6) (10)

Table E–17. Estimated yield strengths (F_y) for grades of steel above the Floor 9 (continued).

Grade F_y (ksi)	Estimated Mill test report F_y (ksi) (1)	Estimated actual F_y (ksi) (2)	Thickness Range	Steel Source	Notes
Notes:					
1	This F_y is the estimated average F_y on that would have been reported on the mill test reports, had they been available. For WF shapes, it represents the value from a specimen taken from the web.				
2	Estimated average mill test report F_y corrected for rate and location effects. For a WF shape, this represents the value appropriate for the flange.				
3	Based on reported literature properties for plates (Galambos 1978:Table 3; citing Baker 1969) and rolled shapes: (Alpsten 1975: Fig. 13; Galambos 1978). Estimated flange F_y reduced by 2.4 ksi (AISI 1973:Table 22).				
4	F_y = 42 ksi steel substituted with F_y = 45 ksi (Soldano 2002 †).				
5	Based on averages from Yawata mill test reports (Symes 1969b §; Barkshire 1969a §; White 1969c §).				
6	Based on Laclede mill test reports (Table E–11) and conversations with Laclede metallurgists (Brown 2002).				
7	Use of A 441-modified vs. WEL-TEN based on White memo (White 1969a).				
8	F_y = 85 ksi and F_y = 90 ksi steel substituted with F_y = 100 ksi (Symes 1969a; White 1968).				
9	Based on typical values from manufacturer reports.				
10	Assumed to be chemically identical to A242 angles.				

Core (Welded Box Columns)

In the absence of any confirming mill test reports, the best estimate of yield strength for the core columns is 12 percent higher than specified value (Baker 1969), also listed in Table E–17.

Core (Rolled Wide flange Shapes)

Given the tonnages of wide flange shapes supplied, it is likely that Yawata supplied all the rolled core columns, but NIST has found no confirming evidence of this. Furthermore, NIST has found no open literature information on chemistry or typical mechanical properties of Yawata rolled shapes. In the absence of mill records or steel mill source identification, the best estimate of the yield strength for the expected average mill test report F_y for core rolled shapes is 20 percent higher than the specified minimum yield strength, as detailed by Alpsten (1972) and corrected for the difference between flanges and webs (AISC 1973), summarized in Table E–17.

E.5.4 Sources of Information

Preliminary searches used open literature sources of information, including trade journals to locate information on the various companies and steels involved in construction. Table E–18 lists the journals examined, and the strategy for locating WTC specific information. As mentioned, Table E–16 lists similar information for the databases and search strategies used to locate WTC information.

Table E–18. Trade journals examined for WTC steel information.

Journal	Search Method
Acier Stahl Steel	1966 to 1972 Tables of Contents.
Civil Engineer-ASCE	1965 to 1973 Index on WTC.
Engineering News Record Also, see compilation volume of all articles published (ENR 1972)	1967 to 1973 Index on WTC, New York City.
The Iron Age	1966 to 1968 Index on Japan, WTC, structural steel, fabricator and steel company name.
Iron and Steel	Page-by-page for 1968 to 1971.
Iron and Steel Engineer	1967 to June 1968 Table of Contents, Dateline column, Industry news column. Index is not topical.
Japan's Iron and Steel Industry 1967-1970	1967 to 1970 page-by-page.
Metal Construction	Page-by-page.
Metal Progress	Page-by-page.
Modern Steel Construction	Tables of Contents.
Nihon Kinzoku Gakkaishi (J. Jap. Inst. Metals)	Cursory, WTC era.
Steel	1966 to 1969 Index on Japan, WTC, fabricator and steel company name.
Transactions of the Iron and Steel Institute of Japan	1965 to 1969 Table of Contents and news pages.
Stahlbau	1966 to 1973 Index under Hochbau.
Steelways	–
Structural Engineer	1966 to 1972 cursory.
Welding Design and Fabrication	Cursory.
West of Scotland Iron and Steel Institute Journal	1966 to 1969 Tables of Contents.

After identifying the fabrication companies, NIST contacted Laclede Steel Corporation, Nippon (formerly Yawata) Steel, PACCAR (formerly Pacific Car and Foundry), Montague Betts, Dovell Engineering, and several former employees of Stanray Pacific and Pacific Car and Foundry. NIST did not attempt to contact fabricators that were only involved in the lower floors (Atlas Machine and Iron Works, Levinson, Mosher, and Drier). Table E–19 summarizes these contacts and information. Most of the information in this report came from the archives of LERA.

Initially, NIST had hoped to find the mill test reports for the steel used, which would have provided complete yield (F_y) and tensile strength and chemistry information for all the steels. Each fabricating company, as part of the quality control program required by their contract with PONYA, supplied this information to Tishman, the general contractor, to SHCR, the structural engineers, and to PONYA. Unfortunately, Laclede, Montague-Betts (Davis 2003 †), PACCAR (Bangert 2002 †) (the new name of Pacific Car and Foundry), SHCR (Magnussen 2002 †), and Tishman (Christensen 2003 †) all confirm that they have no mill test reports from that era.

Table E–19. Sources examined for mill test reports and other construction information, other than the (LERA) archives.

Contact	Background	Result
Laclede Steel Corporation David McGee Larry Hutchison	Laclede fabricated the trusses for the floor panels.	During Nov. 2002 NIST personnel visited Laclede, which shared material from its archive, including two mill test reports.
Ronald Symes Former Chief Engineer, PC&F	PC&F fabricated the perimeter columns.	Symes did not retain any WTC documents relating to steel properties, but he did have information on welding
Nicholas Soldano Former general manager, PC&F	–	Soldano provided information on steel substitutions, but had no WTC documents.
D. Bangert, VP for facilities PACCAR	PACCAR owned Pacific Car & Foundry before selling it in 1974.	PACCAR retained no records relating to any aspect of PC&F
Nippon Steel USA Tomokatsu Kobayashi, VP	Nippon Steel formed by the merger of Yawata and Fuji Steel, which together supplied most of the Japanese steel.	Nippon located several 1960s era data sheets for Yawata WEL-TEN steels, but no mill test reports for steels used in the WTC.
Mitsui USA, Janet Garland	Mitsui imported the steel for PC&F	Mitsui has no WTC records.
Carl Lojic, former president, Joseph Tarkan, former Chief Engineer, Stanray Pacific	Stanray Corp closed its fabricating business in 1969, and has apparently gone out of business.	Neither Lojic nor Tarkan retained any documents from the project.
Corus Construction & Industrial Homi Sethna	Corus (formerly British Steel) owns the works that rolled the thicker plate for the welded core columns.	Corus was unable to locate any records from the WTC era.
Tony Wall, President, Dovell Engineering	Dovell was the detailer for Stanray Pacific.	The Northridge earthquake damaged their building. During clean-up they disposed of all WTC documents.
William Betts, CEO Montague-Betts	Montague-Betts fabricated all rolled shapes above the 9th floor.	Six years after completion, Montague-Betts destroyed, as per company policy, all records relating to the WTC construction.
Marubeni-Itochu Steel Tadashi Yaegashi Chief Administrative Officer	Marubeni-Itochu succeeded Marubeni-Iida, which imported the Yawata steel for Montague-Betts.	"All sales transactions going back to the 1960's have been destroyed"
SGS US Testing Company Rich Franconeri	SGS succeeded US Testing and The Superintendence Co., both of which inspected the Japanese steel.	SGS was unable to locate any documents from that era.
Skilling, Ward, Magnussen, and Barkshire (SWMB); Jon Magnussen, partner	SWMB is the successor to the structural engineering firm that designed the towers.	SWMB retained no WTC records. They transferred everything to LERA. NIST has access to these records.
Tishman Realty and Construction; Linda Christensen	Tishman was the general contractor for the construction.	"[O]ur archive facility has standing orders that any and all files over seven years in age are to be destroyed."

NIST also contacted several of the inspection companies (Franconeri 2003 †) and the steel mills (Sethna 2003 †) and steel importing companies (Garland 2004 †; Yaegashi 2003†), as well as Crest Steel, which some Stanray Pacific communications mention (Steinberg 2002 †). All confirmed that they retained no records relating to steel for the WTC.

NIST investigators located six pages of mill test reports for PC&F in the LERA archives, and several individual mill test reports in the Laclede archives.

E.6 CONTEMPORANEOUS CONSTRUCTION SPECIFICATIONS

Section E.4, Contemporaneous Steel Specifications, traces the sources and grades of steel used to fabricate structural steel components for the WTC towers. This section supplements that by extending further into the construction process, specifically adding information on the fabrication (welding) of components and the erection of the buildings.

E.6.1 Fabrication of the Various Components

Floor Trusses

Laclede Steel manufactured the trusses for the floor panels for both WTC 1 and WTC 2 from steel they made at their mill in Alton, Il. The chords of the trusses were fabricated from hot-rolled angles, while the web was from hot-rolled round bar. The web and the chord angles were joined by resistance welding (Laclede 1969).

Little information is available on the standards used for fabrication of the floor trusses. However, floor joist standards existed since 1929. The AISC Steel Construction Manual (1972) *Standard Specifications for Open Web Steel Joists* specifies that 36 ksi and 50 ksi minimum yield strength steel are permitted in such bar joists, and that “Joint connections and splices shall be made by attaching the members to one another by arc or resistance welding or other approved methods.” A Technical Digest from the Steel Joist Institute (Somers 1980) also confirms the use of resistance welding.

Exterior Wall Columns and Spandrels

The perimeter column panels, fabricated by PC&F, comprise three important sub assemblies: the columns, the spandrels, and the seats. A *Welding Design and Fabrication* article (1970a) describes the fabrication sequence, which began with forming the inside wall of the panels (using a butt joint to link the spandrel plates to the inner column webs), followed by the addition of the flanges and outer web plate of the columns by six simultaneous submerged arc welds. PC&F constructed a 16-station automated production line to keep up with the schedule of 55,800 tons of perimeter column panels between November 1967 and August 1970, an average of 1,400 tons per month.

The construction contract states that the submerged arc electrodes used in the WTC were purchased to the requirements of ASTM Standard A 558 “Specification for Bare Mild Steel Electrodes and Fluxes for Submerged Arc Welding.” This standard was withdrawn in 1969, and was replaced by an equivalent American Welding Society (AWS) Standard A 5.17 “Bare Mild Steel Electrodes and Fluxes for Submerged Arc Welding.” The period 1965 to 1969 was one of transition, during which AWS assumed the responsibility of maintaining the standards for welding filler materials. Because the contract was awarded in 1967, the fabrication was likely started with the requirements of the 1965 version of the

ASTM Standard (ASTM A 558-65T, jointly published by AWS as AWS A 5.17-65T), but later perimeter column panels may have included some minor changes associated with the conversion to the 1969 version of the AWS Standard (AWS A 5.17-69). Distorted columns were straightened in the conventional manner by heating just after column assembly, so any low-strength areas in the steel plates and changes in microstructure should not be interpreted solely in terms of the airplane impact and subsequent fires.

The *Welding Design and Fabrication* article (1970b) further states that PC&F inspected the perimeter column panel welds using either ultrasonic, or visual and magnetic particle techniques.

The inner wall assembly (the spandrels and inner plates of the perimeter column panels) was joined with complete joint penetration welds according to the requirements of AWS D 2.0 “Specifications for Welded Highway and Railway Bridges.” This probably refers to the 1966 version of AWS D 2.0. They may have chosen this standard over D 1.0 “Code for Welding in Building Construction” because, at the time, D 1.0 was limited to steel strengths under 60 ksi (Fenton 1966). AWS D 2.0 specifies various dimension and strength requirements for the assemblies and their welds (e.g., paragraphs 302 and 403). This standard, like most standards, lags the steel technology of the time. Thus, it seems to be mostly designed around the application of fairly old steels, like A 7, A 36, and A 373. However, newer steels, such as the higher strength steels used in the WTC towers, could be used after formal approval.

Once the inner wall was ready, the columns were assembled from outer web plates, butt plates, diaphragm plates, and flange plates (Welding Design 1970a). Once assembled and preheated, the plates were joined in the main fillet weld gantry, a station that made six, 0.75 in (19 mm) fillet welds simultaneously along the length of the perimeter column panel. Then the panel was jacked 90 degrees, and the other six fillet welds were made along the length of the panel. At full production, this gantry laid down 2,900 lb (1,300 kg) of weld metal a day. These large fillet welds started 6 in. (150 mm) from the ends of the columns, so manual welding was used to finish the welding of the ends and to make any repairs.

Core (Welded Box Columns)

Stanray Pacific Corp. fabricated the welded core columns in both buildings above floor 9. Like PC&F, they used large assembly fixtures and triple submerged arc welding stations to achieve high production rates. Review of some of the correspondence generated during the initial stages of the fabrication shows the level of attention to welding and inspection details needed to meet the requirements of PONYA and SHCR as described below.

A September 1967 draft of the contract between PONYA and The United States Testing Laboratory (a third-party inspector) lists the documentation that would be required of the work at Stanray Pacific Corp (White 1967c §). This contract prescribes daily and weekly written reports of components that are accepted, those that are rejected, and a summary of any problems, with copies going both to the construction manager and to SHCR. In addition, a weekly report was sent with all the chemical and physical (mechanical) tests performed. The inspectors checked the various steps from plate delivery (checking heat number, specification conformance and condition), through fabrication (alignment, 100 percent visual inspection of the welds, and selection of regions for non-destructive testing), to final inspection (perpendicularity of milled ends, overall length, cleaning, and marking). PONYA also had a procedure to inspect the steel from all sources. The procedure included double-checking the mill certificates by performing a tensile test and a check analysis on 1 out of 10 heats selected at random (Monti 1967b §). The requirements were still higher for steel with strengths above 50 ksi or from foreign sources. The welding procedures, welders and welding operators were qualified in accordance with requirements of Appendix D of AWS Codes D1.1-66 and D 2.0-66. The welding electrodes for manual metal arc welding conformed to ASTM A 233-64T, E60 and E70 series (also AWS A 5.1-64T). Mild steel electrodes and fluxes for submerged arc welding conformed to ASTM A 588-65T (also AWS A 5.17-65T) and to Section 1.17.3 of the AISC Specification for Structural Steel Buildings.

By October 1967, welders were being qualified, magnetic particle inspector qualification was being discussed (based on a minimum of 40 hours of training), and chemical analysis of the steel was underway (Chauner 1967a). The level of inspector oversight continued to increase until by November 10 "U.S. Testing inspectors are all over the place and recording a lot of information" (Chauner 1967b). The level of attention to detail increased even more after a surprise visit to Stanray by Hugh Gallagher, a PONYA inspector, on November 20, 1967 (Gallagher 1967).

While reading the correspondence, one senses that toward the beginning of the contracts, the various fabricators faced major (and perhaps unexpected) challenges introduced by both the tight production schedule and PONYA and SHCR's strict quality requirements.

Connections (Bolts and Welds)

The Port Authority contract allowed the use of ASTM A 307, A 325, and A 490 fasteners. The WTC Design Standards book (p. DS1-6) calls for the use of ASTM A 325 bolts with no indication of type. According to the standard, they would have therefore been supplied as Type 1. As in the contemporary version of ASTM A 325, Type 1 bolts in 1970 had F_y = 120 ksi for diameters up to and including 1 in, and F_y = 105 ksi for larger diameters. ASTM A 325-70 does differ significantly from ASTM A 325-02 in several ways. In particular, the specification for Type 2 bolts was withdrawn in 1991. ASTM A 325-02 also admits three new chemistries for Type 1 bolts. In ASTM A 325-02, the specification for Type 1 Carbon Steel bolts most closely approximates the Type 1 bolts of A 325-70. Table E–20 compares the chemistry requirements of the two standards. A 325-70 also admits a slightly wider range of acceptable hardness, which is currently in Table 3 of A 325-02.

Table E–20. Comparison of chemistry requirements for ASTM A 325 "Standard Specification for High-Strength Bolts for Structural Steel Joints, including Suitable Nuts and Plain Hardened Washers" between 1970 and 2002 standards

Element	ASTM A 325-70 (% mass fraction) Maximum	ASTM A 325-02 (% mass fraction) Maximum
C	0.27	0.28–0.55
Mn	0.47	0.57
P	0.048	0.048
S	0.058	0.058
Si	–	0.13–0.32

Key: C, carbon; Mn, manganese; P, phosphorus; S, sulfur; Si, silicon.
Note: Data are for product, not heat, analysis. Mechanical property requirements are identical between versions.

Spandrels of adjacent perimeter column panels were attached together with high-strength bolted shear connections. Adjacent spandrels were butted to each other with splice plates on the inside and outside (Fig. E–3). For floors 9 to 107, each spandrel was connected to the splice plates with anywhere from 6 to 32 bolts, depending on design load. Splice plates were all 36 ksi steel regardless of spandrel grade. Bolts for all connections between spandrels conformed to ASTM A 325. Minutes of a May 1967 (Feld 1967a) meeting between PC&F, PONYA, and Koch, state that no A 490 bolts were to be used for the spandrel splice plates, and that only A 325 bolts were to be used there. "Bow-tie" spandrels in trees below the floor 9 were connected with heavy 42 ksi splice plates with A 325 or A 490 bolts in friction connections.

Perimeter columns were bolted via the butt plates to those immediately above and below, with four bolts in the upper stories and six bolts in the lower stories. Other than at the mechanical floors, panels were staggered (Fig. E–4) so that only one third of the units were spliced in any one story. At the mechanical floors, every column contained a splice, and columns were welded together as well as bolted.

Seats for the trusses that supported the floor were welded to spandrels in the perimeter column panels and to channels or core columns at the central core. The trusses were positioned on the seats and held in place with construction bolts until welded to the seats. The construction bolts generally remained in place after welding.

Construction (On-site Assembly)

During fabrication, Karl Koch Erecting Co. used a combination of bolting, shielded metal arc (SMA) welding (E7018), and gas metal arc welding (semiautomatic Fab Co 71 with CO_2 shielding) to join the components (Welding Design 1970b). The E7018 low-hydrogen SMA electrode would likely have been produced to ASTM Standard A 233-64T (also published by AWS as A 5.1-64T), then AWS Standard A 5.1-69 for the later parts of the fabrication. The 3/32 in. (2.4 mm) diameter Fab Co 71 (sic, probably should be FabCO 71, a trademark of Hobart Brothers Company) was an E70T-1 flux cored arc (FCA) electrode and would likely have been produced according to ASTM A 559 (withdrawn in 1969), then AWS A5.20-69. Higher-strength SMA electrodes (ASTM A 316 until 1969, then AWS A 5.5-69) were also permitted by the contract. More than 48,000 lb (22,000 kg) of electrodes were used in each of the towers (Welding Design 1970b). Koch used a combination of visual and ultrasonic inspection on the joints. They estimated that rework would cost three times as much as the original weld, so they inspected early and often to minimize any rework. One reason that rework was so expensive is that some welds took as many as 200 passes, so they wanted to catch any problems before the later passes made access more difficult.

Perhaps the most common construction standard for buildings of the period was AWS D 1.0 "Welding in Building Construction" (Fenton 1966). This document was subject to frequent revisions by the responsible committee. Some versions that may have been specified for parts of the WTC towers were the versions published in 1966, 1967, and 1968. The 1967 and 1968 revisions addressed issues such as the details on the use of multiple-electrode submerged arc welding, more requirements on qualification of the welders (especially tack welders), and the addition of radiographic inspection. Many of these revisions may have been driven by the needs of the WTC design. Because the D2.0 code referenced in the discussion on fabrication of perimeter column panels above only covers the use of submerged arc and shielded metal arc welds (unless through special application of Section 5), use of D1.0 (specifically through the use of Section 502) might have been the easiest way to cover the use of FabCO 71 electrode.

Incidentally, the apparent misspelling of FabCO 71 in one of the references points out the problem of inconsistencies in some of the references. The likely explanations include both authors' faulty memories of some details, but also changes that occurred after an article (perhaps based on the near-term construction plans) went to press. An apparent example of the later case involves the plan to use electroslag welding to fabricate the "trees," the branching columns that formed the transition from the 10 ft (3 m) spacing of columns in the lobby area to the 40 in. (1 m) spacing of columns for all the upper floors. Gillespie's book (1999) describes the fabrication of these trees by electroslag welding. However, Koch's book (2002) describes their inability to get the electroslag process operating under field conditions (in a location described as the "belly band," halfway up between the front doors and the branching of the trees), so they welded all these large joints manually.

Examination of the perimeter columns shipped to NIST revealed arc welds at the ends of the trusses, where they were attached to the columns during erection. These welds supplemented the bolt attachment at the seats, and were probably produced by gas metal arc or shielded metal arc electrodes.

E.7 REFERENCE LISTS

E.7.1 References from Publicly Available Sources

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The text identifies references that are private communications or unpublished works that are not bound by any material transfer agreement with the symbol, †. All contemporaneous memoranda referring to Laclede are from the Laclede archives in Alton, Il. NIST obtained Yawata documents from Nippon Steel USA, New York office.

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E.7.3 References from Nonpublic Sources

The text denotes with the symbol § references that are private communications or unpublished works bound by material transfer agreements.

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Morris, R. E. 1969. Letter to James White, SHCR, with attached mill test report for Fuji Steel Plate that appears in other documents. September 10.

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Walton, W. E. 1968. Letter to Malcolm Levy, PONYA, with attached ultrasonic and metallurgical report (Magnaflux Corp) on plate of “British” (i.e., Colvilles) steel. Details chemical analysis, weld quality and (poor-quality) micrographs. February 8.

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White, James. 1967c. Memo to PONYA. September 1.

Yamada, H. 1967. Memo from Marubeni-Iida, the firm that imported the Fuji steel for Stanray Pacific, to PONYA, listing 1,441 tons of A 36 and A 572 grade 42 steel plates up to 3 in. thick. Plates are 36 ft 3/4 in. long and 36 in. to 94 5/8 in wide. July 18.

Attachment 1
STEEL COMPANIES INVOLVED IN THE WORLD TRADE CENTER

Most of the fabrication firms that worked on the steel for the World Trade Center (WTC) are no longer in business. This section summarizes the contributions of each of the major steel firms involved, and their current status.

1.1 ATLAS MACHINE AND IRON WORKS

Contract WTC212

Atlas fabricated the 27 in. by 32 in. perimeter box columns, spandrels, and X-bracing below the 4th floor (Feld 1971) (13,600 tons). This contract was the first major use of electroslag welding in the United States (Feld 1971).

Most recent address:
Atlas Machine and Iron Works
13951 Lee Highway
Gainesville, VA 22065
Arthur X. Miles, President and Registered Agent

The Virginia Corporation Commission indicates that Atlas went out of business in 1999. The address is at the intersection of US 29 and I-66 in Gainesville, Virginia. A drive past the site on November 24, 2002, confirmed that it is inactive.

1.2 DRIER STRUCTURAL STEEL

Drier fabricated the foundation load distribution system (base plates and grillages) (Feld 1971). No information is available on its current status.

1.3 DOVELL ENGINEERING

Dovell was the detailer for Stanray Pacific. (The detailer makes the detailed fabrication drawings of the columns and beams.)

Current Address
9901 Paramount Blvd Suite 202
Downey, CA 90241
562-927-4770

Dovell President Tony Wall (Wall 2002 †) indicated that the former owner, who was active in the WTC project, is not in a position to provide details of the WTC project.

1.4 GRANITE CITY STEEL

Granite City fabricated the electrical/telephone ducts and the floor deck system (Feld 1971).

1.5 HOBART BROTHERS CO./ITW

Hobart provided the electrodes used for on-site erection by Karl Koch Erecting Company

ITW purchased it several years ago, but it still maintains its headquarters in Ohio.

Current Address
400 Trade Square East
Troy, OH 45373
www.hobartbrothers.com

1.6 KARL KOCH ERECTING COMPANY

Koch erected the towers (McAllister 2002).

Skanska, an international construction company, purchased Koch in 1982. Karl Koch III is still alive, and recently wrote a book "Men of Steel" that includes information about the project (Koch 2002).

1.7 LACLEDE STEEL CO.

Contract WTC226

Laclede fabricated the trusses for the floor system (Feld 1971). It entered bankruptcy on November 30, 1998, but re-emerged in January 2001 only to reenter bankruptcy again July 27, 2001. Currently a group of former employees has purchased the assets.

Current address
211 N Broadway
St Louis, MO 63102
314-425-1400

1.8 LEVINSON STEEL

Contract WTC230

Levinson fabricated the below-grade area (12,000 tons of 14WF sections), the plaza, and the damping units (Feld 1971). Metals USA acquired Levinson in March 1998. The www.metalsusa.com Web site does not list any information on Levinson, however. Metals USA went bankrupt in August 2001, but was reported to be emerging from bankruptcy on October 31, 2002.

1.9 MONTAGUE-BETTS

Contract WTC226

Montague-Betts fabricated all the rolled columns and beams in the core of both towers, 25,900 tons (Feld 1971). Their contract was for "all rolled columns and beams, including cover-plated sections throughout both towers…including horizontal trusses on 2nd floor... and exterior wall steel above 107th floor and the weldments for supporting future T.V. masts," (Feld 1971).

Current address of former owners:
1619 Wythe Rd
PO Box 11929
Lynchburg, VA, 24501
William Davis, President
434-522-3200

William Davis (2002 †), son of the founder (now age 91), confirmed that they furnished all the rolled beams for the core of both towers as well as the antenna base. Montague-Betts closed its steel fabrication business in 1992, though the family still owns a majority interest in one steel fabrication business in Lynchburg.

1.10 MOSHER STEEL

Mosher fabricated the elevator core framing system to the 9th floor (Feld 1971) (13,000 tons).

Trinity Industries acquired Mosher Steel in November 1973, which is still in business. Rodengen's (2000, p. 58) book has only a partial chapter on Mosher, and only notes that it "shipped more than 13,000 tons of steel for the lower portion…"

1.11 PACIFIC CAR AND FOUNDRY

Contract WTC214

Pacific Car and Foundry fabricated the perimeter column panels from the 9th to 107th floors (Feld 1971), 55,800 tons. It changed its name to PACCAR in 1972. As PACCAR, they manufacture Kenworth and Peterbilt trucks.

Contact Info:
PACCAR Inc.
777 106th Avenue N.E.
Bellevue, WA 98004
Telephone 425-468-7400; Fax 425-468-8216

Dick Bangert (2002 †) (VP for facilities) confirmed that PACCAR sold the structural steel division "years ago" and has no records from that business. Ron Symes (2002 †), chief engineer for PC&F during the WTC construction, confirmed that the division was sold in 1974. The PACCAR corporate history (Groner 1981) reports that the WTC contract was not profitable for the Structural Steel Division because it had estimated the job based on shipping the completed sections by barge to New York, but were unable to obtain insurance to do that. As a result, they had to ship by rail, which nearly doubled the shipping costs. These losses, plus concessions to settle strikes in 1969 and 1970 sent the division into a decline from which it never recovered. Nicholas Soldano (2002 †), former general manager, remembered that the metals recycler Schnitzer bought the Seattle property where the perimeter columns were fabricated.

1.12 PITTSBURGH-DES MOINES STEEL

Pittsburgh-Des Moines (PDM) fabricated the perimeter bifurcation columns from the 4th to the 9th floors, 6,800 tons (Feld 1971). The bifurcation columns are also referred to as the “tuning forks” or the trees. Civil Engineering (1970) reported that Lukens Steel “supplied seven basic grades of carbon and alloy plate steels for use in the welded ‘trees… steels meet yield strength requirements from 36,000 to 65,000 min psi.” Reliance Steel and Aluminum (www.rsac.com) acquired PDM Steel Service Centers in July 2001.

1.13 STANRAY PACIFIC CORP

Stanray Pacific fabricated the welded core box columns and built-up beams above the 9th floor, 31,100 tons (Feld 1971).

The California business portal report indicates that the company is no longer in business (Record # C0388500). According to its annual reports, the parent corporation, Stanray (1969, 1970), decided to close the Stanray Pacific (based in Los Angeles, California) subsidiary during 1969. Joe Tarkan, Stanray Pacific chief engineer for the WTC contract, confirmed this (Tarkan 2002 †).

Attachment 2
NOTES ON ASTM STANDARDS FOR STRUCTURAL STEEL

This attachment summarizes the important aspects of the relevant standards that governed the structural steel supplied and compares contemporary (current) and contemporaneous (1960s) standards. In general, the differences between the contemporaneous and contemporary standards are minor, and are usually additions or deletions of individual steel chemistries or small changes in test protocol. However, because of these changes, it is possible that a steel that met a construction-era version of a standard might not meet that same standard today, because the chemistry or elongation requirements have changed. This statement should not be interpreted to mean that the steel in question as used was unsuitable, however.

The ASTM International defines a standard as "a document that has been developed and established with the consensus principles of the Society and that meets the approval requirement of ASTM procedures and regulations." A standard may be a document that specifies the properties of a material, as in the case of steel standard specifications such as A 36. Other standards are test methods that define the way in which the properties in a specification must be measured. An example of this is A 370, which defines the test methods for establishing the strength of steel. An important aspect of ASTM standards is that they are consensus documents, established by committees where membership is open to all individuals and organizations. Except for military construction, the U.S. Government does not establish structural steel standards for the industry. Instead, the ASTM committees that establish steel standards are required to have balanced membership among producers, users, and independent experts. The standards they produce allow the producers and consumers to efficiently specify materials, without requiring them to include all possible properties and methods in a contract. This report, to avoid confusion with other uses, will use the term "standard" to refer to all ASTM documents, regardless of their status as Specifications, Test Methods, Terminology Standards, or Practices.

The ASTM issues its standards annually in a multi-volume "Annual Book of ASTM Standards," but revises an individual standard only when the committee in charge sees a need. ASTM does require that standards be reauthorized every five years, even if they have not been revised. The designation of a standard, for example A 36-66, comprises two parts. The first (for example "A 36") is a shorthand for the general chemistry and mechanical property requirements, in the case of structural steels. Following the designation is a two digit number denoting the most recent revision year of the standard (for example "-66," which denotes a substantial revision in 1966). The steel fabrication contracts stipulated that the appropriate standards were those in effect in September 1966. In some cases the relevant standard was not revised in 1966, and so bears a prior year revision mark.

An individual ASTM standard does not contain all the information to uniquely characterize the steel. Instead, there is a "chain of standards" that defines the properties of the steel. The WTC steel contracts allowed the use of steels that conformed to certain ASTM standards (e.g., A 36, A 242, A 441, A 514). These standards define the mechanical and chemical properties of the steels, but in turn reference other standards that define how those properties shall be measured. For instance, all the steel standards, then and now, require that the steel conform to ASTM A 6 ("Requirements for Delivery of Structural Steel"), which specifies, among many things, the dimensional tolerances of plates and rolled shapes. The rest of this section describes the minor differences between the ASTM standards that governed structural steel used for construction of the WTC, and those that exist today.

2.1 A 6-65 VERSUS A 6-02

ASTM A 6-65 "Standard Specification for General Requirements for Delivery of Rolled Steel Plates, Shapes, Sheet Piling and Bars for Structural Use," specifies the tolerances for structural steel. Both versions specify that mechanical properties shall be determined in accord with A 370. At some point ASTM editorially amended the title of the standard to its present version "Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling." A 6-02 is a much longer and more complex document than A 6-65.

For determining mechanical properties, A 6-65 specifies the size and shape of test specimens, while A 6-02 references (similar) specimens in A 370. Table 2–1 summarizes the significant differences in determining mechanical properties between A 6-65 and A 6-02. Two differences are particularly significant. A 6-65 specifies that steels shall be tested in the rolling direction (longitudinally), but A 6-02 requires most plates to be tested in the transverse direction. The location of specimens from shapes is also different: in A 6 they are always taken from the web, but in A 6-02 for the large shapes used for columns, the specimen is taken from the flange. Typically, because the flange is thicker than the web, the flange yield stress will be less than the web yield stress (Alpsten 1975, AISC 1974). In summary, to conform to A 6, most A 36 specimens for the WTC projects would have been tested full thickness. Core column steels over 1.5 in thick would have been permitted to use the round 0.5 in. (12.7 mm) diameter specimen because of their thickness. Thin perimeter column plates would have been tested full thickness.

In terms of chemistry, A 6-65 does not require any special method be used to determine the chemistry of the steel. In contrast, A 6-02 specifies that chemistry is to be determined in accord with ASTM A 751 ("Standard Test Methods, Practices, and Terminology for Chemical Analysis of steel products"). A 6-65 requires the mill test report to state the percentages of carbon, manganese, phosphorus and sulfur, as well as any element required by the individual standard. To that list, A 6-02 adds silicon, nickel, chromium, molybdenum, copper, vanadium, and niobium (referred to as columbium in the U.S. steel industry). The chemistry requirements have also been moved between standards. A 6-65 specifies two types of chemical analysis. The so-called ladle analysis is conducted at the steel mill on the steel before rolling. "Check" or product analyses are conducted on representative samples taken from the finished structural product All of the contemporaneous steel standards (e.g., A 36-66, etc) specify compositions determined in both ladle and check analyses, where the check analyses are slightly relaxed from the ladle analyses. In contemporary standards, the check analysis values (now called product analysis) have been removed from the standards to a single table in A 6-02. A spot check of the some of these for A 36-01 and A 242-01 indicates that the values listed in Table B of A 6-02 ("Permitted Variations in Product Analysis") are identical to the values listed under check analysis in the contemporaneous steel standards of the 1960s.

Table 2–1. Differences in specimen sampling requirements between A 6-65 and A 6-02e.

Shape	Specimen Location	Orientation	Specimen type and size
		A 6-65	
Beams, channels or zees	Web (Sec. 6.4)	Longitudinal (Sec. 6.3) Full-thickness (Sec. 6.5)	
Shapes or plates except alloy steel plates over 1.5 in. thick	Generally specified as corner in product specifications, but no apparent restrictions on position within thickness for non-full-thickness specimens.	Longitudinal (Sec. 6.3) Full-thickness (Sec. 6.5)	18 in. long specimen with 8 in. gage length or straight-sided specimen. For t>1.5 in. can use 0.505 in. diameter round specimen with 2 in. gage length
Alloy steel plates 0.75<t<=1.5 in.	“	Longitudinal	May use a round specimen with d = 0.505 in. very similar to A 370 02 Fig. 4
Alloy steel plates >1.5 in. thick	“	Longitudinal	May use a round specimen with d = 0.505 in. very similar to A 370 02 Fig. 4
		A 6-02e	
Shapes: $t \leq 0.75$ in.	If w>6 in. from the flange, otherwise from the web (Sec. 11.3.2)	Full thickness (Sec. 11.5.1) Longitudinal (Sec. 11.2)	8 in. or 2 in. gage length flat specimen A 370 Fig. 3
Shapes: t > 0.75 in.	“	“	0.5 in. diameter round specimen (A 370 Fig. 4) or full thickness flat specimen (A 370 Fig. 3) if desired
Plates: $t \leq 0.75$ in.	Corner (Sec. 11.3.1)	Full thickness (Sec. 11.5.1) Transverse if w>24 in. (Sec. 11.2)	8 in. or 2 in. gage length specimen A 370 Fig. 3
Plates: t>0.75 in.	“	“	0.5 in. diameter round specimen (A 370 Fig. 4) or full thickness flat specimen (A 370 Fig. 3) if desired

Steel products have a natural variability in mechanical properties. Because the mill test for yield and tensile strength represents only one or two specimens, it is possible that tests conducted on the finished product may yield properties that differ from the mill test report. Sometimes these tests will yield values that are lower than the appropriate standard specification. Should a specimen taken as part of the investigation exhibit a yield point or strength less than the applicable standard, this does not imply that the steel as a whole did not meet the standard. A 6-02 makes this quite clear:

> X2.1 The tension testing requirements of Specification A 6/A 6M are intended only to characterize the tensile properties of a heat of steel for determination of conformance to the requirements of the material

> specifications. These testing procedures are not intended to define the upper or lower limits of tensile properties at all possible test locations within a heat of steel. It is well known and documented that tensile properties will vary within a heat or individual piece of steel as a function of chemical composition, processing, testing procedure and other factors. It is, therefore, incumbent on designers and engineers to use sound engineering judgement (sic) when using tension test results shown on mill test reports. The testing procedures of Specification A 6/A 6M have been found to provide material adequate for normal structural design criteria.

Thus, the results of contemporary tension tests on WTC steels can only be used to assert that the steel in question is of a quality that could reasonably be expected to meet a given ASTM standard. It may be that an individual tension test might result in a measured yield point less than that acceptable in the standard. As long as the measured yield point is close to the specified minimum, the steel in question probably met the requirements of the standard.

2.2 A 370-67 VERSUS A 370-02

ASTM A 370, "Standard Methods and Definitions for Mechanical Testing of Steel Products," controls the methods used for mill acceptance testing of heats (or plates) of steel. Aside from minor revisions in 1966, to incorporate A 443 ("Method of Notch Toughness of Turbine and Generator Steel Forgings") A 370-67 is identical to A 370-66.

By and large A 370-67 and A 370-02 are very similar. Although the section numbers are different, much of the text is unchanged over the past 35 years. Table 2–2 summarizes the important differences between the two documents as they relate to tensile testing. As long as the loading rates are specified as the maximum rate in A 370-02, the test results will also meet A 370-67.

2.3 E 6-66 VERSUS E 6-99$^{\varepsilon 2}$

ASTM E 6, "Standard Terminology Relating to Methods of Mechanical Testing," defines the technical terms used in the various mechanical testing standards. The definitions of elastic limit, elongation, gage length, Poisson's ratio, proportional limit, reduction of area, and tensile strength are word-for-word identical in the two standards. The definitions of yield point and yield strength differ textually, but not in spirit. Table 2–3 summarizes the textual differences between the two versions.

Table 2–2. Differences between A 370-67 and A 370-02.

A 370-67	A 370-02
Section 10d suggests that tests defined in terms of strain rate are acceptable, but not feasible with production grade equipment	Section 7.4. specifically allows tests defined in terms of strain rate
No such language.	Note 2 specifically disallows tests in load control
No restriction on minimum extension rate for tests	Section 7.4.1 requires that minimum speed for testing shall not be less than one-tenth of the maximum rate for determining yield point or yield stress
No such language	Allows maximum testing rate to be less than 100,000 psi, min
	Section 13 (Determination of yield point) has different language but is similar in spirit
Section 12(b)(1) specifies a so-called "divider method" for measuring yield point.	Absent from Section 13
In section 13 (determination of yield strength) the order of the methods is reversed.	
In Section 13 the extension under load method may "be used only when the product specification permits."	No such recommendation
Section 13 allows the yield point to be reported as the yield strength if the load drop occurs before the specified offset is reached.	No such allowance
A Class B1 extensometer is required for all offset method determinations of yield strength.	Section 13.2.2 allows the use of a Class B2 extensometer for determining yield strength if the offset is ≤0.2 %

Table 2–3. Differences in the definitions of yield point and yield stress in ASTM E 6.

E 6-66	E 6-99$^{\varepsilon 2}$
Yield Point	
"[FL^{-2}] the first stress in a material, less than the maximum attainable stress, a which an increase in strain occurs without an increase in stress Note—It should be noted that only materials that exhibit the unique phenomenon of yielding have a 'yield point.'"	"YP [FL^{-2}], n – a term used, by E 8 and E 8M, for the property which is now referred to as upper yield strength." "Upper yield strength UYS, [FL^{-2}], n –in a uniaxial test, the first stress maximum (stress at first zero slope) associated with discontinuous yielding at or near the onset of plastic deformation."
Yield Strength	
"[FL^{-2}] The stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain. the deviation is expressed in terms of strain." Notes on the offset and total extension under load methods follow.	"YS or S_y [FL^{-2}], n–the engineering stress at which, by convention, it is considered that plastic elongation of the material has commenced. This stress may be specified in terms of (*a*) a specified deviation from a linear stress-strain relationship, (*b*) a specified total extension attained, or (*c*) maximum or minimum engineering stresses measured during discontinuous yielding." Discussion of the offset and specified extension under load methods follows, as well as discussion of upper and lower yield strengths, differences between the results of the two methods and of rate effects.

2.4 A 36-66 VERSUS A 36-01

All chemistry requirements of Table 2 are identical for carbon, manganese, sulfur, and phosphorus. A 36-01 requires that steel for plates and shapes other than group 1 be killed or semi-killed, while A 36-66 only requires "where improved notch toughness is important, the material may be specified to be silicon killed fin grain practice." As a consequence, A36-01 limits silicon to 0.4 percent for all sizes of plates and shapes, while A 36-66 only specifies silicon for plates with t>1.5 in. A 36-66 requires the material to pass a bend test defined as "The bend test specimens shall stand being bent cold through 180 deg without cracking on the outside of the bent portion, to an inside diameter which shall have a relation to the thickness of the specimens as prescribed in Table IV." The bend test is absent from A 36-01. A 36-66 requires that the steel be made by "open-hearth, basic-oxygen, or electric-furnace" A 36-01 has no such requirements. The elongation requirements differ between the two standards. A 36-66 has relaxed elongation requirements for thicker plates that are missing from A 36-01, and does not differentiate between plates and shapes for elongation requirements. Other than these differences, the standards are identical.

2.5 A 242-66 VERSUS A 242-01

The yield and tensile requirements are unchanged in the two standards, but the chemistry requirements differ substantially. A 242-66 admits high and low carbon variants. A 242-01 admits only a low carbon, low manganese type. During the WTC construction era, A 242 was revised to include the Type 1 variant of A 242-01. Table 2–4 compares the chemistry requirements between the two standards. Another difference is that A 242-01 prescribes the method for determining the atmospheric corrosion resistance, while A 242-66 only states, "If the steel is specified for materially greater atmospheric corrosion resistance than structural carbon steel with copper, the purchaser should so indicate and consult with the manufacturer." The elongation requirements are relaxed for thicker plates and shapes in A 242-66. The current standard also adds some required elongations when specimens with 2 in. gage length are tested. Finally, A 242-01 no longer mandates that steel pass a bend test. Requirements for bend testing are now included as a non-mandatory appendix in A 6-02.

Table 2–4. Differences in chemistry requirements between A 242-66 and A 242-01.

Element	A 242-66	A 242-66	A 242-01 (Type 1)
C (max.)	0.22	0.15	0.15
Mn (max.)	1.25	1.4	1.00
S (max.)	0.05	0.05	0.05
P (max.)	NR	NR	NR
Cu (min.)	NR	NR	0.20

Key: C, carbon; Cu, copper; Mn, manganese; NR, no requirement; P, phosphorus; S, sulfur.
Note: Compositions expressed in % mass fraction.

The A 242 steel that Laclede supplied for the floor trusses would have met the chemistry requirements of A 242-66, but would not meet the chemistry requirements of A 242-01, because of its elevated carbon content. In terms of its load-carrying capacity, these differences are irrelevant, however.

2.6 A 441-66 VERSUS A 572-01

ASTM A 441 was withdrawn in 1989. A 441-66 and A 572-01 are similar in several ways. Both are standards for vanadium-containing steels with minimum yield points greater than those specified in A 36. To some degree it can be argued that A 572 replaced A 441. The carbon, manganese, and silicon levels in both standards are similar but not identical. However, in terms of chemistry, most steels that met A 441-66 would probably meet A 572-01. A 572-01 admits a wider range of minimum yield points in much thicker sections as well, see Table 2–5.

Table 2–5. Differences between A 441-66 and A 572-01.

A 441-66 YP (ksi)	Thickness t (in.)	A 572-01 YP (ksi)
40	4 in. <*t*<=8 in.	
	t <= 6 in.	42
42	1.5 in.<*t*<=4 in.	
	t<=4 in.	50
	t<=2 in.	55
46	3/4 in.<*t*<=1.5 in.	
	t<=1.25 in.	60
	t<=1.25 in.	65
50	*t*<=3/4 in.	

2.7 A 514-65 VERSUS A 514-00A

A 514-65 differs from A 514-00a at dozens of points. Table 2–6 summarizes the substantial ones. Unlike standards such as A 36, which have simple, non-proprietary chemistry requirements, each variant chemistry in A 514 represents a single mill's 100 ksi steel. For instance, Brockenbrough and Johnson (1968) identify A 514 Grade F as USS T1, A 514 Grade B as USS T1 Type A, and A 514 Grade H as USS T1 Type B.

2.8 YIELD POINT VERSUS YIELD STRENGTH

Both E 8 and A 370 distinguish between yield point and yield strength. For steels of interest to the investigation, all standards for steels with yield strength under 80 ksi, whether contemporary or contemporaneous, specify yield point instead of yield strength. ASTM E 6-99$^{\varepsilon 2}$ (Standard Terminology Relating to Methods of Mechanical Testing) defines them as follows:

- **yield point**, YP [FL^{-2}], n – a term used, by E 8 and E 8M, for the property which is now referred to as upper yield strength.

- **upper yield strength**, UYS, [FL^{-2}], n –in a uniaxial test, the first stress maximum (stress at first zero slope) associated with discontinuous yielding at or near the onset of plastic deformation.

Table 2–6. Differences in ASTM A 514-65 and A 514-00a.

	A 514-65	**A 514-00a**
Sampling requirements	One tension test from each of two plates from each lot (Sec. 10.2) Brinell hardness from all plates not tension-tested (Sec. 7.1)	One tension test from every plate in each lot (Sec. 8.1) Brinell hardness may be substituted for plates 3/8 in. and under, with tension test from at least two plates (Sec. 7.2)
Test specimen orientation	No special requirement	Plates over 24 in. wide must be tested in the transverse direction (8.1)
Strength	$t \le 3/4$ in. 115 ksi ≤ TS ≤ 135 ksi 3/4 in. < $t \le 2.5$ in. 115 ksi ≤ TS ≤ 135 ksi 2.5 in. < $t \le 4$ in. 105 ksi ≤ TS ≤ 135 ksi (Table 2)	$t \le 3/4$ in. 110 ksi ≤ TS ≤ 130 ksi 3/4 in. <$t \le 2.5$ in. 110 ksi ≤ TS ≤ 130 ksi 2.5 in. <$t \le 6$ in. 105 ksi ≤ TS ≤ 130 ksi (Table 2)
Elongation in 2 in. (%)	2.5 in.<t≤4 in.: 17 % special elongation reduction allowances for plates under 5/16 in. (Sec. 6.2)	2.5 in. <$t \le 6$ in.: 16 % No such allowance
Chemistry (compositions expressed in % mass fraction)	Admits Types D, G *Type D* 0.13-0.2C 0.4-0.7Mn, 0.035P, 0.04S 0.2-0.35Si, 0.85-1.2Cr, 0.15-0.25Mo 0.04-0.1Ti, 0.2-0.4Cu, 0.0015-0.005 B *Type G* 0.15-0.21C, 0.8-1.1Mn, 0.035P, 0.04S, 0.5-0.9Si, 0.5-0.9Cr, 0.4-0.6Mo 0.05-0.15Zr, 0.0025 Max B	Types D, G absent Admits new types J, K, M, P, Q, R, S, T.
Chemistry	Most S allowables are 0.04 %	Most S allowables are 0.035 %

- **yield strength**, YS or S_y [FL-2], n –the engineering stress at which, by convention, it is considered that plastic elongation of the material has commenced. This stress may be specified in terms of (*a*) a specified deviation from linear stress-strain relationship, (*b*) a specified total extension attained, or (*c*) maximum or minimum engineering stresses measured during discontinuous yielding.

The definitions of yield point and yield strength differ textually, but not semantically, between ASTM E 6-99$^{\varepsilon 2}$ and E 6-66, and are contrasted in Sec. 0 and Table 2–7.

In terms of mechanical properties, it matters little whether yield point or yield strength is specified. Almost certainly the yield point of plain carbon steels (like A 440 and A 36) will exceed the yield strength by only 1 KSI to 4 ksi, because they typically exhibit a yield drop after yielding. Of the relevant standards, only A 514 specifies steel in terms of yield strength. Both contemporary and contemporaneous version of A 36, A 242, A 441, and A 572 specify yield point rather than yield strength. The AISC Manual of Steel Construction (AISC 1973, p.1-3) treats them identically:

> As used in the AISC Specification, “yield stress” denotes either the specified minimum yield point (for those steels that have a yield point) or specified minimum yield strength (for those steels that do not have a yield point).

Table 2–7. Methods for determining Yield Point and Yield Strength in ASTM A 370.

A 370-67	A 370-02
Yield Point	
"Drop of the beam" method Section 12(a)(1)	"Drop of the beam" method Section 13.1.1
Position of the knee Section 12(a)(2)	Position of the knee Section 13.1.2
Total extension under load (at a suggested strain of $\varepsilon = 0.005$) Section 12(b)(2)	Total extension under load (at a suggested strain of $\varepsilon = 0.005$) Section 13.1.3
"Divider method" Section 12(b)(1)	
Yield Strength	
Offset method with no suggested value but with an example that uses $\varepsilon = 0.002$ Section 13(b)	Offset method with no suggested value but with an example that uses $\varepsilon = 0.002$ Section 13.2.1
Extension under load with no required or suggested strain value: "this approximate method be used only when the product specification permits" Section 13(a)	Extension under load with no suggested strain, but with an example that uses of $\varepsilon = 0.005$ Section 13.2.2

A 370-02 permits three different methods for measuring yield point and two methods for yield strength, summarized in Table 2–2. The "drop of the beam" method applies to testing machines that prescribe the loading rate, rather than the extension rate.

Interestingly, neither A 370-67 nor A 370-02 mandates a specific value of the total extension under load determining either yield point or yield stress when using the total extension under load method. It does suggest a value of $\varepsilon = 0.005$, but does so in a nonmandatory note. Furthermore, A 370-67 does not require the mill to report which method it used for measuring yield point. Neither A 6-65 nor A 370-67 has any requirements as to the contents of a mill test report. A 6-02 does have a detailed section on Test Reports, however.

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Appendix F
INTERIM REPORT ON INVENTORY AND IDENTIFICATION OF STEELS RECOVERED FROM THE WTC BUILDINGS

F.1 PURPOSE OF REPORT

The purpose of Project 3 of the National Institute of Standards and Technology (NIST), World Trade Center (WTC) Investigation, Mechanical and Metallurgical Analysis of Structural Steel, is to analyze structural steel available from WTC 1, 2, and 7 to determine the metallurgical and mechanical properties and quality of the metal, weldments, and connections and to provide these data to other investigation projects. (For test plan details, see http://wtc.nist.gov/media/WTCplan_new.htm#proj3.) The properties determined under this project will be used in two ways:

- Properties will be correlated with the design requirements of the buildings to determine if the specified steel was in place in the towers.
- Properties will be supplied for other projects in the Investigation as input for models of building performance.

This interim report is an output of Task 1 of Project 3. Task 1 is defined in the NIST plan as "Collect and catalog the physical evidence (structural steel components and connections) and other available data, such as specifications for the steel, the location of the steel pieces within the buildings, and the specified steel properties."

F.2 SCOPE OF REPORT

The Task 1 report comprises three parts:

- Tower Design – Structural Steel Documents.
- Contemporaneous Structural Steel and Construction Specifications.
- Inventory and Identification of Steels Recovered from the WTC Buildings. This appendix covers part 3; Appendix E presents the structural design of the WTC towers and the specifications used for the steel and construction of the buildings.

Part 1, which is covered in Appendix E of this Progress Report, describes the tower structure and critical structural elements to be characterized in Project 3. This includes the structural design and properties specified by the structural engineers for columns, floor systems, and connections.

Part 2, also covered in Appendix E, describes the contemporaneous (late 1960s era) specifications for various types and grades of steel designated by the ASTM International, the American Institute of Steel Construction, and other national and international organizations. It also includes information from numerous suppliers of the steel for the towers. The structural steel for the towers was supplied through at least a dozen contracts to suppliers and fabricators. Substantial understanding of the consistency, quality, and actual strength of the steel (as opposed to specified minimum values) can be gained if the production practices and quality control procedures used by the various steel suppliers are understood. Practices and data from the numerous WTC steel suppliers have been investigated and are reported for both structural steel and construction practices. In addition, this information has been used to estimate typical mechanical property values for many of the grades of steel. These typical values can serve as a guide for the properties to be inserted into the finite element models of building performance and as a point of comparison for actual properties measured on the recovered steel.

Part 3, covered in this appendix, documents the steel recovered for the WTC Investigation. Approximately 236 pieces of WTC steel were available for study at NIST. These pieces represent a small fraction of the steel examined at the various recovery yards where the steel was sent as the WTC site was cleared.

F.3 BACKGROUND INFORMATION RELATED TO RECOVERY OF WTC STRUCTURAL STEEL

Beginning in October 2001, members of the Federal Emergency Management Agency (FEMA), American Society of Civil Engineers (ASCE), Building Performance Assessment Team (BPAT), members of the Structural Engineers Association of New York (SEAoNY), and Professor A. Astaneh-Asl of the University of California, Berkeley, California, with support from the National Science Foundation, began work to identify and collect WTC structural steel from the various recovery yards where debris, including the steel, was taken during the cleanup effort. Dr. J. Gross, a structural engineer at NIST and a member of the FEMA/ASCE BPAT, was involved in these early efforts.

There were four major sites where debris from the WTC buildings was shipped during the clean-up effort in which the volunteers worked. These were:

- Hugo Nue Schnitzer, Inc., Fresh Kills Landfill in Staten Island, New Jersey;
- Hugo Nue Schnitzer East, Inc., Claremont Terminal in Jersey City, New Jersey;
- Metal Management, Inc., in Newark, New Jersey; and
- Blanford and Co. in Keasbey, New Jersey.

The volunteers searched through unsorted piles of steel and other debris for pieces from the WTC buildings, specifically searching for (McAllister 2002):

- Exterior column panels and interior core columns from WTC 1 and WTC 2 that were exposed to fire and/or impacted by the aircraft;
- Exterior column panels and interior core columns from WTC 1 and WTC 2 directly above and below the impact zones;
- Badly burned pieces from WTC 7;
- Connections from WTC 1, WTC 2, and WTC 7 (e.g., seat connections, single-shear plates, and column splices);

- Bolts in all conditions;
- Floor trusses, including stiffeners, seats, and other components; and
- Any pieces that in the engineers' professional opinion might be useful.

Once identified for recovery, the samples were marked as "SAVE" and given an alphanumeric code relative to the recovery yard from which they came and an accession number. Some pieces were not saved in their entirety, but instead, small portions were removed, hereafter called coupons. (Coupons were also removed in the field for WTC 5, held at Gilsanz Murray Steficek, LLP [GMS, LLP], and later brought to NIST.)

Facing concern that the identified steel may not be properly preserved in the recovery yards, NIST arranged for the steel to be shipped to its campus in Gaithersburg, Maryland, starting in March 2002. Professor Astaneh-Asl also granted NIST permission to take custody of the steel that he had personally marked. Before the samples were shipped to the NIST campus, environmental testing for asbestos and analysis of the paint for lead was conducted. Volunteers from SEAoNY, with assistance from additional NIST personnel, continued their presence at the recovery yards and identified, catalogued, and shipped steel specimens to NIST through October 2002. The structural components recovered now constitute the material base from which samples are being removed for further evaluation and or testing relative to the fire and structural response of the WTC buildings as part of the WTC Investigation.

Structural steel elements were also collected and held by the Port Authority of New York and New Jersey (PANYNJ) in Hanger 17 located at John F. Kennedy International Airport (JFK). The main goal of the Port Authority project was to decontaminate and preserve the steel, as well as other WTC artifacts, for future exhibits and memorials. A complete listing of the pieces held by PANYNJ can be found in the Preservation and Inventory Report prepared by Voorsanger and Associates Architects, PC (Voorsanger 2002). NIST personnel visited the hanger and identified 12 additional pieces that were considered important to the Investigation. Six of these samples were moved whole to the Gaithersburg campus. The remaining pieces had portions removed and sent to NIST, with the bulk of the structural element remaining at JFK.

F.4 STRUCTURAL ELEMENTS RECOVERED FROM THE WTC BUILDINGS

F.4.1 Present Location and Labeling of Structural Steel Elements

At present, NIST possesses 236 labeled samples from the WTC buildings. While the majority of the NIST-held samples reside on the Gaithersburg campus, some samples were shipped to the Boulder campus for mechanical property testing following initial documentation.

As samples were delivered, overall images of the pieces were taken for record-keeping purposes. An example is shown in Fig. F–1. Samples are identified by their original alphanumeric identification codes assigned by SEAoNY to be consistent with the FEMA report. However, there were cases in which two different codes were found on one piece. In these instances, if the pieces were already undergoing

Figure F–1. Characteristic "overall" view of the samples taken for each piece received. Sample shown here is C-14.

documentation procedures, the first code noted was used. Samples that arrived lacking a code were labeled as part of the U series. Additionally, samples brought from Hanger 17 at JFK maintained their "B"-series labels provided in the Voorsanger report (Voorsanger 2002).

Attachment 1 is a complete list of each sample received, in alphanumeric order, with its classification, a brief description of the component, and the location of the piece on the NIST campus. These samples range from full exterior column panels to pieces of bolts and bags of glass and other debris fragments. The pieces were classified into one of eight categories:

Classification	No. of Pieces	Symbol
Exterior column panel sections (flat wall or corner)	94	C, CC, or Cn
Bowtie pieces	2	BT
Rectangular built-up box column (not perimeter column)	11	RB
Wide flange sections	44	W
Floor trusses	23	J
Channels	25	Ch
Coupons from WTC 5	7	Cn5
Miscellaneous (isolated bolts, floor hanger components, or other)	30	B,H,O

Attachment 2 lists the pieces separated by type, and Attachment 3 displays characteristic photographs of the various pieces.

F.4.2 Identification of WTC Structural Steel Elements

Information from Leslie E. Roberts Associates indicates that all structural steel pieces in WTC 1 and WTC 2 were uniquely identified by stampings (recessed letters and numbers) and/or painted stencils (Faschan 2002). NIST has been successful in finding these identification markings on many of the perimeter panel sections, core columns, and other wide flange members. Of the 94 pieces of perimeter panel labeled in Attachment 1, 90 distinct panels were observed. (The other four pieces of perimeter column had become separated from the main panel during salvage and were subsequently labeled C-13a, C-16a, C-28b, and K-16a.) At this time, of the 90 panels, 41 distinct exterior column panels have been identified and 1 partially identified. Tables F–1 and F–2 list these samples, respectively, with Fig. F–2 showing the relative locations of the identified exterior panels within the top third of the buildings. Significantly more pieces were recovered from WTC 1 than WTC 2. Table F–3 lists the 12 core columns in NIST's possession that have been positively identified through their stampings. An additional sample, C-83, is also listed in this group. Though no markings were found on the piece, the shape and dimension of this sample are in conformance with the design drawings for core columns and it has a similar appearance to core column C-90. Additionally, there are 13 pieces of wide flange sections that have stampings and/or markings with different codes that are not presently understood (see Table F–4). NIST is still investigating the identification of these pieces.

The positive identification of the structural elements was made possible by deciphering the stampings and/or stencils found on them. During the fabrication process, the exterior panel sections were stamped at the bottom of the center column on the inside face. These stampings indicated the building, center column line number, and floors spanned by the columns. The core columns had stampings placed at the

Table F–1. Identified exterior column panel pieces from WTC 1 and WTC 2.

NIST Name	Type	Description	Bldg.	Column	Floors	Derrick Division
B-1024	C	Full panel	WTC 2	154	21 – 24	NA
B-1043	C	Full panel	WTC 2	406	40 – 43	NA
B-1044	C	Full panel	WTC 2	409	40 – 43	NA
C-10	C	Full panel	WTC 1	451	85 – 88	5x
C-13 C-13a	CC C	Rectangular column with spandrel Partial of single column	WTC 2 WTC 2	200 159	90 – 92 90 – 92	569 569
C-14	C	1 column, lower 1/3	WTC 2	300	85 – 87	570
C-18	C	3 columns, bottom 2/3	WTC 2	230	93 – 96	NA
C-22	C	3 columns, lower 1/2	WTC 1	157	93 – 96	69
C-24	C	3 columns, upper 1/3	WTC 2	203	74 – 77	NA
C-25	C	1 column, lower 1/2	WTC 1	206	89 – 92	69
C-40	C	2 columns, lower 2/3	WTC 1	136	98 – 101	6x
C-46	C	Nearly full panel	WTC 2	157	68 – 71	569
C-48	C	Nearly 2 full columns	WTC 2	442	91 – 94	NA
C-55	C	1 column, lower 1/3	WTC 1	209	94 – 97	NA
C-89	C	2 full columns	WTC 2	215	12 – 15	NA
C-92	C	1 column, lower 1/3	WTC 2	130	93 – 96	NA
C-93	C	1 column, lower 1/3	WTC 1	339	99 – 102	NA
CC	C	2 full columns	WTC 1	124	70 – 73	NA
K-1	C	3 columns, lower 1/3	WTC 1	209	97 – 100	NA
K-2	C	1 column, lower 2/3	WTC 1	236	92 – 95	NA
M-2	C	Full panel	WTC 1	130	96 – 99	63
M-10a	C	3 columns, middle section 1/3	WTC 2	209	82 – 85	NA
M-10b	C	3 columns, lower 1/2	WTC 2	206	83 – 86	569
M-20	C	2 columns, lower 1/3	WTC 1	121	99 – 102	63
M-26	C	Full panel	WTC 1	130	90 – 93	6x
M-27	C	2 columns, lower 3/4	WTC 1	130	93 – 96	63
M-28	C	3 columns, lower 1/4	WTC 2	345	98 – 101	NA
M-30	C	2 columns, lower 1/3	WTC 1	133	94 – 97	65
N-1	C	2 full columns	WTC 1	218	82 – 85	NA
N-7	C	Full panel	WTC 1	127	97 – 100	NA
N-8	C	Full panel	WTC 1	142	97 – 100	67
N-9	C	Nearly full panel	WTC 1	154	101 – 104	69
N-10	C	2 columns, lower 2/3	WTC 1	115	89 – 92	6x
N-12	C	2 full columns	WTC 1	206	92 – 95	69
N-13	C	3 columns, lower 1/3	WTC 1	130	99 – 102	63
N-99	C	Nearly full panel	WTC 1	148	99 – 102	67

Table F–1. Identified exterior column panel pieces from WTC 1 and WTC 2 (continued).

NIST Name	Type	Description	Bldg.	Column	Floors	Derrick Division
N-101	C	Full panel	WTC 1	133	100 – 103	65
S-1	C	2 columns, lower 1/3	WTC 1	433	79 – 82	47
S-9	C	Full panel	WTC 1	133	97 – 100	NA
S-10	C	2 columns, lower 1/2	WTC 1	224	92 – 95	NA
S-14	C	Full panel	WTC 2	218	91 – 94	557

Key: NA, information not available.
Note: "x" in Derrick Division: Unreadable.

Table F–2. Partially identified exterior column panel from WTC 1 or WTC 2.

NIST Name	Type	Description	Bldg.	Column	Floors
C-117	C	3 columns, lower 1/3	NA	NA	100 – 104

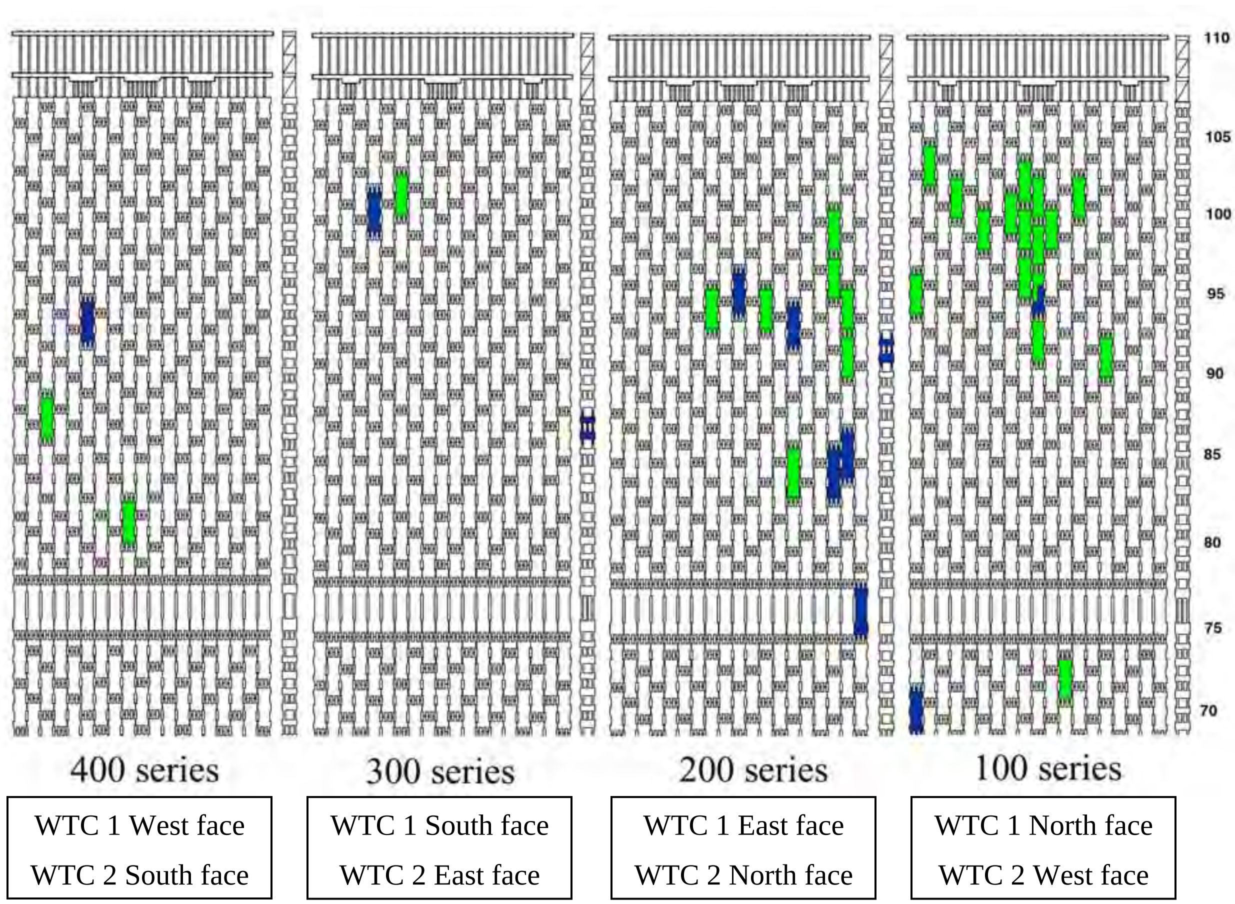


Figure F–2. Location of the exterior panels recovered from the top third of WTC 1 and 2.

Table F–3. Identified pieces of core column material from WTC 1 and WTC 2.

NIST Name	Type	Description	Bldg.	Column	Floors	Derrick Division	FY (ksi)
B-1011	RB	Heavy rectangular column	WTC 1	508	51 – 54	55	36
B-6152-1	RB	Heavy rectangular column	WTC 1	803	15 – 18	52	36
B-6152-2	RB	Heavy rectangular column	WTC 1	504	33 – 36	51	36
C-83[a]	RB	Heavy rectangular column	NA	NA	NA	NA	NA
C-88a	RB	Heavy rectangular column	WTC 2	801	80 – 83	550	42
C-88b	RB	Heavy rectangular column	WTC 2	801	77 – 80	550	42
C-90	RB	Heavy rectangular column	WTC 2	701	12 – 15	549	36
C-30 or S-12	W	Wide flange section	WTC 2	1008	104 – 106	NA	36
C-65 or S-8	W	Wide flange section	WTC 1	904	86 – 89	52	36
C-71	W	Wide flange section	WTC 1	904	77 – 80	NA	36
C-80	W	Wide flange section	WTC 1	603	92 – 95	51	36
C-155	W	Wide flange section	WTC 1	904	83 – 86	52	36
HH or S-2	W	Wide flange section	WTC 1	605	98 – 101	53	42

a. C-83 was not positively identified but due to similar size and shape was deemed a core column.
Key: NA, information not available.

Table F–4. Other built-up box columns and wide flange sections from WTC 1 and WTC 2 with ambiguous stampings and/or markings.

NIST Name	Type	Description	Markings
C-79	RB	Thin rectangular column	101A 81 – 85 – 87 – 92 52
C-101	RB	Thin rectangular column	78A 10 27 50
C-154	RB	Thin rectangular column	825: 107 – 108 52
C-26	W	Three connected wide flange sections	604/605 107 64 50
C-44	W	Wide flange section	59 S 563
C-45	W	Wide flange section	16 S2 563 Fy 50
C-60	W	Wide flange section	193 S1 69
C-61	W	Wide flange section	150 S 69
C-62	W	Wide flange section	224 (S) <48> Fy 50
M-17	W	Wide flange section	163 (9) 62 Fy 36
M-23	W	Wide flange section	F 2010
M-37	W	Wide flange section	130 (8x – 92) <50>
M-38	W	Wide flange section	Fy 42

Note: “x”, unreadable.

Figure F–3. Example of stampings on the interior base of the middle column for each panel.

lower end of the component near the connector. The building was typically represented as "A" for WTC 1 and "B" for WTC 2. An example of a stamping found on an exterior column is shown in Fig. F–3, where the stamping indicates that the piece was from WTC 2, with center column line number 206, spanning floors 83 through 86. Core column material was found to have similar markings (Fig. F–4). Other stampings have also been found on the flanges of the perimeter columns that indicated the column type (Fig. F–5 and Table F–5) as well as the specified minimum yield strength of the column. Additional stampings are located on the flanges, but are not yet understood.

NIST is still investigating the significance of these codes. All of these stampings typically reside within 1 meter from the bottom of the column.

Figure F–4. Example of stampings placed on one end of a core column.

a)

b)

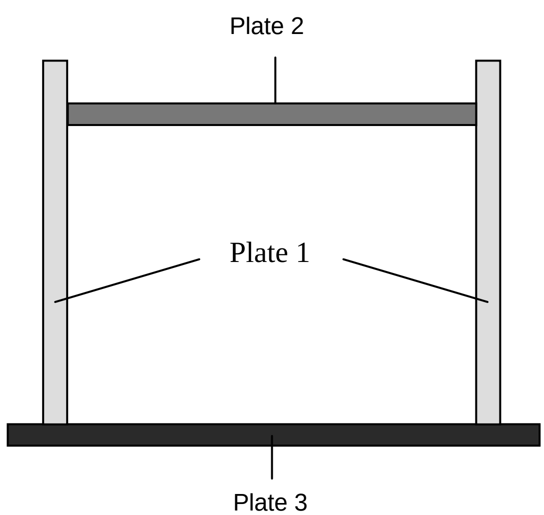


Figure F–5. (a) Example of stamping placed on flange indicating the column type (120), and (b) schematic indicating the various plates corresponding to Table F–5.

Table F–5. Examples of column types with corresponding plate gages.

Column Type	Plate 1 (in.)	Plate 2 (in.)	Plate 3 (in.)
120	1/4	1/4	1/4
121	5/16	1/4	1/4
122	3/8	1/4	1/4
123	7/16	1/4	1/4
124	1/2	1/4	1/4
125	9/16	1/4	1/4
126	5/8	1/4	1/4
128	3/4	1/4	1/4
129	13/16	5/16	5/16
133	1-1/16	3/8	3/8
149	2-1/16	11/16	11/16
150	2-1/8	3/4	3/4
152	2-1/4	3/4	3/4
334	1-1/8	3/8	3/8
335	1-3/16	7/16	7/16
520	1/4	1/4	1/4
522	3/8	1/4	1/4

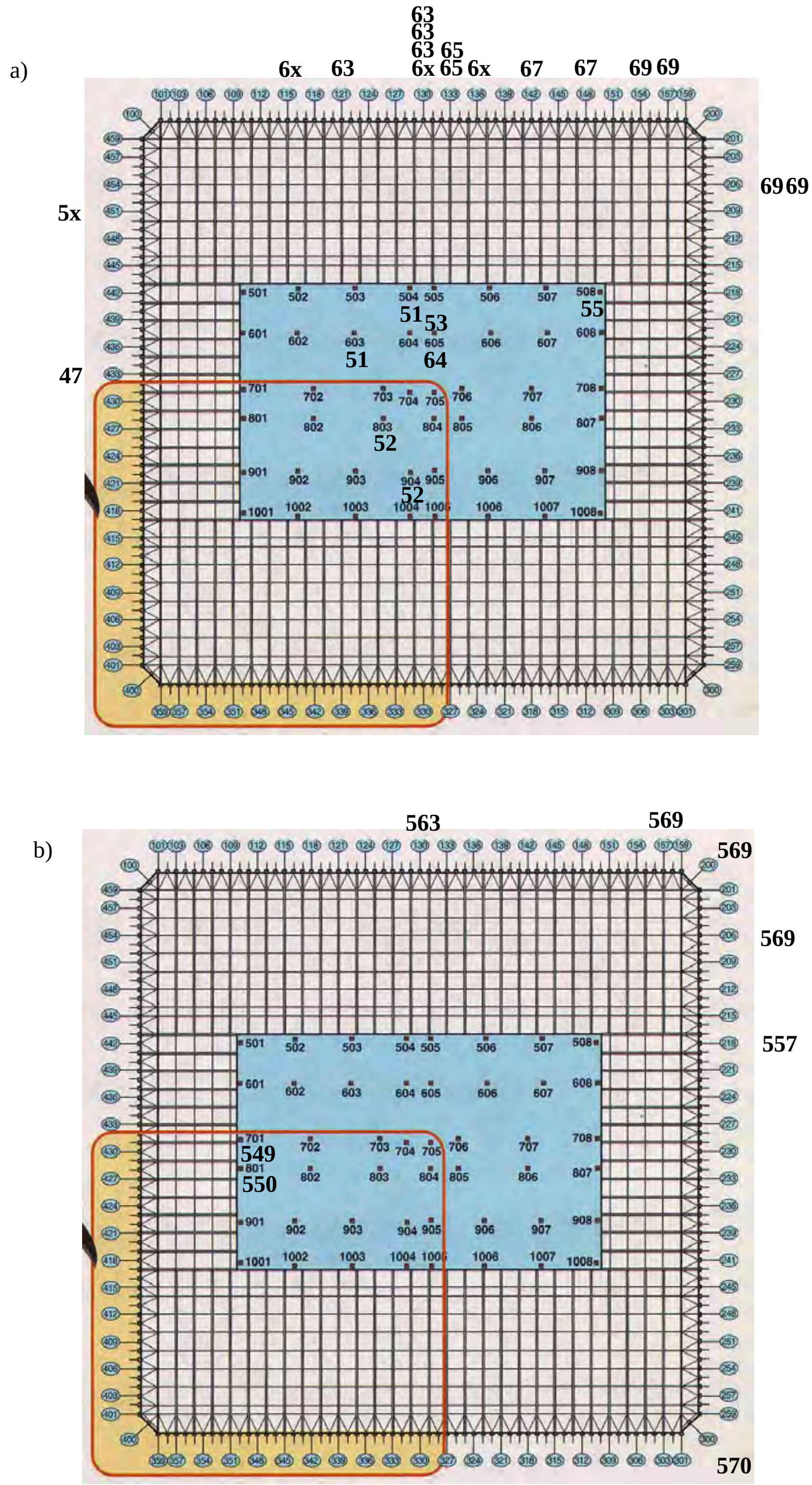
Each of the structural elements was additionally stenciled in white or yellow lettering with similar building information. For the exterior panel sections, the stenciling was located on or near the lower spandrel on the interior face. Figure F–6 (a) shows a typical stenciling found on a perimeter panel, indicating this piece was in WTC 2, with center column line number 300, spanning floors 85 through 87. For the core columns, both stenciling and handwritten codes have been observed on the recovered pieces. Figure F–6 (b) shows one of these stencilings from a core column located in WTC 1.

Also seen in Fig. F–6 (a) are two other indicators, 3T and <570>, found on the exterior panel sections. These markings are the estimated piece tonnage (1 ton equals approximately 907 kg) and the erector's derrick division number, respectively. This information was also stamped on some of the core column pieces (see Fig. F–4). The erector, Karl Koch Erecting Co., Inc., assigned derrick divisions 47 through 70 for WTC 1 and derrick divisions 547 through 570 for WTC 2 (PONYA 1967). Each division was assigned to a specific area of the building and shared a crane with other nearby derrick divisions. Therefore, a single crane may have lifted pieces from derrick divisions 65, 67, and 69. Figure F–7 shows the derrick division numbers that hoisted the specific columns for both buildings, according to the derrick numbers found on structural elements with positive identification (also shown in Tables F–2 and F–3).

Of the 41 positively identified exterior panels, 25 had specific markings giving all the information needed (building, column, floors) to locate the structural element within the buildings from one or both codes (i.e., stampings or stencils). The flange stampings, which indicated the specified yield strength and column type, were used to confirm the findings (Tables F–6 and F–7). The only deviation noted was that 100 ksi steel was substituted for the 85 ksi and 90 ksi grades that were specified. This can be observed in a)

b)

Figure F–6. (a) Characteristic stenciling found on the lower portions of the exterior column panels for sample C-14. (b) Characteristic stenciling found on an interior core column for sample B-6152.



Source: McAllister 2002.

Figure F–7. Schematic showing the derrick divisions that hoisted the specific columns for (a) WTC 1, and (b) WTC 2.

Table F–6 for samples B-1043, B-1044, C-10, and M-10b. This substitution is consistent with (PANYNJ) documents of the construction period, indicating that 100 ksi steel was used for all steel specified as 85 ksi or 90 ksi. (See Appendix C, Contemporaneous Structural Steel and Construction Specifications.)

Sixteen other panels were positively identified using a combination of the stampings, including the specified minimum yield strength (Table F–8) and column type (Table F–9), the stenciled derrick division number (Table F–8), or association to another panel, as follows:

- **C-10**: The stampings indicated that the center column line number was 451 and the panel spanned floors 85 through 88, but the building identification information was obscured by a weld bead. The building can be identified by a derrick division number in the 50 series, which corresponds to WTC 1 (Fig. F–7). (Note that the flange stampings indicated that the steel used is 100 ksi, while the building design drawings indicated that 85 ksi was specified. As mentioned above, substitution of the specified 85 ksi, as well as the 90 ksi grades, by 100 ksi steel was approved.)

- **C-24**: This piece was readily identifiable as a mechanical or service floor due to the non-uniform width of the columns. Unfortunately, only the upper portion of the panel was recovered, and thus no stampings were found. However, the end connections to these floors were welded in addition to the typical bolting. In doing so, the end plate and a small portion of the column from the panel above this piece remained after the collapse, and the stamping of “B 203 77-78” identifying the panel above this sample was clearly visible.

- **C-55**: The stampings indicated that the center column line number was 209 and the panel spanned floors 94 through 97, however, no building information was observed. By reviewing the flange stampings (Table F–8), the piece was determined to belong to WTC 1.

- **C-92**: Stenciling on the piece indicated that it was from WTC 2, floors 93 through 96. However, the center column line number was partially obscured, with 13x visible. By reviewing the flange stampings (Tables F–8 and F–9), the piece center column line number was determined to be 130.

- **C-93**: The stampings indicated that the center column line number was 339 and the panel spanned floors 99 through 102; however, no building information was observed. By reviewing the flange stampings (Table F–8), the piece was determined to belong to WTC 1.

- **CC**: The stampings indicated that the center column line number was 124 and the panel spanned floors 70 through 73; however, no building information was observed. By reviewing the flange stampings (Table F–8), the piece was determined to belong to WTC 1.

- **K-1**: The stampings indicated that the center column line number was 209 and the panel spanned floors 97 through 100; however, no building information was observed. By reviewing the flange stampings (Table F–8), the piece was determined to belong to WTC 1.

Table F–6. Specified and observed minimum yield strengths for positively identified exterior column panels.[a]

NIST Name	Bldg	Column	Floors	Specified Minimum Yield (ksi)			Stamping Observed		
				Column 1	Column 2	Column 3	Column 1	Column 2	Column 3
B-1024	WTC 2	154	21-24	50	50	50	NA	50	NA
B-1043	WTC 2	406	40-43	85	90	90	100	100	100
B-1044	WTC 2	409	40-43	85	80	85	100	80	100
C-10	WTC 1	451	85-88	85	85	90	100	100	100
C-13 or S-11 and C13a or S-19	WTC 2	200	90-92	100	100	100	100	NA	NA
C-14 or S-18	WTC 2	300	85-87	100	100	100	NA	NA	NA
C-18	WTC 2	230	93-96	55	55	55	55	55	55
C-22	WTC 1	157	93-96	80	75	80	80	NA	80
C-24	WTC 2	203	74-77	100	100	100	NA	NA	NA
C-25	WTC 1	206	89-92	80	80	80	80	NA	NA
C-40	WTC 1	136	98-101	60	60	55	NA	60	55
C-46	WTC 2	157	68-71	80	70	65	80	NA	65
C-48 or S-5	WTC 2	442	91 - 94	65	65	65	NA	65	NA
C-55	WTC 1	209	94-97	70	70	70	NA	70	NA
C-89	WTC 2	215	12 - 15	50	50	55	NA	NA	NA
C-92	WTC 2	130	93 - 96	60	60	60	60	NA	NA
C-93	WTC 1	339	99 - 102	60	60	60	NA	60	NA
CC	WTC 1	124	70-73	50	50	50	NA	50	50
K-1 or K-13	WTC 1	209	97-100	60	60	60	60	60	60
K-2 or K-40	WTC 1	236	92-95	65	65	65	NA	65	NA
M-2	WTC 1	130	96-99	55	55	55	55	55	55
M-10a	WTC 2	209	82-85	85	85	85	NA	NA	NA
M-10b	WTC 2	206	83-86	85	85	85	100	100	NA
M-20	WTC 1	121	99-102	55	55	55	NA	55	55
M-26	WTC 1	130	90-93	50	55	50	NA	55	50
M-27	WTC 1	130	93-96	50	55	55	50	55	NA
M-28	WTC 2	345	98 - 101	70	70	70	NA	NA	NA
M-30	WTC 1	133	94-97	55	55	55	NA	55	55
N-1	WTC 1	218	82-85	70	75	75	70	75	NA
N-7 or M-3	WTC 1	127	97-100	55	55	60	55	55	60
N-8 or M-7	WTC 1	142	97-100	60	60	60	NA	60	NA
N-9 or M-8	WTC 1	154	101-104	55	55	55	55	55	NA
N-10 or M-15	WTC 1	115	89-92	55	55	55	NA	55	55
N-12 or M-13	WTC 1	206	92-95	75	75	75	NA	75	75
N-13 or M-14	WTC 1	130	99-102	55	55	55	NA	NA	NA
N-99 or M-16	WTC 1	148	99-102	65	65	65	65	65	NA
N-101 or M-21	WTC 1	133	100-103	55	55	55	55	55	55
S-1 or EE	WTC 1	433	79-82	70	70	70	NA	70	70
S-9 or C-63	WTC 1	133	97-100	55	55	55	55	55	55
S-10 or C-17	WTC 1	224	92-95	70	70	70	70	70	NA
S-14 or C-20	WTC 2	218	91-94	65	65	70	65	65	70

a. Columns 1, 2, and 3 are viewed left to right as viewed from the inside of the building.
Key: NA, information not available.

Table F–7. Specified and observed column types for positively identified exterior column panels.[a]

NIST Name	Bldg	Column	Floors	Specified Column Type			Stamping Observed		
				Column 1	Column 2	Column 3	Column 1	Column 2	Column 3
B-1024	WTC 2	154	21-24	149	150	152	149	150	152
B-1043	WTC 2	406	40-43	335	334	334	335	334	334
B-1044	WTC 2	409	40-43	335	335	335	335	335	335
C-10	WTC 1	451	85-88	120	120	120	120	120	120
C-13 or S-11 and C13a or S-19	WTC 2	200	90-92	120	520	120	120	NA	NA
C-14 or S-18	WTC 2	300	85-87	122	522	120	NA	NA	NA
C-18	WTC 2	230	93-96	120	120	120	120	120	120
C-22	WTC 1	157	93-96	120	120	120	120	NA	120
C-24	WTC 2	203	74-77	325	325	325	Bottoms missing		
C-25	WTC 1	206	89-92	120	120	120	120	NA	NA
C-40	WTC 1	136	98-101	121	121	121	NA	121	121
C-46	WTC 2	157	68-71	126	128	129	126	NA	129
C-48 or S-5	WTC 2	442	91 - 94	120	120	120	NA	120	NA
C-55	WTC 1	209	94-97	120	120	120	NA	120	NA
C-89	WTC 2	215	12 - 15	147	145	143	NA	NA	NA
C-92	WTC 2	130	93 - 96	124	123	123	124	NA	NA
C-93	WTC 1	339	99 - 102	121	121	121	NA	121	NA
CC	WTC 1	124	70-73	133	133	133	NA	133	133
K-1 or K-13	WTC 1	209	97-100	120	120	120	120	120	120
K-2 or K-40	WTC 1	236	92-95	120	120	120	NA	120	NA
M-2	WTC 1	130	96-99	122	122	122	122	122	122
M-10a	WTC 2	209	82-85	120	120	120	NA	NA	NA
M-10b	WTC 2	206	83-86	120	120	120	120	120	NA
M-20	WTC 1	121	99-102	120	120	120	NA	120	120
M-26	WTC 1	130	90-93	125	125	125	NA	125	125
M-27	WTC 1	130	93-96	124	123	123	124	123	NA
M-28	WTC 2	345	98 - 101	120	120	120	NA	NA	NA
M-30	WTC 1	133	94-97	123	123	123	NA	123	123
N-1	WTC 1	218	82-85	123	123	123	123	123	NA
N-7 or M-3	WTC 1	127	97-100	121	121	121	121	121	121
N-8 or M-7	WTC 1	142	97-100	121	121	121	NA	121	NA
N-9 or M-8	WTC 1	154	101-104	120	120	120	120	120	NA
N-10 or M-15	WTC 1	115	89-92	125	125	125	NA	125	125
N-12 or M-13	WTC 1	206	92-95	120	120	120	NA	120	120
N-13 or M-14	WTC 1	130	99-102	121	121	120	NA	NA	NA
N-99 or M-16	WTC 1	148	99-102	120	120	120	120	120	NA
N-101 or M-21	WTC 1	133	100-103	120	120	120	120	120	120
S-1 or EE	WTC 1	433	79-82	123	123	123	NA	123	123
S-9 or C-63	WTC 1	133	97-100	122	122	122	122	122	122
S-10 or C-17	WTC 1	224	92-95	120	120	120	120	120	NA
S-14 or C-20	WTC 2	218	91-94	120	120	120	120	120	120

a. Columns 1, 2, and 3 are viewed left to right as viewed from the inside of the building.
Key: NA, information not available.

Table F–8. Specified minimum yield strengths (ksi) from WTC 1 and WTC 2, along with the observed stampings, used to positively identify some exterior column panels.[a]

NIST NAME	Markings	Column Line	Floors	Derrick Division	If WTC 1 Column 1	If WTC 1 Column 2	If WTC 1 Column 3	If WTC 2 Column 1	If WTC 2 Column 2	If WTC 2 Column 3	Observed Column 1	Observed Column 2	Observed Column 3	Confirmed identification	Full Identification
C-10	451: 85 - 88	451	85 - 88	5x	90	85	85	80	80	80	100	100	100	**WTC 1**	A451: 85-88
C-55	209: 94 - 97	209	94 - 97	NA	**70**	**70**	**70**	60	60	60	NA	**70**	NA	**WTC 1**	A209: 94-97
C-92	B13x: 93-96	130	93 - 96	NA	"B" indicates WTC 2			**60**	**60**	**60**	**60**	NA	NA	**130**	B130: 93 - 96
		139	93 - 96	NA				65	60	60					
C-93	339: 99 - 102	339	99 - 102	NA	60	60	60	65	65	60	NA	**60**	NA	**WTC 1**	A339: 99 - 102
CC	124: 70 - 73	124	70 - 73	NA	**50**	**50**	**50**	55	55	55	NA	**50**	**50**	**WTC 1**	A124: 70-73
K-1	209: 97 - 100	290	97 - 100	NA	**60**	**60**	**60**	55	55	55	**60**	**60**	**60**	**WTC 1**	A209: 97-100
K-2	236: 92 - 95	236	92 - 95	NA	**65**	**65**	**65**	60	60	60	NA	65	NA	**WTC 1**	A236: 92-95
M-2	x - 9x <63>		90's	63	3 columns of column type 122						55	55	55		A130: 96-99
M-30	x33: 94 - 97	133	94 - 97	65	**55**	**55**	**55**	60	60	60	NA	**55**	**55**	**WTC 1, 133**	A133: 94-97
		233	94 - 97		60	60	60	55	55	55					
		333	94 - 97		55	55	55	60	60	55					
		433	94 - 97		65	65	65	55	55	50					
N-1	2x8: 82 - 85	218	82 - 85	5x	**70**	**75**	**75**	60	60	65	**70**	**75**	NA	**WTC 1, 218**	A218: 82-85
		248	82 - 85		Column line 248 spans either floors 81 - 84 or 84 - 87										
N-7	127: 97 - 100	127	97 - 100	NA	**60**	**55**	**55**	60	65	60	**60**	**55**	**55**		
N-12	x06: 92 - 95	106	92 - 95	69	65	65	65	65	65	70	NA	**75**	**75**		A206: 92-95
		206	92 - 95		**75**	**75**	**75**	65	65	65				**WTC 1, 206**	
		306	92 - 95		65	65	65	65	65	70					
		406	92 - 95		70	70	70	70	70	70					
S-10 or C-17	224: 92 - 95	224	92 - 95	NA	**70**	**70**	**70**	60	60	60	**70**	**70**	NA	**WTC 1**	A224: 92-95

a. Columns 1, 2, and 3 are left to right viewed from inside the building.

Table F–9. Specified column types of exterior panels from WTC 1 and WTC 2, along with the observed stampings, used to positively identify some exterior column panels.[a]

NIST	Markings	Column	Floors	If WTC 1			If WTC 2			Observed			Confirmed
NAME		Line		Column 1	Column 2	Column 3	Column 1	Column 2	Column 3	Column 1	Column 2	Column 3	identification
C-10	451: 85 - 88	451	85 - 88	120	120	120	120	120	120	120	120	120	Inconclusive
C-55	209: 94 - 97	209	94 - 97	120	120	120	120	120	120	NA	120	NA	Inconclusive
C-92	B13x: 93-96	130	93 - 96	"B" indicates WTC 2			**124**	**123**	**123**	**124**	NA	NA	130
		139	93 - 96				123	124	124				
C-93	339: 99 - 102	339	99 - 102	121	121	121	121	121	121	NA	121	NA	Inconclusive
CC	124: 70 - 73	124	70 - 73	133	133	133	133	133	133	133	133	NA	Inconclusive
K-1	209: 97 - 100	290	97 - 100	120	120	120	120	120	120	120	120	120	Inconclusive
K-2	236: 92 - 95	236	92 - 95	120	120	120	120	120	120	NA	120	NA	Inconclusive
M-2	x - 9x <63>		90's	3 columns of having 55 ksi						122	122	122	Inconclusive
M-30	x33: 94 - 97	133	94 - 97	123	123	123	123	123	123	123	123	NA	233 and 433
		233	94 - 97	120	120	120	120	120	120				eliminated
		333	94 - 97	123	123	123	123	123	123				
		433	94 - 97	120	120	120	120	120	120				
N-1	2x8: 82 - 85	218	82 - 85	123	123	123	123	123	123	NA	123	123	218
		248	82 - 85	Column line 248 spans either floors 81 - 84 or 84 - 87									
N-7	127: 97 - 100	127	97 - 100	121	121	121	121	121	121	121	121	121	Inconclusive
N-12	x06: 92 - 95	106	92 - 95	122	122	122	122	122	122	120	120	NA	106 and 306
		206	92 - 95	120	120	120	120	120	120				eliminated
		306	92 - 95	122	122	122	122	122	122				
		406	92 - 95	120	120	120	120	120	120				
S-10 or C-17	224: 92 - 95	224	92 - 95	120	120	120	120	120	120	NA	120	120	Inconclusive

a. Columns 1, 2, and 3 are left to right viewed from inside the building.

- **K-2**: The stampings indicated that the center column line number was 236 and the panel spanned floors 92 through 95; however, no building information was observed. By reviewing the flange stampings (Table F–8), the piece was determined to belong to WTC 1.

- **M-2:** No information was available from the stampings at the base of the middle column, and very little information was recovered from the stenciling on the spandrel. A derrick division number of <63> was observed, placing the element in WTC 1 (Table F–8). The only other information was – 9, indicating that some portion of the panel was located in the 90s-floor-level range. The flange stampings from the recovered piece specified that all three columns were of the 122 type, with FY 55 ksi steel. In addition, columns 1 and 3 had floor truss seats, while column 2 had gusset plates for the diagonal bracing straps. Reviewing the building design drawings, it was found that five panels meet the 122 column type, with 55 ksi steel in the 90s range (Table F–10). Of these, only two panels had columns 1 and 3 with floor truss seats (130: 96 through 99 and 330: 96 through 99). As shown in Fig. F–7, the derrick division of <63> identifies the panel as 130: 96 through 99.

- **M-10a**: The sample was identified solely by association to another panel (bolted spandrel connection). The sample M-10 retrieved by SEAoNY was actually composed of pieces from two different exterior column panels (Fig. F–8). Therefore, with the positive identification of M-10b via the stampings and stencils, M-10a's connection to it allowed its identification as WTC 2,209: 82 through 85.

- **M-28**: The stampings indicated that the center column line number was 345 and the panel was located in WTC 2. However, the markings of the floors spanned were partially obscured; 9x – 1xx. By reviewing the building design drawings, the only panel that could fit spanned floors 98 through 101.

- **M-30**: The stampings found were x33 94-97, where the "x" signifies missing information due to a weld bead running across this area. Thus, the building and exact center column line numbers were unknown. However, a derrick division number of <65> was visible on the interior spandrel. From this information, as well as the specified minimum yield strength (Table F–8) and column type (Table F–9), M-30 was determined to belong to WTC 1, with a center column line number of 133.

- **N-1**: The stampings indicated that the columns spanned floors 82 through 85; however, no building information was observed, and a weld bead ran through the middle of the center column line number, yielding only 2x8. By reviewing the building plans, only column line 218 spanned the floors specified, and the flange stampings (Tables F–8 and F–9) indicated that the piece belonged to WTC 1.

- **N-7**: The stampings indicated that the center column line number was 127 and the panel spanned floors 97 through 100, however, no building information was observed. By reviewing the flange stampings (Table F–8), the piece was determined to belong to WTC 1.

- **N-12**: The stampings found were x06 92-95 where the x signifies missing information due to a weld bead running across this area. Thus, the building and exact center column line numbers were unknown. However, a derrick division number of <69> was visible on the

Table F–10. Information used to determine the identification of exterior panel M-2.

1) 5 panels in WTC 1, floors 90-99 that meet the criterion of 3 columns[a] with column type 122 and 55 ksi

PANEL NUMBER			SPANDREL 1				SPANDREL 2				SPANDREL 3			
Center Col #	Story @ Splice		FLOOR	SEAT DETAIL			FLOOR	SEAT DETAIL			FLOOR	SEAT DETAIL		
	Lower	Upper		COL 1	COL 2	COL 3		COL 1	COL 2	COL 3		COL 1	COL 2	COL 3
127	94	97	97	5110	1411	5210	96	5110	1411	5210	95	5110	1411	5210
130	96	99	99	1411	5210	1411	98	1411	5210	1411	97	1411	5210	1411
330	96	99	99	1411	5210	1411	98	1411	5210	1411	97	1411	5210	1411
133	97	100	100	5210	1411	5110	99	5210	1411	5110	98	5210	1411	5110
333	97	100	100	5210	1411	5110	99	5210	1411	5110	98	5210	1411	5110

2) Only two panels that meet the additional criterion of columns 1 and 3 having truss seat attachments and column 2 having gusset plate attachments

- Seat detail 5110 and 5120 are gusset plates for diagonal bracing straps
- Seat detail 1411 are truss seat attachments

PANEL NUMBER			SPANDREL 1				SPANDREL 2				SPANDREL 3			
Center Col #	Story @ Splice		FLOOR	SEAT DETAIL			FLOOR	SEAT DETAIL			FLOOR	SEAT DETAIL		
	Lower	Upper		COL 1	COL 2	COL 3		COL 1	COL 2	COL 3		COL 1	COL 2	COL 3
130	96	99	99	1411	5210	1411	98	1411	5210	1411	97	1411	5210	1411
330	96	99	99	1411	5210	1411	98	1411	5210	1411	97	1411	5210	1411

3) Derrick Division suggests that panel came from North face of WTC 1, i.e., panel in the 100-series

PANEL NUMBER			SPANDREL 1				SPANDREL 2				SPANDREL 3			
Center Col #	Story @ Splice		FLOOR	SEAT DETAIL			FLOOR	SEAT DETAIL			FLOOR	SEAT DETAIL		
	Lower	Upper		COL 1	COL 2	COL 3		COL 1	COL 2	COL 3		COL 1	COL 2	COL 3
130	96	99	99	1411	5210	1411	98	1411	5210	1411	97	1411	5210	1411

a. Columns 1, 2, and 3 are left to right viewed from inside the building.

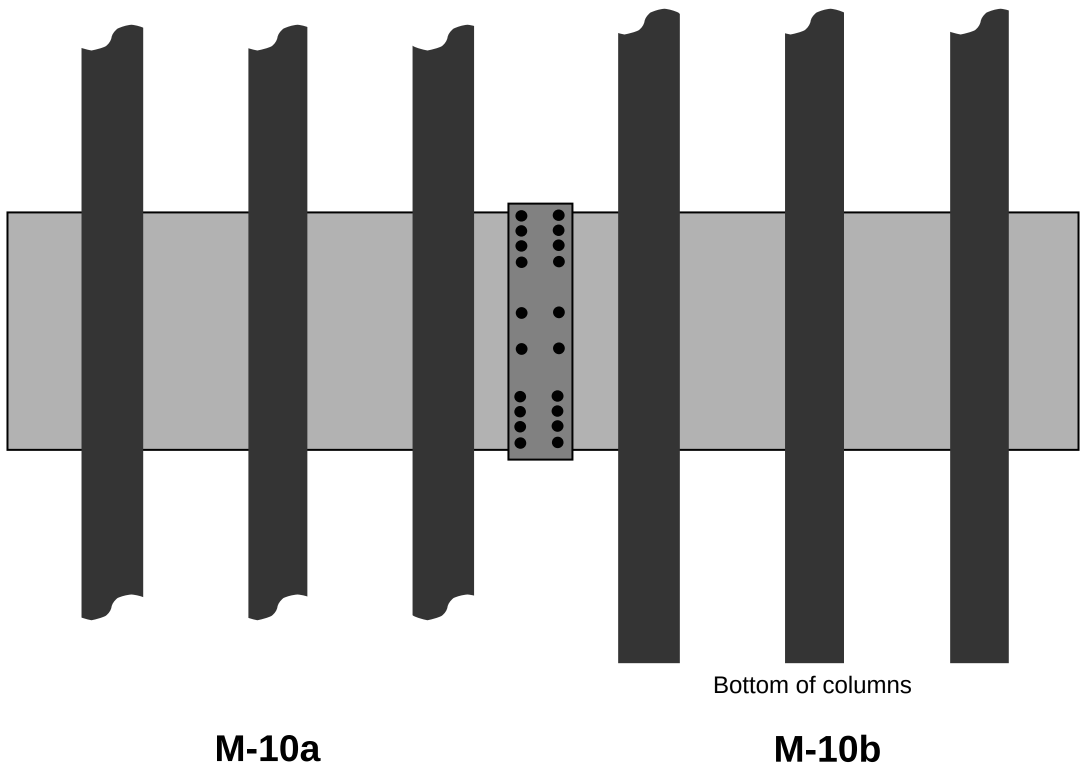


Figure F–8. Schematic showing the sample M-10 as two separate exterior column panels, M-10a and M-10b.

interior spandrel. From this information, as well as the specified minimum yield strength (Table F–8) and column type (Table F–9), it was determined that N-12 belonged to WTC 1, with a center column line number of 206.

- **S-10 or C-17**: The stampings indicated that the center column line number was 224 and the panel spanned floors 92 through 95, however, no building information was observed. By reviewing the flange stampings (Table F–8), the piece was determined to belong to WTC 1.

In addition to the overall images taken for record-keeping purposes, the exterior column panels were mapped to indicate how much of the panel was recovered after the collapse. Figure F–9 displays schematics of typical exterior panels recovered, and Figs. F–10 and F–11 show these maps, with the recovered portion indicated, for the identified samples from WTC 1 and WTC 2, respectively. Special note should be given to the fact that these diagrams are drawn as if viewed from the outside of the building. B-1043, B-1044, and C-24 were samples located at the mechanical floors of the building. C-13 and C-13a (pieces of the same exterior panel) and C-14 were exterior wall panels located at the corner of the building.

For the 12 samples identified as core column material (Table F–3), all but 2 were clearly marked. Sample C-30 had markings that clearly indicated the building and column; however, the floors were partially

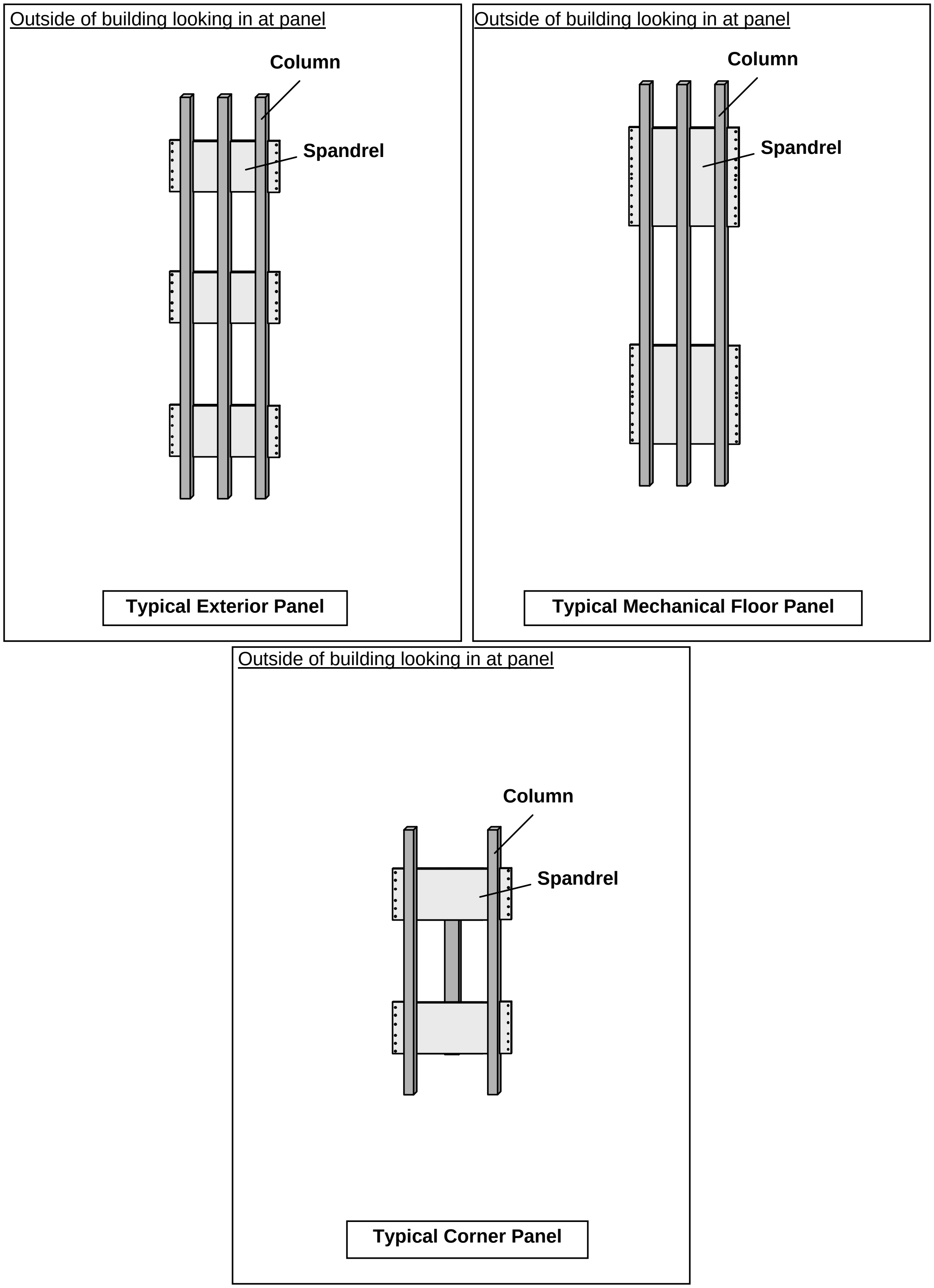


Figure F–9. Schematics displaying the various types of exterior column panels.

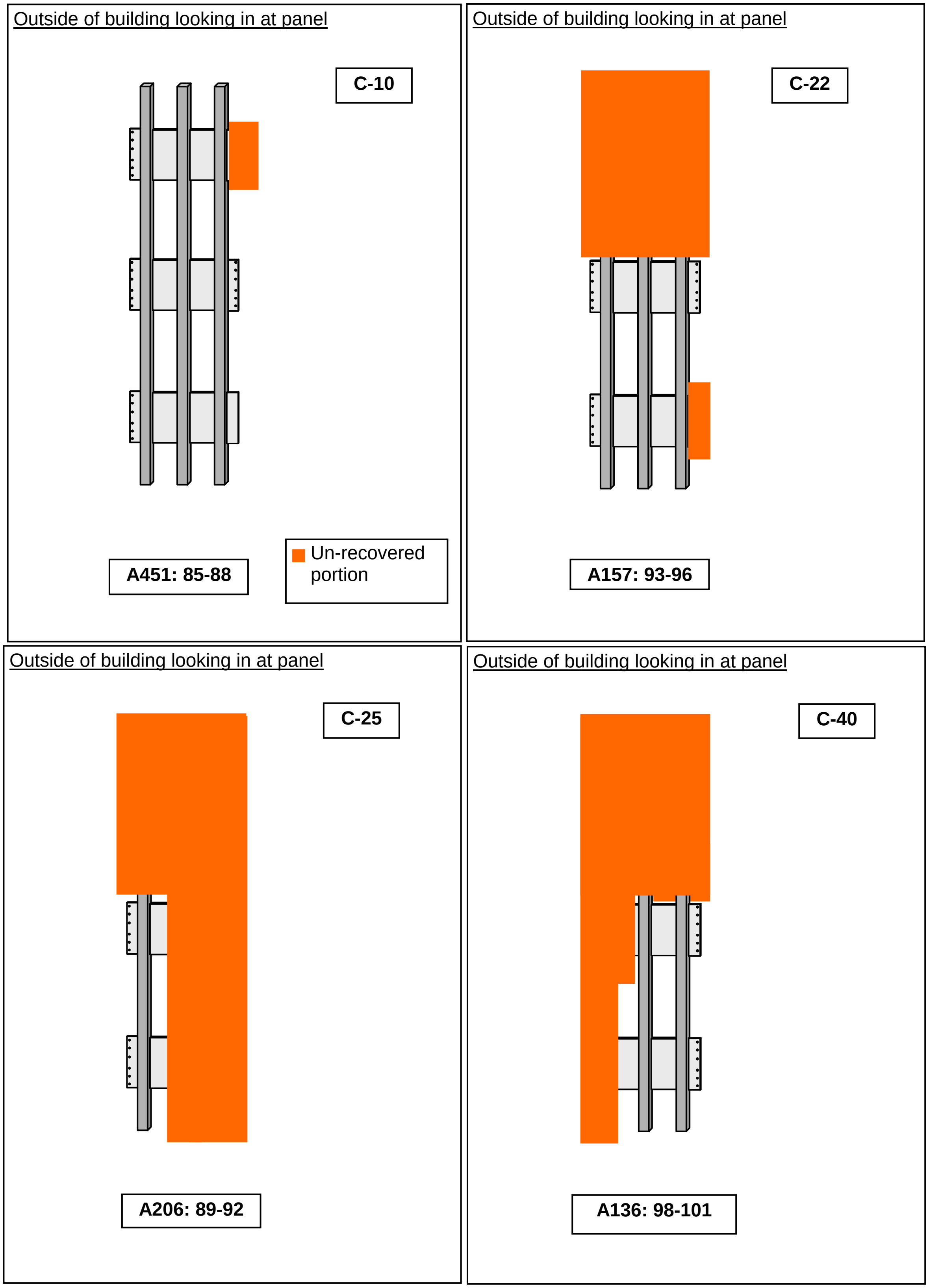


Figure F–10. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 1.

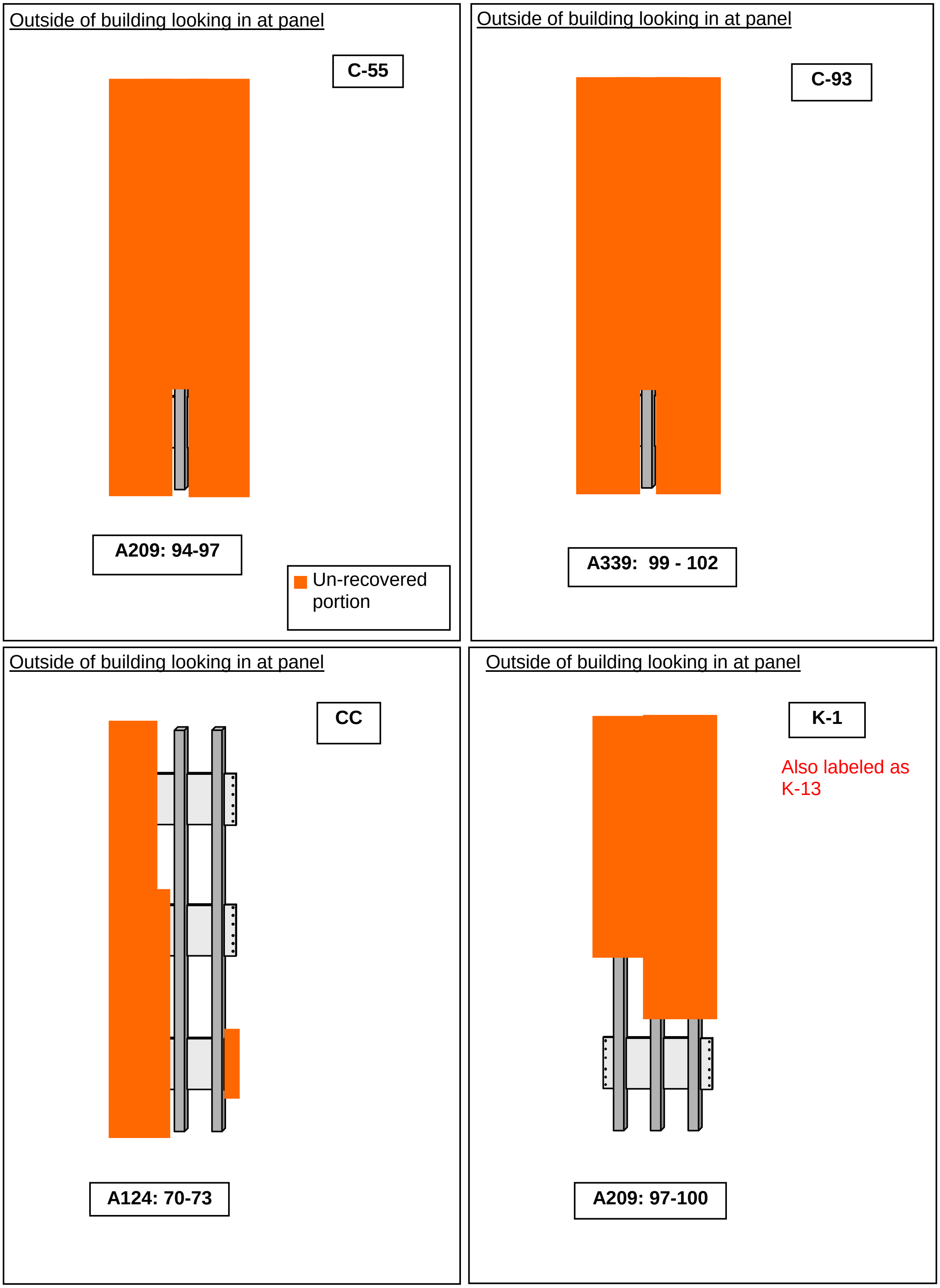


Figure F–10. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 1 (continued).

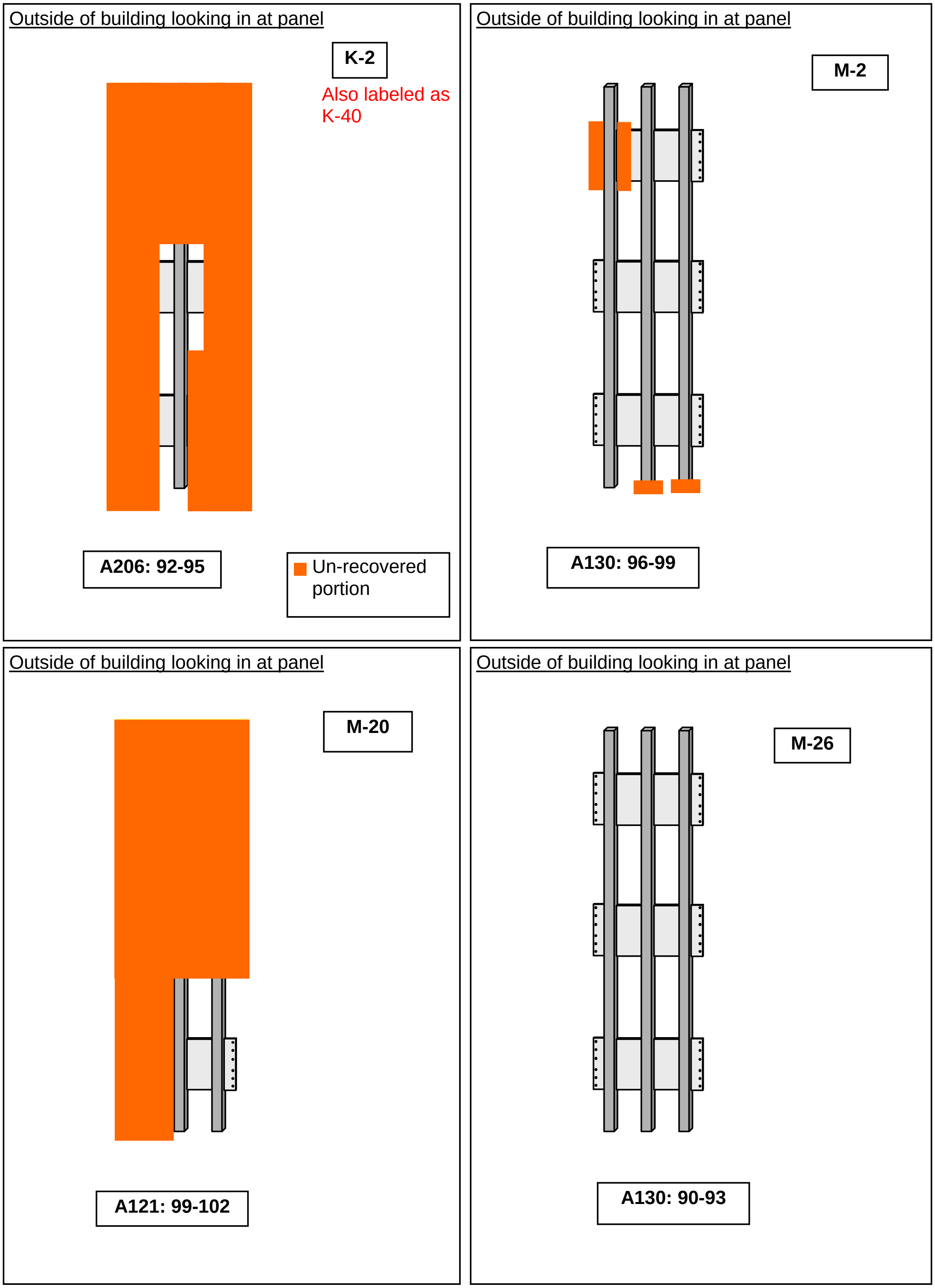


Figure F–10. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 1 (continued).

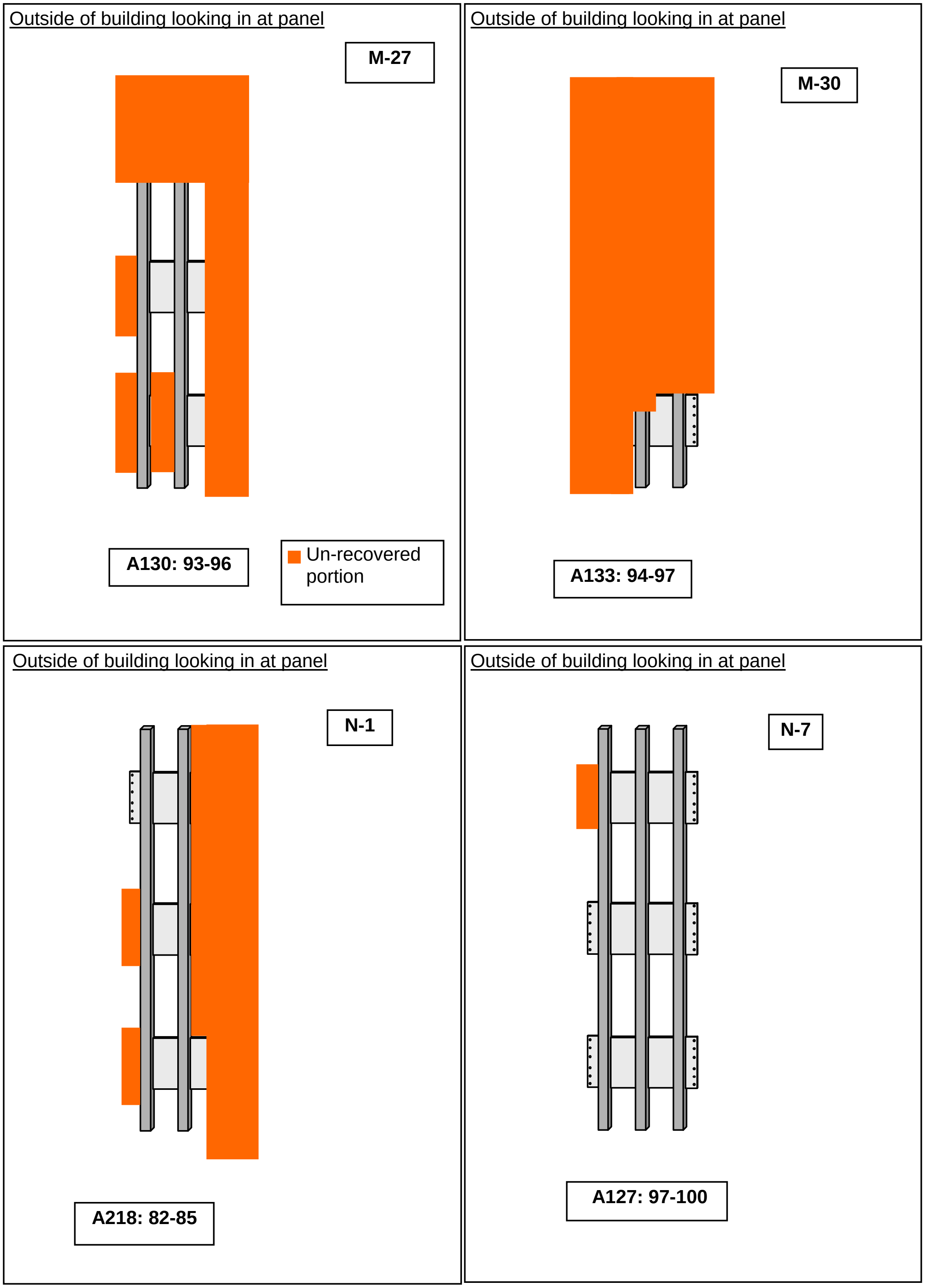


Figure F–10. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 1 (continued).

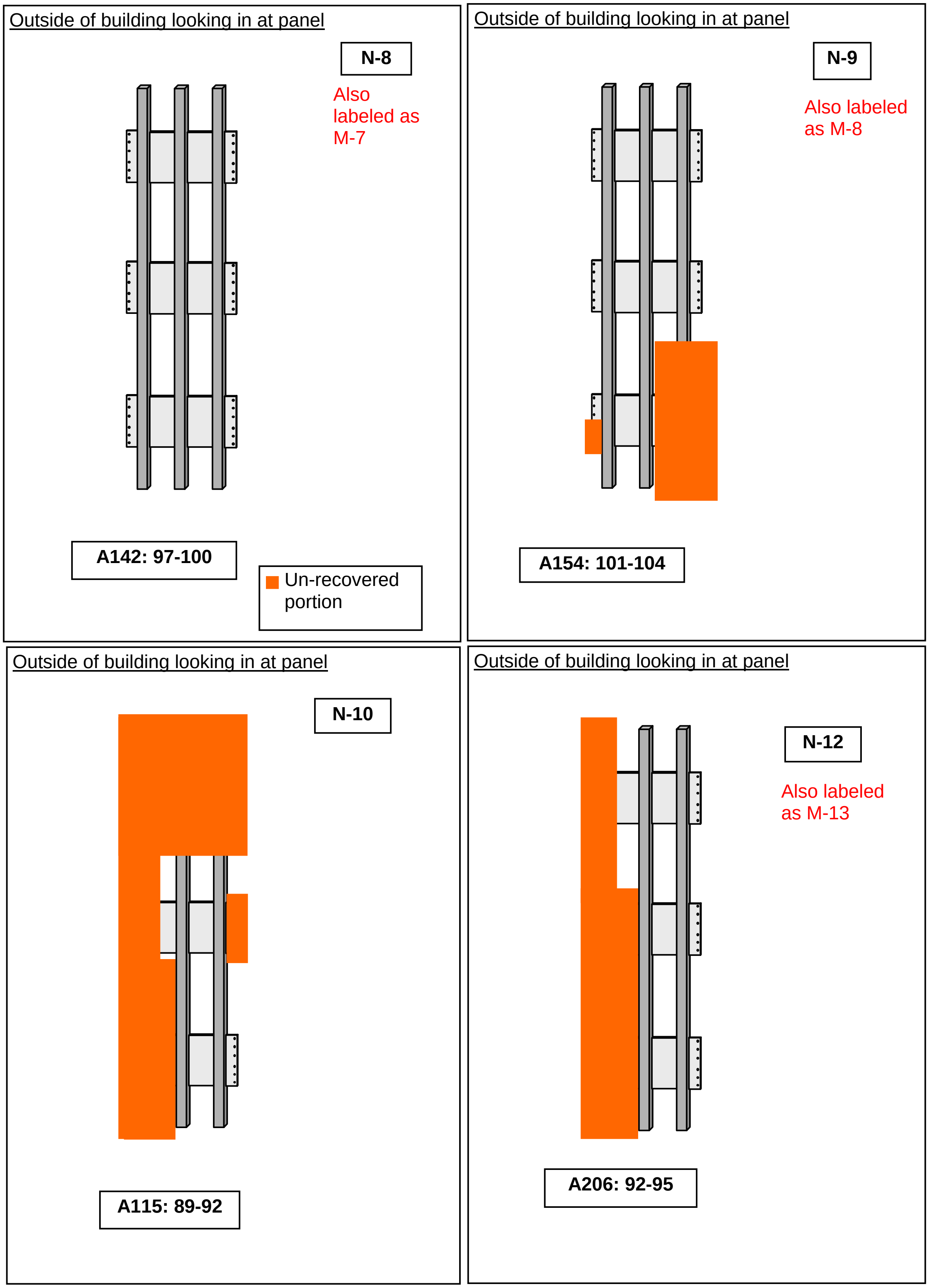


Figure F–10. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 1 (continued).

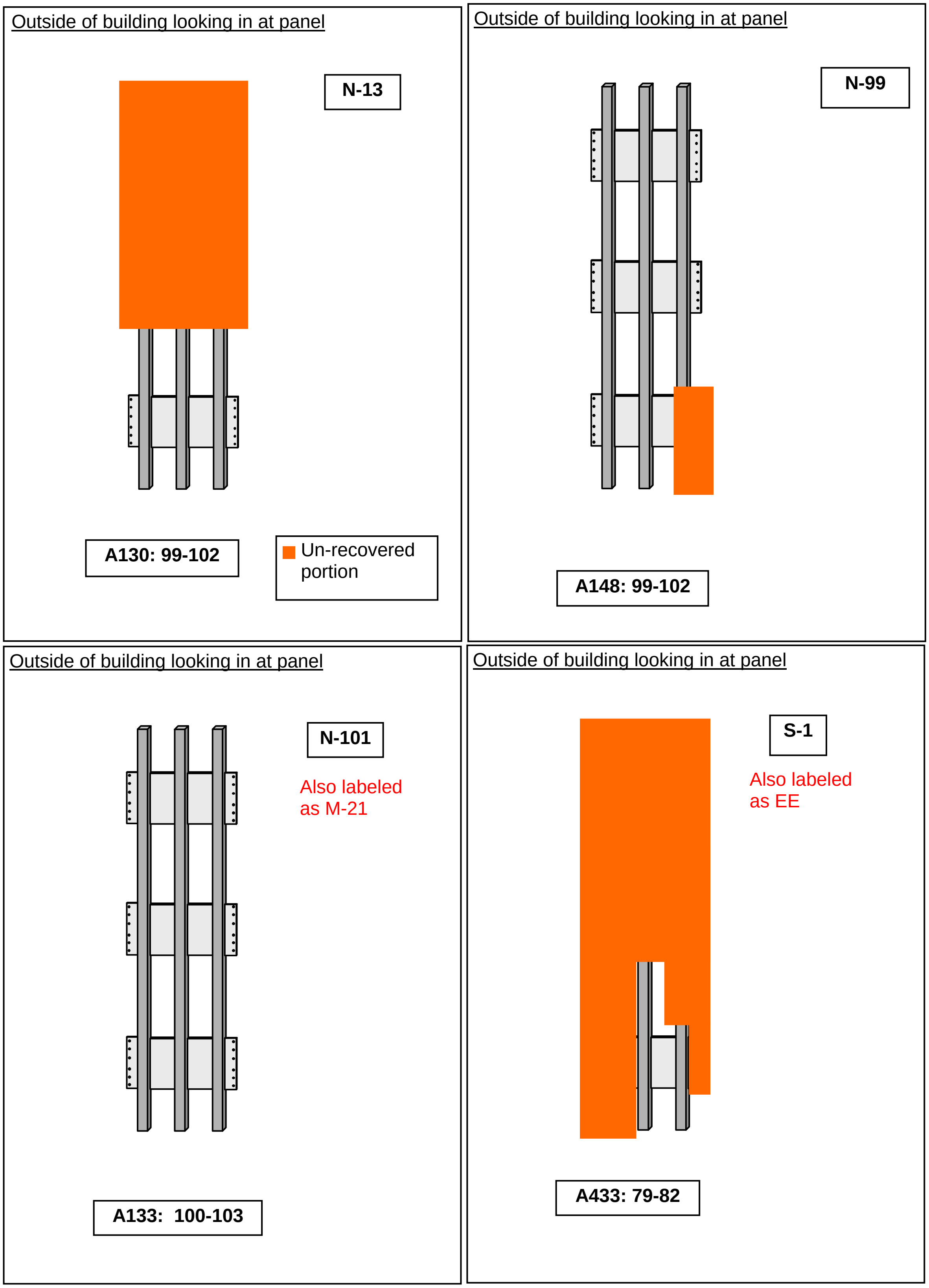


Figure F–10. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 1 (continued).

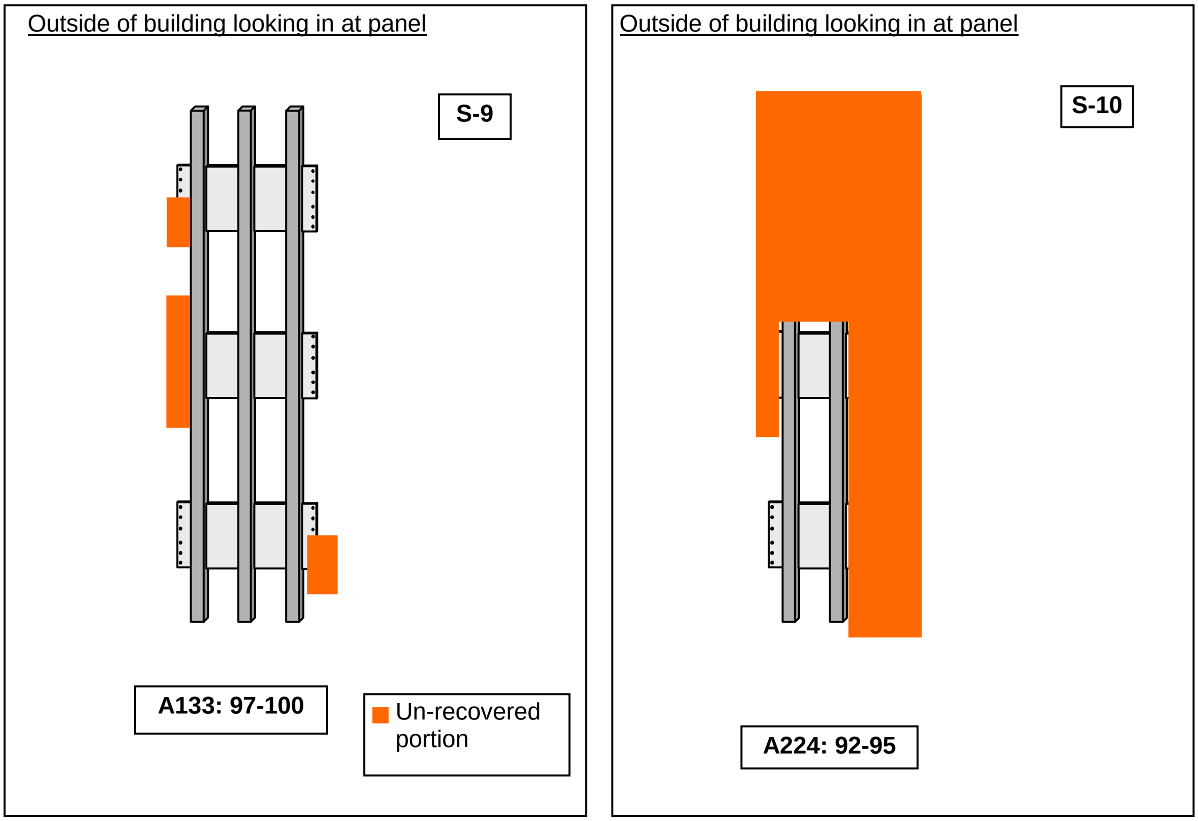


Figure F–10. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 1 (continued).

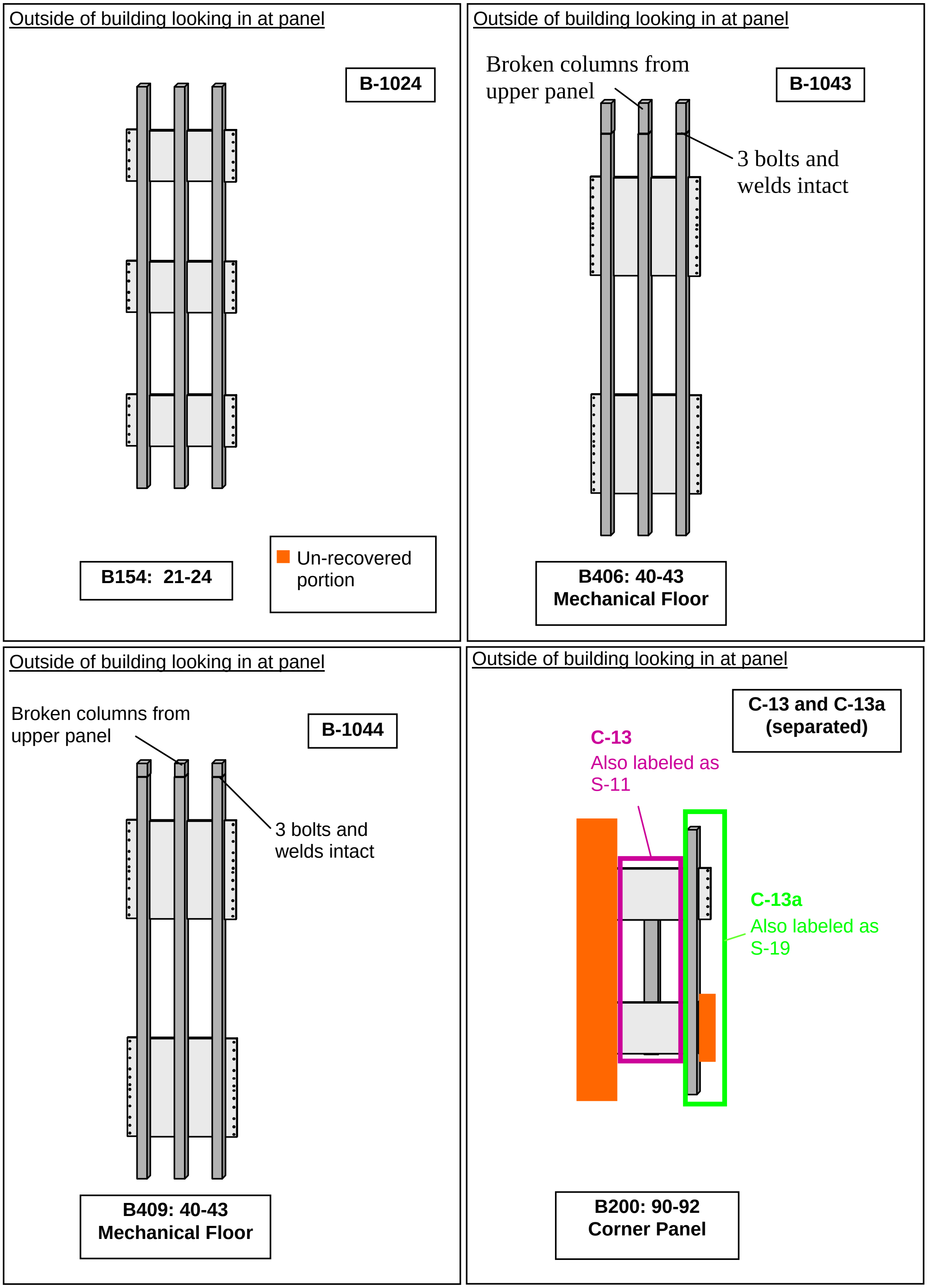


Figure F–11. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 2.

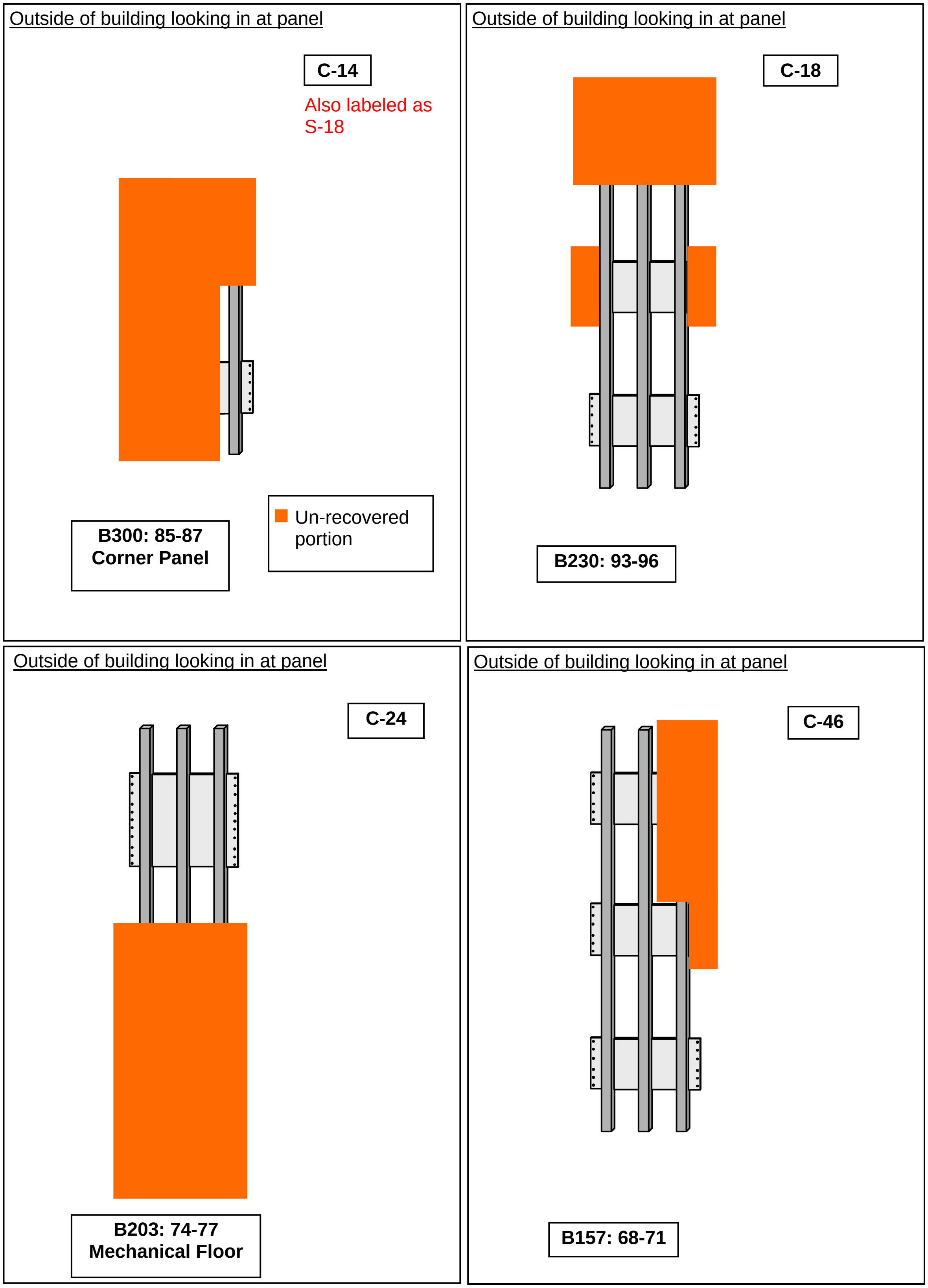


Figure F–11. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 2 (continued).

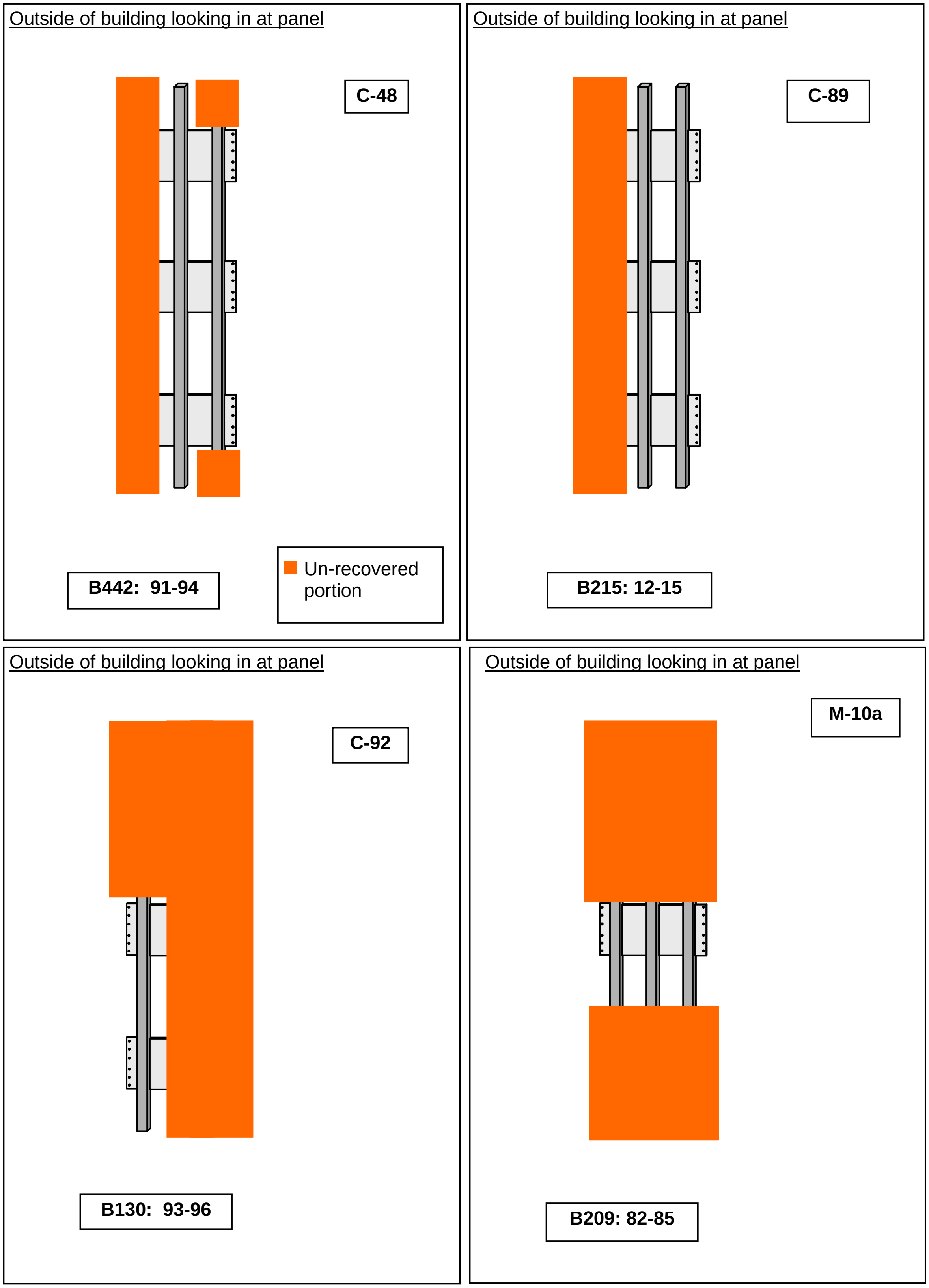


Figure F–11. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 2 (continued).

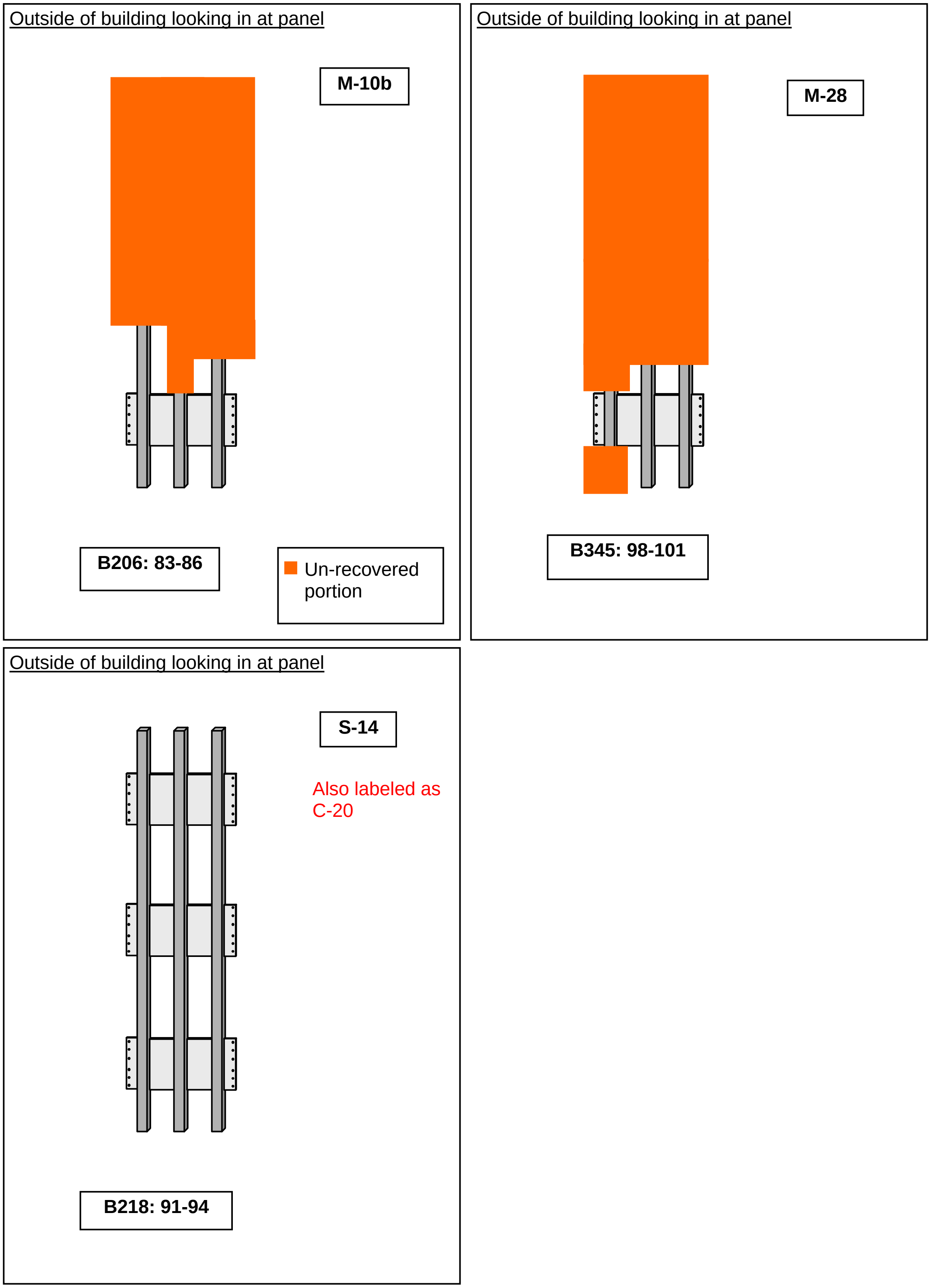


Figure F–11. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 2 (continued).

obscured: “x04 – 10x”. As the 24 ft section has both connector ends, it spanned only two floors and fit with the floor levels of 104–106. The second sample was C-88b, which did not have any stampings or markings, but was welded to C-88a (identified by stampings). A final sample, C-83, was also found among this group. While no markings were found on the sample, it was recorded as a core column due to its shape, which was very similar to C-90.

There were 13 other wide flange sections that had stampings and/or markings that did not correspond to the code as discussed above (Table F–4). Instead, there were typically three distinct grouping of numbers and/or letters. Two examples are:

Sample C-44: “59 S 563”

Sample M-17: “163 9 62”

Given the position of the last grouping and the numbers typically found there, this is probably the derrick division. The first two most likely indicate the as-built locations of the pieces within the building. NIST is still investigating the identification of these samples.

Floor trusses were also recovered; however, attempts to identify their specific as-built locations within the buildings were not successful. No stampings were found. Of the 23 pieces held by NIST, 8 are of significant size but are badly tangled and twisted as a result of the collapse and subsequent handling of the material. The remaining pieces consist of shorter sections of chord and rod material in addition to welded sections that connected the trusses to the floor seats.

At present, there are seven samples from WTC 5, all in the GZ-series (see Attachment 1.2.9). These are coupons that were removed at the WTC site and held by GMS, LLP. They were subsequently sent to NIST once the Investigation officially began.

No structural elements have been positively identified from WTC 7. However, the columns were fabricated from conventional 36 ksi, 42 ksi, and 50 ksi steel that complied with ASTM specifications.

F.5 STRUCTURAL STEEL ELEMENTS OF SPECIAL IMPORTANCE

Of the 41 exterior column panels and 12 core columns positively identified, many were considered especially important to this Investigation. Two major categories of steel are considered to be of special value:

- Samples located in or around the floors impacted by the airplane
- Samples that can represent 1 of 14 grades of steel specified for the exterior columns, 1 of 4 grades of steel specified for the core columns, and 1 of the 2 grades of steel for the floor trusses

F.5.1 Samples Located in or Around the Floors Impacted by the Airplane

Interpretation of the photographic evidence revealed that damage to WTC 1 due to aircraft impact occurred from floor 94 to floor 99 and was bounded by columns 111 through 152. For WTC 2, the impact area was lower with damage found from floor 77 to floor 85. While the damage appears to be bordered by column lines 411 and 440, columns closer to the southeast corner of the building may also have been affected. However, few images were obtained where smoke is not obscuring this portion of the south face of WTC 2 to complete the analysis. From this information, NIST was able to determine which perimeter panels and core columns could be used to comment on damage and possible failure mechanisms in this area. Figure F–12 shows the sample overlay of the exterior panels in NIST's possession in and around the impact zone of WTC 1. Sample C-80, a core column, was also identified as residing near the impact zone. The recovered portion of each column is approximately represented in this image. Unfortunately, there were no similar corresponding exterior panels for WTC 2, but two core columns were recovered, (Fig. F–13). Later reports will describe the type of damage and failure mechanisms associated with each sample.

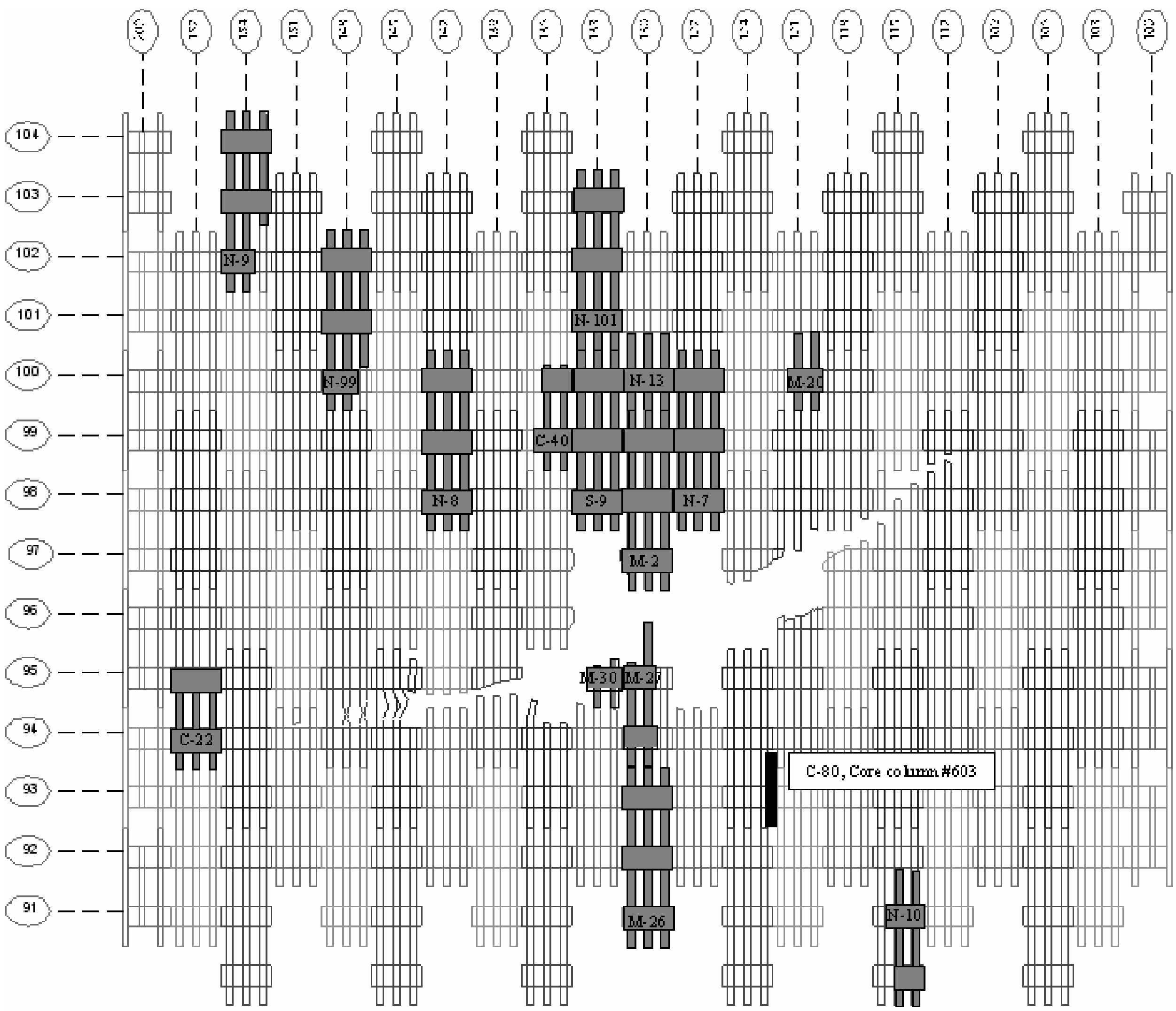


Figure F–12. Interpreted column damage, from photographic evidence, to WTC 1, with overlay of samples in NIST's possession.

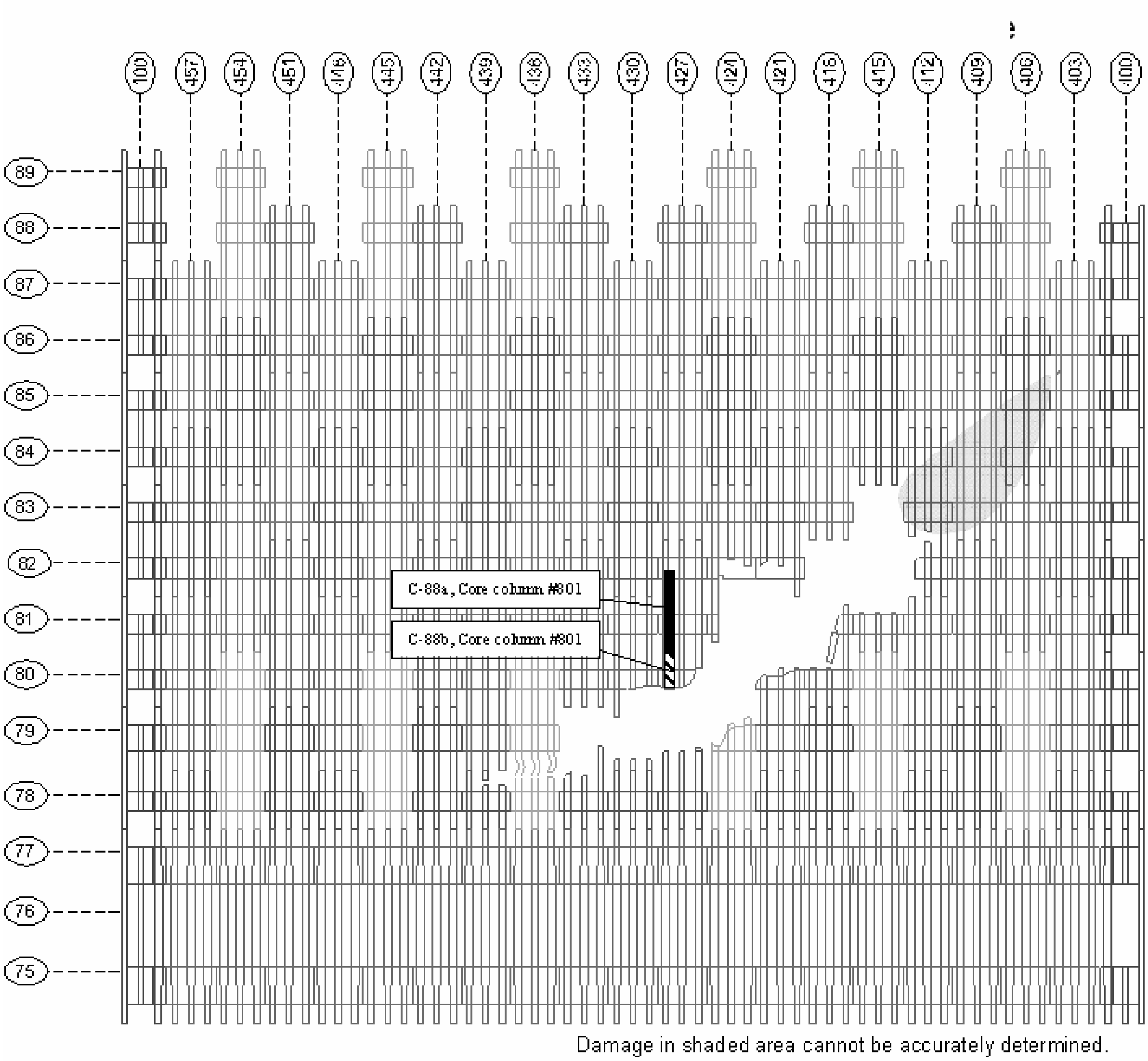


Figure F–13. Interpreted column damage, from photographic evidence, to WTC 2, with overlay of samples in NIST's possession.

F.5.2 Samples Representing the Various Types of Steel Specified in the Design Drawings

The other grouping of samples that was deemed important was that which belonged to one of the different grades of steel specified in the buildings' construction. The following minimum yield strengths, in ksi (1 ksi equals 1,000 pounds per square inch), were specified for each structural element:

- Columns of the exterior panels: 36, 42, 45, 46, 50, 55, 60, 65, 70, 75, 80, 85, 90, and 100
- Core columns: 36, 42, 46, and 50
- Floor truss material: 36 and 50

From the recovered steel, sufficient representative samples from each important class of steel groups are available for a full examination (i.e., chemical, metallurgical, and mechanical property analyses) to investigate why and how WTC 1 and WTC 2 collapsed following the initial impact of the aircraft. From Table F–11, it can be seen that 10 of the 14 types of steel specified for the columns are represented, and 10 of the 12 grades of spandrel material have been identified. Additionally, sample ASCE-3 (as-built location in the building not identified) has a flange stamping of 45 for the minimum yield requirement, which would increase the total number of perimeter column material types to 11. One important note is that from the observed stampings of the recovered elements and other documents (see Appendix C), it appears that 100 ksi steel was substituted for the 85 ksi and 90 ksi grades in the construction of the exterior panels (Table F–6). Considering both column and spandrel material, samples of all grades specified for the perimeter panels are available. While only two of the four grades of steels were obtained (36 ksi and 42 ksi) for the core columns (Table F–3), 99 percent of the total number of core columns were fabricated from these two grades. For the floor truss material, the samples could not be identified as to their precise, as-built locations within the buildings. However, initial chemical and mechanical property analyses have shown that both minimum yield strength materials specified have been recovered. Characterization of these samples will be covered extensively in a later report.

F.6 SUMMARY

NIST has 236 samples from the WTC buildings, the majority belonging to WTC 1 and WTC 2. These samples represent roughly 0.25 percent to 0.5 percent of the 200,000 tons of structural steel used in the construction of the two towers. NIST believes the collection of steel from the WTC towers is sufficient for the Investigation. This assertion is drawn from the following two statements. First, recovery of material from locations in or near the impact and fire damaged regions of WTC 1 and WTC 2 was remarkably good, including four exterior panels directly hit by the airplane and three core columns located within these areas. Second, sufficient representative samples exist for all 14 grades of exterior panel material, 2 grades of the core column material (which represents 99 percent, by total number, of columns), and both grades for the floor truss material.

This report identifies the structural steel elements recovered from the WTC towers. Later reports will determine the physical and mechanical properties of the steels and weld metal and the characteristics of the metal, weldments, and connections from WTC buildings. Additionally, a damage assessment/failures mode examination of the recovered structural steel elements will be performed. This information will be utilized in an effort to determine why and how WTC 1 and WTC 2 collapsed following the initial impact of the aircraft.

Table F–11. Listing of recovered exterior column panels with specified minimum yield strengths and thicknesses for columns[a] and spandrels.

NIST NAME	Bldg	Center Column #	Stories located at splice: Lower	Stories located at splice: Upper	Panel Type	COLUMN 1 Column Type	COLUMN 1 FY (ksi)	COLUMN 2 Column Type	COLUMN 2 FY (ksi)	COLUMN 3 Column Type	COLUMN 3 FY (ksi)	LOWER SPANDREL Thickness (in)	LOWER SPANDREL FY (ksi)	MIDDLE SPANDREL Thickness (in)	MIDDLE SPANDREL FY (ksi)	UPPER SPANDREL Thickness (in)	UPPER SPANDREL FY (ksi)
B-1024	WTC 2	154	21	24	300	152	50	150	50	149	50	1.25	36	1.25	36	1.25	36
CC	WTC 1	124	70	73	300	133	50	133	50	~~133~~	~~50~~	0.5625	36	0.5625	36	0.5625	36
M-26	WTC 1	130	90	93	300	125	50	125	55	125	50	0.375	36	0.375	36	0.375	36
M-27	WTC 1	130	93	96	300	~~123~~	~~55~~	123	55	124	50	0.375	36	0.375	36	~~0.375~~	~~36~~
M-2	WTC 1	130	96	99	300	122	55	122	55	122	55	0.375	36	0.375	42	0.375	36
M-30	WTC 1	133	94	97	300	123	55	123	55	~~123~~	~~55~~	0.375	36	~~0.375~~	~~36~~	~~0.375~~	~~42~~
C-18	WTC 2	230	93	96	300	120	55	120	55	120	55	0.375	45	0.375	42	~~0.375~~	~~42~~
N-9	WTC 1	154	101	104	300	120	55	120	55	120	55	0.375	42	0.375	36	0.375	36
M-20	WTC 1	121	99	102	300	120	55	120	55	~~120~~	~~55~~	0.375	42	~~0.375~~	~~42~~	~~0.375~~	~~36~~
N-13	WTC 1	130	99	102	300	120	55	121	55	121	55	0.375	42	~~0.375~~	~~42~~	~~0.375~~	~~36~~
N-101	WTC 1	133	100	103	300	120	55	120	55	120	55	0.375	42	0.375	36	0.375	36
S-9	WTC 1	133	97	100	300	122	55	122	55	122	55	0.375	36	0.375	42	0.375	36
N-10	WTC 1	115	89	92	300	125	55	125	55	~~125~~	~~55~~	0.375	36	0.375	42	~~0.375~~	~~42~~
C-40	WTC 1	136	98	101	300	121	55	121	60	~~121~~	~~60~~	0.375	42	0.375	36	~~0.375~~	~~42~~
C-89	WTC 2	215	12	15	300	143	55	145	50	~~147~~	~~50~~	1.375	36	1.375	36	1.375	36
N-7	WTC 1	127	97	100	300	121	60	121	55	121	55	0.375	42	0.375	42	0.375	42
C-92	WTC 2	130	93	96	300	~~123~~	~~60~~	~~123~~	~~60~~	124	60	0.375	42	0.375	42	~~0.375~~	~~42~~
C-93	WTC 1	339	99	102	300	~~121~~	~~60~~	121	60	~~121~~	~~60~~	0.375	42	~~0.375~~	~~42~~	~~0.375~~	~~42~~
K-1	WTC 1	209	97	100	300	120	60	120	60	120	60	0.375	42	~~0.375~~	~~42~~	~~0.375~~	~~42~~
K-2	WTC 1	236	92	95	300	~~120~~	~~60~~	120	60	~~120~~	~~60~~	0.375	42	0.375	42	~~0.375~~	~~42~~
N-8	WTC 1	142	97	100	300	121	60	121	60	121	60	0.375	42	0.375	42	0.375	42
C-48	WTC 2	442	91	94	300	120	65	120	65	~~120~~	~~65~~	0.375	45	0.375	45	0.375	42
N-99	WTC 1	148	99	102	300	120	65	120	65	120	65	0.375	45	0.375	42	0.375	42
S-14	WTC 2	218	91	94	300	120	70	120	65	120	65	0.375	46	0.375	45	0.375	45
M-28	WTC 2	345	98	101	300	120	70	120	70	120	70	0.375	45	~~0.375~~	~~45~~	~~0.375~~	~~45~~
C-55	WTC 1	209	94	97	300	~~120~~	~~70~~	120	70	~~120~~	~~70~~	0.375	46	~~0.375~~	~~45~~	~~0.375~~	~~45~~
S-10	WTC 1	224	92	95	300	~~120~~	~~70~~	120	70	120	70	0.375	50	0.375	46	~~0.375~~	~~45~~
S-1	WTC 1	433	79	82	300	123	70	123	70	~~123~~	~~70~~	0.4375	50	~~0.4375~~	~~46~~	~~0.4375~~	~~45~~
N-1	WTC 1	218	82	85	300	~~123~~	~~75~~	123	75	123	70	0.4375	50	0.375	50	0.375	50
C-46	WTC 2	157	68	71	300	129	65	128	70	126	80	0.625	65	0.625	65	0.5625	65
N-12	WTC 1	206	92	95	300	120	75	120	75	~~120~~	~~75~~	0.375	50	0.375	50	0.375	46
C-22	WTC 1	157	93	96	300	120	80	120	75	120	80	0.375	65	0.375	60	~~0.375~~	~~60~~
C-25	WTC 1	206	89	92	300	~~120~~	~~80~~	~~120~~	~~80~~	120	80	0.375	55	0.375	55	~~0.375~~	~~55~~
B-1044	WTC 2	409	40	43	400	335	85	335	80	335	85	0.9375	60	n/a	n/a	0.9375	50
M-10a	WTC 2	209	82	85	300	120	85	120	85	120	85	~~0.4375~~	~~60~~	0.375	60	~~0.375~~	~~60~~
M-10b	WTC 2	206	83	86	300	120	85	120	85	120	85	0.375	60	~~0.375~~	~~60~~	~~0.375~~	~~55~~
C-10	WTC 1	451	85	88	300	120	85	120	85	120	90	0.375	60	0.375	60	0.375	60
B-1043	WTC 2	406	40	43	400	334	90	334	90	335	85	0.9375	65	n/a	n/a	0.9375	50
C-24	WTC 2	203	74	77	400	325	100	325	100	325	100	~~0.5624~~	~~70~~	n/a	n/a	0.5625	80
C-13 and C-13a	WTC 2	200	90	92	210	120	100	520	100	~~120~~	~~100~~	n/a	n/a	0.375	70	0.375	70
C-14	WTC 2	300	85	87	210	120	100	~~522~~	~~100~~	~~122~~	~~100~~	n/a	n/a	0.375	75	~~0.375~~	~~75~~

a. Columns 1, 2, and 3 are left to right viewed from inside the building.
Strike through indicates section is missing.

F.7 REFERENCES

F.7.1 References from Publicly Available Sources

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PONYA (Port of New York Authority). 1967. to Steel Fabricators, Detailers, and Erectors on "Shop Drawing Procedures and Marking Systems," May 1.

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Attachment 1
DATA ON RECOVERED WTC STEEL

1.1 DATABASE OF RECOVERED STEEL

Table 1–1. List of all WTC steel elements recovered for NIST investigation.

In FEMA report?	NIST Name	Type	Brief Description	Markings	Bldg	Column	Floors	Location
Y	C-67	C	1 column, rest unknown					205
Y	C-68	C	1 column, upper 1/2					205
Y	C-69	W	Wide flange					205
Y	C-70 (formerly U-9)	W	Wide flange					205
Y	C-71	W	Wide flange	904A 77-80	WTC 1	904	77 - 80	PL
Y	C-72b	W	Wide flange					205
Y	C-73	C	1 column, upper 1/2					205
Y	C-74	W	Wide flange					205
Y	C-75	C	portion of 1 column and spandrel, rest unknown					236
Y	C-76	W	Wide flange					205
Y	C-77	C	2 columns from different panels attached at spandrel, 1/3rd of each					205
Y	C-78 (formerly U-8)	W	Wide flange					205
Y	C-79	RB	Rectangular column, FEMA reported possible core column	101A 81 - 85 - 87 -92 52	WTC 1			PL
Y	C-80	W	Wide flange, FEMA reported possible core columns	603A 92-95 <51>	WTC 1	603	92-95	PL
Y	C-81	W	Wide flange					205
Y	C-82	W	Wide flange					205
Y (NSF)	C-83	RB	Heavy rectangular column, FEMA reported as possible core column	No ID, similar to other core column				PL
Y (NSF)	C-84	C	1 full column					PL
Y (NSF)	C-85	W	Wide flange					205
Y	C-87	W	Thick Wide flange					205
Y	C-88a	RB	Not typical column section, both webs are same length, FEMA reported possible core column	801B 80-83	WTC 2	801	80-83	PL
	C-88b		Welded to above piece	801B 77-80	WTC 2	801	77-80	PL
	C88c (formerly U-22)	O	Broke off C-88					PL
Y (NSF)	C-89	C	2 full columns	B 215: 12 - 15	WTC 2	215	12 - 15	PL
Y (NSF)	C-90	RB	Heavy rectangular column, FEMA reported as possible core column	701B 12 - 15	WTC 2	701	12 - 15	PL
Y	C-91	Ch	Channel					236
Y	C-92	C	Partial of single column	B13x: 93-96	WTC 2	130	93 - 96	PL
Y	C-93	C	Partial of single column	339: 99 - 102	WTC 1	339	99 - 102	PL
	C-94	O	May be some type of brace, rectangular box construction					PL
	C-95	Ch	Channel					236
	C-96	Ch	Channel					236
	C-97	Ch	Channel					236
	C-98	Ch	Channel					236
	C-99	Ch	Channel					236
	C-100	J	Possible angle from a floor truss					PL
	C-101 (formerly U-16)	RB	Similar to corner column, but much thinner	78A 10 27 50				PL
	C-102	C	Partial of single column					205
	C-103	O	Square-tube construction					PL
	C-104	J	Possible angle from a floor truss					PL
	C-105	Ch	Channel					236
	C-106 (formerly U-18)	J	Small piece of floor truss					202
	C-107 (formerly U-19)	Ch	Channel					236
	C-108	B	Three sheared bolts					Lab
	C-109	B	Single bolt sheared					Lab
	C-110	B	Bolt and nut					Lab
	C-111	B	Bolt and washer					Lab
	C-112	B	Single bolt sheared					Lab
	C-113	B	Two sheared bolts with washers					Lab
	C-114	B	Sheared bolt with nut					Lab
	C-115	J	Pig-tailed piece from floor truss					Lab
	C-116	H	Damper					Lab
	C-117	C	3 columns, lower 1/3	101-104				PL
	C-118	Ch	Channel					236
	C-119A	O	Square-tube construction					PL
	C-119B	O	Square-tube construction					PL
	C-120	O	Square-tube construction					PL
	C-121	O	Square-tube construction					PL
	C-122	J	Piece of floor truss					PL
	C-123	W	Small Wide flange					205
	C-124	Ch	Channel					236
	C-125	Ch	Channel					236
	C-126	W	Wide flange					205
	C-128	Ch	Channel					B
	C-129	Ch	Channel					236
	C-130	W	Wide Flange					205
	C-131	J	Small portion of floor truss with cement					202

Table 1–1. List of all WTC steel elements recovered for NIST investigation (continued).

In FEMA report?	NIST Name	Type	Brief Description	Markings	Bldg	Column	Floors	Location
	C-132	J	Piece of floor truss					PL
	C-133	C	1 column, bottom 1/3rd of unknown location					205
	C-134	Ch	Channel					236
	C-135	O	May be some type of brace, rectangular box construction					PL
	C-137a	J	Piece of floor truss					PL
	C-137b	J	Piece of floor truss					PL
	C-137c	J	Piece of floor truss					PL
	C-137d	J	Piece of floor truss					PL
	C-137f	J	Piece of floor truss					PL
	C-138	W	Small wide flange					205
	C-139	Ch	Channel					236
	C-140	J	Piece of angle					PL
	C-141	Ch	Channel					236
	C-142	W	Wide flange					205
	C-143	Ch	Channel					236
	C-144	Ch	Channel					236
	C-145	Ch	Channel					236
	C-146a	O	Mangled ball of steel and concrete					202
	C-146b	J	Piece of floor truss					PL
	C-147	Ch	Channel					236
	C-148	Ch	Channel					236
	C-149	J	Piece of floor truss					PL
	C-150	W	Wide flange					205
	C-151	J	Piece of floor truss					PL
	C-152	Ch	Channel					236
	C-153	Ch	Channel					236
	C-154	RB	Thin rectangular beam with supports	825: 107-108 52				PL
	C-155 (formerly U-5)	W	Wide flange	904A 83-86	WTC 1	904	83-86	PL
	C-156 (formerly U-17)	O	Square-tube construction					PL
Y	CC	C	2 full columns	124: 73-70	WTC 1	124	70-73	PL
Y	DD	C	1 Column, spans 1 floor and has end plates on both ends					205
Y	FF	C	Single, thick column					205
	GZ-1	Cn5	Received from D. Sharp, coupon from Bldg #5					Lab
	GZ-2	Cn5	Received from D. Sharp, coupon from Bldg #5					Lab
	GZ-3	Cn5	Received from D. Sharp, coupon from Bldg #5					Lab
	GZ-4	Cn5	Received from D. Sharp, coupon from Bldg #5					Lab
	GZ-5	Cn5	Received from D. Sharp, coupon from Bldg #5					Lab
	GZ-6	Cn5	Received from D. Sharp, coupon from Bldg #5					Lab
	GZ-7	Cn5	Received from D. Sharp, coupon from Bldg #5					Lab
Y	HH or S-2	W	Wide flange, FEMA reported possible core column	605A 98-101	WTC 1	605	98-101	PL
floors in report)	K-1 or K-13	C	3 columns, lower 1/3rd	209: 97-100	WTC 1	209	97-100	202
Y	K-2 or K-40	C	1 column, lower 2/3rds	236: 92-95	WTC 1	236	92-95	PL
Y	K-10	Cn	Flange coupon received from Gross, July 29, 2002					Lab
Y	K-11	Cn	Flange coupon received from Gross, July 29, 2002					Lab
Y	K-12	Cn	Flange coupon received from Gross, July 29, 2002					Lab
Y	K-13	Cn	Flange coupon received from Gross, July 29, 2002					Lab
Y	K-14	Cn	Flange coupon received from Gross, July 29, 2002					Lab
Y	K-15	Cn	Flange coupon received from Gross, July 29, 2002					Lab
Y	K-16	C	1 full column, thick, looks very corroded					PL
	K-16a (formerly U-23)	C	Fell off of K-16 while moving					PL
Y	K-18	Cn	Flange coupon received from Gross, July 29, 2002					Lab
Y	K-19a	Cn	Flange coupon received from Gross, July 29, 2002					Lab
Y	K-19b	Cn	Flange coupon received from Gross, July 29, 2002					Lab
Y	K-50a	O	Rectangular slab of steel with bolts, received from D. Sharp, SEAoNY					Lab
Y	K-50b	O	Rectangular slab of steel with bolts, received from D. Sharp, SEAoNY					Lab
Y	K-50c	O	Rectangular slab of steel with bolts, received from D. Sharp, SEAoNY					Lab
Y	M-2	C	Full panel	-9 <63>	WTC 1	130	96-99	PL
Both are in report but listed separately	M-4 or M-5	C	3 columns, upper 2/3rds					205
Y	M-10a	C	3 columns, unknown location	B209: 82-85	WTC 2	206	82-85	PL
Y	M-10b	C	3 columns, lower 1/2	B206: 83-86	WTC 2	206	83-86	PL
Y	M-11	W	Wide flange					205
Y	M-17	W	Wide flange or I-beam, 1' flange, 2' web, 50 ft to 60 ft long	163 (9) 62				205
	M-17a (formerly U-24)	O	Fell off of M-17 while moving					202

Table 1–1. List of all WTC steel elements recovered for NIST investigation (continued).

In FEMA report?	NIST Name	Type	Brief Description	Markings	Bldg	Column	Floors	Location
	M-18	RB	Large box beam, 19 in. x 21 in. x 17.5 ft long					205
	M-19	C	2 columns, upper 1/3rd					205
	M-20	C	2 columns, lower 1/3rd	A121: 99-102	WTC 1	121	99-102	PL
	M-22	RB	Large box beam, 19 in. x 26.5 in. x 9.5 ft long, etc.					205
	M-23	W	Possibly part of Wide flange or I-beam	F 2010				PL
	M-24	Ch	Channel					236
	M-25	J	Small piece of floor truss					202
	M-26	C	3 full columns	A130: 90-93	WTC 1	130	90-93	PL
	M-26 associated	B	8 bolts and a nut					Lab
	M-27	C	2 columns, lower 3/4ths	A130: 93-96	WTC 1	130	93-96	202
	M-28	C	3 columns, lower 1/4th	B345: 9x - 1xx	WTC 2	345	98 - 101	PL
	M-29	O	5 ft piece of strapping					202
	M-30	C	2 columns, lower 1/3rd	_33: 94-97	WTC 1	133	94-97	202
	M-30 associated	O	Pieces of glass, plexiglass, other rubble					Lab
	M-31	J	Pieces of floor truss					Lab
	M-32	J	Pieces of floor truss					Lab
	M-33	W	Wide flange					205
	M-34	Ch	Channel					B
	M-35	CC	Corner column					205
	M-36	J	Thick angle					PL
	M-37	W	Wide flange	130 (8?–92) <50>				205
	M-38	W	Wide flange	Fy 42				PL
Y	N-1	C	2 full columns	2_8: 82-85	WTC 1	218	82-85	PL
Y	N-3	C	1 column, upper 1/2					236
Y	N-4	C	1 column, middle 1/3rd					236
Y	N-5	O	Part of spandrel plate with bolts					PL
Y	N-6 (formerly U-2)	C	1 column, length of spandrel, crushed					236
Y (as M-3)	N-7 or M-3	C	3 full columns	127: 97-100	WTC 1	127	97-100	PL
Y (as M-7)	N-8 or M-7	C	Full panel	A142: 97-100	WTC 1	142	97-100	PL
Y (as M-8)	N-9 or M-8	C	Almost full panel, missing lower 1/3rd of 1 column	A154: 101-104	WTC 1	154	101-104	PL
Y (as M-15)	N-10 or M-15	C	2 columns, lower 2/3rds	A115: 89-92	WTC 1	115	89-92	PL
Y (as M-9)	N-11 or M-9	C	3 columns, upper 2/3rds					205
Y (as M-13)	N-12 or M-13	C	2 full columns	_06: 92-95	WTC 1	206	92-95	PL
Y (as M-14)	N-13 or M-14	C	3 columns, lower 1/3rd	A130: 99-102	WTC 1	130	99-102	B
Y (as M-16)	N-99 or M-16	C	Almost full panel, missing lower 1/3rd of 1 column	A148: 99-102	WTC 1	148	99-102	PL
	N-101 or M-21	C	3 full columns	A133: 100-103	WTC 1	133	100-103	PL
Y (as C-19)	N-N or C-19	C	1 column, lower 1/2					205
Y (as EE)	S-1 or EE	C	2 columns, lower 1/3rd	A433: 79-82	WTC 1	433	79-82	PL
Y (as C-50)	S-3 or C-50	C	1 column, unknown 1/2					205
Y (as C-63)	S-9 or C-63	C	Full panel	A133: 97-100	WTC 1	133	97-100	PL
Y (as C-17)	S-10 or C-17	C	2 columns, lower 1/2	224: 92-95	WTC 1	224	92-95	PL
Y (as C-20)	S-14 or C-20	C	Full panel	B218: 91-94	WTC 2	218	91-94	PL
	SM-2	W	I-beam					205
Y (as N-2)	T-1 or N-2	J	Floor truss material					202
	U-6	C	3 columns, upper 1/4					236
	U-15	C	Partial of single column					205
	U-25	O	Unknown Wide flange with concrete	<North> 84-155 A8 Div 2				205
Y	W-14A or A	W	Heavy Wide flange					205
Y	W-14B	W	Heavy Wide flange					PL

Key: 202, Bldg 202, high bay; 205, Bldg 205, parking lot; PL, Bldg 202, parking lot; 236, Bldg 236, parking lot; B, bolt; BT, bowtie section of exterior wall; C, flat wall, exterior column panel section; CC, corner panel section of exterior wall; Ch, channel; Cn, coupon of exterior column; Cn5, coupon from WTC 5; H, hanger; J, floor truss; NSF, pieces contributed by A. Asteneh salvaged under NSF contract; O, other; RB, rectangular, built-up box column; W, wide flange section; Lab, Bldg 223, Rm B253; JFK, Hanger 17, JFK Airport; JFK/PL, Main piece at JFK, portion at NIST.

Table 1–2. List of identified exterior panel sections.

In FEMA report?	NIST Name	Type	Brief Description	Markings	Bldg	Column	Floors
	B-1024	C	3 full columns	B154: 21-24	WTC2	154	21-24
	B-1043	C	Mechanical floor, 3 full columns	B406: 40-43	WTC2	406	40-43
	B-1044	C	Mechanical floor, 3 full columns	B409: 40-43	WTC2	409	40-43
Y	C-10	C	Full panel	451: 85-88	WTC1	451	85-88
Y	C-13 or S-11	CC	Single rectangular column with large spandrels	B200: 90-92	WTC2	200	90-92
Y	C-13a or S-19	C	Partial of single column	B200: 90-92	WTC2	200	90-92
Y	C-14 or S-18	C	1 column, lower 1/3rd	B300: 85-87	WTC2	300	85-87
Y	C-18	C	3 columns, bottom 2/3rds	B230: 93-96	WTC2	230	93-96
Y	C-22	C	3 columns, lower 1/2, mangled	A157: 93-96	WTC1	157	93-96
Y	C-24	C	3 columns, upper 1/2, columns change dimensions	B203: 74-77	WTC2	203	74-77
Y	C-25	C	1 column, lower 1/2	A206: 89-92	WTC1	206	89-92
Y	C-40	C	2 columns, lower 2/3rds	A136: 98-101	WTC1	136	98-101
Y	C-46	C	Nearly 3 full columns	B157: 68-71	WTC2	157	68-71
Y	C-48 or S-5	C	Nearly 2 full columns	B442: 91-94	WTC2	442	91 - 94
Y	C-55	C	1 column, lower 1/3rd	209: 94-97	WTC1	209	94-97
Y (NSF)	C-89	C	2 full columns	B215: 12 - 15	WTC2	215	12 - 15
Y	C-92	C	Partial of single column	B13x: 93-96	WTC2	130	93 - 96
Y	C-93	C	Partial of single column	339: 99 - 102	WTC1	339	99 - 102
Y	CC	C	2 full columns	124: 73-70	WTC1	124	70-73
Does not match	K-1 or K-13	C	3 columns, lower 1/3rd	209: 97-100	WTC1	209	97-100
Y	K-2 or K-40	C	1 column, lower 2/3rds	236: 92-95	WTC1	236	92-95
Y	M-2	C	Full panel	-9 <63>	WTC1	130	96-99
	M-10a	C	3 columns, 1/3rd, not labeled but attached to M-10b	B209: 82-85	WTC2	209	82-85
Y	M-10b	C	3 columns, lower 1/2	B206: 83-86	WTC2	206	83-86
	M-20	C	2 columns, lower 1/3rd	A121: 99-102	WTC1	121	99-102
	M-26	C	3 full columns	A130: 90-93	WTC1	130	90-93
	M-27	C	2 columns, lower 3/4ths	A130: 93-96	WTC1	130	93-96
	M-28	C	3 columns, lower 1/4th	B345: 9x - 1xx	WTC2	345	98 - 101
	M-30	C	2 columns, lower 1/3rd	_33: 94-97	WTC1	133	94-97
Y	N-1	C	2 full columns	2_8: 82-85	WTC1	218	82-85
Y (as M-3)	N-7 or M-3	C	3 full columns	127: 97-100	WTC1	127	97-100
Y (as M-7)	N-8 or M-7	C	Full panel	A142: 97-100	WTC1	142	97-100
Y (as M-8)	N-9 or M-8	C	Almost full panel, missing lower 1/3rd of 1 column	A154: 101-104	WTC1	154	101-104
Y (as M-15)	N-10 or M-15	C	2 columns, lower 2/3rds	A115: 89-92	WTC1	115	89-92
Y (as M-13)	N-12 or M-13	C	2 full columns	_06: 92-95	WTC1	206	92-95
Y (as M-14)	N-13 or M-14	C	3 columns, lower 1/3rd	A130: 99-102	WTC1	130	99-102
	N-99 or M-16	C	Almost full panel, missing lower 1/3rd of 1 column	A148: 99-102	WTC1	148	99-102
	N-101 or M-21	C	3 full columns	A133: 100-103	WTC1	133	100-103
	S-1 or EE	C	2 columns, lower 1/3rd	A433: 79-82	WTC1	433	79-82
Y	S-9 or C-63	C	Full panel	A133: 97-100	WTC1	133	97-100
Y	S-10 or C-17	C	2 columns, lower 1/2	224: 92-95	WTC1	224	92-95
Y	S-14 or C-20	C	Full panel	B218: 91-94	WTC2	218	91-94

Table 1–3. List of partially identified exterior panel sections.

In FEMA report?	NIST Name	Type	Brief Description	Markings	Bldg	Column	Floors
	C-117	C	3 columns, lower 1/3	101-104	NA		101-104

Table 1–4. List of unidentified exterior panel sections.

In FEMA report?	NIST Name	Type	Brief Description	Location
Y	C-28B (formerly U-4)	CC	Corner column, in 2 pieces	205
	M-35	CC	Corner column	205
Y	AA (formerly U-7)	C	2 full columns, thick walled	PL
Y (NSF)	ASCE-2	C	1 full column, C3, only 2 spandrels that are large	PL
Y (NSF)	ASCE-3	C	1 column, bottom 1/3rd of left column	PL
Y	BB	C	Single, thick column	205
Y	C-11	C	2 columns, upper 2/3rds	205
Y	C-15 (formerly U-20)	C	Partial of single column	205
Y	C-16	C	1 column, upper 1/3rd	205
Y	C-16a	C	Fell off during moving of C-16	205
Y	C-28 (formerly U-1)	C	1 column of unknown location	205
Y	C-32	C	1 column, upper 1/3rd	236
Y	C-41	C	1 column, lower 2/3rds	205
Y	C-43	C	1 column, lower 1/2	205
	C-47	C	3 columns, upper 1/2	236
Y	C-49 or S-6	C	portion of 1 column	236
Y	C-51	C	2 columns, upper 1/2	205
Y	C-52	C	1 column, upper 2/3rds	205
Y	C-54	C	1 column,small piece with extended outer web	205
Y	C-64	C	1 column with a lot missing	205
Y	C-67	C	1 column, rest unknown	205
Y	C-68	C	1 column, upper 1/2	205
Y	C-73	C	1 column, upper 1/2	205
Y	C-75	C	portion of 1 column and spandrel, rest unknown	236
Y	C-77	C	2 columns from different panels attached at spandrel, 1/3rd of each	205
Y (NSF)	C-84	C	1 full column, stampings on front face	PL
	C-102	C	Partial of single column	205
	C-133	C	1 column, bottom 1/3rd of unknown location	205
Y	DD	C	1 Column, spans 1 floor and has end plates on both ends	205
Y	FF	C	Single, thick column	205
Y	K-16	C	1 full column, thick, looks very corroded	PL
	K-16a (formerly U-23)	C	Fell off of K-16 while moving	PL
Both are in report but listed separately	M-4 or M-5	C	3 columns, upper 2/3rds	205
	M-19	C	2 columns, upper 1/3rd	205
Y	N-3	C	1 column, upper 1/2	236
Y	N-4	C	1 column, middle 1/3rd	236
Y	N-6 (formerly U-2)	C	1 column, length of spandrel, crushed	236
Y (as M-9)	N-11 or M-9	C	3 columns, upper 2/3rds	205
Y (as C-19)	N-N or C-19	C	1 column, lower 1/2	205
Y (as C-50)	S-3 or C-50	C	1 column, unknown 1/2	205
	U-6	C	3 columns, upper 1/4	236
	U-15	C	Partial of single column	205
Y	K-10	Cn	Flange coupon received from Gross, July 29, 2002	Lab
Y	K-11	Cn	Flange coupon received from Gross, July 29, 2002	Lab
Y	K-12	Cn	Flange coupon received from Gross, July 29, 2002	Lab
Y	K-13	Cn	Flange coupon received from Gross, July 29, 2002	Lab
Y	K-14	Cn	Flange coupon received from Gross, July 29, 2002	Lab
Y	K-15	Cn	Flange coupon received from Gross, July 29, 2002	Lab
Y	K-18	Cn	Flange coupon received from Gross, July 29, 2002	Lab
Y	K-19a	Cn	Flange coupon received from Gross, July 29, 2002	Lab
Y	K-19b	Cn	Flange coupon received from Gross, July 29, 2002	Lab
	B-5004	BT	Bowtie section	JFK/PL
	B-5007	BT	Bowtie section	JFK/PL

Table 1–5. List of identified core columns.

In FEMA report?	NIST Name	Type	Brief Description	Markings	Bldg	Column	Floors
	B-1011	RB	Heavy rectangular column	508A: 51-54 <55>	WTC1	508	51-54
	B-6152-1	RB	Heavy rectangular column	803A: 15-18 <52>	WTC1	803	15-18
	B-6152-2	RB	Heavy rectangular column	504A: 33-36	WTC1	504	33-36
NSF	C-83	RB	Heavy rectangular column, FEMA reported as possible core column	No ID found, but similar to core column size and shape			
	C-88a	RB	Not typical column section, both webs are same length, FEMA reported possible core column	801B 80-83	WTC2	801	80-83
	C-88b		Welded to above column	801B 77-80	WTC2	801	77-80
NSF	C-90	RB	Heavy rectangular column, FEMA reported as possible core column	701B 12 - 15	WTC2	701	12 - 15
	C-30 or S-12	W	Wide flange	1008B x04 - 10x	WTC2	1008	104 - 106
	C-65 or S-8	W	Wide flange	904A (86-89) <52>	WTC 1	904	86-89
Y	C-71	W	Wide flange	904A 77-80	WTC1	904	77 - 80
	C-80	W	Wide flange, FEMA reported possible core columns	603A 92-95 <51>	WTC 1	603	92-95
	C-155 (formerly U-5)	W	Wide flange	904A 83-86	WTC1	904	83-86
	HH or S-2	W	Wide flange, FEMA reported possible core columns	605A 98-101	WTC1	605	98-101

Table 1–6. List of built-up box beams and wide flange sections with ambiguous stampings.

NIST Name	Type	Brief Description	Markings	Location
Markings but no knowledge of this coding				
C-79	RB	Rectangular column, FEMA reported possible core column	101A 81 - 85 - 87 -92 52	PL
C-101 (formerly U-16)	RB	Similar to corner column, but much thinner	78A 10 27 50	PL
C-154	RB	Thin rectangular beam with supports	825: 107-108 52	PL
C-26	W	Three connected Wide flanges	604 & 605 (107) <64> Fy 50	PL
C-44	W	Wide flange, FEMA reported possible core columns	59 S 563	PL
C-45	W	Wide flange, FEMA reported possible core columns	16 S2 563 Fy 50	PL
C-60	W	Wide flange, S-shaped	193 S1 57	PL
C-61	W	Wide flange	150 S 69	PL
C-62	W	Wide flange	224 (S) <48> Fy 50	PL
M-17	W	Wide flange or I-beam, 1ft flange, 2 ft web, 50-60 ft long	163 (9) 62 Fy 36	205
M-23	W	Possibly part of Wide flange or I-beam	F 2010	PL
M-37	W	Wide flange	130 (8?–92) <50>	205
M-38	W	Wide flange	Fy 42	PL

Table 1–7. List of unidentified wide flange sections.

In FEMA report?	NIST Name	Type	Brief Description	Location
	B-1022	W	Thick wide flange with severe bend	205
	B-1075	W	Wide flange	205
Y	C-29 (formerly U-10)	W	Wide flange	205
Y	C-35	W	Wide flange	205
Y	C-69	W	Wide flange	205
Y	C-70 (formerly U-9)	W	Wide flange	205
Y	C-72b	W	Wide flange	205
Y	C-76	W	Wide flange	205
Y	C-78 (formerly U-8)	W	Wide flange	205
Y	C-81	W	Wide flange	205
Y	C-82	W	Wide flange	205
Y (NSF)	C-85	W	Wide flange	205
Y	C-87	W	Thick Wide flange	205
	C-123	W	Small Wide flange	205
	C-126	W	Wide flange	205
	C-130	W	Wide flange	205
	C-138	W	Wide flange	205
	C-142	W	Wide flange	205
	C-150	W	Wide flange	205
Y	M-11	W	Wide flange	205
	M-18	RB	Large box beam	205
	M-22	RB	Large box beam	205
	M-33	W	Wide flange	205
	SM-2	W	Wide flange	205
Y	W-14A or A	W	Heavy Wide flange	205
Y	W-14B	W	Heavy Wide flange	PL

Table 1–8. List of recovered floor truss material.

In FEMA report?	NIST Name	Type	Brief Description	Location
Y	C-53	J	Floor truss	PL
Y	C-53B	J	Floor truss	PL
	C-100	J	Possible angle from a floor truss	PL
	C-104	J	Possible angle from a floor truss	PL
	C-106 (formerly U-18)	J	Small piece of floor truss	202
	C-115	J	Pig-tailed piece from floor truss	Lab
	C-122	J	Piece of floor truss	PL
	C-131	J	Small portion of floor truss with cement	202
	C-132	J	Piece of floor truss	PL
	C-137a	J	Piece of floor truss	PL
	C-137b	J	Piece of floor truss	PL
	C-137c	J	Piece of floor truss	PL
	C-137d	J	Piece of floor truss	PL
	C-137f	J	Piece of floor truss	PL
	C-140	J	Piece of angle	PL
	C-146b	J	Piece of floor truss	PL
	C-149	J	Piece of floor truss	PL
	C-151	J	Piece of floor truss	PL
	M-25	J	Small piece of floor truss	202
	M-31	J	Pieces of floor truss	Lab
	M-32	J	Pieces of floor truss	Lab
	M-36	J	Thick angle from floor truss	PL
Y (as N-2)	T-1 or N-2	J	Floor truss	202

Table 1–9. List of recovered channel material.

In FEMA report?	NIST Name	Type	Brief Description	Location
Y	C-91	Ch	Channel	236
	C-95	Ch	Channel	236
	C-96	Ch	Channel	236
	C-97	Ch	Channel	236
	C-98	Ch	Channel	236
	C-99	Ch	Channel	236
	C-105	Ch	Channel	236
	C-107 (formerly U-19)	Ch	Channel	236
	C-118	Ch	Channel	236
	C-124	Ch	Channel	236
	C-125	Ch	Channel	236
	C-128	Ch	Channel	B
	C-129	Ch	Channel	236
	C-134	Ch	Channel	236
	C-139	Ch	Channel	236
	C-141	Ch	Channel	236
	C-143	Ch	Channel	236
	C-144	Ch	Channel	236
	C-145	Ch	Channel	236
	C-147	Ch	Channel	236
	C-148	Ch	Channel	236
	C-152	Ch	Channel	236
	C-153	Ch	Channel	236
	M-24	Ch	Channel	236
	M-34	Ch	Channel	B

Table 1–10. List of material from WTC 5.

In FEMA report?	NIST Name	Type	Brief Description	Location
	GZ-1	Cn5	Coupon from Bldg #5	Lab
	GZ-2	Cn5	Coupon from Bldg #5	Lab
	GZ-3	Cn5	Coupon from Bldg #5	Lab
	GZ-4	Cn5	Coupon from Bldg #5	Lab
	GZ-5	Cn5	Coupon from Bldg #5	Lab
	GZ-6	Cn5	Coupon from Bldg #5	Lab
	GZ-7	Cn5	Coupon from Bldg #5	Lab

Table 1–11. List of miscellaneous material.

In FEMA report?	NIST Name	Type	Brief Description	Location
	C-18 Associated	B	One washer and nut	Lab
	C-108	B	Three sheared bolts	Lab
	C-109	B	Single bolt sheared	Lab
	C-110	B	Bolt and nut	Lab
	C-111	B	Bolt and washer	Lab
	C-112	B	Single bolt sheared	Lab
	C-113	B	Two sheared bolts with washers	Lab
	C-114	B	Sheared bolt with nut	Lab
	M-26 associated	B	8 bolts and a nut	Lab
	C-116	H	Damper	Lab
	B-1044-1	O	Piece of crushed metal decking assoc with B-1044	202
	B-2150	O	Pieces of aluminum sheathing	202
	C88c (formerly U-22)	O	Broke off C-88	PL
	C-94	O	May be some type of brace, rectangular box construction	PL
	C-103	O	Square-tube construction	PL
	C-119A	O	Square-tube construction	PL
	C-119B	O	Square-tube construction	PL
	C-120	O	Square-tube construction	PL
	C-121	O	Square-tube construction	PL
	C-135	O	May be some type of brace, rectangular box construction	PL
	C-146	O	Mangled ball of steel and concrete	202
	C-156 (formerly U-17)	O	Square-tube construction	PL
Y	K-50a	O	Rectangular slab of steel with bolts, received from D. Sharp, SEAoNY	Lab
Y	K-50b	O	Rectangular slab of steel with bolts, received from D. Sharp, SEAoNY	Lab
Y	K-50c	O	Rectangular slab of steel with bolts, received from D. Sharp, SEAoNY	Lab
	M-17a (formerly U-24)	O	Fell off of M-17 while moving	202
	M-29	O	5 ft piece of strapping	202
	M-30 associated	O	Pieces of glass, plexiglass, other rubble	Lab
Y	N-5	O	Plate with bolts	PL
	U-25	O	Unknown Wide flange with concrete	205

Table 1–12. Strength/gage combination of columns recovered by NIST.

Flange FY (ksi)	Flange Gage (in.)	Number of Columns Recovered and Identified by NIST
45	1.75	1
50	0.5	2
50	0.5625	2
50	1.0625	2
50	1.8105	1
50	2.0625	1
50	2.125	1
50	2.25	1
50	2.5	1
50	2.625	1
55	0.25	12
55	0.3125	5
55	0.375	6
55	0.4375	3
55	0.5625	3
55	1.375	1
55	1.6875	1
60	0.25	5
60	0.3125	6
60	0.375	1
60	0.5	1
65	0.25	7
65	0.375	1
65	0.8125	1
70	0.25	7
70	0.4375	2
70	0.75	1
75	0.25	3
75	0.4375	2
80	0.25	3
80	0.625	1
80	1.1875	1
85 – 100	0.25	12
85 – 100	0.5625	3
85 – 100	1.125	2
85 – 100	1.1875	3

Table 1–13. Strength/gage combinations of spandrels recovered by NIST.

Spandrel FY (ksi)	Spandrel Gage (in.)	Number of Spandrels Recovered by NIST
36	3/8	16
36	9/16	3
36	1 1/4	3
36	1 3/8	3
42	3/8	24
45	3/8	7
46	3/8	4
50	3/8	5
50	7/16	2
50	15/16	2
55	3/8	2
60	3/8	6
60	15/16	1
65	3/8	1
65	9/16	1
65	5/8	2
65	15/16	1
70	3/8	2
75	3/8	1
80	9/16	1

1.2 REPRESENTATIVE PICTURES OF RECOVERED WTC STEEL

Figure 1–1. Exterior column panel, sample C-46 shown.

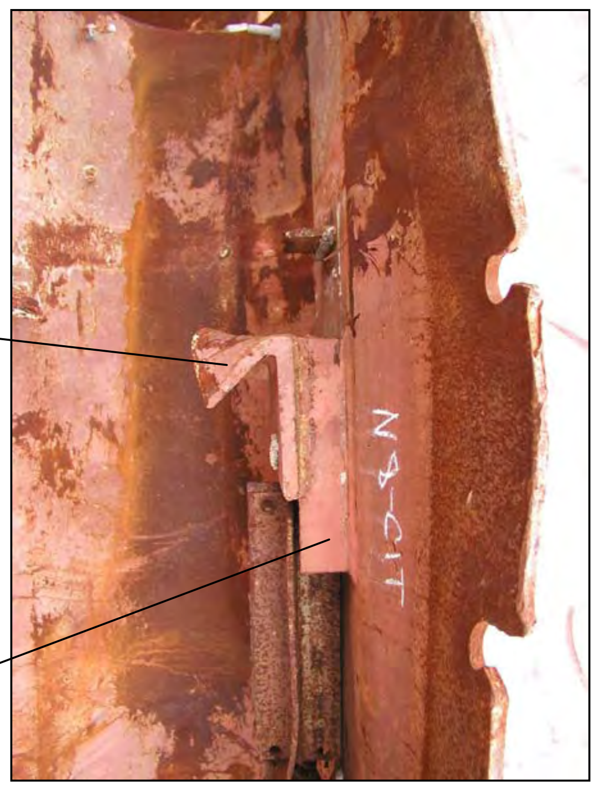

Figure 1–2. Floor truss seats shown from sample N-8.

Seat

Damping unit with missing viscoelastic material. Damping unit has been bent into an upright position.

Gusset plate for viscoelastic damping unit attachment to column interior.

Figure 1–3. Damping Unit shown from sample N-8.

Welded gusset plate used in place of seat on alternate column/spandrel intersections. One method used to attach diagonal bracing strap to exterior wall

Figure 1–4. Gusset plate shown from sample N-8.

Diagonal bracing strap attached directly to exterior column

On Sample C-25

Sample M-29

Figure 1–5. (left) Diagonal bracing strap shown on sample C-25, (top), and single strap labeled M-29 (bottom).

B-5004 at JFK

B-5004 portion cut and moved to NIST campus

Figure 1–6. Bowtie section of exterior wall.

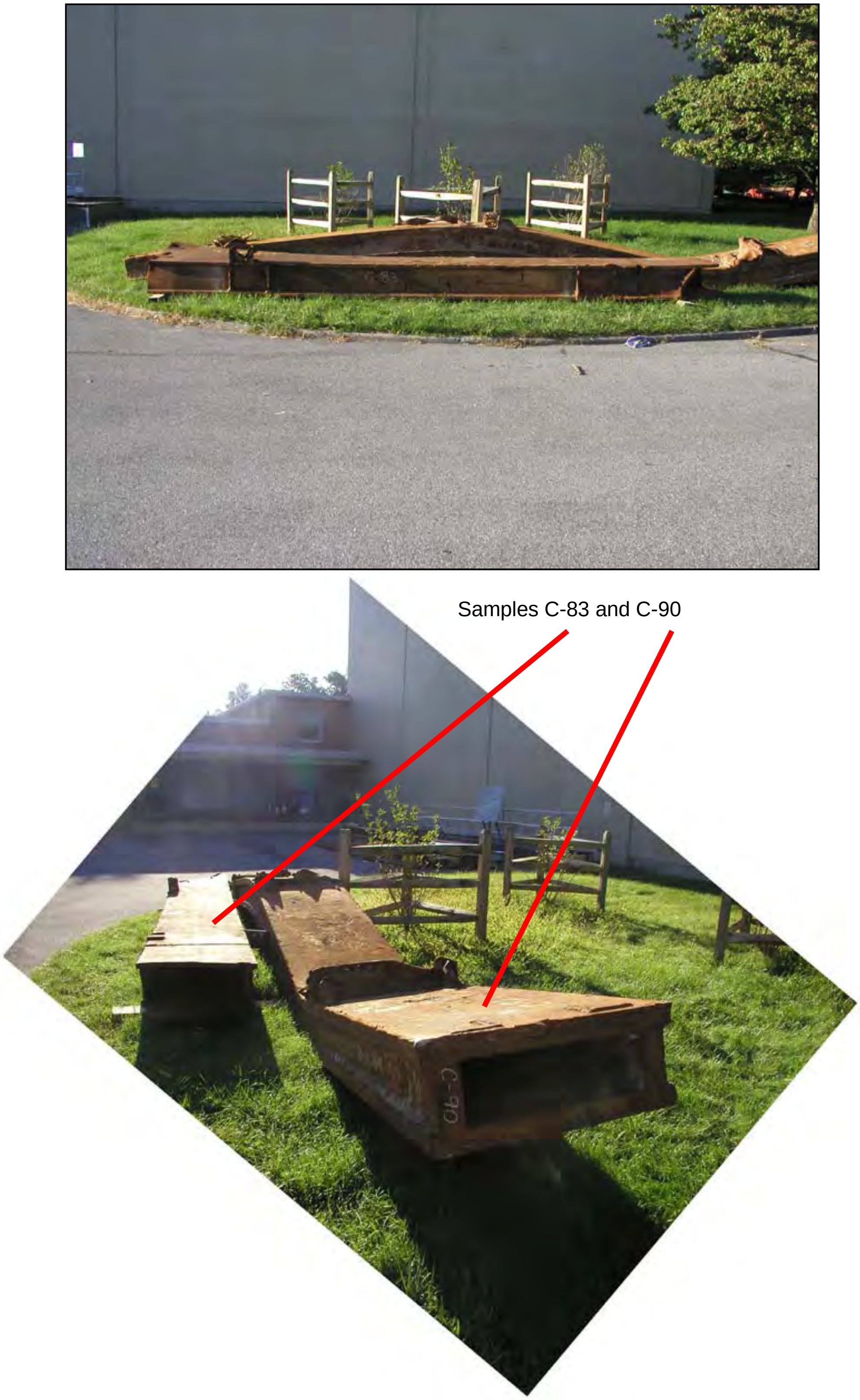

Figure 1–7. Recovered rectangular built up box sections used as core columns.

Sample C-65

Sample C-80

Figure 1–8. Recovered wide flange sections used as core columns.

Figure 1–9. Other recovered wide flange sections, shown is sample C-42.

Figure 1–10. Recovered floor truss material, shown are portions of sample C-53.

Figure 1–11. Recovered inner channel material used to connect floor trusses to core columns; shown is sample C-129.

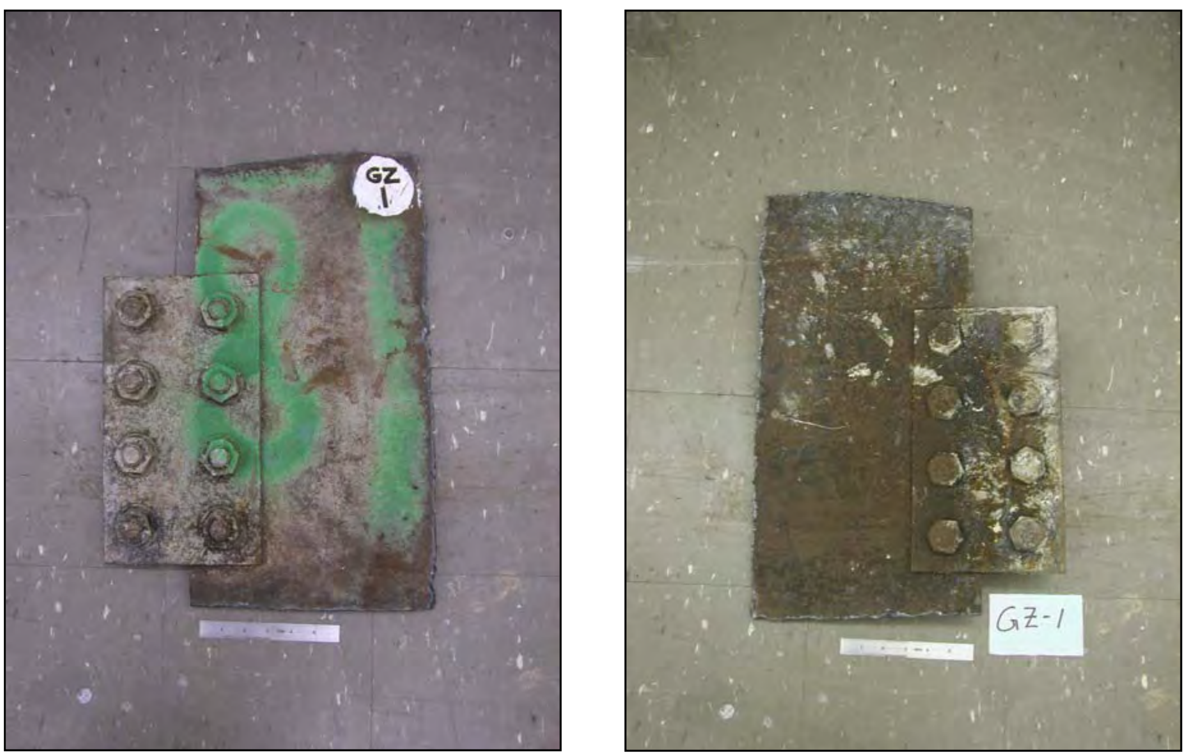

Figure 1–12. Coupons removed in the field from WTC 5; shown is sample GZ-1.

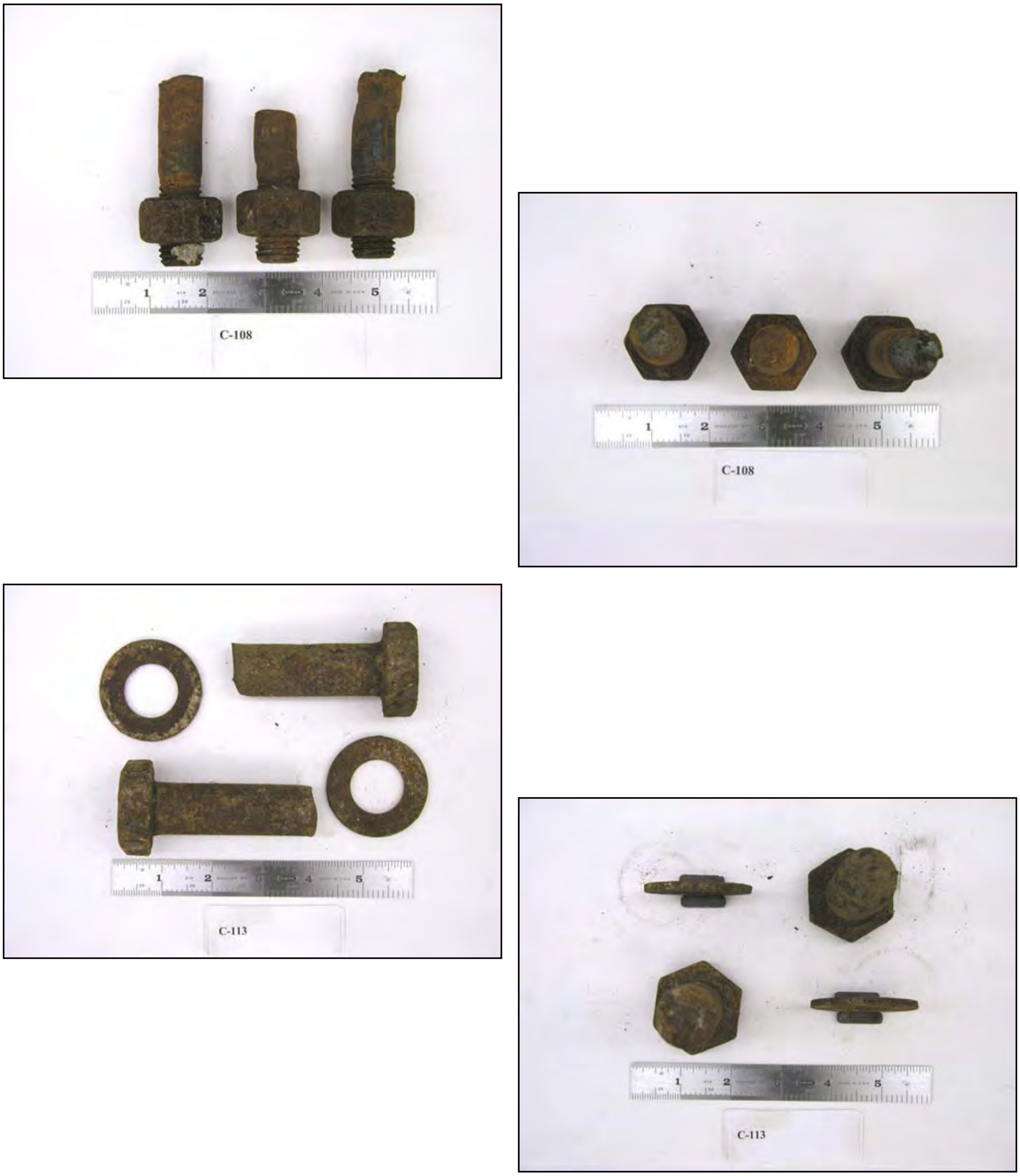

Figure 1–13. Examples of recovered bolts from various samples.

Square tubular piece
Sample C-103

Rectangular tubular piece
Sample C-135

Assorted pieces from within column
Sample M-30 Associated

Figure 1–14. Examples of miscellaneous materials recovered.

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Appendix G
INTERIM REPORT ON SIGNIFICANT FIRES IN WTC 1, 2, AND 7 PRIOR TO SEPTEMBER 11, 2001

Fires occurred in World Trade Center (WTC) 1, 2, and 7 prior to September 11, 2001. This appendix documents the facts of significant fires in the building after first occupancy as they relate to the performance of the automatic sprinkler, manual suppression, fire detection, and smoke purge systems. The ultimate goal of this review was to identify from New York City Fire Department (FDNY) records significant but not well known fires for further study.

G.1 BACKGROUND

The fire protection engineering department of the Port Authority of New York and New Jersey (PANYNJ) maintained records of all significant fire events in the WTC buildings. These records were lost in the collapse of the towers.

Two significant fire events involving WTC 1 are well known. On February 14, 1975, a fire started on the 11th floor of WTC 1. Workers reported the fire to WTC police headquarters. When police reached the fire floor, they reported a serious fire and ordered the heating, ventilating, and air conditioning (HVAC) system be placed into the smoke purge mode. Fire spread through unprotected floor openings in utility closets. Fire damage occurred on floor 10 through floor 19. Approximately 800 m^2 (9,000 ft^2) of the floor 11 contents were destroyed or damaged. At that time, sprinklers had not been installed in the office spaces. However, fire barriers divided the floor into quadrants. The fire on floor 11 was confined to the southeast quadrant. Fire damage on other floors was confined to the utility closets. The fire was extinguished by FDNY. More details about this fire incident can be found in Powers (1975), Lathrop (1975), and a report that is being prepared for the National Institute of Standards and Technology (NIST) by Hughes Associates.

At 12:18 PM on February 26, 1993, a bomb exploded in an underground parking garage of the WTC complex. The explosion occurred on the B2 level in the area of the garage under WTC 3 and adjacent to WTC 1. The explosion resulted in a loss of normal electric power in WTC 1 and WTC 2. HVAC systems shut down. Smoke spread throughout WTC 1 and to a lesser extent in WTC 2. More details about this fire can be found in Isner and Klein (1993a, 1993b). The only historic record of smaller fire incidents in WTC 1, 2, and 7 known to this investigation are the fire reports and fire investigation reports prepared by the FDNY. These reports were provided to NIST by FDNY for use in this investigation.

G.2 FDNY FIRE REPORTS AND FIRE INVESTIGATION REPORTS

The FDNY released 397 Bureau of Operations Fire Reports and 112 Bureau of Fire Investigation Records (Fire Marshals' Reports) which served as the basis for a summary of the fire history in the WTC 1, 2, and 7. NIST obtained reports of fires for the period of 1970–2001 and fire investigation records between 1977 and 2001 for WTC 1, 2, and 7, which in total, consisted of over 500 documents on which to report.

These records included all responses to fires in buildings 1, 2, and 7 by the FDNY. All of these records consist of standardized forms that may be supplemented with other materials. Many were for minor fire events, such as fires that were extinguished by occupants before FDNY arrival. These were not of interest for this investigation. The records of significant fires were identified.

Significant fire incidents were those that exercised the fire suppression systems, specifically multiple sprinklers or multiple standpipes (with or without the activation of at least one sprinkler). These fires will be discussed individually, organized by the building in which they occurred. In addition to these fires, generalized facts relating to those fires involving the use of one standpipe line and one sprinkler and the use of one standpipe line will be provided throughout this report. As an aside, the majority of fire records for significant fires documented the performance of the detectors and sprinkler systems, but almost all reports lacked information about the performance of the smoke purge system.

G.2.1 Fire Record Forms

Depending upon the type and date of the incident, a specific fire report form was used by the FDNY to document the incident. For each type of emergency responded to by the FDNY, responders either completed a form that would describe a structural fire (BF–24) or a form that would describe any other type of emergency (BF–25), such as a nonstructural fire, transportation fire, and/or any other non-fire emergency. For this historical summary, only those events logged and organized under the structural fire form, 345 documents total, were of interest and used. A structural fire form is a one-page document (unless additional information is recorded on separate sheets) that gives valuable information about the fire event on various subjects, including:

- Alarm–the date and time of the received alarm
- Injuries and casualties–the numbers of each for the incident
- Extinguishment–details of the sprinkler and standpipe performance
- Ignition–information on the equipment involved in ignition, the form of the ignition source, the material type and form that was ignited, and the ignition factor (cause)
- Structure–information on the class of construction, the use of the building, and its status (vacant, occupied, under construction, etc.)
- Fire origin–the fire location and classification
- Fire extension–the means of fire extension and number of buildings/vehicles involved
- Damage–information on the damage done by flame, smoke, and water
- Detectors–the type, power source and performance of the detectors in the fire area

Each subject of the incident is given a set of codes or numbers that correspond to any incident, and in order to read the fire records successfully, an understanding of the codes is necessary (see

Attachments G-A.1, G–A.2 and G–A.3). For the nonstructural B-25 record forms, the only fire related subjects included are the injury and casualty numbers, ignition, and structure information.

Depending upon the date of the fire incident, certain information is lacking from the structural fire form. Before 1980, a different record form for structural fire incidents was used which left out the following subjects: fire extension, damage, detectors, and portions of the ignition data. Because of this, detection data are not available for the majority of the fires occurring before 1980.

G.2.2 Overview of Fire Incidents 1970–2001 from FDNY Records

Table G–1 contains the categorization of all structural fire incidents contained in the FDNY records for WTC buildings 1, 2, and 7 available to this investigation. The table contains information on the category of fire incident (whether or not the detection and/or sprinkler systems activated), the time period over which the fires occurred, the numbers of records in that category, and a descriptive statement about the category.

Table G–1. Categorization of WTC 1, 2, and 7 fires from FDNY records.

WTC 1			
Category	**Dates**	**Number**	**Generalization of Incidents**
No detection, no sprinkler	1980–2001	66	Unattended food/appliances, overheated elevator equipment, discarded material, welding operations, electrical failure and suspicious fires
No detection information and no sprinklers	1970–1979	79	Trash can fires, discarded material, food on stove, electrical failure, overheated equipment
Detection, no sprinklers	1980–2000	57	Unattended food/appliances, overheated elevator equipment, discarded material, welding operations, electrical failure
[Detection] and sprinklers	1977–1999	18	Suspicious, electrical failure, discarded material
WTC 2			
Category	**Dates**	**Number**	**Generalization of Incidents**
No detection, no sprinkler	1980–1999	37	Discarded material, welding too close, overheated equipment, suspicious, elevator motor
No detection information and no sprinklers	1975–1979	40	Discarded material, fire in office furniture, trash can fires
Detection, no sprinklers	1981–1999	40	Food on stove, small elevator fire, electrical failure, suspicious, overheated equipment
[Detection] and sprinklers	1977–2000	5	Mechanical failure, suspicious
WTC 7			
Category	**Dates**	**Number**	**Generalization of Incidents**
No detection, no sprinkler	2000	1	Trash can fire/discarded material
Detection, no sprinklers	1990	1	Electrical switch on floor – explosion
[Detection] and sprinklers	1988	1	Suspicious

All FDNY records provided to NIST, unless the records were not readable, contained relevant information about the type and performance of the suppression system. Because of this, reports of incidents in which the sprinkler system activated can range from 1970 to 2001. When the table lists “[detection]” in brackets, this is meant to symbolize that either detection was present or no information on detector performance was included on the form (as is the case with the older records). An attempt was made to compare all investigation records with the fire reports, especially those which activated the suppression system. Looking at the records in Table G–1, it is clear that only 24 fires activated the sprinkler system from 1970–2001 from all three buildings. Many of the other structural fires without sprinkler activation were labeled as suspicious, trash can fires, electrical failures, unattended food/appliances, or overheated equipment.

In order to report on significant structural fires occurring in WTC 1, 2, and 7, the FDNY records had to be reviewed for those incidents that activated sprinklers, detectors, or were extinguished by hose line and those smaller fires that self-extinguished or could be extinguished using a fire extinguisher. The structural fire incidents without detection information (before 1980), had to be reviewed to locate any fires that activated the sprinkler system.

The retrofit installation of sprinklers into WTC buildings 1 and 2 was accomplished in two phases. During the first phase in 1976, sprinkler risers/mains were installed throughout WTC 1 and WTC 2. Sprinklers were installed to protect corridors, storage rooms, lobbies, and certain tenant/PANYNJ spaces. In the second phase of the retrofit from 1983 to 2001, sprinklers were installed in all remaining places in the complex (PACO 2002; shown in Attachment G–B). Prior to the retrofit only the sub-grade areas and selected hazard areas were protected by automatic sprinklers. This retrofit proceeded throughout the buildings as much as practical when other renovations of the office spaces were underway, such as when change of tenants occurred.

After the installation of the sprinkler risers in 1976, tenants had the option of providing sprinklers or compartmentation for fire protection in compliance with Local Law 5. It was therefore possible that during the period of time when retrofit installation of sprinklers was under way, a fire that occurred may or may not have been in an area protected by automatic sprinklers.

The forms used by the FDNY after 1987 give a detailed description of the event and whether or not a system was present at the time of the fire; however, a fire recorded before 1987 will give data only on the number of sprinklers opened. Because of this, an effort was made to look through all reports, especially those that mentioned detection performance, in order to identify fires involving the use of standpipe lines by the FDNY as an alternate indication of a significant fire.

The next section of the report will highlight significant fires occurring in WTC 1, 2, and 7. The significant fires will be described individually by WTC building, and organized by the date on which they occurred in the building. In addition to these significant fires, (1) the fires that activated one sprinkler head and involved the use of one standpipe and (2) the fires that involved the use of only one standpipe, due to the number of incidents, will be generalized as to the nature of the incidents and the procedures followed by the FDNY.

G.2.3 Fire Incidents Occurring in WTC 1

After reviewing all the FDNY records of fire incidents in WTC building 1 since 1970, the significant fires were selected. There were 12 significant fires found for WTC 1, and the fire reports are included in Attachment G–A.4. Table G–2 provides a summary of the fire incident information from FDNY records, which is followed by individual paragraphs about each incident.

Table G–2. Significant fires in WTC 1 extinguished by sprinklers and/or multiple standpipe lines.

Significant Fire	Incident Date	Fire Location	# Sprinklers Activated	# Standpipes Used	Cause of Fire	Material Ignited
1	9/9/77	B-6 level storage room	2	0	None listed	Not listed
2	9/23/77	Dumpster on B-4 level	2	0	Not classified	Trash/waste
3	10/16/81	19th floor office area	-	2	Discarded material	Furniture
4	12/23/83	2 dumpsters on B-4 level	2	1	Suspicious	Trash/waste
5	1/27/85	Office space on mezzanine level (Floor 2)	2	1	Incendiary	Trash/waste
6	9/10/85	Garbage dumpster in service elevator lobby on floor 43	2	1	Suspicious	Trash/waste
7	11/1/85	Storage closet on B-4 level	3	1	Suspicious	Supplies/ stock
8	6/7/86	Dumpster fire on floor 106, compactor room on floor 107	2	1	None listed	Trash/waste
9	9/30/91	Office on B-4 level	≥1	2	Discarded material	Trash/waste
10	11/19/91	Electrical closet on floor 93	0	2	Short circuit	Electrical wire or cable insulation
11	7/23/92	Level B-5 at the power distribution panel	0	2	Electrical failure	Electrical wire or cable insulation
12	11/10/99	Computer room on floor 104	3	≥1	None listed	Plastics, electronic equip

Key: ≥ symbol denotes that at least one of the units of the suppression system was used (and not specifically identified by the fire report); - indicates that the report acknowledges 0 sprinklers open; however, due to the date of the fire, the space may not have had a sprinkler system installed.

Significant Fire #1

On September 9, 1977, at 11:04 p.m., the FDNY received an alarm for a fire in the B-6 level storage room at the address of WTC 1. The fire activated two sprinklers, and was noted to be extinguished before the FDNY's arrival.

Significant Fire #2

Another fire occurred on September 23, 1977, at 11:48 p.m., in a dumpster on the B-4 level of WTC 1. This fire also activated two sprinklers, and the FDNY noted that the fire had been extinguished prior to their arrival.

In both cases, no injuries or casualties resulted from these fires, and the damage was confined to the area of origin.

Significant Fire #3

Six years later, on October 16, 1981, at 7:12 p.m., a fire occurred on floor 19 of WTC 1. The FDNY noted that they used two standpipe lines to extinguish the fire and that one person was evacuated from the scene. Again, the fire report notes that no sprinklers opened, but does not note whether or not sprinklers were present at the time of the fire. Given the date of the incident, sprinklers are not expected to be located on floor 19. The fire was caused by discarded material and involved furniture in an office area of the floor.

Significant Fire #4

Six years later on December 23, 1983, at 2:50 a.m., the FDNY responded to an alarm of fire and heavy smoke conditions on the B-4 level of WTC 1. The FDNY found two dumpsters fully involved in separate locations on the same floor and noted that the two activated sprinklers extinguished a major portion of the fire. The FDNY extinguished the rest of the flames by stretching hose from the standpipe system. Again, no injuries or casualties resulted from this fire. The cause noted on the report was suspicious and the damage was confined to the origin of the fire.

Significant Fire #5

On January 27, 1985, at 8:53 p.m., the FDNY was called for a fire located in an unoccupied office on the mezzanine level of WTC 1. Two sprinklers contained the incendiary (involving arson) fire consuming trash paper/waste. When the FDNY arrived, they extinguished the remaining fire with one standpipe line. Building and content damage was confined to less than 15 percent of the space. Also, no injuries or casualties were reported.

Significant Fire #6

Eight months later on September 10, 1985, at 4:05 p.m., the Port Authority Police informed the FDNY on arrival of a sprinkler flow and smoke condition on floor 43. A medium smoke condition was report by the FDNY on floor 43, where a fire was extinguished by two sprinklers. The fire report notes the use of one standpipe line; however, this was used during the overhaul process. This fire originated suspiciously in a garbage dumpster in a service elevator lobby. There was no building or content damage as well as no injuries or casualties reported.

Significant Fire #7

On November 1, 1985, at 4:05 a.m., the FDNY was called for another suspicious fire producing heavy smoke on the B-4 level under WTC 1 and WTC 2. This fire occurred in a storage closet of the men's bathroom, and the FDNY noted that three sprinklers activated to keep the fire under control until their arrival. Upon arrival, the FDNY extinguished the remaining fire in the closet area with one standpipe line. Again, the damage was noted to be confined to the area of origin.

Significant Fire #8

Less than a year later, on June 7, 1986, at 9:49 a.m., the FDNY received an alarm for a heavy smoke condition on floor 110. For this call, fires were burning in two separate places; a garbage dumpster on floor 106 and the compactor room on floor 107. Sprinklers were noted in operation in both locations and seemed to control the fires, until the FDNY could complete extinguishment with one standpipe line on floor 106. There was no report of injuries or casualties for the previous two fires.

Significant Fire #9

An additional fire occurred in WTC 1 where multiple standpipe lines were used along with the activation of the sprinkler system. This fire occurred on September 30, 1991, at 6:32 p.m., in an office on the B4 level. The fire report noted that the sprinkler system operated; however, there is no mention of how many sprinklers or even their activation in the Operations/Comments section of the report. Two 1 3/4 in. or larger hose lines were used by the FDNY to extinguish this fire. The cause of the fire was abandoned material (cigarette) igniting boxes/carton material in an office. The fire damage was confined to the area of origin and smoke damage was confined to the floor. There was one uniformed officer injured and no civilian injuries or casualties.

Significant Fire #10

A fire occurred on November 19, 1991, at 6:27 pm., and two 2 1/2 in. standpipe hose lines were used by the FDNY. The FDNY responded to WTC 1 for this fire due to a report of fire and smoke condition in electrical closets on possibly four floors (floors 93–96) and an alarm transmitted from floors 93–98. According to the fire report, the sprinklers were in service, but did not operate for this fire. The noted cause of this fire was a short circuit and the material that was ignited was electrical wire or cable insulation. The fire and smoke damage was confined to its area of origin (electrical closet). Two occupants were removed from stalled elevators during this incident, and occupants were evacuated from the scene, although an exact number is not given. Also, two occupants were injured and required first aid.

Significant Fire #11

The FDNY responded to WTC 1 on July 23, 1992, at 10:02 p.m., due to a transformer fire on the 5th sub basement level. Firefighters found a fire situation in a large power distribution panel, where a firefighter was knocked unconscious by a shock blast from the panel. Similar to the fire in November of 1991, two 2-1/2 in. standpipe hose lines were used by the FDNY on this fire. The cause of the fire was an electrical failure and the material ignited was electrical wire or cable insulation. No appreciable damage is noted. As mentioned earlier, one firefighter was injured as well as three civilians.

Significant Fire #12

The final fire associated with WTC 1 was one that occurred on November 10, 1999, at 11:01 p.m., in a computer room on floor 104. The FDNY noted that the fire was “knocked down” by three sprinklers when they arrived and they completed extinguishment with a line extended from the standpipe. The flame damage was confined to the area of origin and computer equipment was involved in fueling the fire. There was one injury and no casualties reported in the FDNY record for this fire.

Table G–2 presents the 12 significant fires in WTC 1. Five of the 12 fires occurred on the basement levels and two occurred on the upper levels (above floor 100). The causes of these significant fires include suspicious, discarded materials, and electrical failures.

G.2.4 Fire Incidents Occurring in WTC 2

Table G–3 presents the significant fire occurring in WTC 2. There were three significant fires found for WTC 2, and the fire reports are included in Attachment G–A.5. Table G–3 provides a summary of the fire incident information from FDNY records, which is followed by individual paragraphs about each incident.

Table G–3. Significant fires in WTC 2 extinguished by sprinklers and/or multiple standpipe lines.

Significant Fire	Incident Date	Fire Location	# Sprinklers Activated	# Standpipes Used	Cause of Fire	Material Ignited
1	5/19/75	Floor 32	-	3	Incendiary	Trash/waste
2	4/12/77	Duct work over grill in restaurant on floor 107	2	0	None listed	Duct work
3	3/22/93	Fan motor room on floor 108	2	0	Mechanical failure	Not classified

- Indicates that the report acknowledges 0 sprinklers open, however due the date of the fire, the space may not have had a sprinkler system installed.

Significant Fire #1

A fire occurred on May 19, 1975, at 9:38 p.m., on floor 32 of WTC 2. The FDNY noted that they used three standpipe lines to extinguish the fire and that the Port Authority reported occupants trapped on floors 31 and 32. The fire report notes that no sprinklers opened, but does not note whether or not sprinklers were present at the time of the fire. Given the date of the incident, sprinklers are not expected to be located on floors 31 and 32. The fire was labeled as incendiary and involved trash/waste. The FDNY stated that the fire involved the core area of the floor and was confined to that area. Over 20 people (civilians and uniformed personnel) were injured by this incident.

Significant Fire #2

On April 4, 1977, at 1:15 p.m., the FDNY was called to WTC 2 for a fire in the duct work over the grills in a restaurant on floor 107. The FDNY record on this fire noted that the fire was extinguished prior to its arrival. The damage was confined to the area of origin, and the fire caused no injuries or casualties.

Significant Fire #3

The second fire occurred on March 22, 1993, at 8:39 a.m., and caused a smoke condition on floor 108. The fire activated two sprinklers due to an overheated bearing in a fan motor room on floor 108. The damage to the area did not exceed 15 percent of the space, and there were no injuries or casualties reported.

Table G–3 presents the three significant fires in WTC 2. No fires were discovered in WTC 2 where multiple sprinklers or standpipes were used with another suppression system. Two of the three fires occurred on the upper levels (above floor 100) and the other occurred on floor 32. The causes of these significant fires included incendiary and mechanical failures.

G.2.5 Additional Fires Involving the Deployment of Standpipe Lines in WTC 1 and 2

The fires described in this section (31 in total) involve the use of one standpipe, with and without the activation of one sprinkler for WTC 1 and WTC 2. Four of the 31 reports describe fires that were extinguished with one sprinkler and one standpipe line (see Attachment G–A.6.1). Three of these fires were located in WTC 1 between the years of 1986–1991 and the other in WTC 2 in 1981. Two of these fires occurred in basement levels, one occurred on floor 106 of WTC 1, and the last on floor 5 in WTC 1. In some of the fire reports, the FDNY noted that the sprinkler controlled the fire, and the standpipe was used to actually extinguish the remaining fire. Half of the fires were labeled as incendiary/suspicious, one was an electrical failure, and the last was unknown.

In addition, 27 of the 31 fire reports describe fires that were extinguished using one standpipe line (see Attachment G–A.6.2). Twenty of these fires occurred in WTC 1 and the other seven occurred in WTC 2. A majority of these fires (19) are labeled as incendiary/suspicious or unknown, while the other causes of the fires are attributed to short circuits, abandoned material/cigarette, welding close to combustibles, and a mechanical failure. The dates of occurrence for these fires range from 1973–1999, with a majority (23) occurring between the years of 1973–1985. These fire incidents did not result in any casualties, but five civilians and one uniformed officer were injured.

Two of the 27 fires involved a 300 person (April 19, 1980) and a 1,500 person (April 17, 1981) evacuation. These will be described in further detail. On April 19, 1980, at 2:06 p.m., the FDNY received reports of an activated smoke detector in the return air duct on floor 106 of WTC 1. The FDNY also received reports of heavy smoke on floor 106, light smoke on floor 109, and heavy odor of smoke in stairways A and B. The report notes that while only one standpipe was used, approximately 300 people were evacuated from the Windows on the World restaurant on floor 107 via stairway C (which was clear of smoke). The fire cause was labeled as abandoned or discarded material and involved plastic material. This fire did not cause any injuries or casualties.

On April 17, 1981, at 9:18 a.m., the FDNY was informed of a fire on floor 7 and a smoke condition on floors 7 through 11 of WTC 1. The FDNY hooked up one standpipe and extinguished the fire located in an air conditioning unit in the “MER” room on floor 7. The cause of this fire was labeled as a mechanical failure. The fire report notes that the Port Authority personnel reported an evacuation of approximately 1,500 people from floors 9 through 23. However, no injuries or casualties were reported from this fire.

G.2.6 Fire Incidents Occurring in WTC 7.

Table G–4 presents the significant fire occurring in WTC 7. There was one significant fire found for WTC 7, and the fire report is included in attachment G–A.7. Table 4 provides a summary of the fire incident information from FDNY records, which is followed by an individual paragraph on the incident.

Table G–4. Significant fires in WTC 7 extinguished by sprinklers and/or multiple standpipe lines.

Significant Fire	Incident Date	Fire Location	# Sprinklers Activated	# Standpipes Used	Cause of Fire	Material Ignited
1	5/20/88	Construction shanties on floor 3	Multiple, # not listed	1	Suspicious	Shanties

Significant Fire #1

In WTC 7, a fire occurred on May 20, 1988, at 12:38 a.m., in the construction shanties on floor 3. Although the fire report does not specifically note the number of sprinklers that activated, the operations notes state that Ladder Truck 10 found the sprinklers (noting more than one) in operation and shut them down. The FDNY had to complete the extinguishment by stretching a line from the standpipe to the fire source. This fire is noted by the report as being suspicious in nature and the flame damage was confined to the area of origin.

It is possible that the fire incidents that were not specifically highlighted, especially those in the areas without sprinklers, involved other methods of extinguishment before FDNY arrival, such as a WTC houseline (pre-connected standpipe hose), hand extinguisher, or bucket of water, as noted on some of the FDNY reports. All other fires, the majority, included in other categories were either self-extinguished, extinguished prior to FDNY arrival (by staff, etc.), or a hand extinguisher was used by the FDNY.

G.3 SUMMARY

In summary, 16 significant fires occurred in WTC 1, 2, and 7, with 12 occurring in WTC 1, three in WTC 2, and one in WTC 7. In addition to these, 31 fires occurred in WTC 1 and WTC 2, which involved the use of one standpipe (with or without the activation of one sprinkler). Of these additional 31 fires, 23 occurred in WTC 1 and eight occurred in WTC 2. The following paragraphs will summarize findings from the 16 significant fires that occurred in all three buildings.

After reviewing the 16 significant fires, trends developed relating to the time of day that the fires occurred. Overall, 12 of the 16 fires occurred between the hours of 6 p.m. and 4 a.m. The fires that occurred during office hours (between 7 a.m. and 6 p.m.) included a dumpster fire in the floor 43 elevator lobby (WTC 1), a dumpster fire on floor 106 (WTC 1), a kitchen fire on floor 107 (WTC 2), and a bearing overheating in the fan motor room on floor 108 (WTC 2). Almost all of the incendiary (arson) and suspicious fires (5 out of 6 fires) and unclassified or unlisted fires (4 out of 5 fires) occurred after business hours (before 7 a.m. and after 6 p.m.).

In addition to the time of day of the fire, trends in the cause of the fire and the materials involved in the fire can be highlighted. Of the 16 fires and their causes, five were labeled as unlisted or unclassified, six as suspicious or incendiary, two as discarded material, and three as an electrical failure or mechanical failure. For the material involved in the fire, eight reports noted trash, waste, and supplies; two reported not listed or not classified; one reported furniture; three reported electrical equipment; one reported duct work; and one reported shanties were the material involved in the fire.

Lastly, the location of the fires throughout the buildings was of interest. Of the 16 fires, four fires were concentrated above floor 100 or and six fires were located in the basement. The others (6 fires) were spread throughout the rest of the building.

G.4 ATTACHMENTS TO THIS FIRE HISTORY

Attachments G–A.1 through G–A.7 are included as a supplement to this report. The first three sections, G–A.1 through G–A.3 are explanations of the numeric codes used in the fire reports by the FDNY. Attachment G–A.1 is included to explain the codes for the fire reports produced prior to and including 1980, Attachment G–A.2 is included to explain the fire reports produced from 1981 to May 31, 1987, and Attachment G–A.3 is included to explain the fire reports produced from June 1, 1987, to the present. The report code explanations are divided into the same sections as the fire report and give short descriptions for the numbers used in the fire report under each section. For example, if the ignition factor for a fire occurring in 1990 was given a number code of 54, the reader can find that the cause of the fire was a "short circuit, ground fault."

Attachments G–A.4 through G–A.7 include the actual fire reports produced by the FDNY on the significant fires highlighted in the sections above. The reader can use Attachments G–A.1 through G–A.3 (depending upon the date of the fire) to read the fire reports in more detail than what is provided in this fire history report.

G.5 CONCLUSIONS

From the information contained in FDNY fire reports and fire investigation records provided to NIST, 47 fires occurred in WTC building 1, 2, and 7 that were of sufficient size and duration to activate multiple sprinklers or were estimated by NIST to be capable of doing so, over the time period the buildings were occupied. This total does not include the major 1975 office fire in WTC 1 or the 1993 bombing.

The records indicate that in areas protected by automatic sprinklers, no fire activated more than 3 sprinklers. Three sprinklers would provide coverage for a floor area of approximately 63 m^2 (675 ft^2). This area is much smaller than the 800 m^2 (9,000 ft^2) damaged by the 1975 fire in an office space unprotected with automatic sprinklers.

Many of the fires that occurred were recorded as suspicious or unknown in cause, occurred during off-peak work hours, and involved materials such as trash or paper-based supplies. In cases where sprinklers were activated, the FDNY records indicated that the sprinklers either extinguished the fire completely or aided in controlling the spread.

G.6 REFERENCES

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PACO Group. 2002. *World Trade Center General Description of All Building Systems and the Capital Program.* August.

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Attachment G-A.1

Explanation of Numeric Codes Used on Fire and Emergency Reports -
Prior to 1980

CLASSIFICATION OF BUILDING BY USE

COMMERCIAL

01 Bank
02 Brewery
03 Coal Pocket
04 Department Store
05 Electric Power Plant
06 Factory:Multi occupancy
07 Factory: Single Occupancy
08 Foundry
09 Freight Depot
10 Garage: Non-Storage
11 " " Storage
12 Gas Works
13 Lumber Yard
14 Motor Vehicle Repair Shop
15 Office Building
16 Oil Selling Station
17 Oil Storage Plant
18 Pier, Wharve, Dock, Bulkhead Building
19 Restaurant, Diner
20 Shed, Newsstand, Shanty
21 Shipyard, Drydock
22 Stable
23 Steam Generating plant
24 Store Building, Taxpayer
25 Warehouse, Storehouse
26 Store Building & Private Dwelling
39 Other Commercial

PUBLIC

40 Airport Building
41 Asylum
42 Bridge
43 Bus Terminal
44 Church, Synagogue
45 Dance Hall, Banquet Hall
46 Dispensary, Clinic
47 Ferry Terminal
Government Building (Not otherwise classified);
48 City
49 Intersate
50 Federal
51 Foreign
52 State
53 Hospital, Infirmary
54 Nursing Home
55 Railroad Station
56 School; College, University
57 " Private High
58 " Public High
59 " Public Junior High
60 " Private Elementary
61 School: Public Elementary
62 " Children's Nursery
63 " Other
64 Television Studio
65 Theatre, Legitimate
66 Theatre, Motion Picture
67 Transit System - Station Structure
68 Tunnel
69 Other Public

RESIDENTIAL

80 Apartment Hotel "A"
81 Apartment House "A"
82 Boarding House, Rooming House "B"
83 Convent, Rectory, Monastery, etc.
84 Dormitory-School Club, Lodge
85 Hotel "B"
86 Lodging House "B"
87 Motel
88 Pvt. Dwelling: 1 Family
89 " Two Family
90 Tenement: New Law "A"
91 " Old Law "A"
92 Converted Dwelling "A"
99 Other Residential

BUILDING STATUS

1 Occupied
2 Partly Occupied, Good Condition
3 Partly Occpuied, Deteriorating
4 Vacant
5 Under Demolition
6 Under Construction

DAMAGE (to Building or Contents)

0 None-No Appreciable
1 Light 0-15%
2 Medium 16%-49%
3 Heavy 50% & Greater

AREA FIRE ORGIN -FLOOR

00 Outside Building
01 1st Floor
to
94 94th and Higher
95 Attic
96 Roof
97 Basement
98 Cellar
99 Sub-cellar

AREA FIRE ORGIN-ROOM OR AREA

10 Area Not in Building
11 Attic
12 Awning
13 Balcony
14 Basement
15 Bathroom Toilet
16 Bedroom, Sleeping Area
17 Ceiling
18 Cellar
19 Chimney
20 Classroom Lecture Area
21 Closet
22 Cockloft
23 Court-Exterior
24 Court-Interior
25 Dining Room, Dining Area
26 Duct-Air Conditioning
27 Duct-Exhaust
28 Flooring
29 Furnace Room
30 Hallway-Private
31 Hallway-Public
32 Incinerator Closet or Room
33 Kitchen, Cooking Area
34 Living Room
35 Lobby
36 Machinery Room
37 Office Area
38 Operating Laboratory Area
39 Partition
40 Porch
41 Projection Booth
42 Recreation Area
43 Roof
44 Sales Showroom Display Area
45 Shaft-Duct, Pipe
46 Shaft-Dumbwaiter
47 Shaft-Elevator
48 Shaft-Exterior Light
49 Shaft-Interior Light
50 Shaft-Vent
51 Shipping Receiving Loading Area
52 Stage
53 Stairway
54 Storage Room Area
55 Vacant-Room, Apartment or area
56 Work Area Workroom
57 Other Areas, Not Classified (State area)

AREA FIRE ORIGIN-OCCUPANCY CLASSIFICATION

COMMERCIAL

Factory:
00 Chemicals
Clothing:
01 Dresses
02 Undergarment
03 Other (State Type)
04 Dry Cleaning Laundry
05 Electrical Products
06 Food & Drink Products
07 Furniture
08 Furs,Fur Goods
Hats:
09 Men's
10 Women's
11 Leather, Leather Products
12 Machine Shop Metal works
13 Paints
14 Paper Products
15 Petroleum Products
16 Plastics, Rubber
17 Printing&Allied Industries
18 Shoes
19 Textiles
20 Toy or Doll
21 Woodworking
22 Other Factories not classified(state type)
Store:
23 Auto Accessories
24 Bakery
25 Butcher
26 Candy, Cigar, Stationery
27 Clothing
28 Department, large
29 Department, small(5&10)
30 Dry Cleaner & Tailor
31 Drug
32 Electrical Appliances
33 Fruit & Vegetables
34 Furniture
35 Grocery,Dairy,Delicatessen
36 Haberdashery
37 Ladies Accessories
38 Laundry
39 Paint Hardware

(2)

40 Restaruant Luncheonette
41 Shoe
42 Shoe Repair
43 Super Market
44 Tavern
45 Other Stores not classified(state type)
Garages:
46 Non Storage
47 Storage
48 Oil Selling Station
49 Motor Vehicle Repair Shop
50 Office Building
Warehouse:
51 Film
52 Paper, Rags,Fibre
53 Other(state type)
54 Freight Depot
55 Pier
56 Shipyard
57 Lumber Yard
58 Shed, Newstand, Shanty,etc.
59 Other Commericial Building Occupancies, not classified(state type)

RESIDENTIAL

60 Apartment Hotel, Multiple Dwelling"A"
61 Apartment House, Multiple Dwelling"A"
62 Boarding House, Rooming House Multiple Dwelling "B"
63 Hotel, Multiple Dwelling "B"
64 Lodging House,Multiple Dwelling"B"
65 Private Dwelling
66 Rectory,Convent, Monastery
67 Tenement House, New Law, Multiple Dwelling "A"
68 Tenement House, Old Law, Multiple Dwelling "A"
69 Other Residential, not classified (state type)

PUBLIC

70 Airport
71 Cabaret, Banquet Hall
72 Church
73 Dance Hall
74 Hospital
75 Motion Picture Theatre
76 N.Y. Transit System-Station
77 Passenger Depot
78 School
79 Theatre
80 T.V. Studio
81 Other Public, not classified (state type)

MANNER EXTENSION

00 Confined to area of origin
01 Cockloft
02 Door or opening between rooms
03 Floor
04 Hall Stairway
05 Partition
06 Pipe Recess
07 Shaft-Dumbwaiter
08 Shaft-Elevator
09 Shaft-Air,Light, Chute,Duct, etc.
10 Ceiling
11 Window
12 Other (state how)

CLASSIFICATION BY TYPE FIRE OR EMERGENCY

TRANSPORTATION FIRES

87 Ship, Vessel
88 Motor Vehicle
89 Other Transportation (state type)

NON-STRUCTURAL FIRES

86 ADV (Abandoned/ Derelict Motor Vehicle)
90 Bonfire
91 Brush, Grass
92 Demolition Wood, Building Site
93 Dump, Land Fill
94 Rubbish-Outside Building
95 Manhole
96 N.Y. Transit System-Yard Roadway, Ties, etc.
97 Railroad-Yard, Roadway, Ties, etc.
98 Tunnel, Bridge
99 Other Non-Structural, not classified (state type)

EMERGENCY

02 Chimney
03 Elevator, Escalator
04 Explosives Escort
05 First Aid - Assist Person(s)
06 First Aid - Resucitation
07 Marine
08 Precarious Condition Signs, Trees,etc.
09 Subway-Railroad
10 Water Leak
11 Bomb-Unexploded, Scare
12 Collapse-Cave in
13 Collision-Vehicular Incident
14 Controlled Fire, Permitted
15 Flood Condition-Broken Water Main
16 Incinerator
17 Leak-Fuel Oil, Gasóline etc.
18 Leak-Illum, Gas, Flam, Vapor
19 Lightning
20 Oil Burner
21 Person Locked in, Locked out
22 Power - Electrical
23 Pressure Rupture
24 Refrigerant Leak
25 Smoke Condition, Odor, Fumes
26 Sprinkler
27 Steam Discharge
28 Other

(3)

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Attachment G-A.2

Explanation of Numeric Codes Used on Fire and Emergency Reports -
From 1981 to May 31, 1987

Type of Report
1 Structural.
2 Transportation Fire.
3 Non Structural Fire.
4 Emergency Response.
5 False Alarm.
6 Additional data (for 8F 24A)

How Reported
1 Street Box Manual.
2 Telephone.
3 Verbal.
4 Class 3—Manual. PFA.
5 Class 3—Valve. PFA.
6 Class 3—Other Automatic. PFA.
7 Class 3—ERS.
8 Street Box—ERS.
9 Class 3—Manual. FONY.
10 Pre-recorded Alarm.

Initial Alarm
0 Special Call Other Than Engine Only—No Chief.
1 Box (Street or Class 3).
5 Special Call Engine Only—No Chief.
8 Special Call (Chief Operated).
9 Still.

Highest Alarm
0 Initial Alarm.
1 More than the Initial Alarm & less than 3 Engine & 2 Lader Co. at work.
2 2nd Alarm.
3 3rd Alarm.
4 4th Alarm.
5 5th Alarm.
6 Simultaneous.
7 Signal 7-5.

How Extinguished
0 Before Arrival.
1 Hand Extinguishers.
2 Sprinkler Heads (State Number).
3 Booster Stream.
4 Low Pressure Hydrant Stream.
5 One-1½" or larger hoseline from a pumping unit or a standpipe outlet, regardless of line termination (Controlling Nozzle, Deckpipe, Stang Multi Versal, Ladder Pipe, T/L, Foam Nozzle, etc.).
6 Two-1½" or larger hoselines as above.
7 Three-1½" or larger hoselines as above.
8 Four or more 1½" or larger hoselines as above.
9 Other (State How).

Ignition Stage—Termination Stage
2 Smolder Stage, before any flame.
3 Flame Stage.
0 Undetermined or not reported.

EQUIPMENT INVOLVED IN IGNITION

Heating Systems
11 Central heating unit.
12 Water heater.
13 Fixed, stationary local heating unit.
14 Indoor fireplace.
15 Portable local heating unit.
16 Chimney, gas vent flue.
17 Chimney connector, vent connector.
18 Heat transfer system.
19 Heating systems not classified above.
10 Heating system, undetermined.

Cooking Equipment
21 Fixed, stationary surface unit.
22 Fixed, stationary oven.
23 Fixed, stationary food warming appliance.
24 Deep fat fryer.
25 Portable cooking, warming unit.
26 Open fire grill.
27 Grease hood, duct.
29 Cooking equipment not classified above.
20 Cooking equipment, undetermined.

Air Conditioning, Refrigeration Equipment
31 Central air conditioning, refrigeration equipment.
32 Water cooling device, tower.
33 Fixed, stationary local refrigerator unit.
34 Fixed, stationary local air conditioning unit.
35 Portable air conditioning, refrigeration unit.
39 Air conditioning, refrigeration equipment not classified above.
30 Air conditioning, refrigeration equipment, undetermined.

Electrical Distribution Equipment
41 Fixed wiring.
42 Transformer, associated overcurrent or disconnect equipment.
43 Meter, meter box.
44 Power switch gear, overcurrent protection devices.
45 Switch, receptacle, outlet.
46 Lighting fixture, lampholder, ballast, sign.
47 Cord, plug.
48 Lamp, light bulb.
49 Electrical distribution equipment, not classified above.
40 Electrical distribution equipment, undetermined.

Appliances, Equipment
51 Television, radio, phonograph.
52 Dryer.
53 Washing machine.
54 Floor care equipment.
55 Separate motor, generator.
56 Hand tools.
57 Portable appliance designed to produce controlled heat.
58 Portable appliance designed not to produce heat.
59 Appliances, equipment not classified in 51 through 58.
50 Appliances, equipment, undetermined.

Special Equipment
61 Electronic equipment.
62 Vending machine, drinking fountain.
63 Office machine.
64 Biomedical equipment device.
65 Separate pump, compressor.
66 Combustion engine.
67 Conveyor.
68 Printing press.
69 Special equipment, not classified above.
60 Special equipment, undetermined.

Processing equipment
71 Furnace, oven, kiln.
72 Casting, molding, forging equipment.
73 Heat treating equipment.
74 Working, shaping machine.
75 Coating machine.
76 Painting equipment.
77 Chemical process equipment.
78 Waste recovery equipment.
79 Processing equipment, not classified above.
70 Processing equipment, undetermined.

Service, Maintenance Equipment
81 Incinerator.
82 Bearing, brake.
83 Rectifier, charger.
84 Tarpot, tar kettle.
85 Arc, oil lamp.
86 Elevator.
87 Torches.
89 Service, maintenance equipment, not classified above.
80 Service, maintenance equipment, undetermined.

Other Object, Exposure Fire
91 Separate, removed exposure.
92 Separate, detached exposure.
93 Separate, adjoining exposure.
94 Attached, protected exposure.
95 Attached, unprotected exposure.
96 Vehicle.
98 No equipment involved—
99 Other object, exposure fire not classified above.
90 Other object, exposure fire.
00 Equipment involved in ignition undetermined or not reported.

FORM OF HEAT OF IGNITION

Heat from Fuel-Fired, Fuel-Powered Object
11 Spark, ember, flame escaping from gas fueled equipment.
12 Heat from gas fueled equipment.
13 Spark, ember, flame escaping from liquid fueled equipment.
14 Heat from liquid fueled equipment.
15 Spark, ember, flame escaping from solid fueled equipment.
16 Heat from solid fueled equipment.
17 Spark, ember, flame escaping from equipment: fuel not known.
18 Heat from equipment; fuel not known.
19 Heat from fuel-fired, fuel-powered object, not classified above.
10 Heat from fuel-fired, fuel-powered object, undetermined.

Heat from Electrical Equipment Arcing, Overloaded
21 Water caused short circuit arc.
22 Short circuit arc from mechanical damage.
23 Short circuit arc from defective, worn insulation.
24 Unspecified short circuit arc.
25 Arc from faulty contact, loose connection, broken conductor.
26 Arc, spark from operating equipment or switch.
27 Heat from overloaded equipment.
28 Fluorescent light ballast.
29 Heat from electrical equipment arcing overloaded, not classified above.
20 Heat from electrical equipment arcing, overloaded undetermined.

Heat from Smoking Material
31 Cigarette.
32 Cigar.
33 Pipe.
39 Heat from smoking material, not classified above.
30 Heat from smoking material, undetermined.

Heat from Open Flame, Spark
41 Cutting torch operation.
42 Welding torch operation.
43 Torch operation, other than cutting and welding.
44 Candle, taper.
45 Match.
46 Lighter.
47 Open fire.
48 Backfire from internal combustion engine.
49 Heat from open flame, spark, not classified above.
40 Heat from open flame, spark, undetermined.

Heat from Hot Object
51 Heat, spark from friction.
52 Molten, hot material.
53 Hot ember, ash.
54 Electric lamp.
55 Rekindle, reignition.
56 Heat from properly operating electrical equipment.
57 Heat from improperly operating electrical equipment.
59 Heat from hot object, not classified above.
50 Heat from hot object, undetermined.

Heat from Explosive, Fireworks
61 Explosive.
62 Blasting agent.
63 Fireworks.
64 Paper cap, party popper.
65 Model rocket, not amateur rocketry.
66 Incendiary device.
69 Heat from explosive, fireworks, not classified above.
60 Heat from explosive, fireworks, undetermined.

Heat from Natural Source
71 Sun's heat.
72 Spontaneous ignition chemical reaction.
73 Lightning discharge
74 Static discharge
79 Heat from natural source, not classified above.
70 Heat from natural source, undetermined.

Heat Spreading from Another Hostile Fire (Exposure)
81 Heat from direct flame, convection currents.
82 Radiated heat.
83 Heat from flying brand, ember, spark
84 Conducted heat
89 Heat spreading from another hostile fire, not classified above.
80 Heat spreading from another hostile fire, undetermined.

Other Form of Heat of Ignition
97 Multiple forms of heat of ignition.
99 Other form of heat of ignition.
00 Form of heat of ignition undetermined.

TYPE OF MATERIAL IGNITED

Gas
11 Natural gas.
12 LP-city gas (LP and air mix).
13 Manufactured gas.
14 LP-gas.
15 Anesthetic gas.
16 Acetylene
17 Specialty gas other than anesthetic.
19 Gas not classified above.
10 Gas.

Flammable, Combustible Liquid
21 Class IA flammable liquid
22 Class IB flammable liquid
23 Gasoline.
24 Class IC flammable liquid.
25 Class II combustible liquid.
26 Class IIIA combustible liquid.
27 Class IIIB combustible liquid.
29 Flammable, combustible liquid, not classified above
20 Flammable, combustible liquid, undetermined.

Volatile Solid, Chemical
31 Fat, grease (food).
32 Grease (nonfood).
33 Polish.
34 Adhesive, resin, tar.
35 Applied paint, varnish
36 Combustible metal.
37 Solid chemical (specify type)
38 Radioactive material
39 Volatile solid, chemical, not classified above.
30 Volatile solid, chemical, undetermined.

Plastic
41 Polyurethane.
42 Polystyrene.
43 Polyvinyl
44 Polyacrylic.
45 Polyester.
46 Polyolefin.
49 Plastic, not classified above.
40 Plastic, undetermined.

Natural Product
51 Rubber.
52 Cork
53 Leather
54 Grass, leaves, hay, straw
55 Grain, natural fiber
56 Coal, coke, briquettes, peat.
57 Food starch
58 Tobacco
59 Natural product, not classified above
50 Natural product, undetermined.

Wood, Paper
61 Growing wood.
62 Felled but unsawn wood.
63 Sawn wood.
64 Wood shavings.
65 Hardboard, plywood.
66 Fiberboard (low density material), wood pulp.
67 Paper, untreated, uncoated.
68 Cardboard.
69 Wood, paper, not classified above.
60 Wood, paper, undetermined

Fabric, Textile, Fur
71 Man-made fabric, fiber, finished goods.
72 Cotton, rayon, cotton fabric, finished goods.
73 Wool, wool mixture fabric, finished goods.
74 Fur, silk, other fabric, finished goods.
75 Wig.
76 Human hair.
79 Fabric, textile, fur, not classified above.
70 Fabric, textile, fur, undetermined.

Material Compounded with Oil
81 Linoleum.
82 Oil cloth.
83 Treated and/or coated paper.
84 Waterproof canvas
85 Oily rags.
86 Asphalt treated material.
89 Material compounded with oil, not classified above.
80 Material compounded with oil, undetermined.

Other Type of Material Ignited
97 Multiple types of material first ignited.
98 Type of material not applicable.
99 Type of material not classified above.
00 Type of material undertermined or not reported.

FORM OF MATERIAL IGNITED

Structural Component, Finish
11 Exterior roof covering, surface, finish.
12 Exterior sidewall covering, surface, finish.
13 Exterior trim, appurtenances.
14 Floor covering, surface.
15 Interior wall covering, surface items permanently affixed to wall and door surface.
16 Ceiling covering, surface.
17 Structural member, framing.
18 Thermal, acoustical insulation within wall, partition or floor/ceiling space.
19 Structural component, finish, not classified above.
10 Structural component, finish, undetermined.

Furniture
21 Upholstered sofa, chair, vehicle seats.
22 Nonupholstered chair, bench.
23 Cabinetry.
24 Ironing board.
25 Appliance housing or casing.
29 Furniture not classified above.
20 Furniture, undetermined.

Soft Goods, Wearing Apparel
31 Mattress, pillow.
32 Bedding, blanket, sheet, comforter.
33 Linen, other than bedding
34 Wearing apparel not on a person.
35 Wearing apparel on a person.
36 Curtain, blind, drapery, tapestry
37 Goods not made up.
38 Luggage
39 Soft goods, wearing apparel, not classified above.
30 Soft goods, wearing apparel, undetermined

Adornment, Recreational Material
41 Christmas tree
42 Decoration for special event.
43 Book
44 Magazine, newspaper, writing paper
45 Toy, game
46 Awning, canopy.
47 Tarpaulin, tent.
49 Adornment, recreational material, not classified above.
40 Adornment, recreational material, undetermined

Suppliers, Stock
51 Box, carton, bag.
52 Basket, barrel.
53 Pallet, skid (not in use).
54 Rope, cord, twine, yarn.
55 Packing, wraping material.
56 Bale storage.
57 Bulk storage.
58 Cleaning supplies.
59 Supplies, stock not classified above
50 Supplies, stock, undetermined.

Power Transfer Equipment, Fuel
61 Electrical wire, cable insulation.
62 Transformer.
63 Conveyor belt, drive belt, V-belt.
64 Tire.
65 Fuel.
69 Power transfer equipment, fuel, not classified above.
60 Power transfer equipment, fuel, undetermined.

General Form
71 Agricultural product.
72 Fence, pole.
73 Fertilizer.
74 Growing, living form.
75 Rubbish, trash, waste
76 Cooking materials.
77 Sign.

Special Form
81 Dust, fiber, lint
82 Pyrotechnics, explosives
83 Atomized, vaporized liquid.
84 Chips.
85 Palletized material, material stored on pallets
86 Gas or liquid in or from pipe or container
87 Rolled material.
88 Adhesive.

Other Form of Material
97 Multiple form of material ignited
98 Form of material not applicable.
99 Form of material not classified above.
00 Form of material undetermined or not reported

IGNITION FACTOR

Incendiary
11 Incendiary, not during civil disturbance
12 Incendiary, during civil disturbance.

Suspicious
21 Suspicious, not during civil disturbance
22 Suspicious, during civil disturbance.

Misuse of Heat of Ignition
31 Abandoned, discarded material.
32 Thawing.
33 Falling asleep.
34 Inadequate control of open fire.
35 Cutting, welding too close to.
36 Children with, child playing.
37 Unconscious, mental, physical impairment
39 Misuse of heat of ignition not classified above
30 Misuse of heat of ignition, undetermined.

Misuse of Material Ignited
41 Fuel spilled, released accidentally.
42 Improper fueling techniques.
43 Flammable liquid used to kindle fire.
44 Washing part, cleaning, refinishing, painting.
45 Improper container.
46 Combustible too close to heat.
47 Improper storage.
48 Children with, child playing.
49 Misuse of material ignited not classified above
40 Misuse of material ignited, undetermined

Mechanical Failure, Malfunction

51 Part failure, leak, break.
52 Automatic control failure.
53 Manual control failure.
54 Short circuit, ground fault.
55 Other electrical failure.
56 Lack of maintenance, worn out.
57 Backfire.
59 Mechanical failure, malfunction not classified above.
50 Mechanical failure, malfunction, undetermined.

Design, Construction, Installation Deficiency

61 Design deficiency.
62 Construction deficiency.
63 Installed too close to combustibles.
64 Other installation deficiency.
65 Property too close to.
69 Design, construction, installation deficiency not classified above.
60 Design, construction, installation deficiency, undetermined.

Operational Deficiency

71 Collision, overturn, knockdown.
72 Accidentally turned on, not turned off.
73 Unattended.
74 Overloaded.
75 Spontaneous heating.
76 Improper startup, shutdown procedures.
79 Operational deficiency not classified above.
70 Operational deficiency, undetermined.

Natural Condition

81 High wind.
82 Earthquake.
83 High water, including floods.
84 Lightning.
89 Natural condition not classified above.
80 Natural condition, undetermined.

Other Ignition Factor

91 Animal.
92 Rekindled from a previous fire.
99 Other ignition factor not classified above.
00 Ignition factor undetermined or not reported.

Construction Class

0 No Building Involved.
1 Fireproof Structure
2 Fire Protected Structure.
3 Non-fireproof Structure.
4 Wood Frame Structure.
5 Metal Structure.
6 Heavy Timber Structure.

Classification of Building By Use—Commercial

01 Bank.
02 Brewery.
03 Coal Pocket.
04 Department Store.
05 Electrical Power Plant.
06 Factory: Multi Occupancy.
07 Factory: Single Occupancy.
08 Foundry.
09 Freight Depot.
10 Garage: Non-Storage.
11 Garage: Storage.
12 Gas Works.
13 Lumber Yard.
14 Motor Vehicle Repair Shop.
15 Office Building.
16 Oil Selling Station.
17 Oil Storage Plant.
18 Pier, Wharve, Dock, Bulkhead Building.
19 Restaurant, diner.
20 Shed, Newsstand, Shanty.
21 Shipyard, Drydock.
22 Stable.
23 Steam Generating Plant.
24 Store Building, Taxpayer.
25 Warehouse, Storehouse.
26 Store Building & Private Dwelling.
39 Other Commercial.

Classification of Building By Use—Public

40 Airport Building.
41 Asylum.
42 Bridge.
43 Bus Terminal.
44 Church, Synagogue.
45 Dance Hall, Banquet Hall.
46 Dispensary, Clinic.
47 Ferry Terminal.
Government Buildings—(Not otherwise classified):
48 City.
49 Interstate.
50 Federal.
51 Foreign.
52 State.
53 Hospital, Infirmary.
54 Nursing Home.
55 Railroad Station.
56 School: College, University.
57 School: Private High.
58 School: Public High.
59 School: Public Jr. High.
60 School: Private Elementary.
61 School: Public Elementary.
62 School: Children's Nursery.
63 School: Other.
64 Television Studio.
65 Theatre, Legitimate.
66 Theatre, Motion Picture.
67 Transit System—Station Structure.
68 Tunnel.
69 Other Public.

Residential

80 Apartment Hotel "A."
81 Apartment House "A."
82 Boarding House, Rooming House "B."
83 Convent, Rectory, Monastery, etc.
84 Dormitory—School, Club, Lodge.
85 Hotel "B."
86 Lodging House "B."
87 Motel.
88 Private Dwelling: One Family.
89 Private Dwelling: Two Family.
90 Tenement: New Law "A."
91 Tenement: Old Law "A."
92 Converted Dwelling "A."
99 Other Residential.

Building Status

1 Occupied.
2 Partly Occupied, Good Condition.
3 Partly Occupied, Deteriorating.
4 Vacant.
5 Under Demolition.
6 Under Construction.

Damage (to Building or Contents)

0 None.
1 1 to 15%.
2 16 to 49%.
3 50% or Greater.

Area Fire Origin—Floor

00 Outside Building.
01 1st Floor.
to
94 94th and Higher.
95 Attic.
96 Roof.
97 Basement.
98 Cellar.
99 Sub-cellar.

Area Fire Origin—Room or Area

10 Area Not in Building.
11 Attic.
12 Awning.
13 Balcony.
14 Basement.
15 Bathroom Toilet.
16 Bedroom, Sleeping Area.
17 Ceiling.
18 Cellar.
19 Chimney.
20 Classroom Lecture Area.
21 Closet.
22 Cockloft.
23 Court—Exterior.
24 Court—Interior.
25 Dining Room, Dining Area.
26 Duct—Air Conditioning.
27 Duct—Exhaust.
28 Flooring.
29 Furnace Room.
30 Hallway—Private.
31 Hallway—Public.
32 Incinerator Closet or Room.
33 Kitchen, Cooking Area.
34 Living Room.
35 Lobby.
36 Machinery Room.
37 Office Area.
38 Operating Laboratory Area.
39 Partition.
40 Porch.
41 Projection Booth.
42 Recreation Area.
43 Roof.
44 Sales Showroom Display Area.
45 Shaft—Duct, Pipe.
46 Shaft—Dumbwaiter.
47 Shaft—Elevator.
48 Shaft—Exterior Light.
49 Shaft—Interior Light.
50 Shaft—Vent.
51 Shipping Receiving Loading Area.
52 Stage.
53 Stairway.
54 Storage Room Area.
55 Vacant—Room, Apartment or Area.
56 Work Area, Workroom.
57 Other Areas, Not Classified (state area)

Area Fire Origin—Occupancy Classification—Commercial

Factory:

99 Chemicals.

Clothing:

01 Dresses.
02 Undergarment.
03 Other (state type).
04 Dry Cleaning Laundry.
05 Electrical Products.
06 Food & Drink Products.
07 Furniture.
08 Furs, Fur Goods.

Hats:

09 Men's.
10 Women's.
11 Leather, Leather Products.
12 Machine Shop Metal Works.
13 Paints.
14 Paper Products.
15 Petroleum Products.
16 Plastics, Rubber.
17 Printing & Allied Industries.
18 Shoes.
19 Textiles.
20 Toy or Doll.
21 Woodworking.
22 Other Factories Not Classified (state type).

Store:

23 Auto Accessories.
24 Bakery.
25 Butcher.

4

Area Fire Origin—Occupancy Classification—(continued)

Store:

26 Candy, Cigar, Stationery.
27 Clothing.
28 Department, Large.
29 Department, Small (5&10).
30 Dry Cleaner & Tailor.
31 Drug.
32 Electrical Appliances.
33 Fruit & Vegetables.
34 Furniture.
35 Grocery, Dairy, Delicatessen.
36 Haberdashery.
37 Ladies Accessories.
38 Laundry.
39 Paint Hardware.
40 Restaurant Luncheonette.
41 Shoe.
42 Shoe Repair.
43 Super Market.
44 Tavern.
45 Other Stores Not Classified (state type).

Garages:

46 Non Storage.
47 Storage.
48 Oil Selling Station.
49 Motor Vehicle Repair Shop.
50 Office Building.

Warehouse:

51 Film.
52 Paper, Rags, Fibre.
53 Other (state type).
54 Freight Depot.
55 Pier.
56 Shipyard.
57 Lumber Yard.
58 Shed, Newsstand, Shanty, etc.
59 Other Commercial Building Occupanices, Not Classified (state type).

Residential

60 Apartment Hotel, Multiple Dwelling "A."
61 Apartment House, Multiple Dwelling "A."
62 Boarding House, Rooming House, Multiple Dwelling "B."
63 Hotel, Multiple Dwelling "B."
64 Lodging House, Multiple Dwelling "B."
65 Private Dwelling.
66 Rectory, Convent, Monastery, etc.
67 Tenement House, New Law, Multiple Dwelling "A."
68 Tenement House, Old Law, Multiple Dwelling "A."
69 Other Residential, Not Classified (state type)

Public

70 Airport.
71 Cabaret, Banquet Hall.
72 Church.
73 Dance Hall.
74 Hospital.
75 Motion Picture Theatre.
76 N.Y. Transit System—Station.
77 Passenger Depot.
78 School.
79 Theatre.
80 T.V. Studio.
81 Other Public, Not Classified (state type).

Manner Extension

00 Confined to area of origin.
01 Cockloft.
02 Door or Opening Between Rooms.
03 Floor.
04 Hall Stairway.
05 Partition.
06 Pipe Recess.
07 Shaft-Dumbwaiter.
08 Shaft—Elevator.
09 Shaft—Air, Light, Chute, Duct, etc.
10 Ceiling.
11 Window.
12 Other (state how).

Number of Occupancies

01 1 Occupancy.
02 2 Occupancies.
99 99 or more Occupancies.

Buildings

0 did not spread beyond building of origin.
1 1 structure or vehicle.
9 9 or more buildings or vehicles.

Note: Form BF-24A must be submitted for each building or vehicle listed in this coded space.

Smoke Detector

0 No detector present.
1 Ionization type, power disconnected or battery removed by occupant.
2 Ionization type, provided early warning.
3 Ionization type, failed to operate, battery powered.
4 Ionization type, failed to operate, line voltage power.
5 Photoelectric type, power disconnected or battery removed by occupant.
6 Photoelectric type, provided early warning.
7 Photoelectric type, failed to operate, battery powered.
8 Photoelectric type, failed to operate, line voltage power.
9 Not possible to determine if detector operated or not.

Classification by Type Fire or Emergency

Transportation Fires

87 Ship, Vessel.
88 Motor Vehicle.
89 Other Transportation (state type).

Non-Structural Fires

86 ADV (Abandoned/Derelict Motor Vehicle).
90 Bonfire.
91 Brush, Grass.
92 Demolition Wood, Building Site.
93 Dump, Land Fill.
94 Rubbish—Outside Building.
95 Manhole.
96 N.Y Transit System—Yard, Roadway, Ties, etc.
97 Railroad—Yard Roadway, Ties, etc.
98 Tunnel, Bridge.
99 Other Non-Structural, Not Classified (state type).

Emergency

02 Chimney.
03 Elevator, Escalator.
04 Explosives Escort.
05 First Aid—Assist Person(s).
06 First Aid—Resuscitation.
07 Marine.
08 Precarious Condition—Signs, Trees, etc.
09 Subway—Railroad.
10 Water Leak.
11 Bomb—Unexploded, Scare.
12 Collapse—Cave In.
13 Collision—Vehicular Incident.
14 Controlled Fire, Permitted.
15 Flood Condition—Broken Water Main.
16 Incinerator.
17 Leak—Fuel Oil, Gasoline, etc..
18 Leak—Illum. Gas, Flam. Vapor.
19 Lightning.
20 Oil Burner.
21 Person Locked In, Locked Out.
22 Power—Electrical.
23 Pressure Rupture.
24 Refrigerant Leak.
25 Smoke Condition, Odor, Fumes.
26 Sprinkler.
27 Steam Discharge.
28 Defective Alarm Device (other than Sprinkler).
29 Smoke Detector.
30 Other.

Power for Equipment

01 1-23 volts A.C.
02 24 volts A.C.
11 1-6 volts D.C.
12 7-12 volts D.C.
15 115 volts A.C.
28 208 volts A.C.
30 220-230 volts A.C.
33 231-330 volts A.C.
34 331 or higher volts AC.
50 25-50 volts A.C.
61 Butane.
62 Coal, Coke, Charcoal, Peat.
63 Fuel Oil, No. 1 or No. 2.
64 Fuel Oil, No. 3 or No. 4.
65 Fuel Oil, No. 5 or No. 6.
66 Gasoline.
67 Kerosene.
68 LN gas (stored as liquid).
69 LP gas (stored as liquid).
70 Natural, or illuminating gas (as a gas).
71 Paper.
72 Propane.
99 Other.

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Attachment G-A.3

Explanation of Numeric Codes Used on Fire and Emergency Reports -
From June 1, 1987 to present

MO9-1 H1-911268-5-R25-115

FIRE RECORD CODE LIST

14.1 TYPE OF REPORT

Code No.
1 Structural
2 Transportation Fire
3 Non-Structural Fire
4 Emergency Response
5 False Alarm
6 Additional data (BF-24A)

14.2 HOW REPORTED

Code No.
10 Telephone
20 Street — Manual
21 Class 3 — Manual
30 Class 3 — P.F.A.
31 Class 3 Valve, Sprinkler — P.F.A.
32 Class 3, Other Automatic — P.F.A.
40
50 Verbal
60 Pre-Recorded Telephone Alarm
70 Tie Line (Direct Line to Dispatcher — Pipeline Corp.)
80 Street Box — ERS
81 Class 3 — ERS
90 Cable Television Link

Note: 1. P.F.A. stands for Private Fire Alarm. These are received using the 3-Box-Terminal Designation.

2. If the alarm was encountered while responding to or returning from another alarm, it is considered a verbal alarm.

3. A pre-recorded telephone alarm (PRTA) is used to designate those telephone alarms received from recording or pre-dialed machines, whether directly to 911, 7-digit telephone or an alarm service.

14.3 INITIAL ALARM

Code No.
0 Special Call Other Than Engine Only — No Chief
1 Box (Street or Class 3)
5 Special Call Engine Only — No Chief
8 Special Call (Chief Operated)
9 Still

14.4 HIGHEST ALARM

Code No.
0 Initial Alarm
1 More than the Initial Alarm & Less than 3 Engines & 2 Ladder Cos. at work.
2 2nd Alarm
3 3rd Alarm
4 4th Alarm
5 5th Alarm
6 Simultaneous
7 Signal 7-5

14.5 BOROUGHS

Code No.
1 Manhattan
2 Bronx
3 Staten Island
4 Brooklyn
5 Queens

14.6 HAZARDOUS MATERIALS

14.6.1 Class to be obtained from D.O.T. required labels or placard or from shipping papers or other documents.

Code No.
00 No Hazardous Materials Involved
11 Class A Explosives
12 Class B Explosives
13 Class C Explosives
15 Blasting Agents
21 Flammable Gases
22 Non-Flammable Gases
23 Poison Gases
24 Chlorine
25 Oxygen
31 Flammable Liquids, flashpoint 100 degrees or less.
33 Combustible Liquids, flashpoint greater than 100 degrees.
41 Flammable Solids
42 Spontaneously Combustible Materials
43 Materials Dangerous when Wet
51 Oxidizers
52 Organic Peroxides
61 Poisons
62 Etiologic (Infectious) Substances
63 Irritants
71 Radioactive I Materials
72 Radioactive II Materials
73 Radioactive III Materials
81 Corrosives
98 Multiple Classes (More than one hazardous material)
99 Other

14.6.2 Amount & Unit — the letter designating the unit of measurement shall follow the two digits indicating the amount. Example: 3000 gals. of gasoline would be correctly coded as 07G in the "Amount" field.

000 No Hazardous Material Involved
01 Less than 1 — F Cubic feet, for gases only
02 1-9 — G Gallon
03 10-49 — M Multiple Units — *Ex:* a spill involves a liquid and a solid
05 100-499 — P Pound
06 500-999 — T Ton
07 1,0000-4,999
08 5,000-9,999
09 10,000-49,999
10 50,000-99,999
11 100,000 and more

14.7 HEATING EQUIPMENT INVOLVED—TYPE OF FUEL USED

Code No.
1. Kerosene
2. L.P.G.
3. Electric
4. Wood
5. Coal
6. Oil
7. Natural Gas
8. Gasoline
9. Other
0. No Heating Equipment involved

14.8 HOW EXTINGUISHED

Code No.
0. Before Arrival
1. Hand Extinguishers
2. Sprinkler Heads (State Number of heads that operated in Operations Section)
3. Booster Stream
4. Low Pressure Hydrant Stream
5. One 1¾" or larger hoseline from a pumping unit or a standpipe outlet, regardless of line termination (Controlling Nozzle, Deckpipe, Stang, Multi Versal, Ladder Pipe, T/L, Foam Nozzle, etc.)
6. Two 1¾" or larger hoselines as above.
7. Three 1¾" or larger hoselines as above.
8. Four or more 1¾" or larger hoselines as above.
9. Other (State How).

14.9 SPRINKLER PERFORMANCE—If sprinklers were present or a factor in this operation, record their performance.

1. Equipment Operated
2. Equipment in service, did not operate
3. Equipment present, fire to small to operate
4. Equipment operated, did not extinguish fire
8. No equipment present
9. Equipment present, not in service. (Record action taken in Operations Section)

14.10 STANDPIPE PERFORMANCE—If a standpipe system was present or used in this operations, record its performance.

1. Standpipe serviceable and used
2. Standpipe present but not used
8. No standpipe present
9. Equipment present, not in service. (Record action taken in Operations Section)

14.11 CONDITION ON ARRIVAL CODES

0 No indication of fire
1 Overheat
2 Smoldering
3 Open Flame
8 Out on Arrival

14.12 EQUIPMENT INVOLVED IN IGNITION

1. HEATING SYSTEMS

10. Solar panel.
11. Central heating unit, furnace
12. Water heater.
13. Woodstove, wall furnaces, fixed local heating unit
14. Indoor fireplace
15. Portable heating unit
16. Chimney, gas vent flue
17. Chimney vent connector
18. Heat transfer system, ducts, pipes.
19. Not classified above.

2. COOKING EQUIPMENT

21. Fixed, stationary surface unit, stove
22. Fixed, stationary oven
23. Fixed, stationary food warming applicance.
24. Deep fat fryer
25. Portable cooking, warming unit
26. Open fired grill
27. Grease hood, duct.
29. Not classified above

3. AIR CONDITIONING, REFRIGERATION EQUIPMENT

31. Central air conditioning, refrigeration equipment
32. Water Cooling device, tower
33. Cold boxes, freezers, refrigerators
34. Fixed, stationary local air conditioning unit
35. Portable air conditioning, regrigeration unit, dehumidifier.
39. Not classified above.

4. ELECTRICAL DISTRIBUTION EQUIPMENT

41. Fixed Wiring, power lines, junction boxes.
42. Transformer, overcurrent or disconnect equipment.
43. Meter, meter box.
44. Power Switch gear, fused, circuit breakers.
45. Switch, receptacle outlet.
46. Lighting fixture, lamp-holder ballast, sign,
47. Cord, plug
48. Lamp, light bulb.
49. Not classified above

5. APPLIANCES EQUIPMENT

50. Dishwasher.
51. Television, radio, sound or picture.
52. Clothes dryers.
53. Washing machine
54. Floor care equipment, vacuum,
55. Separate motor, generator.
56. Hand tools, soldering irons, drills.
57. Controlled heat appliance, electrical blankets, steam irons, heat tapes.
58. Electrical razors, can openers.
59. Not classified above.

6. SPECIAL EQUIPMENT

61. Electronic equipment, radar, x-rays, computer, telephone, transmitter equipment.
62. Vending machine, drinking fountain
63. Office machines.
64. Biomedical equipment, device.
65. Separate pump, compressor, sump pump.
66. Internal combustion engine.
67. Conveyor
68. Printing press.
69. Not classified above

7. PROCESSING EQUIPMENT
71. Furnace, oven, kiln.
72. Casting, molding, forging.
73. Heat treating, quench tank.
74. Working, shaping, machine saws, grinders, sanders, etc.
75. Coating machine, asphalt saturating, rubber spreading machines.
76. Painting, dipping, spraying.
77. Chemical process, distilling.
78. Waste recovery.
79. Not classified above.

8. SERVICE, MAINTENANCE EQUIPMENT
81. Incinerator.
82. Bearing, brake.
83. Rectifier, charger, battery.
84. Tarpot, tar kettle.
85. Arc, oil lamp, gas mantles.
86. Elevator.
87. Torches, bunsen burners.
88. Not classified above.

9. OTHER OBJECTS, EXPOSURE FIRE
96. Vehicle, exhaust systems, vehicle parts.
98. No equipment involved.
99. Other object, Exposure Fire not classified above.

14.13 FORM OF HEAT IGNITION

1. HEAT FROM FUEL-FIRED, FUEL-POWERED OBJECT

The difference between subdivision 11 and subdivision 12 is whether a spark, ember or flame actually excaped from the equipment, or whether it was simply overheating of outside surface of the equipment (or its internal heat) causing the ignition of nearby combustibles.

11. Spark, ember, flame escaping from gas fueled equipment.
12. Heat from gas fueled equipment, pilot lights, normal flames.
13. Spark, ember, flame escaping from liquid fueled equipment.
14. Heat from liquid fueled equipment, pilot lights.
15. Spark, ember, flame escaping from solid fueled equipment.
16. Heat from solid fueled equipment.
17. Spark, ember, flame escaping from equipment, fuel not known.
18. Heat from equipment; fuel not known.
19. Not classified above.

2. HEAT FROM ELECTRICAL EQUIPMENT ARCING, OVERLOADED.
21. Water caused short circuit arc.
22. Short circuit arc from mechanical damage.
23. Short circuit arc from defective, worn insulation.
24. Unspecified short circuit arc.
25. Arc from faulty contact, loose connection, broken conductor.
26. Arc, spark from operating equipment or switch.
27. Heat from overloaded equipment, wires, motors.
28. Fluorescent light ballast.
29. Not classified above

3. HEAT FROM SMOKING MATERIAL
31. Cigarette.
32. Cigar.
33. Pipe.
39. Not classified above

4. HEAT FROM OPEN FLAME, SPARK
41. Cutting torch operation (separating metals).
42. Welding torch operation (joining metals).
43. Blow torches, plumbers torches, Bunsen Burners, soldering, paint stripping.
44. Candle, taper.
45. Match.
46. Lighter (flame type)
47. Campfires, bonfires, warning flares, rubbish fires.
48. Backfire from internal combustion engine.
49. Not classified above.

5. HEAT FROM HOT OBJECT
51. Heat, spark from friction, overheated tires
52. Molten metal, hot forging and hot glass.
53. Hot ember, ash.
54. Electric lamp, light bulbs.
55. Rekindle, reignition.
56. Heat from properly operating electrical.
57. Heat from improperly operating electrical equipment.
59. Not classified above.

6. HEAT FROM EXPLOSIVES, FIREWORKS
61. Explosives, bombs, ammunition.
62. Blasting agent.
63. Fireworks, sparklers.
64. Paper cap, party popper.
65. Model rocket, not amateur rocketry.
66. Incendiary device.
67. Not classified above.

7. HEAT FROM NATURAL SOURCE
71. Sun's heat.
72. Spontaneous ignition, chemical reaction.
73. Lightning discharge.
74. Static discharge.
79. Not classified above.

8. HEAT SPREADING FROM ANOTHER HOSTILE FIRE (EXPOSURE)
81. Heat from direct flame, convection currents.
82. Radiated heat.
83. Heat from flying brand, ember, spark.
84. Conducted heat.
89. Not classified above.

9. OTHER FORM OF HEAT OF IGNITION
97. Multiple forms of heat of ignition.
99. Not classified above.

14.14 TYPE OF MATERIAL IGNITED

1. GAS
11. Natural gas.
12. LP-City Gas (LP and air mix)
13. Manufactured gas.
14. LP-Gas.
15. Anesthetic gas.
16. Acetylene.
17. Specialty gas other than anesthetic.
19. Not classified above.

2. FLAMMABLE, COMBUSTIBLE LIQUID
21. Ethyl ether, pentane and ethylene oxide (Class 1A).
22. Acetone, ethyl alcohol, JP-4 jet fuel and methyl ethyl ketone, (Class 1B)
23. Gasoline.
24. Butyl alcohol, propyl alcohol styrene and turpentine. (Class IC).
25. Kerosene, Fuel Oil 1, 2, 4, 5 and Diesel Fuel
26. No. 6 fuel oil, cottonseed oil and creosote oil, (Class IIIA).
27. Cooking oil, transformer and lubicating oil. (Class IIIB)
29. Not classified above.

3. VOLATILE SOLID, CHEMICAL
31. Fat, grease (food).
32. Grease (nonfood), petroleum jellies.
33. Polish, paraffin, wax.
34. Creosote, pitch, adhesive, resin, tar, gelatin, rosin, asphalt.
35. Applied paint, varnish.
36. Combustible metal magnesium titanium and zirconium.
37. Solid chemical, explosives.
38. Radioactive material.
39. Not classified above

4. PLASTIC
41. RIGID PLASTICS
Included are molded plastics such as appliance cases, floor tile, decorative kitchen laminates.
42. RIGID FOAM PLASTICS
Included are rigid thermal foam insulation for walls and refrigerators.
43. FLEXIBLE PLASTICS
Included is electrical wire insulation.
44. FLEXIBLE FOAMPLASTICS
Included are mattresses, furniture interior foam and carpet pads.
45. FILM PLASTICS
Included are polyethylene trash bags, photographic film and coated wallpaper.
49. Plastic not classified above.

5. NATURAL PRODUCTS
51. Rubber.
52. Cork.
53. Leather.
54. Grass, leaves, hay and straw.
55. Grain, feathers, felt, kapok, hemp, jute, cotton, before processing.
56. Coal, coke, briquettes, peat.
57. Food, starch, excluding fat and grease.
58. Tobacco.
59. Not classified above.

6. WOOD, PAPER
61. Growing wood, tree.
62. Felled but unsawn wood.
63. Finished lumber, finished wood.
64. Wood shavings, sawdust excelsior.
65. Hardboard, plywood.
66. Fiberboard, wood pulp, press board.
67. Paper, untreated, uncoated.
68. Cardboard.
69. Not classified above.

7. FABRIC, TEXTILE, FUR
71. Man-made fabric, fiber, finished goods.
72. Cotton, rayon, cotton fabric.
73. Wool, wool mixture, fabric.
74. Fur, silk, other fabric.
75. Wig.
76. Human hair.
79. Not classified above.

8. MATERIAL COMPOUNDED WITH OIL
81. Linoleum.
82. Oil cloth.
83. Treated and/or coated paper, wax paper.
84. Waterproof canvas.
85. Oily rags.
86. Asphalt treated material.
89. Not classified above.

9. OTHER TYPE OF MATERIAL IGNITED
97. Multiple types of material first ignited.
99. Not classified above.

14.15 FORM OF MATERIAL IGNITED

1. STRUCTURAL COMPONENT, FINISH
11. Exterior roof covering, surface, finish.
12. Exterior sidewall, covering surface, finish, eaves.
13. Doors, porches and platforms.
14. Tile, carpet, rug flooring and stairs.
15. Interior wall covering.
16. Ceiling covering surface.
17. Structural member, framing.
18. Thermal, acoustical insulation, within wall or ceiling.
19. Not classified above.

2. FURNITURE
21. Upholstered sofa, chair, vehicle seats.
22. Nonupholstered chair, bench.
23. Cabinetry, filing cabinets, pianos, dressers, desks, tables and bookcases.
24. Ironing board.
25. Appliance housing or casing.
29. Not classified above.

3. SOFT GOODS, WEARING APPAREL
31. Mattress, pillow.
32. Bedding, blanket, sheet, comforter, heating pad.
33. Linen, towels, tablecloths.
34. Wearing apparel not on a person.
35. Wearing apparel on a person.
36. Curtain, blind, drapery, tapestry.
37. Fabrics, yard goods.
38. Luggage.
39. Not classified above.

4. ADORNMENT, RECREATIONAL MATERIAL
41. Christmas tree.
42. Decoration for special event.
43. Book.
44. Magazine, newspaper, writing paper, files.
45. Toy, game.
46. Awning, canopy.
47. Tarpaulin, tent.
49. Not classified above.

5. SUPPLIES, STOCK
51. Box, carton, bag
52. Basket, barrel.
53. Pallet, skid.
54. Rope, cord, twine, yarn.
55. Packing, wrapping material.
56. Bale storage.
57. Bulk storage.
58. Brooms, brushes, mops, cleaning cloths, cleaning supplies.
59. Not classified above.

6. POWER TRANSFER EQUIPMENT, FUEL
61. Electrical wire, cable insulation.
62. Transformer.
63. Conveyor belt, drive belt, v-belt.
64. Tire.
65. Fuel.
69. Not classified above.

7. GENERAL FORM
71. Argicultural product.
72. Fence, pole.
73. Fertilizer
74. Forests, brush and grass.
75. Film, creosote, rubbish, trash, waste.
76. Cooking materials.
77. Sign.

8. SPECIAL FORM
81. Dust, fiber, lint, sawdust.
82. Pyrotechnics, explosives.
83. Atomized, vaporized liquid.
84. Chips.
85. Material stored on pallets.
86. Accelerants.
87. Rolled material.
88. Adhesive.

9. OTHER FORM OF MATERIAL
97. Multiple form of material ignited.
99. Not classified above.

14.16 IGNITION FACTOR (CAUSE)

1. INCENDIARY
11. Incendiary.
12. Incendiary, during civil disturbance.

2. SUSPICIOUS
21. Suspicious.
22. Suspicious, during civil disturbance.

3. MISUSE OF HEAT OF IGNITION
31. Abandoned, discarded material, cigarette, etc.
32. Thawing.
33. Falling asleep.
34. Inadequate control of open fire.
35. Cutting, welding too close to.
36. Children with matches, lighter, etc.
37. Unconscious, mental, physical impairment.
39. Not classified above.

4. MISUSE OF MATERIAL IGNITED
41. Fuel spilled, released accidentally.
42. Improper fueling technique.
43. Flammable liquid used to kindle fire.
44. Washing part, cleaning, refinishing, painting.
45. Improper container.
46. Combustible too close to heat.
47. Improper storage.
49. Not classified above.

5. MECHANICAL FAILURE, MALFUNCTION
51. Part failure, leak, break.
52. Automatic control failure.
54. Short circuit, ground fault.
55. Other electrical failure.
56. Lack of maintenance, worn out, failure to clean.
57. Backfire.
59. Not classified above.

6. DESIGN, CONSTRUCTION, INSTALLATION DEFICIENCY
61. Design deficiency, catalytic converter failure.
62. Construction deficiency.
63. Installed too close to combustibles.
64. Other installation deficiency.
65. Property too close to, included are exposure fires.
69. Not classified above.

7. OPERATIONAL DEFICIENCY
71. Collision, overturn, knockdown.
72. Accidentally turned on, not turned off.
73. Unattended.
74. Overloaded.
75. Spontaneous heating.
76. Improper startup, shutdown procedures.
79. Not classified above.

8. NATURAL CONDITION
81. High wind.
82. Earthquake.
83. High water, including floods.
84. Lightning.
89. Not classified above.

9. OTHER IGNITION FACTORS
91. Animal.
92. Rekindled from a previous fire.
99. Not classified above.
00. No Fire.

14.17 JUVENILE INVOLVED IN IGNITION

0. Juvenile Not Involved in Ignition or No information that a Juvenile was involved.
1. Juvenile Involved in Ignition.

14.18 CONSTRUCTION CLASS

Code No.
0. No building Involved.
1. Fireproof Structure.
2. Fire Protected Structure.
3. Non Fireproof Structure.
4. Wood Frame Structure.
5. Metal Structure.
6. Heavy Timber Structure.

14.19 CLASSIFICATION OF BUILDING BY USE

COMMERCIAL
592 Bank.
723 Brewery.
895 Coal Storage.
581 Department Store.
615 Electrical Power Plant.
708 Factory: Multi-Occupancy.
709 Factory: Single Occupancy.
771 Foundry.
894 Freight Depot.
882 Garage: Non-Storage.
889 Garage: Storage.
767 Gas Works, Natural Gas Plant.
851 Lumber Yard.
573 Motor Vehicle Repair Shop, Paint Shop.
591 Office Building, State, City, Federal or Commercial
571 Oil Selling Station.
841 Oil Storage Plant.
898 Pier, Wharf, Dock, Bulkhead Building.
161 Restaurant, Diner.
925 Shed, Newstand, Shanty.
781 Shipyard, Drydock.
815 Stable.
614 Steam Generating Plant.
539 Storebuilding, Taxpayer.
891 Warehouse Storehouse.
410 Store Building & Private Dwelling.
580 Other Commercial.

PUBLIC
171 Airport Building.
361 Asylum.
921 Bridge.
173 Bus Terminal.
133 Church, Synagogue.
121 Dance Hall, Banquet Hall.
334 Dispensary, Clinic.
177 Ferry Terminal.
331 Hospital, Infirmary.
311 Nursing Home.
174 Railroad Station, Street Level.
175 Railroad Station, Below Grade.
176 Railroad Station, Above Grade.
241 School: College, University.
215 School: High School.
214 School: Junior High.
213 School: Elementary.
211 School: Children's Nursery.
210 School: Other.
185 Television Studio.
181 Theatre, Legitimate.
183 Theatre, Motion Picture.
170 Transit System—Station Structure.
922 Tunnel.
119 Other Public.

RESIDENTIAL
469 Apartment Hotel "A".
429 Apartment House "A".
439 Boarding House, Rooming House "B".
465 Convent, Rectory, Monastery, etc.
461 Dormitory—School, Club, Lodge.
449 Hotel "B".
430 Lodging House "B".
440 Motel.
411 Private Dwelling: One Family.
414 Private Dwelling: Two Family.
420 Tenement: New Law "A".
423 Tenement: Old Law "A".
422 Converted Dwelling "A".
490 Other Residential.

SPECIAL PROPERTIES
972 Airport Runway.
934 Cemetery.
981 Construction Site.
932 Dump, Landfill.
931 Open Land, Fields.
965 Parking Area, Lot.
963 Pipeline, Power Line Right-of-Way.
962 Public Street.
952 Railroad Switching Yard, marshalling yard.
936 Vacant Lots.
939 Outdoor Property Not Classified.

14.20 BUILDING STATUS CODE

Code	Description
1	Occupied: The building is normally fully occupied or is intended to be fully occupied. A few vacant areas, which are rentable, may exist.
2	Partly Occupied: The building is in good condition and more than 25 percent of the areas are vacant.
3	Partly Occupied, Deteriorating: The building has some vacant areas and these are expected to remain vacant until demolition or alteration because of the condition of the building or its surroundings.
4	Vacant: The building is entirely vacant. (Even if squatters are present.)
5	Under Demolition: The building is in the process of being torn down.
6	Under Construction: The building is under construction and does not have any occupants.
7	Under Construction: The building is partially occupied, whether under a temporary certificate of occupancy or not.

Note: The status code applies to the building, not the fire area. Therefore, codes 1, 2 and 3 may be used whether the fire itself occurred in a vacant or occupied area, and code 1 may apply even if the fire occurred in a vacant area (for example, a fire in a vacant apartment being repainted for a new tenant). The occupied or vacant status of the fire area is now recorded on the "Area of Origin" Code. (see Paragraph 2.19.2)

4

14.21 COMPLEX

11. PUBLIC RECREATION COMPLEX
Included are zoos, amusement parks and general recreation parks.
12. STADIUM, EXHIBITION HALL COMPLEX
Included ballparks, racetracks, sports gardens and armories.
14. CLUB COMPLEX
Included are golf clubs, tennis clubs and country clubs.
20. EDUCATIONAL COMPLEX
Included are schools, colleges and universities.
33. MEDICAL CARE COMPLEX
Included are Hospitals, Medical Centers, Mental Institutions.
34. PRISON COMPLEX
40. BUSINESS WITH RESIDENTIAL COMPLEX
Included are apartments over stores.
41. DWELLING COMPLEX (ONE AND TWO-FAMILY)
42. APARTMENT COMPLEX
44. HOTEL COMPLEX
Included are motels, inns and lodges.
47. MOBILE HOME PARK COMPLEX
58. SHOPPING COMPLEX
Included are department stores malls, discount houses and shopping centers. Also included are groups of business and commercial establishments which may contain theaters and other places of assembly.
59. OFFICE COMPLEX
Included are non-military government office complexes.
61. POWER PRODUCTION COMPLEX
63. MILITARY RESERVATION DEFENSE COMPANY
65. FARM COMPLEX
70. INDUSTRIAL PLANT, MANUFACTURING COMPLEX
80. WAREHOUSE, STORAGE COMPLEX
91. CONSTRUCTION COMPLEX
Included are demolition operations.
93. CAMPSITE COMPLEX
94. WATERFRONT COMPLEX
Included are marinas.
95. RAILROAD TRANSPORT COMPLEX
96. ROAD COMPLEX
Included are highways, streets and all public ways.
97. AIRPORT COMPLEX
98. NO COMPLEX
If other properties meeting the definition for a complex as defined above are identified, they may be indicated by Complex Code 99.

AREA FIRE ORIGIN

14.22 FLOOR CODE NO.

00 OUTSIDE BUILDING
01 1st Floor
to to
94 94th and Higher
95 Attic
96 Roof
97 Basement
98 Cellar
99 Sub-Cellar

14.23 AREA FIRE ORIGIN—OCCUPANCY CLASSIFICATION

00 NOT IN BUILDING

COMMERCIAL

Factory:
99 Chemcials
01 Dresses
02 Undergarment
03 Other (state type)
04 Dry Cleaning Laundry
05 Electrical Products
06 Food and Drink Products
07 Furniture
08 Furs, Fur Goods
09 Men's Hats
10 Women's Hats
11 Leather, Leather Products
12 Machine Shop Metal Works
13 Paints
14 Paper Products
15 Petroleum Products
16 Plastics, Rubber
17 Printing and Allied Industries
18 Shoes
19 Textiles
20 Toy or Doll
21 Woodworking
22 Other Factories not classified (state type)

Store:
23 Auto Accessories
24 Bakery
25 Butcher
26 Candy, Cigar, Stationary
27 Clothing
28 Department, large
29 Department, small (5 & 10)
30 Dry Cleaner & Tailor
31 Drug
32 Electrical Applicance
33 Fruits and Vegetables
34 Furniture
35 Grocery, Dairy, Delicatessan
36 Haberdashery
37 Ladies Accessories
38 Laundry
39 Paint, Hardware
40 Restaurant Luncheonette
41 Shoe
42 Shoe Repair
43 Supermarket
44 Tavern
45 Other Stores not classified (state type)

Garages:
46 Non storage
47 Storage
48 Oil Selling Station
49 Motor Vehicle Repair Shop
50 Office Building

Warehouse:
51 Film
52 Paper, Rags, Fibre
53 Other (state type)
54 Freight Depot
55 Pier
56 Shipyard
57 Lumber yard
58 Shed, Newstand, Shanty, etc.
59 Other Commercial Building Occupancies, not classified (state type)

Residential:
60 Apartment, Hotel, Multiple Dwelling "A"
61 Apartment House, Multiple Dwelling "A"
62 Boarding House, Rooming House, Multiple Dwelling "B"
63 Hotel, Multiple Dwelling "B"
64 Lodging House, Multiple Dwelling "B"
65 Private Dwelling
66 Rectory, Convent, Monastery, etc.
67 Tenement House, New Law, Multiple Dwelling "A"
68 Tenement House, Old Law, Multiple Dwelling "A"
69 Other Residential, not classified (state type)

Public:
70 Airport
71 Cabaret, Banquet Hall
72 Church
73 Dance Hall
74 Hospital
75 Motion Picture Theatre
76 N.Y. Transit System Station
77 Passenger Depot
78 School
79 Theatre
80 T.V. Studio
81 Other Public, not classified (state type)

14.24 AREA OF FIRE ORIGIN

0. MEANS OF EGRESS
01. Hallway, corridor, mall.
02. Exterior stairway.
03. Interior stairway.
04. Escalator.
05. Lobby, entrance way.
09. Not classified above.

1. ASSEMBLY AREA
11. Fixed seats (100 or more persons).
12. Without fixed seats (100 or more persons).
13. With or without fixed seats. (less than 100 persons)
14. Living room, family room, lounge area.
15. Sales, showroom area.
16. Library, art galleries, exhibit.
17. Swimming pool area.
19. Not classified above.

2. FUNCTION AREAS
21. Bedrooms, patient rooms, cells, lockups.
22. Wards, dormitories, barracks.
23. Dining area, lunchroom, cafeteria.
24. Kitchen, cooking area, cloakroom.
25. Laundry area.
28. Health clubs, massage parlors, barber, beauty.

3. FUN AREAS (continued)
31. Laboratrory.
32. Printing or photographic room.
33. First aid, treatment room.
34. Operating room.
35. Electronic, computer, telephone room, telephone booth.
36. Performance, stage area, indoor sports.
37. Projection room, stage light.
38. Process, manufacturing area.
39. Not classified above.

4. STORAGE AREAS
41. Tank, bin, product storage room.
42. Closet.
43. Supply room.
44. Records storage room, vault.
45. Shipping, receiving, loading mail room.
46. Trash or rubbish container, compactor.
47. Garage, carport, vehicle storage.
49. Not classified above.

5. SERVICE FACILITIES
51. Elevator, dumbwaiter.
52. Electrical, plumbing, ventilation shaft.
53. Light shaft.
54. Laundry or mail chute.
55. Duct.
56. Display window.
57. Chimney, flue, stovepipe.
58. Conveyor.
59. Not classified above.

6. SERVICE, EQUIPMENT AREA
61. Machinery room.
62. Heating equipment, water heater area.
63. Switchgear area, transformer vault.
64. Incinerator room area.
65. Maintenance shop, workshop, paint shop, welding shop.
66. Test cell.
67. Enclosure with pressurized air.
68. Enclosure with enriched oxygen atmosphere.
69. Not classified above.

7. STRUCTURAL AREAS, NON-FUNCTIONAL
71. Crawl space, cellar, substructural area.
72. Exterior balcony, open porch.
73. Floor and ceiling assembly concealed floor/ceiling space.
74. Roof and ceiling assembly, concealed roof/ceiling space.
75. Wall assembly, concealed wall space.
76. Exterior wall surface.
77. Exterior roof surface.
78. Awning, overhang.
79. Not classified above.

8. TRANSPORTATION, VEHICLE AREAS
81. Passenger area.
82. Trunk, load carrying area.
83. Engine, running gear, wheel area.
84. Fuel tank, fuel line.
85. Operating, control area, cab, cockpit.
86. Exterior exposed surface.
89. Not classified above.

9. OTHER AREAS OF ORIGIN
91. On or near railroad right of way, embankment.
92. On or near highway, public way, street.
93. Terrace, patio, courtyard.
94. Lawn, field, open area, vacant lot.
95. Wildland area, woods.
97. Multiple location.
98. Vacant room, apartment or area.
99. Not classified above.

14.25 MANNER OF EXTENSION

Code No.
00 Confined to area of origin.
01 Cockloft.
02 Door or opening between rooms.
03 Floor.
04 Hall, Stairway.
05 Partition.
06 Pipe Recess.
07 Shaft-dumbwaiter.
08 Shaft-Elevator.
09 Shaft-air, Light, Chute, Duct, etc.
10 Ceiling.
11 Window.
12 Other (state type).

14.26 NUMBER OF OCCUPANCIES

Codes	Description
01	1 occupancy
02	2 occupancies
•	•
•	•
•	•
99	99 or more occupancies

14.27 BUILDINGS

Code: 0 to 9
0— did not spread beyond building of origin
1—1 structure or vehicles
• •
• •
• •
9—9 or more buildings or vehicles

NOTE: Form BF-24A must be submitted for each building or vehicle listed in this coded space.

14.28 DAMAGE CODES

14.28.1 Percentage Codes
0 No appreciable damage
1 From 1 through 15%
2 From 16 through 49%
3 50% or greater

14.28.2 Extent of Damage Codes: To be used in the Damage Category Boxes "Flame, Smoke and Water".
1. Confined to object or origin.
2. Confined to part of room or area of origin.
3. Confined to room of origin.
4. Confined to fire-rated compartment of origin.
5. Confined to floor of origin.
6. Confined to structure of origin.
7. Extended beyond the structure of origin.
9. No damage of this type.

14.29 SMOKE AND HEAT DETECTOR CODES

14.29.1 Present
1 Present
0 Not Present

14.29.2 Type
1. Smoke
2. Heat

14.29.3 Power Source
1. Battery
2. A/C

14.29.4 Performance
1. In room of fire; operated
2. Not in room of fire; operated
3. In room of fire; did not operate
4. Not in room of fire; did not operate
5. In room; fire too small to operate
6. Did not operate; power source removed
9. Not classified

14.30 POWER FOR EQUIPMENT
01 1-23 volts A.C.
02 24 volts A.C.
11 1-6 volts D.C.
12 7-12 volts D.C.
15 115 volts A.C.
28 208 volts A.C.
30 220-230 volts A.C.
33 231-330 volts A.C.
34 331 or higher volts A.C.
50 25-50 volts A.C.
61 Butane
62 Coal, Coke, Charcoal, Peat
63 Fuel Oil, No. 1 or No. 2
64 Fuel Oil, No. 3 or No. 4
65 Fuel Oil, No. 5 or No. 6
66 Gasoline
67 Kerosene
68 LN gas (stored as liquid)
69 LP gas (stored as liquid)
70 Natural or Illuminating gas (as a gas)
71 Paper
72 Propane
99 Other

14.31 CODE FOR TYPE OF ACTION TAKEN
1. Extinguishment
2. Rescue
3. Investigation
4. Remove Hazard
5. Standby
6. Salvage
7. First Aid
9. Cancelled Enroute

14.32 CLASSIFICATION BY TYPE FIRE OR EMERGENCY

TRANSPORTATION

Code No.
87 Ship, Vessel
88 Motor Vehicle
89 Other Transportation (state type)

NON-STRUCTURAL FIRES

Code No.
84 Explosion, no after fire
85 Outside Spill/Leak with Fire
86 ADV (Abandoned/Derelict Motor Vehicle)
90 Bonfire
91 Brush, Grass
92 Demolition Wood, Building Site
93 Dump, Land Fill
94 Rubbish—Outside Building
95 Manhole
96 N.Y. Transit System—Yard, Roadway, Ties, etc.
97 Railroad Yard, Roadway, Ties, etc.
98 Tunnel, Bridge
99 Other Non-Structural, not classified (state type)

EMERGENCY

Code No.
03 Elevator, Escalator
04 Explosives Escort
05 First Aid—Assist Person(s)
06 First Aid—Resuscitation
07 Marine
08 Precarious Condition—Signs, Trees, etc.
09 Subway, Railroad
10 Water Leak
11 Bomb—Unexploded, Scare
12 Collapse—Cave In
13 Collision—Vehicular Incident
14 Controlled Fire, Permitted
15 Flood Condition—Broken Water Main
16 Incinerator
17 Leak—Fuel Oil, Gasoline, etc.
18 Leak—Illuminating Gas, Flammable Vapor
19 Lightning
20 Oil Burner
21 Person Locked In, Locked Out
22 Power Electrical
23 Pressure Rupture
24 Refrigerant Leak
25 Smoke Condition, Odor, Fumes
26 Sprinkler—Leak, Water Discharge, Damaged Head, etc.
27 Steam Discharge
28 Defective Alarm Device (other then Sprinkler)
29 Smoke Detector
30 Defective Alarm (Sprinkler)—Surge, Work on System, etc.
31 Other

14.33 MOBILE PROPERTY TYPE CODES
11 Automobile
12 Bus
13 Motorcycle, Snowmobile
14 Motorhome
15 Travel Trailer
17 Mobile Home
20 Freight, Road Transport
30 Rail Transport
40 Water Transport
50 Air Transport
60 Heavy Equipment
70 Special Vehicles, Containers
99 Other Mobile Property Types

Attachment G-A.4

Significant fire incidents occurring in WTC 1 (12)

Significant Fire	Incident Date	Fire Location	# Sprinklers Activated	# Standpipes Activated	Cause of Fire	Material Ignited
1	9/9/77	B-6 level storage room	2		None listed	Not listed
2	9/23/77	Dumpster on B-4 level	2		Not classified	Trash/waste
3	10/16/81	19th floor office area	-	2	Discarded material	Furniture
4	12/23/83	2 dumpsters on B-4 level	2	1	Suspicious	Trash/waste
5	1/27/85	Office space on mezzanine level (Floor 2)	2		Incendiary	Trash/waste
6	9/10/85	Garbage dumpster in service elevator lobby on floor 43	2	1	Suspicious	Trash/waste
7	11/1/85	Storage closet on B-4 level	3	1	Suspicious	Supplies/stock
8	6/7/86	Dumpster fire on floor 106, compactor room on floor 107	2	1	None listed	Trash/waste
9	9/30/91	Office on B-4 level	≥1	2	Discarded material	Trash/waste
10	11/19/91	Electrical closet on floor 93	0	2	Short circuit	Electrical wire or cable insulation
11	7/23/92	Level B-5 at the power distribution panel	0	2	Electrical failure	Electrical wire or cable insulation
12	11/10/99	Computer room on floor 104	3	≥1	None listed	Plastics, electronic equip

REPORT - STRUCTURAL FIRE

FOR FIRE RECORDS USE ONLY	TYPE REPORT	MONTH	DAY	YEAR	CD. NO.	HOUR	MINUTE	HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	ADMIN. COMPANY	BLOCK NUMBER	REPORT NUMBER	FILING BATTALION	BADGE NUMBER
	1	09	09	77	1	23	04	01	00	4	1	0	1	0070	5006	0113	0063	01	321

Column groups: DATE AND TIME ALARM RECEIVED; DURATION OF INCIDENT; LOCATION (ALARM / INCIDENT); REPORT

EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	ENG. SQ. MARINE	LADDERS	CODE	NO. SPRINK. LINES OPEN	STANDPIPE LINES USED	BOOSTERS	CAUSE	FIRE MARSHAL	CONSTRUCT	USE	STATUS	DAMAGE TO BLDG.	DAMAGE TO CONTENTS	FLOOR CODE	ROOM OR AREA CODE	OCCUPANCY	MANNER OF EXTENSION
00	00	00	00	00	03	2	0	02	00	0			1	15	1	0	1	99	54	50	00

Column groups: AIDED AND CASUALTIES (CIVILIANS); RESP.; EXTINGUISHMENT; INVESTIGATION; STRUCTURE; AREA FIRE ORIGIN

ADDRESS 1 (NUMBER) World Trade Center (STREET) Man. (BOROUGH)

BUILDING 110 (STORIES) 200x200 (AREA)

(Second Card)

NAME OF OCCUPANT / ROOM/APT. NO: Sgt. Daniels P.A. Police

LEFT IN CHARGE

COMPANIES (TYPE NO. SECT. PTS.)

5 006 0 10	7 001 0 10	7 00# 0 10					

OPERATIONS

Upon arrival at command post was told of fire in B 6 level storage room, operations as follows.

Ladder 1 made necessary imfstagation, located fire, vented, overhauled.

Ladder # checked for extension, vented overhauled.

Engine 6 stretched line from standpipe and stood fast.

Fire was extinguished by sprinkler system before arrival.

BN. 4 suppervised operations on fire floor.

Fire Patrole 2 on the scene.

B.C. Rudy E. DiGeorgio (signature) 1 BN

Rudy E. DiGeorgio (TYPE FULL NAME) 2306 (TIME OF ARRIVAL)

D.C. (TYPE FULL NAME) (TIME OF ARRIVAL) DIV. PAGE 1 OF 1

FOR FIRE RECORDS USE ONLY	TYPE REPORT	MONTH	DAY	YEAR	CO. NO.	HOUR	MINUTE	HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	ADMIN. COMPANY	BLOCK NUMBER	REPORT NUMBER	FILING BATTALION	BADGE NUMBER
	1	09	23	77	1	22	48	00	25	4	1	0	1	0070	5 006	0113	0153	01	187

DATE AND TIME ALARM RECEIVED — DURATION OF INCIDENT — LOCATION: ALARM / INCIDENT — REPORT

EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	ENG. SQ. MARINE	LADDERS	CODE	NO. SPRINK. HDS. OPEN	STANDPIPE LINES USED	CURRENT CONTROL	CAUSE	FIRE MARSHAL	CONSTRUCT.	USE	STATUS	DAMAGE TO BLDG.	DAMAGE TO CONTENTS	FLOOR CODE	ROOM OR AREA CODE	OCCUPANCY	MANNER OF EXTENSION
00	00	00	00	00	03	2	0	02	00	1	63			1	15	1	0	0	99	54	50	00

AIDED AND CASUALTIES: CIVILIANS — RESP. — EXTINGUISHMENT — INVESTIGATION — STRUCTURE — AREA FIRE ORIGIN

ADDRESS 1 World Trade Ctr. (NUMBER, STREET) Man (BOROUGH)

PONYA (NAME OF OCCUPANT) B-4(Lev (ROOM/APT. N)

BUILDING (Second Card) 110 (STORIES) 250 x250 (AREA)

Ptl Lorenza(PONYA) (LEFT IN CHARGE)

COMPANIES (TYPE NO. SECT. PTS.): 5 006 0 05 7 001 0 05

OPERATIONS

Responded to 3-70-10(Manual Alarm)

Upon arrival was informed of fire in Dumpster B-4 level 1 WTC. Ordered investigation and found fire therein, which had been extinguished prior to the arrival of this dept. Operations as follows:

E.6- Rolled up lengths stood fast.

L.1- Search. examination, ventilation of B-4 level, conditions as stated.

B.C. Michael R Porzio 1 (BN.) D.C. ____ DIV. ____ PAGE ____ OF ____

Michael R. Porzio (TYPE FULL NAME) 2251 (TIME OF ARRIVAL) ____ (TYPE FULL NAME) ____ (TIME OF ARRIVAL)

ADMINISTRATIVE COMPANY

BF-24 (1/80)

REPORT — STRUCTURAL FIRE

INCIDENT LOCATION				DATE AND TIME ALARM RECEIVED		INCIDENT DURATION					BOX LOCATION	REPORT			COS. RESP.	
TYPE REPORT	COMMUNITY BOARD	ADMIN COMPANY	BLOCK NUMBER	MONTH DAY YEAR	HOUR MINUTE	HOURS MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	REPORT NUMBER	FUND BATTALION	BADGE NUMBER	ENG SQ ENGINE	LADDER
1	01	5 006	0113	10-16-81	1912	02:30	6	1	7	1	8084	0172	01	531	03	2

AIDED AND CASUALTIES – CIVILIANS						EXTINGUISHMENT		IGNITION						STRUCTURE				AREA FIRE ORIGIN				FIRE EXTENTION				SPECIAL USE		
EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	CODE	SPRINKLER NO. OPEN	STANDPIPE LINES USED	TERM. STAGE	EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	FACTOR	CONSTRUCT	USE	STATUS	BLDG. DAMAGE	CONTS. DMGE.	FLOOR CODE	RM/AREA CODE	OCCUPANCY	MANNER OF	WITHIN BLDG. OCCUPANCYS	OTHER NO. BLDGS.	SMOKE DETECTORS	1	2	3
01	00	00	00	00	6	00	02	3	98	30	71	21	31	1	15	1	1	1	19	37	50	00	01	0	6			

ADDRESS: NUMBER	SUFFIX	PREFIX DIRECT	STREET NAME	TYPE	SUFFIX DIRECT
1			World Trade Center		1

BUILDING: STORIES 99 | AREA 200x200

NAME OF OCCUPANT: U.S. Trading Corp | ROOM, APT. NO.: 1973 | LEFT IN CHARGE: Mr. Guzman (FSD)

EQUIPMENT INVOLVED IN IGNITION: YR | MAKE | MODEL | S/N | POWER

COS: UNIT	PTS.	ITEM			
5 010 20	5 006 10	5 007 05	7 001 20	7 008 20	

OPERATIONS

Responded to alarm box 8084. Upon arrival were informed by Port Authority personnel of a fire in the above location. Initial reports also indicated that the fire had penetrated the skin of the bldg Signal 10-75 was transmitted. Port Authority personnel stretched initial hand line into fire locatio

Eng.10- Stretched standpipe line, extinguished main body of fire, washed down.

Eng.6- Took handline from P.A. personnel, assisted in extinguishment.

Eng.7- Relieved Eng.10 on line, supplied standpipe.

Lad.1- To fire floor, performed primary and secondary searches, vented, overhauled

Lad.8- To 20th floor, performed primary and secondary searches on that and upper floors.

B.C. ____ BN. ____ D.C. ____ DIV. ____ PAGE 1 OF 2

TYPE FULL NAME | TIME OF ARRIVAL | TYPE FULL NAME | TIME OF ARRIVAL

ADMINISTRATIVE COMPANY

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT	DATE AND TIME ALARM RECEIVED					BOX LOCATION	
	MONTH	DAY	YEAR	HOUR	MINUTE	BOROUGH	BOX NO.
6	10	16	81	1912		1	8084

BUILDING NO.: 00

ADDRESS: 1 World Trade Center

IF MOBILE PROPERTY

C.O.S.: 1 001 10

OPERATIONS

Lad.8 (cont.)- Forced door to Office room#2073

Res.1- Checked vent ducts and stairways on and above fire floor, secured passenger elevators serving fire floor.

Patrol#1- On scene, salvage work on 16th and 17th floors.

Patrol#2- On scene, salvage work on 18th floor.

Div.1- D.C. Rossi on scene, in charge of Department operations upon arrival.

B.C.	BN.	D.C.	DIV.	PAGE
	1		1	2 of 2
James J. McKenna	1916	John D. Rossi		
TYPE FULL NAME	TIME OF ARRIVAL	TYPE FULL NAME	TIME OF ARRIVAL	

BF-24 (1/80)-175M-608551 (83)

REPORT — STRUCTURAL FIRE

TYPE REPORT	COMMUNITY BOARD	ADMIN. COMPANY	BLOCK NUMBER	MONTH DAY YEAR	HOUR MINUTE	HOURS MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	REPORT NUMBER	FILING BATTALION	BADGE NUMBER	ENG. SQ. MARINE	LADDER
1	01	5 010	0113	12-23-83	0250	00:25	2	1	0	1	8093	0228	01	075	03	2

(INCIDENT LOCATION; DATE AND TIME ALARM RECEIVED; INCIDENT DURATION; BOX LOCATION; REPORT; COS. RESP.)

EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	CODE	SPRINKLER NO. OPEN	STANDPIPE LINES USED	TERM. STAGE	EQUIP INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	FACTOR	CONSTRUCT.	USE	STATUS	BLDG. DAMAGE	CONTS. DMGE.	FLOOR CODE	RM./AREA CODE	OCCUPANCY	MANNER OF	OCCUPANC'S	NO. BLDGS.	SMOKE DETECTORS	SPECIAL USE 1	2	3
00	00	00	00	00	2	02	01	3	98	40	99	75	21	1	15	1	1	1	99	54	47	00	00	0	0			

(AIDED AND CASUALTIES — CIVILIANS; EXTINGUISHMENT; IGNITION; STRUCTURE; AREA FIRE ORIGIN; FIRE EXTENTION — WITHIN BLDG., OTHER)

ADDRESS

NUMBER	SUFFIX	PREFIX DIRECT	STREET NAME	TYPE	SUFFIX DIRECT
1			World Trade Center		

BUILDING STORIES 99 — AREA 200 X2oo

Port Authority — NAME OF OCCUPANT

ROOM, APT. NO.

P.A.Police — LEFT IN CHARGE

EQUIPMENT INVOLVED IN IGNITION — YR. MAKE MODEL S/N POWER

COS

UNIT	PTS.	ITEM				
5 010	10	7 001	10			

OPERATIONS

Responded to report of fire in B-4 Level , and heavy smoke condition. Found two duppsters fully involved in seperate locations on same floor. Sprinklers extinguished major portion of fire. Siginal 10-41 code 1.

Engine Co. 10 - Streched from standpipe, and extinguished remaining fire.
Ladder Co. 1 - Overhauled, checked for extension and vented.

B.C. Rudolph S. Quatrone — TYPE FULL NAME — Rudolph S. Quatrone

BN. 1st — TIME OF ARRIVAL 02:55

D.C. ____ TYPE FULL NAME ____ DIV. ____ TIME OF ARRIVAL ____ PAGE ☐ OF ☐

BF-24 (1/80)

REPORT — STRUCTURAL FIRE

8000787

TYPE REPORT	COMMUNITY BOARD	ADMIN. COMPANY	BLOCK NUMBER	MONTH	DAY	YEAR	HOUR	MINUTE	HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	REPORT NUMBER	FILING BATTALION	BADGE NUMBER	ENG. SQ. MARINE	LADDER
1	01	5 010	0137	01	27	85	20	53	01	00	2	1	1	1	8084	0293	01	113	02	2

INCIDENT LOCATION; DATE AND TIME ALARM RECEIVED; INCIDENT DURATION; BOX LOCATION; REPORT; COS. RESP.

EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	CODE	SPRINKLER NO. OPEN	STANDPIPE LINES USED	TERM. STAGE	EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	FACTOR	CONSTRUCT.	USE	STATUS	BLDG. DAMAGE	CONTS. DMGE.	FLOOR CODE	RM/AREA CODE	OCCUPANCY	MANNER OF	OCCUPANC'S	NO. BLDGS.	SMOKE DETECTORS	SPECIAL USE 1	2	3
00	00	00	00	00	2	01	00	3	96	66	60	75	11	1	15	1	1	1	02	59	59	00	00	0	0			

AIDED AND CASUALTIES — CIVILIANS; EXTINGUISHMENT; IGNITION; STRUCTURE; AREA FIRE ORIGIN; FIRE EXTENTION WITHIN BLDG. OTHER

ADDRESS

NUMBER	SUFFIX	PREFIX DIRECT	STREET NAME	TYPE	SUFFIX DIRECT
1			World Trade Center		'

BUILDING STORIES: 110 AREA: 200x200

NAME OF OCCUPANT: New York ~~////~~ Port Authority NY,NJ ROOM, APT. NO.

LEFT IN CHARGE: Sgt. Doubraski, PA Police

EQUIPMENT INVOLVED IN IGNITION — YR. MAKE MODEL S/N POWER

COS

UNIT	PTS.	ITEM			
5 006 05	7 001 05	7 010 05			

OPERATIONS

On arrival, found two sprinkler heads containing fire in unoccupied office occupancy on mezzanine level. Units assigned as follows:

Eng. 6: Stretched line, stood fast.
Lad. 1: Located fire, searched, extinguished remaining fire. Shut down sprinkler system.
Lad. 10: Checked for extension.

10-41 signal (code 1) transmitted.

B.C. John J. Spillane 1 BN. D.C. ____ DIV. PAGE __ OF __

John J. Spillane — TYPE FULL NAME; 2055 — TIME OF ARRIVAL; TYPE FULL NAME; TIME OF ARRIVAL

AUG 2 8 2003 JRL

AUG 2 8 2003

8160198

TYPE REPORT	COMMUNITY BOARD	ADMIN. COMPANY	BLOCK NUMBER	MONTH	DAY	YEAR	HOUR	MINUTE	HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	REPORT NUMBER	FILING BATTALION	BADGE NUMBER	ENG. SQ. MARINE	LADDER
1	01	5010	0024	09	10	85	16	05	01	00	4	1	2	1	8084	0119	01	578	7	6

INCIDENT LOCATION · DATE AND TIME ALARM RECEIVED · INCIDENT DURATION · BOX LOCATION · REPORT · COS. RESP.

EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	CODE	SPRINKLER NO. OPEN	STANDPIPE LINES USED	TERM. STAGE	EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	FACTOR	CONSTRUCT.	USE	STATUS	BLDG. DAMAGE	CONTS. DMGE.	FLOOR CODE	RM./AREA CODE	OCCUPANCY	MANNER OF	OCCUPANC'S	NO. BLDGS.	SMOKE DETECTORS	SPECIAL USE 1	2	3
00	00	00	00	00	5	2	1	3	89	00	60	75	21	1	15	1	0	0	43	35	81	00	01	1	2			

AIDED AND CASUALTIES (CIVILIANS) · EXTINGUISHMENT · IGNITION · STRUCTURE · AREA FIRE ORIGIN · FIRE EXTENTION (WITHIN BLDG. / OTHER) · SPECIAL USE

ADDRESS

NUMBER	SUFFIX	PREFIX DIRECT	STREET NAME	TYPE	SUFFIX DIRECT
1			World Trade Center		1

BUILDING STORIES: 99 — AREA: 200 x 200

NAME OF OCCUPANT: Port Authority NY NJ — ROOM, APT. NO.: 43 Fl — LEFT IN CHARGE: Sgt. Albin, PA Police Dept.

EQUIPMENT INVOLVED IN IGNITION

YR.	MAKE	MODEL	S/N	POWER

COS

UNIT	PTS.	ITEM												
5	010	10	5	006	02	7	010	10	7	008	05	7	001	10

OPERATIONS

Upon arrival at the Command Ctr of the World Trade Center, Batt. 1 was informed by the PA Police that, according to their reports, there was a sprinkler flow and smoke condition on the 43 floor. When Ladder 10 (1st due) reported a medium smoke condition on the 43 floor, Batt. 1 transmitted signal 10-76. The fire was located in a garbage dumpster in a service elevator lobby on the 43 F and was extinguished by 2 sprinkler heads with no extension.

Engine 10- Took rolled-ups to 42 floor. Stretched a 2½ line from the standpipe in B stairwell to fire location. Line used for overhauling. Chauffeur manned the Standpipe phone at the CP and assi in communications.

B.C. Kenneth T. McGowan — BN. 1 — TIME OF ARRIVAL 1611

D.C. John J. Hodgens — DIV. 1 — TIME OF ARRIVAL

PAGE 1 OF 2

FIRE RECORDS

TYPE REPORT	DATE AND TIME ALARM RECEIVED (MONTH DAY YEAR HOUR MINUTE)	BOROUGH	BOX LOCATION (BOX NO.)
6	09-10-85 1605	1	8084

8160199

IF MOBILE PROPERTY

NUMBER (INCL. HYPHEN)	STREET NAME
1	World Trade Center

C.O.S. UNIT	PTS.
5 007 02	

OPERATIONS

Engine 6- (teamed with E.10) Assisted E. 10 in Standpipe stretch.

Engine 7- Manned the Standpipe phone in Stairway B on 1st Fl., relayed messages to and from Command Center

Ladder 10- To 43 fl, stairway B, found fire, searched 43 fl (negative) made necessary examination and overhaul of fire area.

Ladder 1-to 44 fl, searched for fire, smoke extension, searched for troubled occupants (negative) Checked top ten fls (101-110) for smoke (Neg.)

Ladder 8- Made secondary search of 43 fl (negative). Searched fls 44 to 49. Searched affected service elevator cars #17 & 29.

Batt. 4- Set up operations post on 43 fl. Communicated with Command Center, supervised operations of units on 43 and 44 fls.

E.4, E.3 (HiRise Unit), L.15 & Res. Co 1 stood fast, took up.
Brooklyn Cos responding on supplemental Boxes 9031 & 9032 at 1617 hrs Batt 31, Batt 32, E. 205, E.XX 279, L.101, L.110 stood fast, took up.
F.C.U. - Established Field Hqtrs, Monitored Communications.

Sprinkler heads (2) replaced by P.A. personnel.

Signal 10-41, Code 1 transmitted, Evidence of separate, previous fire reported to FM McCaffrey.

Responded to Scene: AC Matthew Farrell, Manhattan Borough Commander

Signal	Time	By
10-84	1609	E.10
10-76	1615	Batt 1
P.W.H.	1627	Div 1
U.C.	1640	Div 1

B.C. Kenneth McGowan / 1 BN. D.C. John J. Hodgens / 1 DIV.

Kenneth T. McGowan — 1611 — John Hodgens

TYPE FULL NAME — TIME OF ARRIVAL — TYPE FULL NAME — TIME OF ARRIVAL

PAGE 2 OF 2

FIRE RECORDS

REPORT — STRUCTURAL FIRE

TYPE REPORT	COMMUNITY BOARD	ADMIN. COMPANY	BLOCK NUMBER	MONTH	DAY	YEAR	HOUR MINUTE	HOURS : MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	REPORT NUMBER	FILING BATTALION	BADGE NUMBER	ENG. SQ. MARINE	LADDER
1	01	5 010	0024	11	01	85	0405	00:45	2	1	7	1	8093	0004	01	150	03	2

INCIDENT LOCATION · DATE AND TIME ALARM RECEIVED · INCIDENT DURATION · BOX LOCATION · REPORT · COS. RESP.

AIDED AND CASUALTIES / CIVILIANS: EVACUATED, FIRST AID, INJURED, KILLED, UNIFORMED INJURED	EXTINGUISHMENT: CODE, SPRINKLER NO. OPEN, STANDPIPE LINES USED	IGNITION: TERM. STAGE, EQUIP. INVOLVED, HEAT FORM, MATERIAL TYPE, MATERIAL FORM, FACTOR	STRUCTURE: CONSTRUCT, USE, STATUS, BLDG. DAMAGE	AREA FIRE ORIGIN: CONTS. DMGE., FLOOR CODE, RM./AREA CODE, OCCUPANCY	FIRE EXTENTION: MANNER OF, OCCUPANC'S, WITHIN BLDG., NO. BLDGS. OTHER, SMOKE DETECTORS	SPECIAL USE 1 2 3
00 00 00 00 00	5 03 01	3 98 00 60 50 21	1 15 1 1	1 99 54 50	00 99 0 0	

ADDRESS NUMBER	SUFFIX	PREFIX DIRECT	STREET NAME	TYPE	SUFFIX DIRECT
1&2			World Trade Center		

BUILDING STORIES 90 — 900x500 AREA

NAME OF OCCUPANT ______ ROOM, APT. NO. ______

F.S.D. Gene Raggio
LEFT IN CHARGE

EQUIPMENT INVOLVED IN IGNITION — YR. — MAKE — MODEL — S/N — POWER

COS	UNIT	PTS.	ITEM
	5 010	10	
	5 007	05	
	5 006	05	
	7 010	10	
	7 001	10	

OPERATIONS

Upon arrival to box 8093, at the World Trade Center complex found the following conditions. Proceeded to W.T.C. Command Post and was informed by Port Authority P.D. of heavy smoke condition on B#4 level in area under trade center between nos. 1&2 WTC. Report from companies was for fire in storage closet of mens bathroom in this area , with three sprinkler heads activated keeping fire under control until arrival of F.D. units Co's operated VIZ:

B.C. Joseph L. Barracato BN. 1 D.C. ______ DIV. ______
Joseph L. Barracato
TYPE FULL NAME
0408
TIME OF ARRIVAL
TYPE FULL NAME ______ TIME OF ARRIVAL ______
PAGE 1 OF 2

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT	MONTH	DAY	YEAR	HOUR	MINUTE	BOROUGH	BOX NO.
6	11	01	85	04	05	1	8093

ADDRESS: NUMBER (INCL. HYPHEN) 1&2 — STREET NAME WTC

IF MOBILE PROPERTY . ITEM

YR. MAKE MODEL SERIAL NO. LICENSE STATE

C.O.S. UNIT PTS.

OPERATIONS

E-10 stretched line into fire area from standpipe and extinquished all remaining in closet area

E-7 Stretched line from opposite side of fire and stood fast

E-6 assisted E10 in stretching line and relieved on line and then washdown

L-10 found fire and performed necessary VES and overhaul in area, made primary and secondary search , then up

L-1 performed necessary VES and checked for possible extension in surrounding areas

Times: 10-84	0408
10-75	0410
All Hands	0425
10-41-1	0425

B.C. [signature] BN. 1 D.C. DIV. PAGE 2 OF 2

Joseph L. Barracato — TYPE FULL NAME — 0408 TIME OF ARRIVAL — TYPE FULL NAME — TIME OF ARRIVAL

ADMINISTRATIVE COMPANY

BF-24 (1/80)-175M-608551 (83)

REPORT — STRUCTURAL FIRE

TYPE REPORT	COMMUNITY BOARD	INCIDENT LOCATION: ADMIN. COMPANY	INCIDENT LOCATION: BLOCK NUMBER	DATE AND TIME ALARM RECEIVED: MONTH / DAY / YEAR	HOUR / MINUTE	INCIDENT DURATION: HOURS / MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX LOCATION: BOX NUMBER	REPORT NUMBER	REPORT: FILING BATTALION	BADGE NUMBER	COS. RESP.: ENG. SQ. MARINE	LADDER
1	01	5 010	0024	06-07-86	0949	01:10	8	1	1	1	8084	0076	01	075	04	2

EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	CODE	SPRINKLER NO. OPEN	STANDPIPE LINES USED	TERM. STAGE	EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	FACTOR
00	00	00	00	00	2	02	01	3	00	00	99	75	00

AIDED AND CASUALTIES (CIVILIANS); EXTINGUISHMENT; IGNITION

CONSTRUCT.	USE	STATUS	BLDG. DAMAGE	CONTS. DMGE.	FLOOR CODE	RM./AREA CODE	OCCUPANCY	MANNER OF	OCCUPANC'S	NO. BLDGS.	SMOKE DETECTORS	SPECIAL USE 1	2	3
1	15	1	1	1	94	57	40	00	99	0	2			

STRUCTURE; AREA FIRE ORIGIN; FIRE EXTENTION WITHIN BLDG. / OTHER

ADDRESS: NUMBER 1 | SUFFIX | PREFIX DIRECT | STREET NAME W T C | TYPE | SUFFIX DIRECT 1

BUILDING STORIES 99 — 200 x 200 AREA

NAME OF OCCUPANT: Inhilco Corp — ROOM, APT. NO.: 106-107 — LEFT IN CHARGE: FSD & PA Police

EQUIPMENT INVOLVED IN IGNITION — YR. | MAKE | MODEL | S/N | POWER

COS	UNIT / PTS.	ITEM			
	5 007 10	7 010 10	5 006 05	5 010 05	7 001 05

OPERATIONS

Received EHS alarm for a heavy smoke condition on the 110th floor. While responding received 2nd. source and the box was filled out. Upon arrival found fire in garbage dumpster on the 106th floor. Sprinklers in operation and a heavy smoke condition on 106–110th floors. - Units operated VIZ: Transmitted 10/76.

B.C. [signature] BN. 1 D.C. ____ DIV. ____ PAGE ___ OF ___

TYPE FULL NAME ____ TIME OF ARRIVAL 0154 TYPE FULL NAME ____ TIME OF ARRIVAL ____

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT	MONTH	DAY	YEAR	HOUR	MINUTE	BOROUGH	BOX NO.	BUILDING NO.	TERM. STAGE	EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	CONSTRUCT.	USE	STATUS	BLDG. DAMAGE	CONTS. DMGE.	FLOOR CODE	RM/AREA CODE	OCCUPANCY	MANNER OF	OCCUPANCIES
6	06	07	86	09	49	1	8084	0	3	00	00	99	75	1	15	1	1	1	94	57	40	00	99

DATE AND TIME ALARM RECEIVED — BOX LOCATION — IGNITION — STRUCTURE — AREA OF ORIGIN — FIRE EXTENSION

ADDRESS

NUMBER (INCL. HYPHEN)	SUFFIX	PREFIX DIRECT.	STREET NAME	TYPE	SUFFIX DIRECT.
1			WTC		

IF MOBILE PROPERTY

ITEM

YR.	MAKE	MODEL	SERIAL NO.	LICENSE	STATE

COS.

1 C01 05	5 C03 02						

UNIT PTS.

OPERATIONS

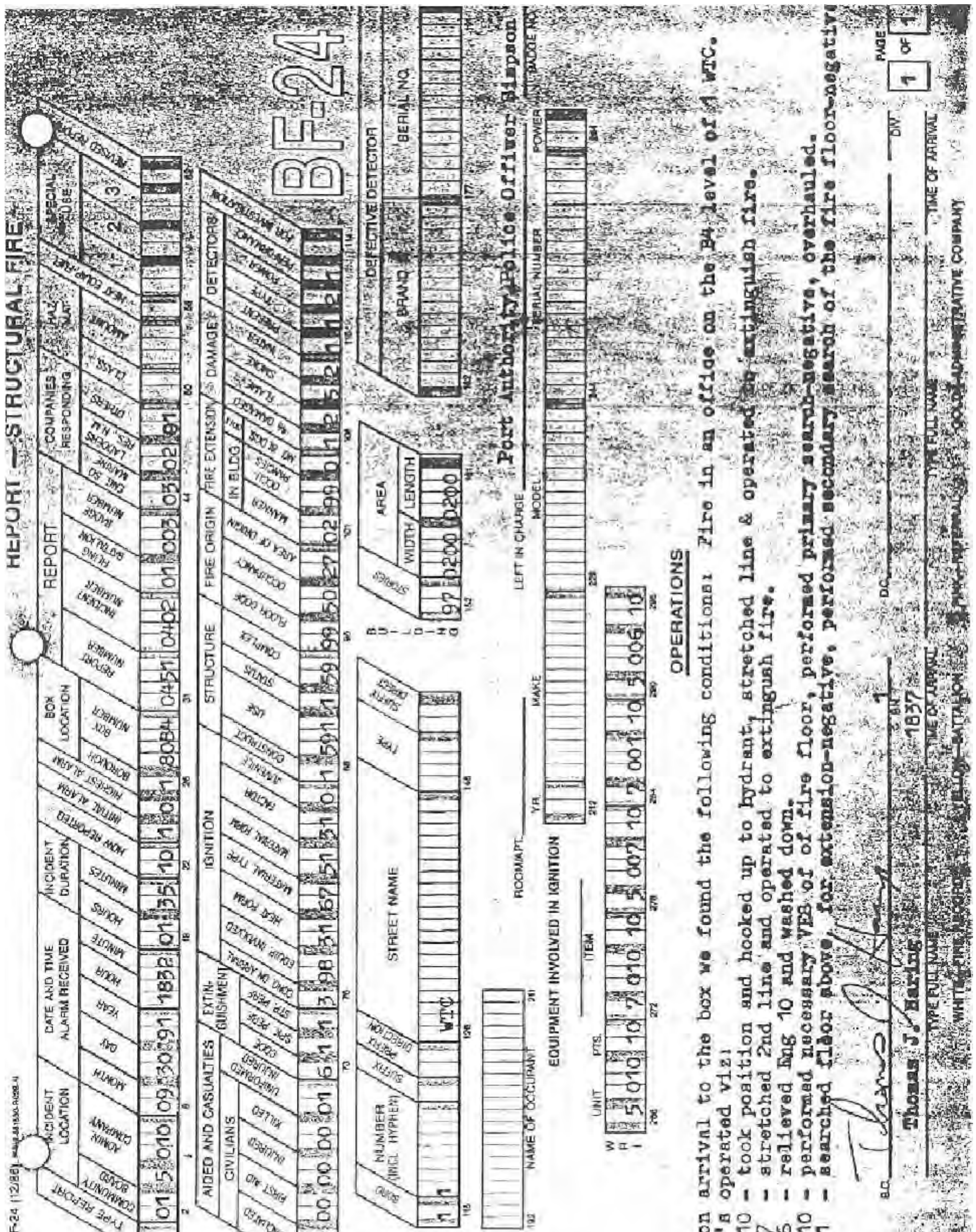
E.7- Upon arrival reported to 106 th floor and stretched from standpipe to fire and extinguished same. Fire in dumpster Assisted L.10 in making search of area. Negative

L.6 Reported to 106th floor. Ordered to stand fast. Then up.
L.10 Reported to 106th Floor. Ordered to stand fast. Then up.
L.10 Reported to 107th Floor. Found fire in compactor room. Sprinkler in operation. Forced entry to sprinkler shut-off room. Shut same. Made necessary search and examination. Then up.
L.1 Reported to 107th Floor. Assisted Ladder 10 in search and examination. Negative searc. Then up.
R-1 Searched floors above fire. Light smoke. Negative. Then up.
L.3/Hi-Rise Unit: Ordered to 10-2
Field Comm Unit: - Ordered to 10-2
Batt. 4: Supervised operations on 106-110 floors. Units E.7, L.10 L.1 and R-1. Then up.

B.C. ________ BN. ________ D.C. ________ DIV. ________ PAGE OF

TYPE FULL NAME — TIME OF ARRIVAL — TYPE FULL NAME — TIME OF ARRIVAL

ADMINISTRATIVE COMPANY



BF-24 (12/86)

REPORT — STRUCTURAL FIRE

BF-24

Port Authority Police Offiver Simpson

OPERATIONS

Upon arrival to the box we found the following conditions: Fire in an office on the B4 level of 1 WTC.
Co's operated viz:
E 10 - took position and hooked up to hydrant, stretched line & operated to extinguish fire.
E 7 - stretched 2nd line and operated to extingush fire.
E 6 - relieved Eng 10 and washed down.
L 10 - performed necessary VES of of fire floor, performed primary search-negative, overhauled.
L 1 - searched flòor above, for extension-negative, performed secondary search of the fire floor-negative

Thomas J. Haring 1837

WHITE—FIRE RECORDS YELLOW—BATTALION PINK—REFERRAL GOLD—ADMINISTRATIVE COMPANY

BF-24

TYPE REPORT	COMMUNITY BOARD	INCIDENT LOCATION: ADMIN. COMPANY	DATE AND TIME ALARM RECEIVED: MONTH / DAY / YEAR / HOUR / MINUTE	INCIDENT DURATION: HOURS / MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOX LOCATION: BOROUGH / BOX NUMBER	REPORT NUMBER	INCIDENT NUMBER	FILING BATTALION	BADGE NUMBER	COMPANIES RESPONDING: ENG. SQ. MARINE / LADDERS RES. H.M. / OTHERS	HAZ MAT: CLASS / AMOUNT	HEAT EQUIP./FUEL	SPECIAL USE	REVISED REPORT
1	01	5 010	11 19 91 1827	03 30	81	1	2	1 8084	02763	0562	01	239	13 08 16	00 000	1	3	0

AIDED AND CASUALTIES — CIVILIANS: EVACUATED / FIRST AID / INJURED / KILLED	UNIFORMED INJURED	EXTINGUISHMENT: CODE / SPR. PERF. / STP. PERF.	IGNITION: COND. ON ARRIVAL / EQUIP. INVOLVED / HEAT FORM / MATERIAL TYPE / MATERIAL FORM / FACTOR / JUVENILE	STRUCTURE: CONSTRUCT / USE / STATUS / COMPLEX / FLOOR CODE	FIRE ORIGIN: OCCUPANCY / AREA OF ORIGIN	FIRE EXTENSION IN BLDG.: MANNER / OCCUPANCIES / NO. BLDGS. / % DAMAGED	DAMAGE: FLAME / SMOKE / WATER	DETECTOR: PRESENT / TYPE / POWER / PERFORMANCE	FOR INVESTIGATION
99 02 02 00	00	6 2 0	3 41 23 43 61 58400 1	591 1 59 93	81 52	09 04 0	1 1 1	0 0 0 0	

ADDRESS: BORO. 1 — NUMBER (INCL. HYPHEN) 1 — SUFFIX — PREFIX DIRECTION — STREET NAME: World Trade Center — TYPE — SUFFIX DIRECT.

BUILDING: STORIES 99 — AREA: WIDTH 0250, LENGTH 0250

DEFECTIVE DETECTOR: BRAND — SERIAL NO.

Port of N.Y. Authority
NAME OF OCCUPANT — ROOM/APT. ____

LEFT IN CHARGE Mr. Cancelliere Bld. Mgr.

EQUIPMENT INVOLVED IN IGNITION — ITEM — YR. — MAKE — MODEL — SERIAL NUMBER — POWER — BADGE NO.

WRI — UNIT / PTS.: 5 010 30 | 5 006 30 | 5 007 30 | 5 004 10 | 5 018 10

OPERATIONS

Cause for Alarm: ERS 1st alarm, transmitted from World Trade Center(WTC) Pol ce Desk,

While responding: Monitored 10-84 by Eng. 10, and WTC Handie Talkie,

On arrival at WTC CP.:

1. Received report of fire in electrial cldsets, on possibl 4 floors,
2. Received reports of smoke condition on 4 floors,
3. Alarms had been transmitted from floors 93 to 98.
4. WTC Police Desk heavily engaged in receipt of phone inqui ries relative to fire.

B.C. ____ ONE BN. ____ D.C. ____ DIV. ____ PAGE 1 OF 8

Lawrence M. Byrnes — TYPE FULL NAME — 1830 TIME OF ARRIVAL — TYPE FULL NAME — TIME OF ARRIVAL

WHITE—FIRE RECORD YELLOW—BATTALION PINK—REFERRAL GOLD—ADMINISTRATIVE COMPANY

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

DATE AND TIME ALARM RECEIVED | BOX LOCATION | IGNITION | STRUCTURE | AREA OF ORIGIN | FIRE EXTENSION

TYPE REPORT | MONTH | DAY | YEAR | HOUR | MINUTE | BOROUGH | BOX NUMBER | INCIDENT NUMBER | BUILDINGS INC. | COND. ON ARRIVAL | EQUIP. INVOLVED | HEAT FORM | MATERIAL TYPE | MATERIAL FORM | CONSTRUCTION | USE | STATUS | FLOOR CODE | OCCUPANCY | AREA CODE | MANNER | OCCUPANCY | REVISED REPORT

ADDRESS: NUMBER (INCL. HYPHEN) | SUFFIX | PREFIX DIRECT. | STREET NAME | TYPE | SUFFIX DIRECTION

WORLD TRADE CENTER

DAMAGE | DETECTORS

SPK. PERFORMANCE | % DAMAGED | FLAME | SMOKE | WATER | PRESENT | TYPE | POWER | PERFORMANCE

World Trade Center

OCCUPANT (Driver) | ROOM/APT. | BUILDING: STORIES | AREA: WIDTH | LENGTH

IF MOBILE PROPERTY: TYPE | YR. | MAKE | MODEL | V.I.N. | LICENSE | STATE

UNIT | PTS.

OPERATIONS

STRATEGY: To attempt to contain the fire situation to electrical closets, and to prevent panic and hysteria in occupancies with large civilian populations, and establish parameters of fire and smoke condition as quickly as possible.

Action taken:

1. On arrival at WTC command post, assumed command of fire situation,
2. Communicated with Ladder Co. 10mwho confirmed fire situationon floors 93, 94, 95 and possibly on 96,
3. Ordered the transmission of signal 10-76 based on: potential for large volume of fire, height to fire area, time involved in units response, difficulty in effecting good voice communications in building,
4. Ordered Manhattan Dispatcher to Alert 10-76 arriving units to assemble in Lobby Staging area, pending unit assignment,
5. Ordered first alarm units to conduct diligent search.
6. Ordered 1st alarm Engine companies to stretch from the "A" stairwell, but not to use water until confrimation of power off was received for electrical closets,

B.C. ONE | D.C. | DIV. 2

B.N.

Lawrence M. Byrnes | 1830

TYPE FULL NAME | TIME ARRIVED | TYPE FULL NAME | TIME ARRIVED

BF-24A (12/86)

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

DATE AND TIME ALARM RECEIVED | BOX LOCATION | IGNITION | STRUCTURE | AREA OF ORIGIN | FIRE EXTENSION

6 11 19 91 1827 1 0084 0562 00 3 41 23 43 61 1 591 1 93 81 52 09 04 0

ADDRESS: 1 1 WORLD TRADE CENTER

DAMAGE | DETECTORS

2 1 1 1 0 0 0 0 0

OCCUPANT (Driver): World Trade Center

BUILDING: STORIES 99 | AREA WIDTH 0250 LENGTH 0250

IF MOBILE PROPERTY: TYPE YR. MAKE MODEL V.I.N. LICENSE STATE

UNIT PTS.

OPERATIONS

7. Confirmed with WTC personnel, engineering staff as to time frame for electrical power removal for effected floors,
8. Denied WTC Police request to iniatete general evacuation of floors "93 to roof level", based on unknowns of: a.smoke conditions,b. stair availability, c.numbers of people that may be involved,
9. Upon notification by WYC Engineering staff of (2) elevators stuck at floor 88 and 101, Ladder Co. 6 assigned task of victim removal, as accompanied by WTC elevator mechanic,
10. As Chief Officers arrived at Command Post they were issued WTC Handie Talkies and their assignments,
11. Called for EMS response for reported burn victim (mechanic with facial burns and for possible nedds,
12. Primary search of upper floors proved to be time consuming due to large floor area and limited Handie Talkie capabalities, conducted by 1st alarm trucks,
13. Rescue Company 1 assigned to follow up secondary search of [illegible]
14. 10-76 Signal proved very effective, provided immediate response of Chief Officers to initiate various sector controls.

B.C. Lawrence M. Byrnes — 1830 — D.C. — DIV. 3

TYPE FULL NAME — TIME ARRIVED — TYPE FULL NAME — TIME ARRIVED

BF-24A (12/86)

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT	DATE AND TIME ALARM RECEIVED: MONTH	DAY	YEAR	HOUR	MINUTE	BOX LOCATION: BOROUGH	BOX NUMBER	INCIDENT NUMBER	BUILDING NO.	IGNITION: COND. ON ARRIVAL	EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	STRUCTURE: CONSTRUCTION	USE	STATUS	AREA OF ORIGIN: FLOOR CODE	OCCUPANCY	AREA CODE	FIRE EXTENSION: MANNER	OCCUPANCY	REVISED REPORT
6	11	19	91	18	27	1	2084	0562	00	3	41	23	43	61	1	591	1	93	81	52	09	04	0

1 2 8 12 17 23 32 37 43 47

ADDRESS

BORO	NUMBER (INCL. HYPHEN)	SUFFIX	PREFIX DIRECT	STREET NAME	TYPE	SUFFIX DIRECTION
1	1			WORLD TRADE CENTER		

57 68 88 93

SPK. PERFORMANCE	DAMAGE: % DAMAGED	FLAME	SMOKE	WATER	DETECTORS: PRESENT	TYPE	POWER	PERFORMANCE
2	1	1	1	0	0	0	0	0

48 53

OCCUPANT (Driver)	ROOM/APT.	BUILDING: STORIES	AREA: WIDTH	LENGTH
World Trade Center		00	0250	0250

94 113 114 123

IF MOBILE PROPERTY

TYPE YR. 128 MAKE 143 MODEL 158 V.I.N. LICENSE STATE

189 195 201 207 213 219 225 231

237 UNIT PTS. 243 249 255 261 261 273 279

OPERATIONS

Chief Officers at scene:

DAC. E. Butler,,,	CW Duty,
DC. H. MeyerS	Div. 1,
DC. V. Dunn	Div. 3,
B.C. L. Byrnes	Batt. ONE,
B.C. R. Ardisson	Batt. 2
B.C, Davison	Batt. 4,
B.C. Roche	Batt. 6,
B.C. Shelley	Batt. 31,
B.C. Wagner	Batt. 32,
B.C. Dawe	Batt. 7,
B.C. Cesark	Safety Battalion.

Personnel Present at Scene:

R. Schhirer	Fire Comm. Laison,
D.C. Basile	EMS Supervisor,
P.O. Tim #1210	WTC Police [illegible]
Lt. Valle	NYPD Pct. 1 supervisor,
Mr. T. Cancelliere	WTC Bldg Mgr.,

(2) Representatitve of Salvation Army.

B.C. ONE B.N. D.C. DIV. 4

Lawrence M. Byrnes 1830

TYPE FULL NAME TIME ARRIVED TYPE FULL NAME TIME ARRIVED

BF-24A (12/86)

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT	DATE AND TIME ALARM RECEIVED (MONTH / DAY / YEAR / HOUR / MINUTE)	BOX LOCATION (BOROUGH / BOX NUMBER)	INCIDENT NUMBER	BUILDING NO.	COND. ON ARRIVAL	EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	CONSTRUCTION	USE	STATUS	FLOOR CODE	OCCUPANCY	AREA CODE	MANNER	OCCUPANCY	REVISED REPORT
6	11 19 91 1827	1 8084	0562	00	3	41	23	43	61	1	591	1	93	81	52	09	04	0

ADDRESS: BORO 1 | NUMBER (INCL. HYPHEN) 1 | STREET NAME WORLD TRADE CENTER

DAMAGE (SPK. PERFORMANCE / % DAMAGED / FLAME / SMOKE / WATER)	DETECTORS (PRESENT / TYPE / POWER / PERFORMANCE)
2 1 1 1 0	0 0 0 0

OCCUPANT (Driver): World Trade Center

ROOM/APT.

BUILDING: STORIES 99 | AREA: WIDTH 0250 | LENGTH 0250

IF MOBILE PROPERTY: TYPE YR. MAKE MODEL V.I.N. LICENSE STATE

UNIT PTS.

OPERATIONS

Time Sequences:

Signal	Time	By Order of:
Box 8084	1827	Dispatcher,
10-84	1830	Batt. 1,
10-76	1833	Batt. 1,
10-44	1842	Batt. 1,
10-84	1845	Div. 1,
10-75	1848	Div. 1,
22-8084	1858	Batt. 1,
10-84	1904	Car. 6B, Chiif Butler,
UC/8084	2008	Car. 6B, via FCU.

Injuries:
[redacted], superficial burns,
[redacted], chest pains.

Unit Citations to be submitted for Ladder Co. 10 and Engine Co. 10.

B.C. ONE B.N. D.C. DIV. PAGE 5 O

Lawdrnce M. Byrnes — TYPE FULL NAME | 1830 — TIME ARRIVED | TYPE FULL NAME | TIME ARRIVED

BF-24A (12/86)

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR

TYPE REPORT	DATE AND TIME ALARM RECEIVED	BOX LOCATION	INCIDENT NUMBER	IGNITION	STRUCTURE
6	11 19 91 1827	1 8084 0562	00	3 41 23 43 61 1 591	1 53 81

ADDRESS: 1 1 WORLD TRADE C

DAMAGE	DETECTORS
21 1 1 0	0 0 0 0

World Trade Center
OCCUPANT (Driver) — ROOM/APT.

IF MOBILE PROPERTY: TYPE YR. MAKE MODEL V.I.N.

UNIT PTS.

OPERATIONS

Unit Operations:

Eng. 10: Responded to the 93rd floor, stretched of 2½" hose, hooked up to standpipe in assisted Lad. 10 in search and size up 94th floors, used a fog nozzle to exti in electrical closets, assisted in eva civilians, connected to buildings stan

Eng. 6: Responded to the 95th floor, stretched 2½" hose, stood fast, assisted in sea of fire area, connected to and suppli siamese,

Eng. 7: Responded to the 93rd floor, stretche 2½" hose from the "B" stairway, open ECC. connected to standpipe and supp

Eng. 4: Responded to the 97th floor, hooked 2½" hose, stood fast, assisted in

Following Engi Comanies reported to Lobby stood fast: Eng. 18, 205, 24, 10

** E-10 also used dry chem on 93

B.C. ____ ONE F.N. ____ D.C. ____

Lawrence M. Byrnes
TYPE FULL NAME

1830
TIME ARRIVED

BF-24A (12/86)

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT | DATE AND TIME ALARM RECEIVED (MONTH, DAY, YEAR, HOUR, MINUTE) | BOX LOCATION (BOROUGH, BOX NUMBER) | INCIDENT NUMBER | BUILDING HOLD | CONDITIONS ON ARRIVAL | EQUIP. INVOLVED | IGNITION (HEAT FORM, MATERIAL TYPE, MATERIAL FORM) | STRUCTURE (CONSTRUCTION, USE, STATUS) | AREA OF ORIGIN (FLOOR CODE, OCCUPANCY, AREA CODE) | FIRE EXTENSION (MANNER, OCCUPANCY) | REVISED REPORT

6 11 19 91 1827 1 8084 0562 00 3 41 23 43 61 1 591 1 93 81 52 09 08 0

ADDRESS: NUMBER (INCL. HYPHEN) 1 — SUFFIX — PREFIX DIRECT — STREET NAME: WORLD TRADE CENTER — TYPE — SUFFIX DIRECT

DAMAGE / DETECTORS: SPK. PERFORMANCE, % DAMAGED, FLAME, SMOKE, WATER, PRESENT, TYPE, POWER, PERFORMANCE

2 1 1 1 0 0 0 0 0

World Trade Center

OCCUPANT (Driver) — ROOM/APT.

BUILDING: STORIES 99 — AREA: WIDTH 0250, LENGTH 0250

IF MOBILE PROPERTY: TYPE, YR., MAKE, MODEL, V.I.N., LICENSE, STATE

UNIT PTS.

OPERATIONS

UNIT OPERATIONS cont:

Lad. 10; Conducted primary search on floors 93, 94, 95, and [illegible], and [illegible]. Unit split into 3 teams, locateddfire condition in electrical distribution closets, called for placement of hand lines, conducted opening up operations on 94th floor to check for fire extension, evacuated unknown member of civilians from floors 94 through 95 during initial operations,

Lad. 1: Conducted primary search on 93rd floor, conducted secondary search on floors 94 to 101, proved to be negative, conducted survey on floors 81 through 92 for water damage to electrical closets due to hhnd-line use, negative.

Lad. 15: Conducted primary and secondary seeached of floors 93 to 97, negative results,

Lad. 8: Conducted examination of floors 92 to 101 for possible fire extension from electrical closets, negative, conducted secondary search on floors 94 and 95,

Lad.110: Performed secondary search of floors 96 through 110, negative results,

B.C. ______ ONE B.N. — D.C. ______ DIV. 7

Lawrence H. Byrnes — 1830
TYPE FULL NAME — TIME ARRIVED — TYPE FULL NAME — TIME ARRIVED

BF-24A (12/85)

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT	MONTH	DAY	YEAR	HOUR	MINUTE	BOROUGH	BOX NUMBER	INCIDENT NUMBER	BUILDING NO.	COND. ON ARRIVAL	EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	CONSTRUCTION	USE	STATUS	FLOOR CODE	OCCUPANCY	AREA CODE	MANNER	OCCUPANCY	REVISED REPORT
6	11	19	91	18	27	1	2084	0552	00	3	41	23	43	61	1	591	1	93	81	52	09	04	0

ADDRESS: BORO 1 | NUMBER (INCL. HYPHEN) 1 | STREET NAME: WORLD TRADE CENTER

SPK. PERFORMANCE	% DAMAGED	FLAME	SMOKE	WATER	PRESENT	TYPE	POWER	PERFORMANCE
2	1	1	1	0	0	0	0	0

OCCUPANT (Driver): World Trade Center | ROOM/APT.: | BUILDING: STORIES 99 | AREA WIDTH 0250 | LENGTH 0250

IF MOBILE PROPERTY: TYPE | YR. | MAKE | MODEL | V.I.N. | LICENSE | STATE

UNIT | FTS.

OPERATIONS

Unit Operations Cont:

Lad. 6: Conducted senondary search 66 floors 105 to 110, negativeresults, removed (2) civilians from stalled elevators on the 88th and 101st floors, no injuries to civilians,or damage to building,

Lad. 20: Conducted secondary search of floors 98,99, and 100 negative resilts,

Lad.131: Conducted search of floors 105 to 110, with negative eesults.

Field Communications Unit: Set up Communications/Command Post in Basement, WTC Command Post,

Battalion 2 supervised Lobby Staging Area,

Battalions 4, 6, and 7 operated on various floors with units so assigned,

Division 3 operated as Operations OIC on various floors.

Division 1 operated at the Command Post as CIC

B.C. [signature] ONE B.N. | D.C. [signature] | DIV.

Lawrence M. Byrnes | 1830 | Harold Meyers | 8

TYPE FULL NAME | TIME ARRIVED | TYPE FULL NAME | TIME ARRIVED

BF-24

INCIDENT LOCATION | DATE AND TIME ALARM RECEIVED | INCIDENT DURATION | BOX LOCATION | REPORT | COMPANIES RESPONDING | HAZ MAT | SPECIAL USE

1 01 7 010 07-23-92 2202 02:30 10 1 1 1 0283 0227 0455 01 233 02 17 00 000 3 0

AIDED AND CASUALTIES — CIVILIANS | EXTINGUISHMENT | IGNITION | STRUCTURE | FIRE ORIGIN | FIRE EXTENSION IN BLDG. | DAMAGE | DETECTOR

00 03003 00 02 6 0 0 3 42 24 43 01 55 0 1 642 1 70 99 32 63 00 99 0 0 0 0 0 0 0 0 0

ADDRESS: NUMBER (INCL HYPHEN) | STREET NAME: WORLD TRADE CENTER | BUILDING: STORIES 99, AREA WIDTH/LENGTH 0200 0400 | DEFECTIVE DETECTOR: BRAND, SERIAL NO.

NAME OF OCCUPANT: WORLD TRADE CENTER

ROOM/APT. 5th sub base.

LEFT IN CHARGE Mr. Tom Cancelliere, Mgr.

EQUIPMENT INVOLVED IN IGNITION: ITEM, YR., MAKE, MODEL, SERIAL NUMBER, POWER, BADGE NO.

UNIT PTS. 5 010 20 5 007 20 5 006 20 5 055 20 5 004 12

OPERATIONS

Response to alarm of Fire, reported as a transformer fire, on the 5th sub basement level. Upon confirmation of fire requested box transmission.

Strategy here is to identify fire area, conduct required primary and secondary search as for effect ventilation of fire area, limit smoke condition build up in building, limit amount and duration of service interuption in the Twin Towers complex.

B.C. [signature] BN. 1 D.C. DIV.

Lawrence M. Byrnes TYPE FULL NAME | 2205 TIME OF ARRIVAL | TYPE FULL NAME | TIME OF ARRIVAL

PAGE 1 OF 6

WHITE – FIRE RECORD YELLOW – BATTALION PINK – REFERRAL GOLD – ADMINISTRATIVE COMPANY

BF-24A (12/86)

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT	DATE AND TIME ALARM RECEIVED (MONTH / DAY / YEAR / HOUR / MINUTE)	BOX LOCATION (BOROUGH / BOX NUMBER)	INCIDENT NUMBER	BUILDING NO. / CODE OR AIRPORT / EQUIP. INVOLVED	IGNITION (HEAT FORM / MATERIAL TYPE / MATERIAL FORM)	STRUCTURE (CONSTRUCTION / USE / STATUS)	AREA OF ORIGIN (FLOOR CODE / OCCUPANCY / AREA CODE)	FIRE EXTENSION (MANNER / OCCUPANCY)	REVISED REPORT
6	07 23 92 2202	1 8093	0455	01 3 42	24 43 61	1 642 1	99 32 65	00 99	0

ADDRESS: NUMBER (INCL. HYPHEN) / SUFFIX / PREFIX / DIRECT. / STREET NAME / TYPE / SUFFIX DIRECTION

STREET NAME: WORLD TRADEECENTER

SPR. PERFORMANCE	DAMAGE (% DAMAGED / FLAME / SMOKE / WATER)	DETECTORS (PRESENT / TYPE / POWER / PERFORMANCE)	OCCUPANT (Driver)	ROOM/APT.	BUILDING: STORIES	AREA: WIDTH	LENGTH
0	0 0 0 0	0 0 0 0	Port Authority NY/NJ		99	0200	0400

IF MOBILE PROPERTY: TYPE / YR. / MAKE / MODEL / V.I.N. / LICENSE / STATE

UNIT PTS.

7 010 20	7 001 20	7 008 20	7 006 20	1 001 20	10082290	5 024 12	5 003 12
5 005 12	7 015 12	7 020 12					

OPERATIONS

Due to high electrical voltage (13000)volts confirmed, no water was used in initially, pending confirmation of power off at the electdrical distribution panel.
Due to large floor area of the 5th floor-sub-basement, responding units were split into teams viz: Ladder Co. 10 and Engine 10 using the K13 stair to approach the fire adea, Ladder 1 to use a second stairway-K12, to access the 5th sub. basement level. These units were takded with pinpointing the fire area, an area of 200' x 400'.

A member of Ladder Co. 1 having found the fire situation in a very large power disfribution panel, attempted to relay information to his officer. Prior to his transmission firefighter was struck by a shock blast generated bye the involved panel. Ladder 1 firefighter knocked unconsciense required a conserted effort to remove to a separate safe area.

Unit Operations.
Engine 10 - Operated on 5th sub level, stretched a 2½" hand line frmmm the standpipe, opersted when power off confirmation received. Company [illegible]

Engine 7 - Operated on fire floor with line off standpipe, opersted under B.C. Demarest, Batt. 4, extinguished fire, used dry chemical extinguishers on fire,

B.C. [signature] 1 B.N. D.C. DIV.

Lawrence M.Byrnes / 2205

TYPE FULL NAME TIME ARRIVED TYPE FULL NAME TIME ARRIVED

CRC 128

BF-24A (12/86)

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT	DATE AND TIME ALARM RECEIVED	BOX LOCATION	INCIDENT NUMBER	BUILDING NO.	COND. ON ARRIVAL	EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	CONSTRUCTION	USE	STATUS	FLOOR CODE	OCCUPANCY	AREA CODE	MANNER	OCCUPANCY	REVISED REPORT
6	07 23 92 2202	1 8093	0455	01	3	42	24	43	61	1	642	1	99	32	63	00	99	0

ADDRESS: WORLD TRADE CENTER

DAMAGE / DETECTORS: 0 0 0 0 0 0 0 0 0

OCCUPANT (Driver): Poet Authority NY/NJ

ROOM/APT.: 5th sub. base.

BUILDING: STORIES 99; AREA WIDTH 0200, LENGTH 0400

IF MOBILE PROPERTY: TYPE YR. MAKE MODEL V.I.N. LICENSE STATE

UNIT PTS.

OPERATIONS

Operation cont:
Engine 6 - Assisted Eng 7 in stretch of and operation of a 2½" line into fire area, performed search of area, overhauled as neceesary,

Engine 55 - Operated with and relieved Eng. 7 on hand line on fire floor, took up hose lines,

Engine 4 - Under supervision of BC. Turnee,B2, transported injured member of Lad. 1 to ambulance on the B1 level of the fire building,relieved Eng. 10 on a hand line, oberhauled, took up hand line,

Engine 24 - Transported Air Cylindeer fire area under supervision of BC. Jackson,

Engine 3 - Ordered to and did provide air cylingers to operating units of the B5 level.
Engin
Engine 5 - Operated as Command Post Company

Stood Fast - Eng9Sat1, Engines 15, 28. 33, 34, 207/Maxi, 284/Sat.3,

B.C. 1 Lawrence M. Byrnes (signature) 1 B.N. D.C. DIV. 3

Lawrence M. Byrnes 2205
TYPE FULL NAME TIME ARRIVED TYPE FULL NAME TIME ARRIVED

CRC 226

BF-24A (12/86)

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT	MONTH	DAY	YEAR	HOUR	MINUTE	BOROUGH	BOX NUMBER	INCIDENT NUMBER	BUILDING NO.	COND. ON ARRIVAL	EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	CONSTRUCTION	USE	STATUS	FLOOR CODE	OCCUPANCY	AREA CODE	MANNER	OCCUPANCY	REVISED REPORT
6	07	23	92	22	02	1	8093	0455	01	3	42	24	43	61	1	892	1	99	32	63	00	99	0

ADDRESS: BORO | NUMBER (INCL. HYPHEN) | SUFFIX | PREFIX DIRECT | STREET NAME: WORLD TRADE CENTER | TYPE | SUFFIX DIRECTION

SPK. PERFORMANCE	% DAMAGED	FLAME	SMOKE	WATER	PRESENT	TYPE	POWER	PERFORMANCE
0	0	0	0	0	0	0	0	0

OCCUPANT (Driver): Port Authority NY/NJ

ROOM/APT.: 5th sub base.

BUILDING: STORIES: 99 | AREA WIDTH: 0200 | LENGTH: 0400

IF MOBILE PROPERTY: TYPE | YR. | MAKE | MODEL | V.I.N. | LICENSE | STATE

UNIT | PTS.

OPERATIONS

Operations cont:

Ladder 1- Operated at the B5 level, conducted seadrh to pinpoint the fire area, concucted a primary search for possible employees trapped. FF. Amodio injured in explosion of 13000 volt distribution panel,

Ladder 10- Performed a search of the B5 level lto identify the fire area, and searched for possible trapped employees, gathered and used dry chemical extinguishers on the fire prior to power removal, overhauled as required,

Ladder 8- Performed a secondary search of the fire area, used dry chemical extinguishers, assisted in overhauling,

Ladder 6- Placed and used portable exhaust fans in stairwells to effect ventilation , took up,

Ladder 15- Supplied spare SCBA cylinders to staging area,

Ladder 20- Supplied spare SCBA cylinders to staging area,

B.C. Lawrence M. Byrnes   1 B.N.   D.C. ______ DIV. ______ PAGE 4

Lawrence M. Byrnes — TYPE FULL NAME   2205 — TIME ARRIVED   ______ TYPE FULL NAME   ______ TIME ARRIVED

CRC 228

BF-24A (12/86)

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT	DATE AND TIME ALARM RECEIVED: MONTH / DAY / YEAR / HOUR / MINUTE	BOX LOCATION: BOROUGH / BOX NUMBER	INCIDENT NUMBER	BUILDING NO.	COND. ON ARRIVAL	EQUIP. INVOLVED	IGNITION: HEAT FORM / MATERIAL TYPE / MATERIAL FORM	STRUCTURE: CONSTRUCTION / USE / STATUS	AREA OF ORIGIN: FLOOR CODE / OCCUPANCY / AREA CODE	FIRE EXTENSION: MANNER / OCCUPANCY	REVISED REPORT
6	07 23 92 2202	1 8093	0455	01	3	42	24 43 61	1 642 1	99 63 63	00 99	0

ADDRESS: BORO 1 | NUMBER (INCL. HYPHEN) | SUFFIX | PREFIX DIRECT | STREET NAME: WORLD TRADE EENTER | TYPE | SUFFIX DIRECTION

DAMAGE: SPK. PERFORMANCE	% DAMAGED	FLAME	SMOKE	WATER	DETECTORS: PRESENT	TYPE	POWER	PERFORMANCE
0	0	0	0	0	0	0	0	0

OCCUPANT (Driver): PodrtAuthority NY/NJ | ROOM/APT.: 5th sub. base. | BUILDING: STORIES 99 | AREA: WIDTH 0200 | LENGTH 0400

IF MOBILE PROPERTY: TYPE | YR. | MAKE | MODEL | V.I.N. | LICENSE | STATE

UNIT | PTS.

OPERATIONS

Operation cont:

Reacue 1- Compapy split to perform several dperations-
1. Assisted in adhinistering first aid to Injured firefighter,
2. Conductéd secondary search of fire area, hegative,
3. Used Thermal Camera to check for possible fire extension,

Rescue 2- Assisted (2) civilian electricians (with SCBA's) to confirm power off in electricalpanel, relayed confirmation of power off to Command Post. Essisted in hand line iperation, assisted in VES of fire floor.

Chief Officers present:

DAC R. Palmer,	CW Duty		
DC. R. Nanson	Div. 1,		
B.C. L. Byrnes	Batt. 1,	B.C. Costa	Batt. 7,
B.C. W.Demarest	Batt. 4,	B.C. Miccio	Batt. 6,
B.C. R.Turner,	Batt. 2,	B.C. Nardone	Batt. 9.
B.C. Jackson	Batt. 35,	B.C. Ross	Batt. 31.

B.C. Lawrence M. Byrnes 1 B.N. D.C. DIV.

Lawrence M. Byrnes (signature) TYPE FULL NAME 2205 TIME ARRIVED TYPE FULL NAME TIME ARRIVED

PAGE 5

CRC 228

BF-24A (12/85)

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT	DATE AND TIME ALARM RECEIVED	BOX LOCATION	INCIDENT NUMBER	BUILDING NO.	IGNITION	STRUCTURE	AREA OF ORIGIN	FIRE EXTENSION	REVISED REPORT
6	07 23 92 22 02	1 3058	0455	01	3 42 2 43 61	1 54 2 1	99 63 63	00 99	0

ADDRESS: BORO 1 — STREET NAME: WORLD TRADE CENTER

SPK. PERFORMANCE	% DAMAGED	FLAME	SMOKE	WATER	DETECTORS PRESENT	TYPE	POWER	PERFORMANCE
0	0	0	0	0	0	0	0	0

OCCUPANT (Driver): Port AuthorityNY/NJ

ROOM/APT.

BUILDING: STORIES 99 — AREA: WIDTH 0200, LENGTH 0400

IF MOBILE PROPERTY: TYPE YR. MAKE MODEL V.I.N. LICENSE STATE

UNIT PTS.

OPERATIONS

Operstions cont:

Greater alarms transmitted dur to magnitude of fire area involved, potential for smoke and heat problems, numbers of employees in building need tosupply and relieve operating personne.

Injuries: FF. Amodio, Lad. 1, concussion, removed to Beckman Hosp.,
FF. Cancel, Lad 10, granted ML.,
FF. Hanson, Lad.10, granted ML.,
FF. Selletti, Lad. 10, Granted ML.

Note: Staff and employees of the New York Port Authority provided excellent guidance and assistance. Help provided included mechanical, ventilation, electrical distribution disciplines which made for a much eesier fire ground operation.

B.C. Lawrenee M. Byrnes B.N. 1 D.C. DIV.

TYPE FULL NAME TIME ARRIVED 2005 TYPE FULL NAME TIME ARRIVED

BF-24 (12. 530-R02B-N

REPORT -- STRUCTURAL FIRE

TYPE REPORT	INCIDENT LOCATION (COMMUNITY BOARD / ADMIN COMPANY)	DATE AND TIME ALARM RECEIVED (MONTH / DAY / YEAR / HOUR / MINUTE)	INCIDENT DURATION (HOURS / MINUTES)	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOX LOCATION (BOROUGH / BOX NUMBER)	REPORT (REPORT NUMBER / INCIDENT NUMBER / FILING BATTALION / BADGE NUMBER)	COMPANIES RESPONDING (ENG SQ MARINE / LADDERS RES. H.M. / OTHERS / CLASS)	HAZ MAT (AMOUNT / HEAT EQUIP.-FUEL)	SPECIAL USE 1 / 2 / 3	REVISED REPORT
1	01 5 010	11 10 99 23 01	01 30	1	0 1	7	1 8084	0103 0648 01 185	03 02 01 63	04F 0	0 0 0	0

AIDED AND CASUALTIES – CIVILIANS (EVACUATED / FIRST AID / INJURED / KILLED / UNIFORMED INJURED)	EXTINGUISHMENT (CODE / SPK. PERF. / STP. PERF.)	IGNITION (COND ON ARRIVAL / EQUIP INVOLVED / HEAT FORM / MATERIAL TYPE / MATERIAL FORM / FACTOR / JUVENILE)	STRUCTURE (CONSTRUCT / USE / STATUS / COMPLEX)	FIRE ORIGIN (FLOOR CODE / OCCUPANCY / AREA OF ORIGIN)	FIRE EXTENSION (IN BLDG.: MANNER / OCCUPANCIES; NO BLDGS)	DAMAGE (% DAMAGED / FLAME / SMOKE / WATER)	DETECTORS (PRESENT / TYPE / POWER / PERFORMANCE / FOR INVESTIGATION)
00 00 01 00 00	2 1 1	2 61 29 43 61 • • 0	1 59 1 1 59	94 81 27	00 01 1	1 2 6 3	1 1 1 1 1

BF-24

ADDRESS

BORO	NUMBER (INCL HYPHEN)	SUFFIX	PREFIX DIRECTION	STREET NAME	TYPE	SUFFIX DIRECT
1	1			WORLD TRADE CENTER		

BUILDING

STORIES	AREA WIDTH	AREA LENGTH
99	200	200

DEFECTIVE DETECTOR

BRAND	SERIAL NO.

NAME OF OCCUPANT: PORT AUTHORITY

ROOM/APT ______

LEFT IN CHARGE MIKE HURLY

BADGE NO. FSD

EQUIPMENT INVOLVED IN IGNITION

ITEM	YR.	MAKE	MODEL	SERIAL NUMBER	POWER

WRI

UNIT	PTS
5 010	10
7 010	10
5 006	02
7 001	10
5 007	05

OPERATIONS

Upon arrival at box 8084 found fire on the 104th floor which was knocked down by three sprinkler heads.
Initially a 10-76 was transmitted until the
source of the smoke condition was found. B-1 then operated with 3 Engines and 2 Trucks and all other units returned to Quarters. Units operated as follows:
L-10 Gained access to building, found smoke condition on 103rd and 104th floors, found fire in computer room on the 104th floor, performed search and overhauling.
E-10 Took hydrant, stretched line to the siamese, stretched line from standpipe, completed extinguishment.
L-1 Assisted L-10, performed search of areas above and adjacent to the fire.
E-6 Took hydrant, assisted E-10 in stretch off the standpipe.

B.C. [signature] — Bc John Akerman (TYPE FULL NAME)

B.N. 01 — TIME OF ARRIVAL 23:07

D.C. ______ (TYPE FULL NAME) — DIV. ______ TIME OF ARRIVAL ______

PAGE 1 OF 2

ADMINISTRATIVE COMPANY

F-24A (12/86) 15M2-861531-R02B-N

REPORT -- ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT | DATE AND TIME ALARM RECEIVED: MONTH, DAY, YEAR, HOUR, MINUTE | BOX LOCATION: BOROUGH, BOX NUMBER | INCIDENT NUMBER | BUILDING NO. | IGNITION: COND. ON ARRIVAL, EQUIP. INVOLVED, HEAT FORM, MATERIAL TYPE, MATERIAL FORM | STRUCTURE: CONSTRUCTION, USE, STATUS | AREA OF ORIGIN: FLOOR CODE, OCCUPANCY, AREA CODE | FIRE EXTENSION: MANNER, OCCUPANCY | REVISED REPORT

6 | 1 1 | 1 0 | 9 9 | 2 3 | 0 1 | 1 | 8 0 8 4 | 0 6 4 8 | 0 0

1 2 8 12 17 23 32 37 43 47

ADDRESS: BORO | NUMBER (INCL HYPHEN) | SUFFIX | PREFIX DIRECT | STREET NAME | TYPE | SUFFIX DIRECTION

1 | 1 | | | | WORLD TRADE CENTER

57 68 88 93

SPK. PERFORMANCE | DAMAGE: % DAMAGED, FLAME, SMOKE, WATER | DETECTORS: PRESENT, TYPE, POWER, PERFORMANCE

48 53

94 OCCUPANT (Driver) 113 ROOM/APT.

BUILDING: STORIES | AREA: WIDTH, LENGTH

114 123

IF MOBILE PROPERTY: TYPE YR. 128 MAKE 143 MODEL 158 V.I.N. LICENSE STATE DM

189 195 201 207 213 219 225 231

237 UNIT PTS. 243 249 255 261 261 273 279

OPERATIONS

E-7 Stretched precautionary line from standpipe.
F.S.D. for World Trade Center - Mr. Mike Hurly.
O.E.M. Lt. Wilson.
[redacted]
F.M. Kregler #361 responded on BFI's own knowledge of prior suspicious activity. Job #11201.

B.C. [signature] 01 B.N.

Bc John Akerman — TYPE FULL NAME

23:07 — TIME ARRIVED

D.C. ____ DIV.

TYPE FULL NAME — TIME ARRIVED

PAGE 2 OF

Attachment G-A.5

Significant fire incidents occurring in WTC 2 (3)

Significant Fire	Incident Date	Fire Location	# Sprinklers Activated	# Standpipes Activated	Cause of Fire	Material Ignited
1	5/19/75	Floor 32	-	3	Incendiary	Trash/waste
2	4/12/77	Duct work over grill in restaurant on floor 107	2		None listed	Duct work
3	3/22/93	Fan motor room on floor 108	2		Mechanical failure	Not classified

DATE AND TIME ALARM RECEIVED | DURATION OF (?) | LOCATION ALARM / INCIDENT | REPORT

FOR FIRE RECORDS USE ONLY | TYPE REPORT | MONTH | DAY | YEAR | CORD. NO. | HOUR | MINUTE | HOURS | MINUTES | HOW REPORTED | INITIAL ALARM | HIGHEST ALARM | BOROUGH

1	05	19	75	1	21	38	03	10	2	2	1	0070	7	010	0033	011	101	195

AIDED AND CASUALTIES: CIVILIANS | RESP. | EXTINGUISHMENT | INVESTIGATION | STRUCTURES & AREA FIRE ORIGIN

02	05	05	00	21	09	4	7	00	03	2	61	1	15	1	1	3	32	31	50	00

ADDRESS 2 World Trade Center Man. — N.Y. S. Banking 32n
NUMBER STREET BOROUGH — NAME OF OCCUPANT ... ROOM/APT

BUILDING 110 STORIES 200' x 200' AREA — Port Authority Patrolman Keese #1191 (?) IN CHARGE

(Second Card)

5 028 0	30	5 007 0	30	5 027 0	30	5 010 0	25	5 009 0	12	5 055 0	15
5 024 0	12	5 017 0	12	6 001 0	05	7 010 0	30	7 001 0	20	7 008 0	20

TYPE NO. SECT. PTS.

OPERATIONS

Upon arrival was informed by Port Authority Police of smoke condition on 32 floor and occupants reported trapped on 31 and 32 floors. Established command post and ordered Engine 6 (28) and Engine 7 to proceed to fire area and operate in separate stairways. Ladders 10 and 15 to transmit immediate report of fire conditions and to make search for occupants and perform necessary duties. Battalion 4 to supervise operations in fire area. Division 1 assumed command upon arrival. Ordered one additional engine and ladde r units. Then transmitted 2nd alarm upon receipt of report from fire area of "heavy fire and smoke condition". Battalion 1 with Eng. 10 and Ladder 8 ordered to operate in third stairway. Additional units put to work as required and to relieve operating units. Fire area involved core area throughout and was confined to same.

Four minor fires in previous two hours were suspected arson and were each reported via dispatcher to fire marshal and same ordered to respond.

E.6(28) Stretched line to 32fl via stairway B. Operated to extinguish major body of fire. Then relieved by E.7.

E.7— Assisted E.6 in stretching and advancing line. Then stretched 2n line to 32nd fl via stairway B and operated to extinguish fire.

(OVER)

B.C. Charles J. Votruba (signature) | D.C. | PAGE 1

Charles J. Votruba | 2140 | Roger Rodriguez | DIV. | of

TYPE FULL NAME | TIME OF ARRIVAL | TYPE FULL NAME | TIME OF ARRIVAL

ADMINISTRATIVE COMPANY

E.27- Stretched line to 32nd fl. Operated to extinguish remaining fire. assisted in search. Overhauled fire area.

L.10- Forced door to 32nd fl- stairway "C". Operated house hose line to assist in making search. Then operated on 32nd fl. continuing search. Made examination. Then assisted in overhauling. Then relieved by L.1

L.1- Made search of floors 33, 34, and 35. Then made examination at 33rd fl for extension of fire.

E.10 (Spec. called)- Ordered to stand fast at 31st fl. Then relieved on hose lines 32nd fl. to assist in overhauling. Then took up lines of E.6 & 7.

L.8- (Spec. called)- Assisted in making secondary search of office occupancies 32nd fl. Then relieved L.10 overhauling fire.

Batt.4- Supervised units initially operating on the 32nd fl.

Batt.1- Ordered to assist in supervision in fire area. Then relieved Batt and supervised secondary search, examination and overhauling.

E.9- Ordered to stand fast. Then made search of 44th fl.

E.55- Relieved operating units on the 32nd fl. where necessary.

E.24- Made examination 20th, 40th and 60th fls. (2 WTC)

E.17- Made search examination 20, 40th, 60th fls (1 WTC)

L.15- Made search of permineter office occupancies 32nd fl. Then made search and examination of 44th fl. Then made final examination of 31st fl core area.

Batt. 32 Supervised examination in #1 WTC. Then supervised examination in #2 WTC 44th and 60th fls.

Mar. 1- Stretched line to gate and ordered to stand fast.

Res. 1- ~~Administered~~ Administered oxygen to Fr. O'Neill E.7. Then made examination for fire extension around perimeter of fire area and 31st fl.

Superpumper System- Superpumper and Sat. 2 ordered to return to qtrs. Sat 1 ordered to stand fast at hook up site then to qtrs/

FCU. Established Field Hdtgs. monitored HT circuits. Transmitted Progress reports. Maintained

MSU- Serviced Units, Exchanged 30 cylinders.

Time of Special Calls: 2150 hrs.

2nd alarm: 2154 hrs.

under control: 2257 hrs.

Chief Officers Operating: BC Votruba, Batt. 1 BC Bagley. Bn

REPORT – STRUCTURAL FIRE

FOR FIRE RECORDS USE ONLY			TYPE REPORT	MONTH	DAY	YEAR	CO. NO.	HOUR	MINUTE	HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	ADMIN. COMPANY	BLOCK NUMBER	REPORT NUMBER	FIRING BATTALION	BADGE NUMBER
			1	04	12	77	1	13	15	00	30	1	1	0	1	0070	5 004	0113	0058	01	999

DATE AND TIME ALARM RECEIVED · DURATION OF INCIDENT · LOCATION: ALARM / INCIDENT · REPORT

AIDED AND CASUALTIES – CIVILIANS · RESP · EXTINGUISHMENT · INVESTIGATION · STRUCTURE · AREA FIRE ORIGIN

EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	ENG. SQ. MARINE	LADDERS	CODE	NO. SPRINK. HDS. OPEN	STANDPIPE LINES USED	SUBSEQ. CONTROL	CAUSE	FIRE MARSHAL	CONSTRUCT.	USE	STATUS	EXTENT TO BLDG.	DAMAGE TO CONTENTS	FLOOR CODE	ROOM OR AREA CODE	OCCUPANCY	MANNER OF EXTENSION
00	00	00	00	00	302		0	02	00	0			1	15	1	0	1107	33	40	00	

ADDRESS 2 World Trade Center (NUMBER, STREET) Man (BOROUGH)

Windows on the World Rest. (NAME OF OCCUPANT) ROOM/APT. NO.

BUILDING (Second Card) 110 (STORIES) 400 x 400 (AREA)

Mr. Sarnelli FSD (LEFT IN CHARGE)

COMPANIES

TYPE	NO.	SECT.	PTS.					
7	001	0	05	5	006	0	05	

OPERATIONS

On arrival found fire to be extinguished pryor to arrival ; Fire was located in duct work over grills in in reatarant on 107 th floor, Bn. 1 notified dispatcher to notify board of Health of possible food contamination from heat smoke and gases in restaurant.

E. 6 Took rolled ups to 107 floor, stretched line and stood fast.

L. 1 Made necessary search and investigation.

F.P. 2 on scene replaced two sprinkler heads.

B.C. William M. Feehan 1 (BN.) D.C. ____

ABC William M. Feehan (TYPE FULL NAME) 1318 (TIME OF ARRIVAL) ____ (TYPE FULL NAME)

BF-24 (12/86) 61530-H02B-N

REPORT – STRUCTURAL FIRE

TYPE REPORT	COMMUNITY BOARD	ADMIN COMPANY	MONTH	DAY	YEAR	HOUR	MINUTE	HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	REPORT NUMBER	INCIDENT NUMBER	FILING BATTALION	BADGE NUMBER	ENG SQ MARINE	LADDERS RES. H.M.	OTHERS	CLASS	AMOUNT	HEAT EQUIP.-FUEL	SPECIAL USE 1	2	3	REVISED REPORT
1	01	5 010	03	22	93	08	39	00	40	10	1	0	1	8087	0286	0058	01	620	03	02	01	00	000	0	00	00	00	0

Incident Location / Date and Time Alarm Received / Incident Duration / Box Location / Report / Companies Responding / Haz Mat / Special Use

1 2 8 18 22 26 31 44 50 55 62

AIDED AND CASUALTIES – CIVILIANS / EXTINGUISHMENT / IGNITION / STRUCTURE / FIRE ORIGIN / FIRE EXTENSION IN BLDG. / DAMAGE / DETECTORS

EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	CODE	SPK. PERF.	STP. PERF.	COND ON ARRIVAL	EQUIP INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	FACTOR	JUVENILE	CONSTRUCT.	USE	STATUS	COMPLEX	FLOOR CODE	OCCUPANCY	AREA OF ORIGIN	MANNER	OCCUPANCIES	NO. BLDGS.	% DAMAGED	FLAME	SMOKE	WATER	PRESENT	TYPE	POWER	PERFORMANCE	FOR INVESTIGATION
00	00	00	00	00	0	1	2	2	19	27	99	99	59	0	1	591	1	59	99	50	61	00	01	0	1	9	9	9	1	1	2	1	0

63 73 76 88 95 101 106 110 114

BF-24

ADDRESS

BORO	NUMBER (INCL HYPHEN)	SUFFIX	PREFIX DIRECTION	STREET NAME	TYPE	SUFFIX DIRECT
1	2			WTC		

115 126 146

BUILDING

STORIES	AREA WIDTH	AREA LENGTH
99	0200	0200

152 161

DEFECTIVE DETECTOR

BRAND	SERIAL NO.

162 177 19

192 NAME OF OCCUPANT 211

ROOM/APT. ACS108

LEFT IN CHARGE FEELY

BADGE NO.

EQUIPMENT INVOLVED IN IGNITION

YR.	MAKE	MODEL	SERIAL NUMBER	POWER

212 229 244 264

WRI

UNIT	PTS.	ITEM									
5 010	05	7 010	05	5 006	05	7 001	05	5 007	05		

266 272 278 284 290 295

OPERATIONS

Upon arrival at Box 8087 found smoke condition on the 108th floor caused by a overheated bearing in a fan motor room ACS-108-4.Smoke Det.caused 2 sprinkler heads to go off.Units operated VIZ:
E10-Hooked up to standpipe on the 108th fl.stoodfast,then up.
L10-Made necessary search and examination,found cause of smoke condition,checked for extension found neg.Then up.
E6-Assisted L10 in search,then up.

1

B.C. Gordan Symon
Gordan Symon
TYPE FULL NAME

BN. 0844
TIME OF ARRIVAL

D.C.
TYPE FULL NAME

DIV.
TIME OF ARRIVAL

PAGE 1 OF 2

WHITE – FIRE RECORD YELLOW – BATTALION PINK – REFERRAL GOLD – ADMINISTRATIVE COMPANY

F-24A (12/86) 15/H2-861531-R02B-N

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT	DATE AND TIME ALARM RECEIVED (MONTH / DAY / YEAR / HOUR / MINUTE)	BOX LOCATION (BOROUGH / BOX NUMBER)	INCIDENT NUMBER	BUILDING NO.
6	05 22 93 0839	1 8087	0058	00

ADDRESS: BORO 1 | NUMBER (INCL. HYPHEN) 2 | STREET NAME WTC

OPERATIONS

E7-Assisted E10 in stretching line,then up.
L1-Assisted L10 in search,then up.

1

B.C. Gordan Symon (signature) 1st B.N.

TYPE FULL NAME: Gordan Symon — TIME ARRIVED: 0844

D.C. ______ DIV. ______

TYPE FULL NAME ______ TIME ARRIVED ______

PAGE 2 OF 2

Attachment G-A.6.1

Additional fire incidents involving the deployment of standpipe lines in WTC 1 and WTC 2 –

- Fires involving the use of one standpipe line and the activation of one sprinkler (4 in total)

BF 24 (1/80)

REPORT – STRUCTURAL FIRE

INCIDENT LOCATION				DATE AND TIME ALARM RECEIVED		INCIDENT DURATION					BOX LOCATION		REPORT		COS. RESP.	
TYPE REPORT	COMMUNITY BOARD	ADMIN. COMPANY	BLOCK NUMBER	MONTH DAY YEAR	HOUR MINUTE	HOURS MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	REPORT NUMBER	FILING BATTALION	BADGE NUMBER	ENG. SQ. MARINE	LADDER
1	01	5 006	0113	09-29-81	0904	01:10	6	1	1	1	8087	0287	01	531	03	3

AIDED AND CASUALTIES – CIVILIANS					EXTINGUISHMENT			IGNITION						STRUCTURE				AREA FIRE ORIGIN				FIRE EXTENTION WITHIN BLDG.				SPECIAL USE		
EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	CODE	SPRINKLER NO. OPEN	STANDPIPE LINES USED	TERM. STAGE	EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	FACTOR	CONSTRUCT.	USE	STATUS	BLDG. DAMAGE	CONTS. DMGE.	FLOOR CODE	RM./AREA CODE	OCCUPANCY	MANNER OF	OCCUPANC'S	NO. BLDGS.	OTHER SMOKE DETECTORS	1	2	3
00	00	00	00	01	5	01	01	3	98	00	67	44	21	1	15	1	1	1	97	54	50	00	01	0	6			

ADDRESS

NUMBER	SUFFIX	PREFIX DIRECT	STREET NAME	TYPE	SUFFIX DIRECT
2			World Trade Center		1

BUILDING STORIES: 99

AREA: 210x210

Ebasco Services
NAME OF OCCUPANT

B6 Storage
ROOM, APT. NO.

Mr. Sarnelli (FSD)
LEFT IN CHARGE

EQUIPMENT INVOLVED IN IGNITION

YR.	MAKE	MODEL	S/N	POWER

COS

UNIT	PTS.	ITEM			
5 010 10	5 006 05	7 001 10	7 015 10		

OPERATIONS

Responded to alarm box 8087. Upon arrival found light fire, medium smoke conditions in storage room (office records) at above address and location. Transmitted signal 10-75. Fire had activated 1 sprinkler head. Fire was confined to and extinguished in said room.

Eng.10- Stretched standpipe line, extinguished fire, washed down.

Eng.6- Relieved Eng.10 on line.

Lad.1- Searched, found fire, shut down sprinkler system, reactivated system, checked for extension, overhauled.

B.C. ______ BN. ______ D.C. ______ DIV. ______
TYPE FULL NAME ______ TIME OF ARRIVAL ______ TYPE FULL NAME ______ TIME OF ARRIVAL ______

PAGE 1 OF 2

ADMINISTRATIVE COMPANY

BF-24 (1/80)-175M-608551 (83)

REPORT — STRUCTURAL FIRE

TYPE REPORT	COMMUNITY BOARD	INCIDENT LOCATION: ADMIN. COMPANY	BLOCK NUMBER	DATE AND TIME ALARM RECEIVED: MONTH	DAY	YEAR	HOUR	MINUTE	INCIDENT DURATION: HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOX LOCATION: BOROUGH	BOX NUMBER	REPORT: REPORT NUMBER	FILING BATTALION	BADGE NUMBER	COS. RESP.: ENG. SQ. MARINE	LADDER
1	01	5 010	0024	08	15	86	04	10	01	10	7	1	0	1	8084	0216	01	572	03	2

AIDED AND CASUALTIES — CIVILIANS: EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	EXTINGUISHMENT: CODE	SPRINKLER NO. OPEN	STANDPIPE LINES USED	IGNITION: TERM. STAGE	EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	FACTOR	STRUCTURE: CONSTRUCT.	USE	STATUS	BLDG. DAMAGE	CONTS. DMGE.	AREA FIRE ORIGIN: FLOOR CODE	RM/AREA CODE	OCCUPANCY	MANNER OF	FIRE EXTENTION WITHIN BLDG.: OCCUPANC'S	OTHER: NO. BLDGS.	SMOKE DETECTORS	SPECIAL USE: 1	2	3
00	00	00	00	01	5	01	01	3	98	00	99	75	00	1	15	1	0	0	94	33	50	00	01	0	2			

ADDRESS

NUMBER	SUFFIX	PREFIX DIRECT	STREET NAME	TYPE	SUFFIX DIRECT
1			World Trade Center		1

BUILDING STORIES: 94   AREA: 250x250

NAME OF OCCUPANT: Windows Of The World Restaurant   ROOM, APT. NO.:   LEFT IN CHARGE: Gene Raggio, Fire Saf. Dir.

EQUIPMENT INVOLVED IN IGNITION

YR.	MAKE	MODEL	S/N	POWER

COS

UNIT	PTS.	ITEM				
5 010 05	7010 05	5 006 05	5 007 05	7 001 05		

OPERATIONS

Upon arrival found alarm for fire in dumpster on 106 th floor. Sprinkler head controled fire.

Eng. 10 Supplied standpipe system, stretched handline & extinguished fire.

Eng. 6 Supplied standpipe system & assisted Eng. 10.

Eng. 7 Made search & examination of floors 98-105.

Lad. 10 Made necessary search, examination & overhaul of fire area.

Lad. 1 Made necessary search & examination Of 106 & 107 floors.

B.C. Kenneth P. Markgraf   BN. 1st   D.C. ____   DIV. ____   PAGE __ OF __

Kenneth P. Markgraf — TYPE FULL NAME   0415 — TIME OF ARRIVAL   ____ TYPE FULL NAME   ____ TIME OF ARRIVAL

TYPE REPORT	COMMUNITY BOARD	ADMIN. COMPANY	MONTH	DAY	YEAR	HOUR	MINUTE	HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	REPORT NUMBER	INCIDENT NUMBER	FILING BATTALION	BADGE NUMBER
1	01 5	010	06	11	87	20	18	01	15	10	1	0	1	8084	0187	0305	01	075

COMPANIES RESPONDING: 03 2

AIDED AND CASUALTIES — CIVILIANS: EVACUATED 00, FIRST AID 00, INJURED 00, KILLED 00, UNIFORMED INJURED 00

EXTINGUISHMENT / IGNITION / STRUCTURE / FIRE ORIGIN / FIRE EXTENSION IN BLDG / DAMAGE / DETECTORS:
5 1 1 3 41 27 43 61 55 0 1 591 1 59 98 50 04 ... 1 1 2 1 1 1 2 1 1

BF-24

ADDRESS — NUMBER (INCL. HYPHEN): 1 — STREET NAME: World Trade Center

BUILDING — STORIES: 94 — AREA: WIDTH 500, LENGTH 500

DEFECTIVE DETECTOR — BRAND — SERIAL NO.

NAME OF OCCUPANT: NY Port Authority

ROOM/APT. ____ LEFT IN CHARGE ____

EQUIPMENT INVOLVED IN IGNITION — ITEM — YR. — MAKE — MODEL — SERIAL NUMBER — POWER

W R — UNIT / PTS.: 5 010 10 5 006 10 7 010 10 7 001 10

OPERATIONS

Responded to report of smoke condition at 1 WTC detoured by Port Authority to 5 WTC. Upon arrival found medium smoke condition at entrance to PATH tubes. Fire was located in escalator motor.

Engine Co. 10- Streched line from standpipe and extinguished fire.
Engine Co. 6- Assisted E-10 in strech adding thier lengths of hose to E-10 s. MPO hooked up to siamese.
Ladder Co. 10- Located fire. Had electric shut down, assisted mechanics in gaining access to motor.
Ladder Co. 1- Operated on fire level and above directed passengers to other PATH entrances.

BC Rudolph S. Quatrone — 1st BN.

Rudolph S. Quatrone — 20:21 TIME OF ARRIVAL

DC ____ DIV. ____ PAGE ☐ OF ☐

WHITE—FIRE RECORDS — YELLOW—BATTALION — PINK—REFERRAL — GOLD—ADMINISTRATIVE COMPANY

BF-24 (12/86) 1500-P-02B-N

REPORT — STRUCTURAL FIRE

TYPE REPORT	INCIDENT LOCATION (COMMUNITY BOARD / ADMIN. COMPANY)	DATE AND TIME ALARM RECEIVED (MONTH / DAY / YEAR / HOUR / MINUTE)	INCIDENT DURATION (HOURS / MINUTES)	HOW REPORTED / INITIAL ALARM / HIGHEST ALARM / BOROUGH	BOX LOCATION (BOX NUMBER)	REPORT (REPORT NUMBER / INCIDENT NUMBER / FILING BATTALION / BADGE NUMBER)	COMPANIES RESPONDING (ENG SQ MARINE / LADDERS RES. H.M. / OTHERS)	HAZ MAT (CLASS / AMOUNT)	HEAT EQUIP./FUEL	SPECIAL USE (1 / 2 / 3)	REVISED REPORT
1	01 5 010	03 24 91 2104	01 25	81 1 7 1	8084	0332 0406 01 380	07 05 05	000	0		

AIDED AND CASUALTIES — CIVILIANS (EVACUATED / FIRST AID / INJURED / KILLED / UNIFORMED INJURED)	EXTINGUISHMENT (CODE / SPK. PERF. / STP PERF.)	IGNITION (COND. ON ARRIVAL / EQUIP. INVOLVED / HEAT FORM / MATERIAL TYPE / MATERIAL FORM / FACTOR / JUVENILE)	STRUCTURE (CONSTRUCT / USE / STATUS / COMPLEX / FLOOR CODE)	FIRE ORIGIN (OCCUPANCY / AREA OF ORIGIN)	FIRE EXTENSION IN BLDG. (MANNER / OCCU. PANCIES / NO. BLDGS)	DAMAGE (% DAMAGED / FLAME / SMOKE / WATER)	DETECTORS (PRESENT / TYPE / POWER / PERFORMANCE)	FOR INVESTIGATION
00 00 00 00 00	5 1 1	3 98 99 67 58 11 0	1 059 1 59	05 59 05	00 01 0	1 2 5 2	1 1 2 1	1

BF-24

ADDRESS: BORO	NUMBER (INCL. HYPHEN)	SUFFIX	PREFIX DIRECTION	STREET NAME	TYPE	SUFFIX DIRECT.
1	1			World Trade Center		

BUILDING: STORIES	AREA WIDTH	AREA LENGTH
99	0250	0250

DEFECTIVE DETECTOR — BRAND — SERIAL NO.

NAME OF OCCUPANT: NYPA

ROOM/APT. 5 th. Floor

LEFT IN CHARGE Sgt. Bleeker NYPA Sgt. 294 BADGE NO.

EQUIPMENT INVOLVED IN IGNITION — YR. — MAKE — MODEL — SERIAL NUMBER — POWER

WRI (UNIT / PTS) — ITEM				
5 010 10	5 007 02	5 004 05	7 010 10	7 001 10

OPERATIONS

Chief officers & Officials on Scene	
D.C. Hassett	MSFC
B.C. Casey	Battalion 1
B.C. Demerest	Battalion 4
FM Corbett	#224
FSD Raggio	NYPA
Sgt. Bleeker	NYPA PD.

Signal	Time	By order of
8084	2104	Disp.
10-76 8084	2110	Bn. 1
75 8084	2126	Div. 1
U.C.	2128	Div. 1
10-41-1	2133	Div. 1

B.C. J. B. Casey — 1 BN.

John B. Casey — TYPE-FULL NAME — 2109 TIME OF ARRIVAL

D.C. — 1 DIV.

Bruce Hassett — TYPE FULL NAME — 2121 TIME OF ARRIVAL

PAGE 1 OF 2

WHITE—FIRE RECORD YELLOW—BATTALION PINK—REFERRAL GOLD—ADMINISTRATIVE COMPANY

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Attachment G-A.6.2

Additional fire incidents involving the deployment of standpipe lines in WTC 1 and WTC 2 –

- Fires involving the use of one standpipe line (27 in total)

DATE AND TIME ALARM RECEIVED						DURATION OF INCIDENT		HOW REPORTED	INITIAL ALARM		LOCATION ALARM / INCIDENT				REPORT		
TYPE	MONTH	DAY	YEAR	HOUR	MINUTE	HOURS	MINUTES			BOROUGH	BOX NUMBER	ADMIN. COMPANY	BLOCK NUMBER	REPORT NUMBER			BATT. NUMBER
1	05	24	73	03	10	01	20	2	1	1	0067	7 010	0033	0068	01		365

INJURED AND CASUALTIES CIVILIANS						RESP.	EXTINGUISHMENT		INVESTIGATION			STRUCTURE				AREA FIRE ORIGIN			
00	00	00	00	00	05	2	6	00	01	0	64		1	15	1	1	0	09	47	50	00

ADDRESS 1 World Trade Center (NUMBER, STREET) Man, (BOROUGH)

World Trade Center (NAME OF OCCUPANT) ROOM APT.

BUILDING 110 (STORIES) 570 x 970 (Below Grade) (AREA)

Mickey Seebeck - Superv. Engr. (LEFT IN CHARGE)

PA - Lt. Newman #21

COMPANIES (TYPE NO. SECT. FIS)

5 006 0 10	5 007 0 02	5 027 0 02	5 010 0 02	7 010 0 10	7 001 0 10

OPERATIONS

On arrival found fire in elevator car (J3) - B2 level, -J-4 area - Heavy smoke condition in adjacent areas, B-2 level. Light smoke cond. in Concourse, on 19th, 38th & 78th floors and various other floors. Fire was confined and extinguished with 1 house line and one F.D. hand line from standpipe. Areas involved with smoke were searched, occupant assisted where required. Operations as follows:

E6- stretch line from standpipe J-4 area, ext. fire in elev. & shaft.
E7 - assist E6 in stretching handline.
E10 - made search upper floors - 77th to 79th(light smoke cond.)
E27 - made search upper floors - 38th fl. & vic.
L10(L18) - report to CP - search & examination of concourse area.
L1 - forced elev. door B-2 level, examine car, overhaul.
B2 - supervise operations of E6 & L1 opening elev. car door & ext. fire B-2 ~~XXX~~ level (284)

B.C. [signature] 1 BN. D.C. [signature] 1 DIV PAGE 1 OF 1

Frank L. Picariello (TYPE FULL NAME) 0312 (TIME OF ARRIVAL) John J. Hart (TYPE FULL NAME) (TIME OF ARRIVAL)

TYPE REPORT	MONTH	DAY	YEAR	HOUR	MINUTE	HOURS	MINUTES	HOW RECEIVED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	ADMIN COMPANY	ROOF NUMBER	ELECT NUMBER	FIRING BATTALION	PAGE NUMBER
1	06	15	73	20	15	00	45	2	1	0	1	0067	7 010	0033	0104	02	325

EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	LENGTH	MARINE	LANGUAGE	CODE	EXTINGUISHMENT	STANDPIPE	EXTENT	CAUSE	FIRE MARSHAL	CONSTRUCTION	USE	STATUS				COST	ROOM OF AREA CODE	OCCUPANCY	MANNER OF EXTENSION
00	00	00	00	01	05	2	9	00	01	1	64		1	15	1	1	1			06	57	50	00

ADDRESS 1 World Trade Center — Manhattan — Unoccupied — 6518
NUMBER / STREET / BOROUGH / NAME OF OCCUPANT / ROOM/AF

BUILDING 110 — 200x200 — Security Guards
STORIES / AREA / LEFT IN CHARGE

COMPANIES

TYPE	NO	SECT	FTS									
5 006	0	10	7 010	0	10							

OPERATIONS

Upon arrival found fire in rubbish in room 6518 On the 65th. floor, fire confined and extinguished.Fire was in an unoccupied office of bldg

E.6: Stretched line off standpipe extinguished. fire,had taken rolled ups to 65th. floor.

L.10: Overhauled,made necessary examination,ventilated.

On Scene: Fire Patrol #2.

Injured Member:Fr. 1st. Vincent Segretto #9050 Lad.10,twist right knee.

Dr. Schwarts notified.No time lost.

B.C. Louis Pike 2 BN. D.C. John J. Hart 1 DIV. PAGE

Louis Pike — 2017 — John J. Hart — 2020 — OF
TYPE FULL NAME / TIME OF ARRIVAL / TYPE FULL NAME / TIME OF ARRIVAL

FIRE RECORDS

REPORT – STRUCTURAL FIRE

FOR FIRE RECORDS USE ONLY | TYPE REPORT | DATE AND TIME ALARM RECEIVED: MONTH, DAY, YEAR | CO. NO. | HOUR | MINUTE | DURATION OF INCIDENT: HOURS, MINUTES | HOW REPORTED | INITIAL ALARM | HIGHEST ALARM | BOROUGH | LOCATION: ALARM: BOX NUMBER | INCIDENT: ADMIN. COMPANY, BLOCK NUMBER | REPORT: REPORT NUMBER, FILING BATTALION, BADGE NUMBER

1 02 26 93 1 0904 01 40 4 1 1 0707 010 0033 0096 01 999

AIDED AND CASUALTIES: CIVILIANS: EVACUATED, FIRST AID, INJURED, KILLED | UNIFORMED INJURED | RESP.: ENG. SQ. MARINE, LADDERS | EXTINGUISHMENT: CODE, NO. SMALL FIRE UNITS, STANDPIPE LINES USED | INVESTIGATION: OUTSIDE ENTITIES, CAUSE, FIRE MARSHALS | STRUCTURE: CONSTRUCT., USE, STATUS | AREA FIRE ORIGIN: DAMAGE TO BLDG, DAMAGE TO CONTENTS, FLOOR CODE, ROOM OR AREA CODE, OCCUPANCY, MANNER OF EXTENSION

00 00 00 00 00 01 4 6 00 01 1 64 06 1 1 15 1 2 3 05 57 50 00

ADDRESS 2 World Trade Center

NUMBER STREET BOROUGH

BUILDING (Second Card) 110 STORIES 200x200 AREA

Port Authority

NAME OF OCCUPANT ROOM/APT. NO.

Mr. Sarnelli(Firesaftey Dir)

LEFT IN CHARGE

COMPANIES: TYPE NO. SECT. FTS.

5 006 15 5 007 20 5 027 12 7 010 20 7 001 20 7 008 10

7 015 10

OPERATIONS

Received Alarm Class 3-70-4,
On Arrival proceeded to the 5th. floor, smoke reported there and floors above.
E.-6 took in rolled-up lengths and masks, proceeded to check out 9th and 10th. floors, then checked 21st,22nd, and 61st. floors.
E-7- found fire,rubbish burning on foyer of 5th. floor, extinguished with line taken from standpipe.
L.-10- examined and overhauled, checked 14th to 17th. floors for smoke.
L.1- checked following floors,9,10,11,61st and 69th. assisted in crowd control.
E-27- checked 10th and 11th. floor and assisted in crowd control.
L-8- checked the 80th thru 99th. floor.
L.-15- checked 100th thru 110th. floors.
Deputy chief Hart was on scene.
BC. Whitney supervised following Cos. L.1-,L.10,E.7,E.27(Batt. 4)
Sgt, Keeghan of FP.#2 was on scene.
Fire was also found on 49th. Floor,Extinguished on arrival, found and checked out by E.7. Fire Marshall Peritti Called, because of suspicious fire.

B.C. Charles M. Blaich (signature) BN. 1 D.C. DIV. PAGE OF

Charles M. Blaich — TYPE FULL NAME | 0907 — TIME OF ARRIVAL | John J. Hart — TYPE FULL NAME | TIME OF ARRIVAL

REPORT - STRUCTURAL FIRE

FOR FIRE RECORDS USE ONLY	TYPE REPORT	DATE AND TIME ALARM RECEIVED: MONTH / DAY / YEAR / CD. NO. / HOUR / MINUTE	DURATION OF INCIDENT: HOURS / MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	LOCATION ALARM: BOX NUMBER	LOCATION INCIDENT: ADMIN. COMPANY / BLOCK NUMBER	REPORT: REPORT NUMBER / FILING BATTALION / BADGE NUMBER
	1	10 30 75 1 0308	00 25	4	1	4	1	0070	7 010 0033	0181 01 445

AIDED AND CASUALTIES CIVILIANS: EVACUATED / FIRST AID / INJURED / KILLED / UNIFORMED INJURED	RESP.: ENG. SQ. MARINE / LADDERS	EXTINGUISHMENT: CODE / NO. SPRINKLERS OPEN / STANDPIPE LINES USED	INVESTIGATION: RUBBISH CONTENTS / CAUSE / FIRE MARSHAL	STRUCTURE: CONSTRUCT / USE / STATUS / DAMAGE TO BLDG. / DAMAGE TO CONTENTS	AREA FIRE ORIGIN: FLOOR CODE / ROOM OR AREA CODE / OCCUPANCY / MANNER OF EXTENSION
00 00 00 00 00	03 2	6 00 01	2 1 64	1 15 1 1 1	32 30 50 00

ADDRESS # 2 World Trade Center (NUMBER, STREET) Manhattan (BOROUGH)

NAME OF OCCUPANT: Port of N.Y. Authority ROOM/APT. NO.

BUILDING 110 (STORIES) 250x250 (AREA)

LEFT IN CHARGE: Fire Safety Director

(Second Card)

COMPANIES: 5 006 0 05 7 010 0 05

TYPE NO. SECT. PTS.

OPERATIONS

Responded to Class 3-70-4

Upon arrival found fire in planter on 32 floor there confined and [illegible] extinguished as follows.

E.6 - Carry in rolled up lengths, hook up to S/P outlet, strecte line and operate on and extinguish fire and necessary wash down.

L.10 - Make necessary examination of 32 and 33 floors and overhaul burnt debris on 32 floor.

B.C. ____ 1st D.C. ____ BN. DIV. PAGE ___ OF ___

Fergus J. McDermott #2

TYPE FULL NAME — TIME OF ARRIVAL — TYPE FULL NAME — TIME OF ARRIVAL

... - STRUCTURAL FIRE

FOR FIRE RECORDS USE ONLY	TYPE REPORT	DATE AND TIME ALARM RECEIVED: MONTH	DAY	YEAR	CO. NO.	HOUR	MINUTE	DURATION OF INCIDENT: HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	LOCATION ALARM: BOX NUMBER	ADMIN. COMPANY	INCIDENT: BLOCK NUMBER	REPORT NUMBER	REPORT: FILING BATTALION	BADGE NUMBER
	1	03	09	76	1	1603		00	30	4	1	0	1	0070	5 006	0010	0124	04	880

AIDED AND CASUALTIES CIVILIANS: EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	RESP.: ENG. SQ. MARINE	LADDERS	EXTINGUISHMENT: CODE	NO. SPRINKLERS OPEN	STANDPIPE LINES USED	RUBBISH CONTAINER	INVESTIGATION: CAUSE	FIRE MARSHAL	STRUCTURE: CONSTRUCT.	USE	STATUS	DAMAGE TO BLDG.	DAMAGE TO CONTENTS	AREA FIRE ORIGIN: FLOOR CODE	ROOM OR AREA CODE	OCCUPANCY	MANNER OF EXTENSION
00	00	00	00	00	03	2	6	00	01	1	64		1	15	1	1	0	80	10	50	00

ADDRESS 1 World Trade Center (NUMBER, STREET) Man. (BOROUGH)

BUILDING 110 (STORIES) (Second Card) 204 x 204 (AREA)

Various (NAME OF OCCUPANT) ROOM/APT. NO.

Lt. John Elliott PAPD (LEFT IN CHARGE)

COMPANIES

5 055 0	10	7 008 0	05				

TYPE NO. SECT. PTS.

OPERATIONS

On arrival found light fire condition in rubbish piled against wall in main concourse which had caused scortching to said wall.

E 55 Stretched line from standpipe outlet and extinguished fire. washed down.

L 8 Examined for extension and overhauled.

Stanley Hirschfield

tanley Hirschfield (TYPE FULL NAME) 4 (BN.) 1608 (TIME OF ARRIVAL) D.C. ______ (TYPE FULL NAME) DIV. ______ (TIME OF ARRIVAL) PAGE 1 OF 1

ADMINISTRATIVE COMPANY

REPORT—STRUCTURAL FIRE

FOR FIRE RECORDS USE ONLY	TYPE REPORT	MONTH	DAY	YEAR	CD. NO.	HOUR	MINUTE	HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	ADMIN. COMPANY	BLOCK NUMBER	REPORT NUMBER	FIRING BATTALION	BADGE NUMBER
	1	10	20	76	1	16	18	00	40	1	1	0	1	0070	5 006	0113	0084	01	10

DATE AND TIME ALARM RECEIVED · DURATION OF INCIDENT · LOCATION: ALARM / INCIDENT · REPORT

AIDED AND CASUALTIES: CIVILIANS · RESP. · EXTINGUISHMENT · INVESTIGATION · STRUCTURE · AREA FIRE ORIGIN

EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	ENG. SQ. MANNED	LADDERS	CODE	NO. SPRINK. HDS. OPEN	STANDPIPE USED	...	CAUSE	...	CONSTRUCT.	USE	STATUS	...	FLOOR	CODE	ROOM OR AREA CODE	OCCUPANCY	MANNER OF EXTENSION
00	00	01	00	00	30	2	6	00	01	1			E	1	5	1 0 0	79				00

ADDRESS 1 World Trade Centre Man.
NUMBER STREET BOROUGH

Port Authority of NY
NAME OF OCCUPANT ROOM/APT. NO.

BUILDING Tower "B" 110 #300 X 300
(Second Card) STORIES AREA

Mr. Sarnelli FSD
LEFT IN CHARGE

COMPANIES

TYPE	NO.	SECT.	PTS.	TYPE	NO.	SECT.	PTS.	TYPE	NO.	SECT.	PTS.
5	007	0	05	5	006	0	02	5	024	0	02
7	001	0	05	7	008	0	02				

OPERATIONS

On arrival was notified by Mr Sarnelli of a rubbish fire on the 79 floor south west Quadrent tower B .

E. 7 Responded to 79 floor stretched line from standpipe, extinguished fire

E. 6 Stretched line and stood fast in fire control center area.

E. 24 Stretched line and stood fast in stairway in lobby area.

L. 1 Performed necessary search,ventilation, and over haul on fire floor.

L. 8 Stood fast with tools and masks in stairway at lobby.

Injured P.O. Keely (port Autority Police) # 1501 removed to Beekman Hospital Via public ambulance. apparent, smoke inhalation.

B.C. Theodore A. Campbell BN. DIV. PAGE OF

Theodore A. Campbell 1620
TYPE FULL NAME TIME OF ARRIVAL TYPE FULL NAME TIME OF ARRIVAL

ADMINISTRATIVE COMPANY

FOR FIRE RECORDS USE ONLY	TYPE REPORT	DATE AND TIME ALARM RECEIVED: MONTH	DAY	YEAR	CO. NO.	HOUR	MINUTE	DURATION OF INCIDENT: HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	LOCATION: ALARM: BOX NUMBER	INCIDENT: ADMIN COMPANY	BLOCK NUMBER	REPORT NUMBER	REPORT: FIRING BATTALION	BADGE NUMBER
	1	06	24	77	1	22	05	01	55	4	0	7	1	0070	5 006	0113	0158	01	531

AIDED AND CASUALTIES: CIVILIANS: EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	ENG SQ. MARINE	RESP.: LADDERS	CODE	EXTINGUISHMENT: NO. SPRINKLER HEADS OPEN	STANDPIPE LENGTHS USED	ASSISTIN CONTROL	INVESTIGATION: CAUSE	FIRE MARSHAL	STRUCTURE: CONSTRUCT.	USE	STATUS	DAMAGE TO BLDG	DAMAGE TO CONTENTS	AREA FIRE ORIGIN: FLOOR CODE	ROOM OR AREA CODE	OCCUPANCY	MANNER OF EXTENSION
00	03	07	00	00	03	4	0	00	01	1	62		1	15	1	1	0	46	31	50	00

ADDRESS 1 World Trade Ctr (NUMBER / STREET) Man (BOROUGH)

Port of N.Y. Authority (NAME OF OCCUPANT) 46th hallw (ROOM/APT. NO.)

BUILDING (Second Card) 110 250 x250 (STORIES) (AREA)

Richard Hintsen(Fire Safety D (LEFT IN CHARGE)

COMPANIES (TYPE NO. SECT. PTS.)

5 006 0 20	5 007 0 12	5 024 0 12	1 001 0 10	7 001 0 20	7 008 0 20	
7 015 0 10						

OPERATIONS

Responded to 3-70-2(Manual Alarm)

While responding Batt.1 notified via dept. radio of special call additional Ladder Co(L.15) due to report of fire 46th fl. Upon arrival was informed of fire 46th fl public hallway near freight elevator. Ordered investigation a nd found fire therein, which had been extinguished prior to the arrival of this dept. Evacuation instituted by Port Authority personnel prior to arriva l of Fire Dept. units. Report of smoke detector operational of the 103rd fl. Fire located between freight elevators 49 & 17.

E.6- Rolled up,lengths to the 44th fl. Connected to standpipe therein and stretched to fire floor(46th) Washed down fire area for overhauling purposes.

E.7- Assisted in stretch, then ordered to search, examination of 53rd to 58th fls. Also Checked 45th fl. report of smoke condition.

E.24- Reported to secondary command post([redacted]). Then ordered to check [illegible] of 53rd to 56th fls. Also checked out smoke detector 103rd fl.

B.C. James J. McKenna — 1 (BN.) — 2208 (TIME OF ARRIVAL)
TYPE FULL NAME

D.C. Matthew J. Farrell — 1 (DIV.) — (TIME OF ARRIVAL)
TYPE FULL NAME

PAGE 1 of 3

ADMINISTRATIVE COMPANY

BF-24A (6/76)

REPORT — Additional Data

Structural Fire, Transportation Fire, Non-Structural Fire or Emergency

FOR FIRE RECORDS USE ONLY	TYPE REPORT	MONTH	DAY	YEAR	CD. NO.	HOUR	MINUTE	BOROUGH	BOX NUMBER
	1	06	24	77	2	22	05	1	0020

DATE AND TIME ALARM RECEIVED — ALARM

COMPANIES — TYPE NO. SECT. PTS.

OPERATIONS

L.1- Init¡ally to the fire floor (46th) operated for overhauling search & examinations ventilation of same. Then search, examination & ventilation of the 107th to 46th fl. Stairway "B".

L.8- Initially to secondary command post.(West St). Then ordered to check 78the to 60th fls search & examination. Also assisted in overhauling fire floor (46th)

L.15- Special called to report to West St. Then trough lobby to secondary command post. Enroute found (2) civilian cleaning personnel(female) had been removed to lobby suffering smoke inhalation.Performed first Aid on injured civilians. Then relieved by Res. 1 with resusitator Then xxx ordered to serachg47th to 52nd fls as (2) injured civilians reported to ha ve worked on the 48th & 50th fls.

Res.1- Relieved L.15 and administered first aid(Inhalation)to injured civlians. Then ordered Office r & remainder of members went to investigate report of smoke & people on the 55th fl. Thence down to 46th fl. Search, exa mination of floors enroute.

Batt.1- Initia lly in command, thenordered to [redacted] set up additional command in lobby, as first aid station.Two public ambulances standing by with (4) resuscitators. Directed search operations of E.7, L.15, Res. 1. Ordered smoke purge 45th to 107 when fire was out.

Batt.2- Ordered to supervise units on the fire floor & report conditions therein. Supervised in part operation of E.6, L.1

Batt.32 - Ordered to supervise operations of units above fire floor.

Div.1- In overhauled command of operations at command post B.1 level.

B.C. James J. McKenna (signature) — BN. — D.C. — DIV. 1 — PAGE 2 of 3

James J. McKenna — TYPE FULL NAME — 2208 TIME OF ARRIVAL — Matthew J. Farrell — TYPE FULL NAME — TIME OF ARRIVAL

BF-24A (6/76)

REPORT — Additional Data

Structural Fire, Transportation Fire, Non-Structural Fire or Emergency

FOR FIRE RECORDS USE ONLY	TYPE REPORT	MONTH	DAY	YEAR	CO. NO.	HOUR	MINUTE	BOROUGH	BOX NUMBER
	1	06	24	77	2	22	05	1	0070

DATE AND TIME ALARM RECEIVED — ALARM

COMPANIES — TYPE NO. SECT. PTS.

OPERATIONS

Act Asst Chief Munk on the scene to supervised overall operations.

Remarks: L. 1 checked all elevator ~~xkx~~ ca rs before leaving scene.
Ordered 10-41 Code 1 due to report from Port Authority police of labor trouble with Maintainence personnel (Temco)

Dept. Photgra pher ordered to scene to take pictures of fire area.

Alarm was turned in by Mr. Nick Cappola, Temco Maintainence
(2) Beekman Ambulances on the scene under Mr. B. John

Injured Civilians:

*Name	Address	Injury	
[redacted]	[redacted]	Smoke inhal.	Treated Beekma & Released
[redacted]	[redacted]	Smoke inhal.	" " "
[redacted]	[redacted]	Smoke Inhal	Received O2 nc removed to Hos

Note : All of the above are employees of the Temco Co.

Sgt. Steve Fox Bd# 264	Burn to Fingers Rt Hand.
Ptl A. Halicker	Burn to Rt Wrist
Ptl Meyers	Smoke inhal
Ptl. Carcaic	Smoke Inhal.

Note: Police members of Port Authority Police Dept. All injured treated at Beekman Hospital & released.

B.C. James J. McKenna BN. D.C. 1 DIV. PAGE OF

James J. McKenna TYPE FULL NAME 2208 TIME OF ARRIVAL Matthew J. Farrell TYPE FULL NAME TIME OF ARRIVAL

ADMINISTRATIVE COMPANY

BF-24 (6/72)-50M-121073(75)

REPORT

FOR FIRE RECORDS USE ONLY			TYPE REPORT	MONTH	DAY	YEAR	CD. NO.	HOUR	MINUTE	HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	ADMIN. COMPANY	BLOCK NUMBER	REP. NU
			1	07	13	77	1	15	30	00	30	1	1	0	1	0070	5 006	0113	0122 01

AIDED AND CASUALTIES — CIVILIANS; RESP.; EXTINGUISHMENT; INVESTIGATION; STRUCTURE; AREA FIRE ORIGIN

EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	ENG SQ MARINE	LADDERS	CODE	AID SPRINK. HSS OPEN	STANDPIPE SIAMESE	PORTABLE EXTING.	CAUSE	FIRE MARSHAL	CONSTRUCT	USE	STATUS	DAMAGE TO BLDG	DAMAGE TO CONTENTS	FLOOR CODE	ROOM OR AREA CODE	OCCUPANCY	MANNER OF EXTENSION
00	00	00	00	00	30	2	5	00	01	0			1	15	1	0	0	09	57	50	00

ADDRESS # 1 World Trade Center Tower B (NUMBER / STREET / BOROUGH)

Port Authority (NAME OF OCCUPANT) ROOM/APT

Mr. Sarnelle FSD. (LEFT IN CHARGE)

BUILDING 110 (STORIES) 600 x 600 (AREA)

(Second Card)

COMPANIES

5 007 0	05	7 001 0	05							

TYPE NO. SECT. PTS.

OPERATIONS

On arrival found fire to be in airconditioning room on the 9th fl

E. 7 Stretched Line, extinguished fire.

L. 1 Made necessary search and overhaul.

B.C. Theodore A. Campbell (TYPE FULL NAME) 1 BN 1535 TIME OF ARRIVAL

D.C. (TYPE FULL NAME) DIV TIME OF ARRIVAL PAGE

ADMINISTRATIVE COMPANY

REPORT – STRUCTURAL FIRE

FOR FIRE RECORDS USE ONLY			TYPE REPORT	DATE AND TIME ALARM RECEIVED: MONTH	DAY	YEAR	CD. NO.	HOUR	MINUTE	DURATION OF INCIDENT: HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	LOCATION: ALARM: BOX NUMBER	INCIDENT: ADMIN. COMPANY	BLOCK NUMBER	REPORT: REPORT NUMBER	FILING BATTALION	BADGE NUMBER
			1	08	08	77	1	11	05	00	20	4	1	[illegible]	1	0070	6 006	00113	0070	04	989

AIDED AND CASUALTIES – CIVILIANS: EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	RESP.: ENG. SQ. MARINE	LADDERS	CODE	EXTINGUISHMENT: NO. SPRINK. HDS. OPEN	STANDPIPE LINES USED	RUBBISH CONTROL	INVESTIGATION: CAUSE	FIRE MARSHAL	STRUCTURE: CONSTRUCT.	USE	STATUS	DAMAGE TO BLDG.	DAMAGE TO CONTENTS	AREA FIRE ORIGIN: FLOOR CODE	ROOM OR AREA CODE	OCCUPANCY	MANNER OF EXTENSION
00	00	00	00	00	03	2	5	00	01	1	62		1	15	1	1	0	35	35	50	00

E-6

ADDRESS 1 World Trade Center (NUMBER, STREET) Man. (BOROUGH)

Various (NAME OF OCCUPANT) n/a (ROOM/APT. NO.)

BUILDING 110 (STORIES) 200 x 200 (AREA)

(Second Card)

Sgt. Brown, 109, P.A.P.D. (LEFT IN CHARGE)

COMPANIES

TYPE	NO.	SECT.	PTS.	TYPE	NO.	SECT.	PTS.
5	006	0	05	7	001	0	05

OPERATIONS

On arrival found fire in accumulated rubbish in elevator lobby of 35th floor which had caused scortching of the walls and ceiling. Port Authority personnel had extinguishef fire with hand extinguishers and standpipe hose. Fire termed suspicious by P.A.PD.

E 6 Responded with rolled- up lengths, hooked-up to standpipe and stood fast.

L 1 Searched, examined for extension and overhauled.

B.C. ABC *James Hallinan* 4 (BN.) D.C. ______ DIV. ______ PAGE 1 OF 1

James Hallinan (TYPE FULL NAME) 1110 (TIME OF ARRIVAL) ______ (TYPE FULL NAME) ______ (TIME OF ARRIVAL)

ADMINISTRATIVE COMPANY

REPORT - STRUCTURAL FIRE

FOR FIRE RECORDS USE ONLY			TYPE REPORT	MONTH	DAY	YEAR	CD. NO.	HOUR MINUTE	HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	ADMIN. COMPANY	BLOCK NUMBER	REPORT NUMBER	FILING BATTALION	BADGE NUMBER
			1	11	03	77	1	1845	00	30	4	1	0	1	8085	5 006	0113	0015	04	480

EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	ENG. SQ. MARINE	LADDERS	CODE	NO. SPRINKLERS OPEN	STANDPIPE LINES USED	RUBBISH CONTRIB.	CAUSE	FIRE MARSHAL	CONSTRUCT.	USE	STATUS	DAMAGE TO BLDG.	DAMAGE TO CONTENTS	FLOOR CODE	ROOM OR AREA CODE	OCCUPANCY	MANNER OF EXTENSION
00	00	00	00	00	03	2	5	00	01	1	62		1	15	1	1	0	58	21	50	00

ADDRESS # 1 World Trade Center Tower 1 Man. (NUMBER / STREET / BOROUGH) — Closet (NAME OF OCCUPANT) — N.A. (ROOM/APT. NO)

BUILDING 110 (STORIES) — 300 X 300 (AREA) — Sgt. Fox P.A. (LEFT IN CHARGE)

(Second Card)

COMPANIES (TYPE NO. SECT. PTS.)

5 006 0	10	7 001 0	05				

OPERATIONS

On arrival found cause of alarm to be fire in janatorial closet in volving rags and rubbish. Batt 4 requested a 10-41 Codel Fire caused searing of paint on walls and ceiling.

E-6 Stretched rolled up lengths and hooked up to standpipe outlet and extinguished fire.

L-1 Made examination of walls and ceiling and ventilated as necessary.

B.C. Stanley Hirschfield — 4th BN. D.C.

Stanley Hirschfield (TYPE FULL NAME) — 1850 (TIME OF ARRIVAL) — TYPE FULL NAME — TIME OF ARRIVAL

DIV. PAGE 1 OF 1

ADMINISTRATIVE COMPANY

FOR FIRE RECORDS USE ONLY	TYPE REPORT	MONTH	DAY	YEAR	CD. NO.	HOUR	MINUTE	HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	ADMIN. COMPANY	BLOCK NUMBER	REPORT NUMBER	FILING BATTALION	BADGE NUMBER
	1	02	13	78	1	20	18	00	45	4	1	0	1	8089	5 006	0113	0071	01	531

DATE AND TIME ALARM RECEIVED — DURATION OF INCIDENT — LOCATION: ALARM / INCIDENT — REPORT

EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	ENG. SQ. MARINE	LADDERS	CODE	NO. SPRINKLERS OPEN	STANDPIPE LINES USED	RUBBISH CONTRIB.	CAUSE	FIRE MARSHAL	CONSTRUCT.	USE	STATUS	DAMAGE TO BLDG.	DAMAGE TO CONTENTS	FLOOR CODE	ROOM OR AREA CODE	OCCUPANCY	MANNER OF EXTENSION
00	00	00	00	00	03	2	5	00	01	1	62		1	15	1	1	0	94	15	50	00

AIDED AND CASUALTIES (CIVILIANS) — RESP. — EXTINGUISHMENT — INVESTIGATION — STRUCTURE — AREA FIRE ORIGIN

ADDRESS 2 World Trade Ctr (NUMBER, STREET) Man (BOROUGH)

Port of N.Y. Authority (NAME OF OCCUPANT) 107 (ROOM/APT. NO.)

BUILDING 110 (STORIES) 200 x200 (AREA)

Sgt. Wescott PONYA Police (LEFT IN CHARGE)

(Second Card)

COMPANIES

TYPE NO.	SECT.	PTS.					
5 006 0110	7 001 0	10					

OPERATIONS

Responded to manual ala rm box 8089-

Upon arrival was informed of fire 107th fl. Ordered investigation and found fire in rubbish & maint. materials therein. Batt. 1 ordered additional Battalion Chief to respond on report of definate fire. Batt.4 responde d.

E.6- Rolled up lengths to the fire floor extinguished remaining fire

L.1- Search, examination of fire floor & floor above. Opened walls for examination. Overhauled burned materials.

Bn.4- Ordered to supervise operations on the fire floor.

Div. 1- Responded to scene, and assumed command.

Note: Batt.1 transmitted 10-41 Code 2 & requested F.M. to respond.

B.C. ______ 1 BN. D.C. ______ 1 DIV. PAGE ___ OF ___

Jmaes J. McKenna (TYPE FULL NAME) 2020 (TIME OF ARRIVAL) Matthew J. Farrell (TYPE FULL NAME) ______ (TIME OF ARRIVAL)

ADMINISTRATIVE COMPANY

FOR FIRE RECORDS USE ONLY	TYPE REPORT	MONTH	DAY	YEAR	CD. NO.	HOUR/MINUTE	DURATION OF INCIDENT HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	ADMIN. COMPANY	BLOCK NUMBER	REPORT NUMBER	FILING BATTALION	BADGE NUMBER
	1	01	18	79	1	2353	00	45	4	1	0	1	8087	5 006	0113	0164	01	352

EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	ENG. SQ. MARINE	LADDERS	CODE	NO. SPRINKLERS OPEN	STANDPIPE LINES USED	RUBBISH CONTRIB.	CAUSE	FIRE MARSHAL	CONSTRUCT.	USE	STATUS	DAMAGE TO BLDG.	DAMAGE TO CONTENTS	FLOOR CODE	ROOM OR AREA CODE	OCCUPANCY	MANNER OF EXTENSION
00	00	00	00	00	03	2	5	00	01	1	62		1	15	1	1	0	06	31	50	00

ADDRESS 2 World Trade Center (NUMBER / STREET) Man. (BOROUGH)

N.Y. State (NAME OF OCCUPANT) ROOM/APT. NO.

BUILDING (Second Card) 110 (STORIES) 150x150 (AREA)

Bon Baily P.A. Police (LEFT IN CHARGE)

COMPANIES

5 006 0 10	~~5 007 0~~	7 001 0 10	7 008 0 10		

TYPE NO. SECT. PTS.

OPERATIONS

Upon arrival was told of fire on the 6th floor, operations as follows.
Ladder 1 made necessary investagation, located the fire, vented, overhauled and searched.
Ladder 8 searched and vented floor above, overhauled.
Engine 6 stretched a line from standpipe and extinguished the fire.
Engine 6 washed down.
Batt. 2 on the scene.
Div. 1 on the scene.

B.C. Joseph A. Hingerton BN. 1 D.C.

Joseph A. Hingerton — TYPE FULL NAME
2355 — TIME OF ARRIVAL
TYPE FULL NAME
TIME OF ARRIVAL
DIV. PAGE 1 OF 1

ADMINISTRATIVE COMPANY

BF-24 (1/80)

TYPE REPORT	COMMUNITY BOARD	ADMIN. COMPANY	BLOCK NUMBER	MONTH	DAY	YEAR	HOUR	MINUTE	HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	REPORT NUMBER	FILING BATTALION	BADGE NUMBER	ENG. SQ. MARINE	LADDER
1	01	5	006	0113	04	19	80	14	06	02	30	1	1	2	1	8086	0156	01	531	07	3

INCIDENT LOCATION · DATE AND TIME ALARM RECEIVED · INCIDENT DURATION · BOX LOCATION · REPORT · COS. RESP.

AIDED AND CASUALTIES – CIVILIANS: EVACUATED / FIRST AID / INJURED / KILLED / UNIFORMED INJURED	300 00 00 00 00
EXTINGUISHMENT: CODE / SPRINKLER NO. OPEN / STANDPIPE LINES USED	5 00 01
IGNITION: TERM. STAGE / EQUIP. INVOLVED / HEAT FORM / MATERIAL TYPE / MATERIAL FORM / FACTOR	3 98 30 40 50 31
STRUCTURE: CONSTRUCT. / USE / STATUS / BLDG. DAMAGE / CONTS. DMGE.	11 15 1 1
AREA FIRE ORIGIN: FLOOR CODE / RM./AREA CODE / OCCUPANCY / MANNER OF	1106 54 59 00
FIRE EXTENTION: WITHIN BLDG. OCCUPANC'S / OTHER NO. BLDGS.	01 0
SMOKE DETECTORS	2
SPECIAL USE 1 / 2 / 3	

ADDRESS: NUMBER 1 · SUFFIX · PREFIX DIRECT · STREET NAME WORLD TRADE · TYPE CNTR · SUFFIX DIRECT 1

BUILDING STORIES 110 · 212 x 212 AREA

Port Authority N.Y. & N.J.
NAME OF OCCUPANT · ROOM, APT. NO.

Mr. John Barrett, F.S.D.
LEFT IN CHARGE

EQUIPMENT INVOLVED IN IGNITION · YR. · MAKE · MODEL · S/N · POWER

COS: UNIT · PTS. · ITEM

5 006 20	5 007 05	5 010 02	7 001 20	7 008 20

OPERATIONS

Upon arrival Batt. 1 received report of activated smoke detector in return air duct 106 floor Tower #1.

Batt. 1 subsequently received reports of heavy smoke on floor 106, light smoke on floor 109, heavy odor of smoke in stairways " A " and " B " leaving only stairway " C " as a clear exit

B.C. James J. McKenna — BN. 1 — D.C. — DIV.
James J. McKenna
TYPE FULL NAME · 1409 TIME OF ARRIVAL · TYPE FULL NAME · TIME OF ARRIVAL

PAGE 1 OF 5

ADMINISTRATIVE COMPANY

BF-24A (1/80)

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT	MONTH	DAY	YEAR	HOUR	MINUTE	BOROUGH	BOX NO.	BUILDING NO.	TERM. STAGE	EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	CONSTRUCT	USE	STATUS	BLDG. DAMAGE	CONTS. DAMGE	FLOOR CODE	RM/AREA CODE	OCCUPANCY	MANNER OF	OCCUPANCIES
6	04	19	80	14	06	1	8086	01	3	98	30	40	50	1	15	1	1	1	106	54	59	00	01

ADDRESS: WORLD TRADE CNTR

IF MOBILE PROPERTY

ITEM

YR. MAKE MODEL SERIAL NO. LICENSE STATE

COS.									
5	006	20	5	007	15	5	010	12	7	001	20	7	008	20

UNIT PTS.

OPERATIONS

from the 107th. floor restaurant. There is an open access stair from 106, opening into 107th. floor restaurant dining area.

At this point fire had not been located. On basis of above, Batt. 1 ordered approx. 300 persons evacuated from the " Windows on the world restaurant " on the 107th. floor via stairway " C " which was clear of smoke. Later stairway " B " was clear of smoke and was made available for evacuation.

On Arrival of D.C. Rossi, Div. 1 Batt. 1advised Him of above and recommended a 2nd. alarm be transmitted, as fire had not been located. Restaurant was being evacuated and all units were now assigned to work. D.C. Rossi transmitted a 2nd. alarm and requested 2 additional Batt. chi

Operations of Cos. are as follows:

Eng. 6 Masks, rolledups, responded to fire floor via freight elevator to 104 fl. via stairway to 106 fl. Met Lad. 1 who had located fire on 106 fl. Hooked 4 lengts of 21/2 hose to standpipe, operated on fire.

B.C. James J. McKenna BN. D.C. DIV. PAGE 2 OF

James J. McKenna 1409

TYPE FULL NAME TIME OF ARRIVAL TYPE FULL NAME TIME OF ARRIVAL

(1) White Copy: Fire Records (2) Canary Copy: Battalion (3) Pink Copy: Administrative Company (4) Goldenrod Copy: Referral

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT	MONTH	DAY	YEAR	HOUR	MINUTES	BOROUGH	BOX NO.	BUILDING NO.	TERM. STAGE	EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	CONSTRUCT.	USE	STATUS	BLDG. DAMAGE	CONTS. DMGE.	FLOOR CODE	RM/AREA CODE	OCCUPANCY	NUMBER OF OCCUPANCIES
6	04	19	80	14	06	1	8086	01	3	98	30	40	50	1	15	1	1	1	106	54	59	00 01

DATE AND TIME ALARM RECEIVED — BOX LOCATION — IGNITION — STRUCTURE — AREA OF ORIGIN — FIRE EXTENSION

ADDRESS: NUMBER (INCL. HYPHENS) 1 — STREET NAME: WORLD TRADE CNTR

IF MOBILE PROPERTY — ITEM

YR. — MAKE — MODEL — SERIAL NO. — LICENSE — STATE

COS — UNIT — PTS.

OPERATIONS

Operations of Cos. continued:

Lad. 1 to 106 floor to investagate activated smoke alarm. searched and located fire, checked floor above for extension, and overhaled.

Eng. 7 Responded to 109 fl. to investigate a sprinkler alarm, found no sprinkler flow, and light smoke condition.

Lad. 8 Responded to 109 fl. with Eng. 7.

Eng. 7, and Lad. 8 then reported to 108, 107, and 106fls. made search and assisted in evacuation of Restaurent.

Eng. 10, Responded to 106 fl. to assist and relieve Eng. 6 on the line.

2nd. alarm units reported to comand post, stood fast then orded to take up by DAC Glasse.

Rescue 1 responded on 2nd. alarm, reported to 107 fl. assisted in evacuation of restaurent, then to 106 fl. to assist Lad. 1, then orded to take up.

B.C. James J. McKenna 1 BN. — D.C. — DIV. — PAGE 3 OF

James J. McKenna 1409

TYPE FULL NAME — TIME OF ARRIVAL — TYPE FULL NAME — TIME OF ARRIVAL

(1) White Copy: Fire Records (2) Canary Copy: Battalion (3) Pink Copy: Administrative Company (4) Goldenrod Copy: Referral

FD-24A (1/80)

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT	MONTH	DAY	YEAR	HOUR	MINUTE	BOROUGH	BOX NO.	BUILDING NO.	...
6	04	19	30	14	06	1	8086	01	...

STREET NAME: 1 WORLD TRADE CENTER

IF MOBILE PROPERTY

OPERATIONS

Operations of Cos. continued:

Chief Officers on scene:

DAC Glasse, City wide command Chief.

D.C. Rossi Dept. chief 1st. Div.

B.C. James J. McKenna Chief 1st. Batt.

B.C. Louis Pike Chief 2nd. Batt.

B.C. John T. Carroll Chief 4th. Batt.

B.C. Edward J. Miller Chief 6th. Batt.

Field Comm. unit, Lieut. Soranno on scene.

Fire Patrol #2 on scene.

W.T.C. fire safety director Mr. John Barrett on scene.

P.A. Patrolmen Cemonuk, and Corbeek on Duty at Command Post.

Building Mechanical System falures noted:

#1 Sprinkler alarm received for unsprinklerd 109 Fl.
#2 Return air duct smoke detectors did not shut fans down.
#3 Heat fused link shut damper in purge system.
#4 107 floor standpipe phone unreliable due to feed back from radios.

B.C. James J. McKenna BN. 1 D.C. DIV.

James J. McKenna — 1409

TYPE FULL NAME — TIME OF ARRIVAL — TYPE FULL NAME — TIME OF ARRIVAL

(1) White Copy: Fire Records (2) Canary Copy: Battalion (3) Pink Copy: Administrative Company (4) Goldenrod Copy: Referral

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

DATE AND TIME ALARM RECEIVED | BOX LOCATION | IGNITION | STRUCTURE | AREA OF ORIGIN | FIRE EXTENSION

WORLD TRADE CNTR

IF MOBILE PROPERTY

MAKE | MODEL | SERIAL NO. | LICENSE | STATE

UNIT | FLG.

OPERATIONS

High Rise Report prepared and fowarded to Div. of Fire Prevention.

Follow up on Mechancal and Electrical malfunctions being made with Port Authority.

See copy of High Rise report for details.

B.C. James J. McKenna 1409 BN. D.C.

James J. McKenna

TYPE FULL NAME | TIME OF ARRIVAL | TYPE FULL NAME | TIME OF ARRIVAL

(1) White Copy: Fire Records (2) Canary Copy: Battalion (3) Pink Copy: Administrative Company (4) Goldenrod Copy: Referral

BF-24 (1/80)

REPORT — STRUCTURAL FIRE

TYPE REPORT	INCIDENT LOCATION: COMMUNITY BOARD	ADMIN. COMPANY	BLOCK NUMBER	DATE AND TIME ALARM RECEIVED: MONTH	DAY	YEAR	HOUR	MINUTE	INCIDENT DURATION: HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOX LOCATION: BOROUGH	BOX NUMBER	REPORT: REPORT NUMBER	FILING BATTALION	BADGE NUMBER	COS. RESP.: ENG. SQ. MARINE	LADDER
1	01	5	006	0113	11	04	80	0123	00	15	6	1	0	1	8093	0026	01	531	03	2

AIDED AND CASUALTIES: EVACUATED	CIVILIANS: FIRST AID	INJURED	KILLED	UNIFORMED INJURED	EXTINGUISHMENT: CODE	SPRINKLER NO. OPEN	STANDPIPE LINES USED	TERM. STAGE	IGNITION: EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	FACTOR	STRUCTURE: CONSTRUCT.	USE	STATUS	BLDG. DAMAGE	CONTS. DMGE.	AREA FIRE ORIGIN: FLOOR CODE	RM./AREA CODE	OCCUPANCY	MANNER OF	FIRE EXTENTION WITHIN BLDG. / OTHER: OCCUPANC'S	NO. BLDGS.	SMOKE DETECTORS	SPECIAL USE: 1	2	3
00	00	00	00	00	5	00	01	3	98	00	60	75	00	1	15	1	0	0	99	56	50	00	01	0	6			

ADDRESS: NUMBER	SUFFIX	PREFIX DIRECT	STREET NAME	TYPE	SUFFIX DIRECT
1			World Trade Center		1

BUILDING STORIES: 99

AREA: 250x250

NAME OF OCCUPANT: N.Y./N.J. Port Authority

ROOM, APT. NO.: Carpenter's Rm.

LEFT IN CHARGE: Sarnelli (FSD)

EQUIPMENT INVOLVED IN IGNITION — YR. / MAKE / MODEL / S/N / POWER

COS: UNIT	PTS.	ITEM			
5 010	10	7 001 05			

OPERATIONS

Responded to alarm box 8093. Upon arrival found cause to be for a rubbish fire at the above location.

Eng.10- Stretched standpipe line, extinguished fire, washed down.

Lad.1- Searched, found fire, vented, checked for extension, overhauled.

B.C. James J. McKenna — BN. 1

James J. McKenna — TYPE FULL NAME

TIME OF ARRIVAL: 0126

D.C. ____ TYPE FULL NAME ____ DIV. ____ TIME OF ARRIVAL ____

PAGE 1 OF 1

ADMINISTRATIVE COMPANY

TYPE REPORT	COMMUNITY BOARD	INCIDENT LOCATION: ADMIN. COMPANY	INCIDENT LOCATION: BLOCK NUMBER	DATE AND TIME ALARM RECEIVED: MONTH-DAY-YEAR	DATE AND TIME ALARM RECEIVED: HOUR MINUTE	INCIDENT DURATION: HOURS MINUTES	HOW REPORTED / INITIAL ALARM / HIGHEST ALARM / BOROUGH	BOX LOCATION: BOX NUMBER	REPORT NUMBER	REPORT: FILING BATTALION / BADGE NUMBER	COS. RESP.: ENG. SQ. MARINE / LADDER
1	01	5 006	0113	02-17-81	0859	00 40	4 1 0 1	8084	0141	01 531	03 2

1 4 8 12 18 22 26 30 34 38 43 45

AIDED AND CASUALTIES – CIVILIANS: EVACUATED / FIRST AID / INJURED / KILLED; UNIFORMED INJURED	EXTINGUISHMENT: CODE / SPRINKLER NO. OPEN / STANDPIPE LINES USED	IGNITION: TERM. STAGE / EQUIP. INVOLVED / HEAT FORM / MATERIAL TYPE / MATERIAL FORM / FACTOR	STRUCTURE: CONSTRUCT. / USE / STATUS / BLDG. DAMAGE / CONTS. DMGE.	AREA FIRE ORIGIN: FLOOR CODE / RM./AREA CODE / OCCUPANCY / MANNER OF	FIRE EXTENTION: WITHIN BLDG. / OTHER: OCCUPANC'S / NO. BLDGS.	SMOKE DETECTORS	SPECIAL USE 1	SPECIAL USE 2	SPECIAL USE 3
00 00 00 00 00	5 00 01	2 31 42 99 99 46	1 15 1 1	0108 10 81	00 99 0	2			

46 52 56 61 66 72 77 82 86 89 95

ADDRESS

NUMBER	SUFFIX	PREFIX DIRECT	STREET NAME	TYPE	SUFFIX DIRECT
1			World Trade Center		1

96 104 106 126 130

BUILDING STORIES 110 (132)

300x300
AREA

Port Authority N.Y. , N.J.
NAME OF OCCUPANT ROOM, APT. NO.

Mr. Sarnelli F.S.D.
LEFT IN CHARGE

EQUIPMENT INVOLVED IN IGNITION

YR.	MAKE	MODEL	S/N	POWER

134 151 166 187

COS

UNIT	PTS.	ITEM			
5 010 05	7 001 05				

188 194 200 206 212 217

OPERATIONS

Upon arrival at box 8084, found fire in air condition duct and filters on 108 floor

Eng. 10 Used house line, and extinguished fire.

Lad. 1 Overhauled and checked for extention.

B.C. James J. McKenna (signature) BN. 1 D.C. DIV. PAGE OF

James J. McKenna 0900
TYPE FULL NAME TIME OF ARRIVAL TYPE FULL NAME TIME OF ARRIVAL

BF-24 (1/80)

REPORT — STRUCTURAL FIRE

TYPE REPORT	COMMUNITY BOARD	INCIDENT LOCATION: ADMIN. COMPANY	BLOCK NUMBER	DATE AND TIME ALARM RECEIVED: MONTH	DAY	YEAR	HOUR	MINUTE	INCIDENT DURATION: HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX LOCATION: BOX NUMBER	REPORT NUMBER	REPORT: FILING BATTALION	BADGE NUMBER	COS. RESP.: ENG. SQ. MARINE	LADDER
1	01	5 006	0113	04	17	81	09	18	01	30	7	1	2	1	8084	0145	01	196	09	3

AIDED AND CASUALTIES: EVACUATED	FIRST AID	CIVILIANS: INJURED	KILLED	UNIFORMED INJURED	EXTINGUISHMENT: CODE	SPRINKLER NO. OPEN	STANDPIPE LINES USED	TERM. STAGE	IGNITION: EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	FACTOR	STRUCTURE: CONSTRUCT.	USE	STATUS	BLDG. DAMAGE	CONTS. DMGE.	AREA FIRE ORIGIN: FLOOR CODE	RM./AREA CODE	OCCUPANCY	MANNER OF	FIRE EXTENTION WITHIN BLDG.: OCCUPANC'S	OTHER: NO. BLDGS.	SMOKE DETECTORS	SPECIAL USE: 1	2	3
99	00	00	00	00	5	00	01	3	31	20	00	00	50	1	15	1	1	1	07	36	59	00	01	0	9			

ADDRESS

NUMBER	SUFFIX	PREFIX DIRECT	STREET NAME	TYPE	SUFFIX DIRECT	BUILDING STORIES	AREA
1			World Trade Center			99	300x300

NAME OF OCCUPANT: Port Authority N.Y., N.J.

ROOM, APT. NO.

LEFT IN CHARGE: Bill Sarneli F.S.D.

EQUIPMENT INVOLVED IN IGNITION: HVAC Unit

YR. | MAKE | MODEL | S/N | POWER

COS

UNIT	PTS.	ITEM							
5 010	10	5 006	10	5 007	10	7 001	10	7 015	10

OPERATIONS

Upon arrival at box 8084, was informed by PA police of fire in MER room on the 7th floor with smoke condition on 7 thru 11 floors, transmitted 2nd alarm.

Eng. 10 Masks, rolledups, responded to 7th floor, hooked up to standpipe, extinguished fire in air condition unit in MER room.

Eng. 6 Masks, rolledups, responded to 7th floor, stood fast.

Eng. 7 Masks, rolledups, responded to 7th floor, stood fast.

Lad. 1 Masks, responded to 7th. floor, V.S.O. air condition unit, checked for extention

Lad. 15 Masks, Vented and searched floors 8 thru 13, checked for extention in AC duct.

Lad. 6 Masks, under direction of BC Moffet, Bn. 4 searched floors 13 thru 18.

B.C. ________ BN. ________ D.C. ________ DIV. ________ PAGE 1 OF 2

TYPE FULL NAME | TIME OF ARRIVAL | TYPE FULL NAME | TIME OF ARRIVAL

ADMINISTRATIVE COMPANY

REPORT — Additional Data

Structural Fire, Transportation Fire, Non-Structural Fire or Emergency

FOR FIRE RECORDS USE ONLY	TYPE REPORT	MONTH	DAY	YEAR	CD. NO.	HOUR	MINUTE	BOROUGH	BOX NUMBER
	1	04	17	81	2	09	18	1	8084

COMPANIES TYPE NO. SECT. PTS.

OPERATIONS
continued

Rescue 1 Masks, searched floors 18 thru 23.

2nd. alarm units:
Engs. 24, 15, 55, 9, 241, and Marine 1 were orded to stand fast at there assigned locations.

Div. 1, D.C. DeCaprio responded and supervised opperations
Chiefs Gormley, Bishop, and Farrell also on scene.
P.A. personell reported evacuation of aprox, 1,500 persons from floors 9 thru 23.
F.C. unit on scene
F.P. 2 on scene.

	TIMES
Box	0918
2-2	0928
U/C	0959

B.C. Peter Costello

Peter Costello — TYPE FULL NAME

BN. 1 — 0920 — TIME OF ARRIVAL

D.C. Angelo DeCaprio — TYPE FULL NAME

DIV. — TIME OF ARRIVAL

PAGE 2 OF 2

ADMINISTRATIVE COMPANY

REPORT

BF-24 (1/80)

TYPE REPORT	COMMUNITY BOARD	ADMIN. COMPANY	BLOCK NUMBER	MONTH	DAY	YEAR	HOUR / MINUTE	HOURS : MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	REPORT NUMBER	FIRE BATTALION	BADGE NUMBER	ENG. SQ. MARINE	LADDER
1	01	5 006	0113	05	05	81	1025	01 : 05	7	1	0	1	8084	0036	01	352	03	2

(INCIDENT LOCATION; DATE AND TIME ALARM RECEIVED; INCIDENT DURATION; BOX LOCATION; REPORT; COS. RESP.)

EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	CODE	SPRINKLER NO. OPEN	STANDPIPE LINES USED	TERM. STAGE	EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	FACTOR	CONSTRUCT.	USE	STATUS	BLDG. DAMAGE	CONTS. DMGE.	FLOOR CODE	RM./AREA CODE	OCCUPANCY	MANNER OF	OCCUPANCIES	NO. BLDGS.	SMOKE DETECTORS	SPECIAL USE 1	2	3
00	00	00	00	00	5	00	01	3	98	00	72	37	21	1	15	1	1	2	02	21	59	00	01	0	0			

(AIDED AND CASUALTIES – CIVILIANS; EXTINGUISHMENT; IGNITION; STRUCTURE; AREA FIRE ORIGIN; FIRE EXTENTION WITHIN BLDG. / OTHER)

ADDRESS: NUMBER 1 — STREET NAME World Trade Center — TYPE 1

STORIES 94 — BUILDING 200X200

AREA Bill Sarnelli F.S.D. — LEFT IN CHARGE

P.A. N.Y. & N.J.

NAME OF OCCUPANT — ROOM, APT. NO.

EQUIPMENT INVOLVED IN IGNITION — YR. — MAKE — MODEL — S/N — POWER

CODS: UNIT 5 006 — PTS. 05 — ITEM 7 001 05

OPERATIONS

Batt-1 Responded to the W.T.C. , upon arrival we were informed of a small fire on the second floor of tower # 1 .

Eng-6 With masks and rolled ups reported to the second floor, where they operated a line from the standpipe on a fire in a storage closet, washed down and took up.

Lad-1 Vented and overhauled in the fire area.

Batt-1 Transmitted a signal 10-41 code 1.

B.C. Joseph A. Hingerton — BN. 1 — TIME OF ARRIVAL 1030

D.C. — TYPE FULL NAME — DIV. — TIME OF ARRIVAL — PAGE OF

ADMINISTRATIVE COMPANY

BF-24 (1/80)

REPORT – STRUCTURAL FIRE

Type Report	Community Board	Admin. Company	Block Number	Month	Day	Year	Hour	Minute	Hours	Minutes	How Reported	Initial Alarm	Highest Alarm	Borough	Box Number	Report Number	Filing Battalion	Badge Number	Eng. Sq. Marine	Ladder
1	01	5 010	0113	11	10	82	20	58	00	25	7	1	0	1	8087	0086	01	150	03	2

Groups: Incident Location; Date and Time Alarm Received; Incident Duration; Box Location; Report; Cos. Resp.

Evacuated	First Aid	Injured	Killed	Uniformed Injured	Code	Sprinkler No. Open	Standpipe Lines Used	Term. Stage	Equip. Involved	Heat Form	Material Type	Material Form	Factor	Construct	Use	Status	Bldg. Damage	Conts. Dmge.	Floor Code	Rm./Area Code	Occupancy	Manner of	Occupanc's	No. Bldgs.	Smoke Detectors	Special Use 1	2	3
00	00	00	00	00	5	00	01	3	98	40	60	75	31	1	15	1	0	0	99	14	47	00	00	0	0			

Groups: Aided and Casualties (Civilians); Extinguishment; Ignition; Structure; Area Fire Origin; Fire Extention (Within Bldg., Other); Special Use

ADDRESS

Number	Suffix	Prefix Direct	Street Name	Type	Suffix Direct
2			World Trade Center		I

BUILDING — Stories: 99

200 X 200
AREA

Port Authority
NAME OF OCCUPANT

ROOM, APT. NO.

Mr. Marchino Fire Safety director
LEFT IN CHARGE

EQUIPMENT INVOLVED IN IGNITION

YR.	MAKE	MODEL	S/N	POWER

COS

UNIT	PTS.	ITEM			
5 010 05	7 001 05				

OPERATIONS

Responded to smoke condition at ~~XXX~~ B-2 level. Building personnel reported heavy smoke condition. found fire in open shaft.

Engine 10- streched line from standpipe, and extinguished fire.

Ladder 1- searched , vented, checked for extension, overhauled.

B.C. Joseph L. Barracato (signature)
Joseph L. Barrasato
TYPE FULL NAME

1st BN.
20:59
TIME OF ARRIVAL

D.C.
TYPE FULL NAME

DIV.
TIME OF ARRIVAL

PAGE OF

ADMINISTRATIVE COMPANY

BF-24 (1/80)

REPORT — STRUCTURAL FIRE

TYPE REPORT	INCIDENT LOCATION			DATE AND TIME ALARM RECEIVED		INCIDENT DURATION	HOW REPORTED / INITIAL ALARM / HIGHEST ALARM / BOROUGH	BOX LOCATION		REPORT			COS. RESP.
	COMMUNITY BOARD	ADMIN. COMPANY	BLOCK NUMBER	MONTH / DAY / YEAR	HOUR / MINUTE	HOURS / MINUTES		BOX NUMBER	REPORT NUMBER	FILING BATTALION	BADGE NUMBER	ENG. SQ. MARINE	LADDER
1	01	5 010	0113	12-13-82	2038	00:30	6 1 0 1	8093	0102	01	150	03	2

AIDED AND CASUALTIES — CIVILIANS: EVACUATED / FIRST AID / INJURED / KILLED / UNIFORMED INJURED	EXTINGUISHMENT: CODE / SPRINKLER NO. OPEN / STANDPIPE LINES USED	IGNITION: TERM. STAGE / EQUIP. INVOLVED / HEAT FORM / MATERIAL TYPE / MATERIAL FORM / FACTOR	STRUCTURE: CONSTRUCT. / USE / STATUS / BLDG. DAMAGE	AREA FIRE ORIGIN: CONTS. DMGE. / FLOOR CODE / RM/AREA CODE / OCCUPANCY	FIRE EXTENTION: MANNER OF / OCCUPANC'S / NO. BLDGS. / SMOKE DETECTORS	SPECIAL USE 1 / 2 / 3
00 00 00 00 00	5 00 01	3 98 00 99 75 00	1 15 1 0	0 98 18 54	00 00 0 0	

ADDRESS — NUMBER	SUFFIX	PREFIX DIRECT	STREET NAME	TYPE	SUFFIX DIRECT
1			World trade Center		1

BUILDING STORIES: 99

AREA: 200x200

NAME OF OCCUPANT:

ROOM, APT. NO.:

LEFT IN CHARGE: Fire Safety Dir. Niedimeirer

EQUIPMENT INVOLVED IN IGNITION — YR. / MAKE / MODEL / S/N / POWER

COS — UNIT	PTS.	ITEM			
5 010 05		7 001 05			

OPERATIONS

Upon arrival box 8093 found following conditions at B-1 level, fire in dumpster and also on loading platform, Units operated viz:

E-10 stretched line from standpipe and extinquished fire

L-1 did necessary overhauling and venting

B.C. Joseph L Barracato BN. 1 D.C. ____ DIV. ____

Joseph L. Barracato — TYPE FULL NAME

2038 — TIME OF ARRIVAL

TYPE FULL NAME — TIME OF ARRIVAL

PAGE 1 OF 1

ADMINISTRATIVE COMPANY

ENG 10

BF-24 (1/80)-175M-608551 (83)

REPORT — STRUCTURAL FIRE

TYPE REPORT	COMMUNITY BOARD	ADMIN COMPANY	BLOCK NUMBER	MONTH	DAY	YEAR	HOUR	MINUTE	HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	REPORT NUMBER	FILING BATTALION	BADGE NUMBER	ENG. SQ. MARINE	LADDER
1	01	5 010	0113	01	04	84	16	25	00	20	2	1	0	1	8084	0028	01	572	03	2

(INCIDENT LOCATION; DATE AND TIME ALARM RECEIVED; INCIDENT DURATION; BOX LOCATION; REPORT; COS. RESP.)

EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	CODE	SPRINKLER NO. OPEN	STANDPIPE LINES USED	TERM. STAGE	EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	FACTOR	CONSTRUCT.	USE	STATUS	BLDG. DAMAGE	CONTS. DMGE.	FLOOR CODE	RM./AREA CODE	OCCUPANCY	MANNER OF	OCCUPANC'S	NO. BLDGS.	SMOKE DETECTORS
00	00	00	00	00	5	00	01	3	00	40	60	75	00	1	15	1	0	1	15	57	50	00	00	0	

(AIDED AND CASUALTIES — CIVILIANS; EXTINGUISHMENT; IGNITION; STRUCTURE; AREA FIRE ORIGIN; FIRE EXTENTION — WITHIN BLDG., OTHER; SPECIAL USE 1 2 3)

ADDRESS

NUMBER	SUFFIX	PREFIX DIRECT	STREET NAME	TYPE	SUFFIX DIRECT	BUILDING STORIES	AREA
1			World Trade Center			110	200x200

NAME OF OCCUPANT

ROOM, APT. NO.

FSD Norcia
LEFT IN CHARGE

EQUIPMENT INVOLVED IN IGNITION — YR. MAKE MODEL S/N POWER

COS	UNIT	PTS.	ITEM
	5 010	05	7 010 05

OPERATIONS

Upon arrival at box was informed of fire in service elevator room on the 15th floor.
Engine 10 extinguished fire.
Ladder 10 checked for extension and overhauled.
one sprinkler head was released.

B.C. Kenneth P Markgraf 1 — BN.

Kenneth P. Markgraf — TYPE FULL NAME

1628 — TIME OF ARRIVAL

D.C. ____ DIV. ____ TYPE FULL NAME ____ TIME OF ARRIVAL ____

PAGE 1 OF 1

ADMINISTRATIVE COMPANY

BF-24 (1/80) 175M-608551 (83)

REPORT — STRUCTURAL FIRE

TYPE REPORT	COMMUNITY BOARD	INCIDENT LOCATION: ADMIN. COMPANY	BLOCK NUMBER	DATE AND TIME ALARM RECEIVED: MONTH-DAY-YEAR	HOUR MINUTE	INCIDENT DURATION: HOURS MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX LOCATION: BOX NUMBER	REPORT NUMBER	REPORT: FILING BATTALION	BADGE NUMBER	COS. RESP.: ENG. SQ. MARINE	LADDER
1	01	5 010	0113	12-28-84	1435	00:25	6	1	1	1	8087	0269	01	150	01	1

AIDED AND CASUALTIES (CIVILIANS): EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	EXTINGUISHMENT: CODE	SPRINKLER NO. OPEN	STANDPIPE LINES USED	IGNITION: TERM. STAGE	EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	FACTOR	STRUCTURE: CONSTRUCT.	USE	STATUS	BLDG. DAMAGE	CONTS. DMGE.	AREA FIRE ORIGIN: FLOOR CODE	RM./AREA CODE	OCCUPANCY	MANNER OF	FIRE EXTENTION WITHIN BLDG.: OCCUPANC'S	OTHER: NO. BLDGS.	SMOKE DETECTORS	SPECIAL USE 1	2	3
00	00	00	00	00	5	00	01	3	00	40	06	75	21	1	15	1	0	1	23	35	59	00	00	0	1			

ADDRESS: NUMBER	SUFFIX	PREFIX DIRECT	STREET NAME	TYPE	SUFFIX DIRECT
2			WORLD TRADE CENTER		1

BUILDING: STORIES 110 — AREA 200' x 200'

NAME OF OCCUPANT ____ ROOM, APT. NO. ____

LEFT IN CHARGE: IRV SMALL, Fire Safety Director

EQUIPMENT INVOLVED IN IGNITION

YR.	MAKE	MODEL	S/N	POWER

COS: UNIT	PTS.	ITEM			
5 010 05	7 010 05				

OPERATIONS

Upon arrival found fire in the freight elevator lobby, 23rd floor.
Units operated as follows:
Engine 10 stretched line from the standpipe and operated line.
Ladder 10 V.E.S. performed necessary overhaul, used can on the fire.

CODE 10-41-CODE 1 Transmitted.

B.C. Joseph L. Barracato — BN. — D.C. ____ DIV. ____

JOSEPH L. BARRACATO (TYPE FULL NAME) — 1440 hrs (TIME OF ARRIVAL) — ____ (TYPE FULL NAME) — ____ (TIME OF ARRIVAL) — PAGE ____ OF ____

BF-24 (1/80)-175M-608551 (83) E-10

REPORT — STRUCTURAL FIRE

TYPE REPORT	COMMUNITY BOARD	ADMIN. COMPANY	BLOCK NUMBER	MONTH	DAY	YEAR	HOUR	MINUTE	HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	REPORT NUMBER	FILING BATTALION	BADGE NUMBER	ENG. SQ. MARINE	LADDER
1	01	5 010	0024	05	10	85	14	49	01	00	2	1	1	1	8084	0149	04	180	85	4

Group headings: INCIDENT LOCATION; DATE AND TIME ALARM RECEIVED; INCIDENT DURATION; BOX LOCATION; REPORT; COS. RESP.

EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	CODE	SPRINKLER NO. OPEN	STANDPIPE LINES USED	TERM. STAGE	EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	FACTOR	CONSTRUCT.	USE	STATUS	BLDG. DAMAGE	CONTS. DMGE.	FLOOR CODE	RM/AREA CODE	OCCUPANCY	MANNER OF	OCCUPANC'S	NO. BLDGS.	OTHER	SMOKE DETECTORS	1	2	3
00	00	00	00	00	5	00	01	0	00	40	00	75	24	1	45	1	0	0	38	31	81	00	00	0	6				

Group headings: AIDED AND CASUALTIES — CIVILIANS; EXTINGUISHMENT; IGNITION; STRUCTURE; AREA FIRE ORIGIN; FIRE EXTENTION — WITHIN BLDG.; SPECIAL USE

ADDRESS: NUMBER | SUFFIX | PREFIX DIRECT | STREET NAME: WORLD TRADE CENTER | TYPE | SUFFIX DIRECT: 1

BUILDING 110 STORIES

200 x200 AREA

NAME OF OCCUPANT ROOM, APT. NO.

Irv Small Fire Safety Director
LEFT IN CHARGE

EQUIPMENT INVOLVED IN IGNITION: YR. MAKE MODEL S/N POWER

COS	UNIT	PTS.	ITEM
	5	010	05
	7	010	05
	7	015	05

OPERATIONS

Upon arrival, advised by FSD that there was a dumpster fire in the hall on the 38th floor, extinguis[h]
prior to FD arrival. Once informed that all fire had been extinguished permission was granted to
building personnell to clear floor area of smoke by use of HVAC system. A 10-41-1 was transmitted f[or]
this box. Operations as follows: B4 set up lobby command post, monitored hard wire communications.
E 10 donned masks, stretched line from stanpipe and completed operations on fire floor.
L 10 " " , made search of 38th floor , overhauled as needed.
L 15 " " , made search of 39th, 40th, 41st floors, all found satisfactory.
All other 10-76 units stood fast at staging area.
10-41-1 transmitted.

B.C. Arthur V. Meadows BN. 4 D.C. [signature] DIV. 1 PAGE 1 OF 1

Arthur V Meadows — TYPE FULL NAME
1423 — TIME OF ARRIVAL
Thomas Kilker — TYPE FULL NAME
TIME OF ARRIVAL

BF-24 (I 79-12SM-808651 (83)

REPORT — STRUCTURAL FIRE

TYPE REPORT / COMMUNITY BOARD / INCIDENT LOCATION / DATE AND TIME ALARM RECEIVED / INCIDENT DURATION / HOW REPORTED / INITIAL ALARM / HIGHEST ALARM / BOROUGH / BOX LOCATION / REPORT / COS. RESP.

1 01 5 06 0024 08 06 85 1747 00 55 6 1 0 1 8084 0071 01 075 03 2

AIDED AND CASUALTIES / CIVILIANS / EXTINGUISHMENT / IGNITION / STRUCTURE / AREA FIRE ORIGIN / FIRE EXTENTION / SPECIAL USE 1 2 3

00 00 00 00 5 00 01 3 07 56 40 18 35 1 15 1 0 1 55 57 59 00 00 0 0

ADDRESS: NUMBER / STREET NAME: World Trade Center

BUILDING: 99

200 X200
AREA

NAME OF OCCUPANT: Port Authority

ROOM, APT. NO.

Irvin Small F.S.D.
LEFT IN CHARGE

EQUIPMENT INVOLVED IN IGNITION: YR. / MAKE / MODEL / S/N / POWER

GOS: UNIT / PTS. / ITEM: 010 10 010 10 5 001 10 5 006 05

OPERATIONS

Responded to ERS alarm ,report of smoke in building. As units were responding informed by dispatcher of numerous reports of smoke from 30 to 55th floor. After searching found cause of smoke to be coming from NER fan room on 55th floor. Fire was caused by welding operation resulten heat caused insulation to ignite on inside of fan room. Units operated viz.

Engine Co. 10 - Streched line from standpipe on 55th floor and extinguished fire.
Ladder Co. 10 - Searched ,overhauled, checked for extension.
Ladder Co. 1-- Searched , checked for extension.
Engine Co. 6 - Searched , maned command post/.

B.C. ~~Rudolph E Quatrone~~ 1st BN D.C. DIV. PAGE OF

Rudolph A Quatrone
TYPE FULL NAME
17:51
TIME OF ARRIVAL
TYPE FULL NAME
TIME OF ARRIVAL

ADMINISTRATIVE COMPANY

INCIDENT LOCATION | DATE AND TIME ALARM RECEIVED | INCIDENT DURATION | BOX LOCATION | REPORT | COMPANIES RESPONDING

1 01 5 010 04 28 89 2258 00 45 10 1 0 1 8093 0405 0503 01 150 03 02 01 09 000 0

AIDED AND CASUALTIES | CIVILIANS | EXTINGUISHMENT | IGNITION | STRUCTURE | FIRE ORIGIN | FIRE EXTENSION IN BLDG | DAMAGE | DETECTORS

00 00 00 00 00 5 0 1 3 98 39 97 34 99 0 1 591 1 59 97 50 42 00 01 0 1 2 2 0

BF-24

ADDRESS: NUMBER (INCL. HYPHEN) 1 — STREET NAME World Trade Center

BUILDING: STORIES 99 — AREA WIDTH 450 LENGTH 450

INELLCO
NAME OF OCCUPANT

ROOM/APT.

LEFT IN CHARGE

EQUIPMENT INVOLVED IN IGNITION — ITEM — YR. — MAKE — MODEL — SERIAL NUMBER — POWER

UNIT PTS.

W R I 5 007 10 5 010 02 7 010 10 7 015 05

OPERATIONS

esponded to WTC Complex and upon arrival found fire in 1WTC, B1 level in lockerroom at that location. Fire
onfined to lockers Nos. 131-132-133-151-152-153-154. Co's operated VIZ:
-7 stretched 2½"line to fire and extinguished all fire,washed down and [illegible]
-10 assisted E7 in stretching line and then stood fast
-10 found fire, performed necessary VES, forced entry into lockers, made primary and secondary searches
in fire room negative, overhauled and then up.
-15 checked fl. above for extension(Negative), also checked adjacent [illegible] and then up.

Joseph L. Barradat [illegible] 2302

BF-24

TYPE REPORT	INCIDENT LOCATION	DATE AND TIME ALARM RECEIVED	INCIDENT DURATION	HOW REPORTED / INITIAL ALARM / HIGHEST ALARM	BOX LOCATION	REPORT	COMPANIES RESPONDING	HAZ MAT
1	01 5 010 12	07 89 1313	01 00	81 1 7 1	8084	0104 0231 01 296	07 09 08 00	000 0

AIDED AND CASUALTIES (CIVILIANS)	EXTINGUISHMENT	IGNITION	STRUCTURE	FIRE ORIGIN	FIRE EXTENSION	DAMAGE	DETECTORS
00 00 04 00 00	0 1	3 98 24 97 19 54 0	1 591 1 59	07 50 61	00 01 0	1 1 2 1	1 1 2 1

ADDRESS: NUMBER (INCL. HYPHEN) 1 1 — STREET NAME: World Trade Center

BUILDING: STORIES 99 — AREA: WIDTH 450, LENGTH 450

DEFECTIVE DETECTOR: BRAND / SERIAL NO.

NAME OF OCCUPANT / ROOM/APT.

LEFT IN CHARGE F.S.D. Gene Raggio

EQUIPMENT INVOLVED IN IGNITION: ITEM / YR. / MAKE / MODEL / SERIAL NUMBER / POWER / BADGE NO.

UNIT	PTS.				
5 01	10	5 007 02	5 006 02	7 010 10	7 015 10

OPERATIONS

Upon arrival to Box 8084 found the following conditions. Light smoke condition in lobby and reported fire on the 7th fl., E10 officer transmitted 10-76 for fire in High rise building. Fire was confirmed on 7th floor in Machinery Equipment Room in air filters of air handling room. Cos. operated VIZ;
E10 stretched 2½ line to air handling room and operated on fire, washed down and then took up.
E-7 assisted E10 in stretching first line to fire floor and then stood fast, then took up.
E-6 responded to 6th floor with roll ups and stoodfast and then took up.
L-10 operated on 7th floor performed necessary search and notifications, searched 10th, 9th, 8th, floor results negative overhauled on fire floor and then up.
L-15 made search of 7th and 8th floor, performed necessary VESand then up.

BC Charles Callahan — Charles Callahan (TYPE FULL NAME) — BN. 1 — 1317 TIME OF ARRIVAL

DC Robert J. Blume — Robert J. Blume (TYPE FULL NAME) — DIV. 1 — 1330 TIME OF ARRIVAL

PAGE 1 OF 1

WHITE—FIRE RECORD YELLOW—BATTALION PINK—REFERRAL GOLD—ADMINISTRATIVE COMPANY

F-24A (12/86)

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT	DATE AND TIME ALARM RECEIVED (MONTH / DAY / YEAR / HOUR / MINUTE)	BOX LOCATION (BOROUGH / BOX NUMBER)
6	12 07 89 1313	1 8084 0231

ADDRESS: BORO 1 NUMBER (INCL. HYPHEN) 1 STREET NAME WTC

OCCUPANT (Driver) ROOM/APT.

IF MOBILE PROPERTY: TYPE YR. MAKE MODEL V.I.N. LICENSE STATE

UNIT PTS.: 7 008 10

OPERATIONS

L-8- search floors 9-40 results negative and then up.
B-6- established operations post on 6th floor., then went ~~to~~ 7th floor MER and directed operation of units. Supervised E10,E6,E4 and L15.
B-4- supervised operations of L15, L10, an L8 on floors above fire units conducted primary and secondary search of floors above, results negative.
Times 10-76 1314 - E10
Under control 1340 - Div 1
Injured Civilians - [redacted] -smoke inhalation
[redacted] smoke inhalation
taken to St. Vincents Hospital

[redacted] - smoke inhalation
[redacted] smoke inhalation
taken to Beekman

EMS Chief Cervo directed operations for EMS
PA operations Commander on scene Capt. Stinmer.

B.C. Charles Callahan 1 B.N. D.C. Robert J. Blume 1 DIV. PAGE 2 OF

Charles Callahan — TYPE FULL NAME — 1317 TIME ARRIVED

Robert J. Blume — TYPE FULL NAME — 1330 TIME ARRIVED

BF-24 (12/86)

REPORT — STRUCTURAL FIRE

BF-24

TYPE REPORT	INCIDENT LOCATION	DATE AND TIME ALARM RECEIVED	INCIDENT DURATION				BOX LOCATION	REPORT				COMPANIES RESPONDING			HAZ MAT		SPECIAL USE	REVISED REPORT
1	01 5 010	08 11 94 1317	02 00	10	1 7 1		8087	0149	0254	01	239	09	08	09	00	000	0 00 00 00	0

AIDED AND CASUALTIES — CIVILIANS	EXTINGUISHMENT	IGNITION	STRUCTURE	FIRE ORIGIN	FIRE EXTENSION	DAMAGE	DETECTORS
00 00 00 00 00	5 2	1 3 98 42 16 97 35 0	2 591 1 59	94 50 51	00 00	0 0 1 6	1 1 1 1 0

ADDRESS: NUMBER (INCL. HYPHEN): 1 2 — STREET NAME: WTC

BUILDING: STORIES 99 — AREA: WIDTH 0200 / LENGTH 0200

DEFECTIVE DETECTOR: BRAND — SERIAL NO.

P.A. — NAME OF OCCUPANT — ROOM/APT.

LEFT IN CHARGE: FSD NARCIA — BADGE NO.

EQUIPMENT INVOLVED IN IGNITION: YR. — MAKE — MODEL — SERIAL NUMBER — POWER

	UNIT	PTS.	UNIT	PTS.	UNIT	PTS.	UNIT	PTS.	UNIT	PTS.
WRI	5 006	20	5 007	12	5 004	12	5 055	07	5 005	07

OPERATIONS

Upon arrival at box 8087,2 WTC found rubbish fire in shaft on the 94th floor.Signal 10-76 was transmitted.Units operated as follows:
E6-Stretched line from standpipe,stood fast,then up.
E4-Operated houseline out of stairway "B"on the 94th floor and extinguished fire in utility shaft,then up.
E7-Assisted L10 in search of upper floors,then up.
E24-Stood fast at Lobby Command post,then up.
E5-Stood fast at Lobby Command Post,then up.

B.C. Lawrence M Byrnes — 01 BN. — D.C. — 1 DIV. — PAGE 1 OF 3

Lawrence M Byrnes (TYPE FULL NAME) — 13:21 TIME OF ARRIVAL — David Corcoran (TYPE FULL NAME) — 13:25 TIME OF ARRIVAL

WHITE—FIRE RECORD YELLOW—BATTALION PINK—REFERRAL GOLD—ADMINISTRATIVE COMPANY

24A (12/86)

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT	DATE AND TIME ALARM RECEIVED (MONTH / DAY / YEAR / HOUR / MINUTE)				BOX LOCATION (BOROUGH / BOX NUMBER)		INCIDENT NUMBER	BUILDING NO.	IGNITION (COND. ON ARRIVAL / EQUIP. INVOLVED / HEAT FORM / MATERIAL TYPE / MATERIAL FORM)	STRUCTURE (CONSTRUCTION / USE / STATUS)	AREA OF ORIGIN (FLOOR CODE / OCCUPANCY / AREA CODE)	FIRE EXTENSION (MANNER / OCCUPANCY)	REVISED REPORT
6	08	11	94	1317	1	8087	0254	00					

1 2 8 12 17 23 32 37 43 47

ADDRESS

BORO	NUMBER (INCL. HYPHEN)	SUFFIX	PREFIX DIRECT	STREET NAME	TYPE	SUFFIX DIRECTION
1	2			WTC		

57 68 89 93

SPK. PERFORMANCE	DAMAGE (% DAMAGED / FLAME / SMOKE / WATER)	DETECTORS (PRESENT / TYPE / POWER / PERFORMANCE)

48 52 94 OCCUPANT (Driver) 113 ROOM/APT.

BUILDING

STORIES	AREA WIDTH	AREA LENGTH

114 123

IF MOBILE PROPERTY

TYPE	YR	MAKE	MODEL	V.I.N.	LICENSE	STATE	DM

128 143 158

189	195	201	207	213	219	225	231
5 224 02	5 024 07	5 028 02	5 003 02	7 010 20	7 001 20	7 015 20	7 008 20
7 101 10	7 003 07	7 005 07	7 018 07	1 001 20			

237 UNIT PTS. 243 249 255 261 261 273 279

OPERATIONS

E3-Stood fast at the Lobby Command Post,then up.
E55-Stood fast at Lobby Command post,then up.
L10-Poerated on the 107th floor,opened various walls and doors,finding no fire or extention,then up.
L15-Assisted R1 in opening up a wall exposing fire in shaft,made necessary search and exam from 94 thru 99th floor(Termination point of shaft)Negative.
L8-Made necessary search and exam of floors 100 thru 106,found negatine.
L5-Stood fast at the Lobby Command Post,then up.
L18-Stood fast at the Lobby Command Post,then up.
L101-Searsh 75,91 and 92nd floor,found negative,then up.
L1-Made necessary search and exam of floors 104 thru 106,then up.
E224-Stood fast at the Lobby Command Post,then up.
R1-Used Thermal-Imaging camera and located fire in shaft on the 94th floor.
Batt4-Supervised operation on the fire floor,then up.
Batt8-Supervised operations on 107 and 74 floors,then up.
Batt2-Stood fast,then up.
FCU-Monitored handi-talkie circuit,progress reports,then up.

B.C. Lawrence M Byrnes 01 B.N.
Lawrence M Byrnes 13:21
TYPE FULL NAME TIME ARRIVED

D.C. [signature] 1 DIV.
David Corcoran 13:25
TYPE FULL NAME TIME ARRIVED

PAGE 2 OF 2

BF-24 (12/..) 530-8026 N

REPORT -- STRUCTURAL FIRE

TYPE REPORT	INCIDENT LOCATION	DATE AND TIME ALARM RECEIVED	INCIDENT DURATION	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX LOCATION	REPORT	COMPANIES RESPONDING	HAZ MAT	HEAT EQUIP. FUEL	SPECIAL USE 1	2	3	REVISED REPORT
1	01 5 010	08 29 99 0747	01 00	8	1	7	1	8084	0320 0128 01 021	03 02 01 00	000	0	0	0	0	

AIDED AND CASUALTIES	EXTINGUISHMENT	IGNITION	STRUCTURE	FIRE ORIGIN	FIRE EXTENSION IN BLDG.	DAMAGE	DETECTORS
00 00 00 00 00	5 0 1	3 98 31 67 78 31 0	1 59 1 1 59	98 81 55 00	01 1 1	1 6 1	1 1 1 4 0

BF-24

ADDRESS: BORO 1 1 — STREET NAME: WORLD TRADE CENTER

BUILDING: STORIES 99 — AREA WIDTH 200 / LENGTH 200

DEFECTIVE DETECTOR: BRAND — SERIAL NO.

NAME OF OCCUPANT: PORT AUTHORITY

ROOM/APT.

LEFT IN CHARGE: BOWSE

BADGE NO.: FSD

EQUIPMENT INVOLVED IN IGNITION: YR. MAKE MODEL SERIAL NUMBER POWER

WRI — UNIT PTS ITEM: 7 010 10 5 010 10 7 001 10 5 007 10 5 006 02

OPERATIONS

Upon arrival at box 8084 found rubbish fire in utility shaft at the B-1 level which dropped down to the B-4 level. Units operated as follows:

L-10 Gained access to the B-1 level, performed search, found fire in shaft, performed overhauling.

E-10 Took hydrant, stretched line to siamese, stretched line from standpipe to utility shaft, extinguished fire at the B-1 level.

L-1 Assisted L-10, performed search of areas above and adjacent to fire, found fire in the shaft at the B-4 level, gained access for stream operation, performed overhauling.

E-7 Took hydrant, assisted E-10 in stretching line on th B-1 level, stretched line from standpipe on B-4 level and extinguished fire that had dropped down from fire on B-1 level.

B.C. [signature] — 01 B.N. — D.C. — D.V.

William Dehm — TYPE FULL NAME — 07:51 TIME OF ARRIVAL — N/A TYPE FULL NAME — 00:00 TIME OF ARRIVAL

PAGE 1 OF 2

ADMINISTRATIVE COMPANY

BF-24A (12/86) 15/H2-861531-R028-N

REPORT -- ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT | DATE AND TIME ALARM RECEIVED: MONTH, DAY, YEAR, HOUR, MINUTE | BOX LOCATION: BOROUGH, BOX NUMBER | INCIDENT NUMBER | BUILDING NO. | COND. ON ARRIVAL | IGNITION: EQUIP. INVOLVED, HEAT FORM, MATERIAL TYPE, MATERIAL FORM | STRUCTURE: CONSTRUCTION, USE, STATUS | AREA OF ORIGIN: FLOOR CODE, OCCUPANCY, AREA CODE | FIRE EXTENSION: MANNER, OCCUPANCY | REVISED REPORT

6 08 29 99 07 47 1 8084 0128 00

1 2 8 12 17 23 32 37 43 47

ADDRESS: BORO | NUMBER (INCL HYPHEN) | SUFFIX | PREFIX DIRECT | STREET NAME | TYPE | SUFFIX DIRECTION

1 1 WORLD TRADE CENTER

57 68 88 93

SPK. PERFORMANCE | DAMAGE: % DAMAGED, FLAME, SMOKE, WATER | DETECTORS: PRESENT, TYPE, POWER, PERFORMANCE

48 53

94 OCCUPANT (Driver) 113 ROOM/APT.

BUILDING: STORIES | AREA: WIDTH, LENGTH

114 123

IF MOBILE PROPERTY: TYPE YR. 128 MAKE 143 MODEL 158 V.I.N. LICENSE STATE DM

189 195 201 207 213 219 225 231

237 UNIT PTS. 243 249 255 261 261 273 279

OPERATIONS

E-6 Took hydrant, assisted E-7 in stretch.

B.C. William Oehm 01 B.N.

D.C. DIV.

PAGE 2 OF 2

William Oehm — TYPE FULL NAME — 07:51 TIME ARRIVED

N/A — TYPE FULL NAME — 00:00 TIME ARRIVED

Attachment G-A.7

Significant fires occurring in WTC 7 (1)

Significant Fire	Incident Date	Fire Location	# sprinklers activated	# standpipes activated	Cause of fire	Material Ignited
1	5/20/88	Construction shanties on floor 3	Multiple, # not listed	1	Suspicious	Shanties

BF-24 (12/86)

REPORT — STRUCTURAL FIRE

TYPE REPORT	COMMUNITY BOARD	ADMIN. COMPANY	MONTH	DAY	YEAR	HOUR	MINUTE	HOURS	MINUTES	HOW REPORTED	INITIAL ALARM	HIGHEST ALARM	BOROUGH	BOX NUMBER	REPORT NUMBER	INCIDENT NUMBER	FILING BATTALION	BADGE NUMBER	ENG. SQ. MARINE	LADDERS RES. H.M.	OTHERS	CLASS	AMOUNT	HEAT EQUIP./FUEL	1	2	3	REVISED REPORT
1	01	5 010	05	20	86	00	38	01	30	3	2	1	2	1 0084	0256	0008	01	075	13	11	08							

INCIDENT LOCATION; DATE AND TIME ALARM RECEIVED; INCIDENT DURATION; BOX LOCATION; REPORT; COMPANIES RESPONDING; HAZ MAT; SPECIAL USE

EVACUATED	FIRST AID	INJURED	KILLED	UNIFORMED INJURED	CODE	SPK. PERF.	STP. PERF.	COND ON ARRIVAL	EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	FACTOR	JUVENILE	CONSTRUCT	USE	STATUS	COMPLEX	FLOOR CODE	OCCUPANCY	AREA OF ORIGIN	MANNER	BLDG. PARTIES	NO. BLDGS	% DAMAGED	FLAME	SMOKE	WATER	PRESENT	TYPE	POWER	PERFORMANCE	FOR INVESTIGATION
00	06	00	00	00	5	1	1	3	98	97	97	97	21	0	1	591	1	59	03	50	39	00	99	0	1	6	6	6	1	1	2	1	1

AIDED AND CASUALTIES (CIVILIANS); EXTINGUISHMENT; IGNITION; STRUCTURE; FIRE ORIGIN; FIRE EXTENSION (IN BLDG.); DAMAGE; DETECTORS

BF-24

ADDRESS

BORO	NUMBER (INCL. HYPHEN)	SUFFIX	PREFIX DIRECTION	STREET NAME	TYPE	SUFFIX DIRECT
1	7			WTC		

BUILDING

STORIES	AREA WIDTH	AREA LENGTH
48	0400	0400

DEFECTIVE DETECTOR

BRAND	SERIAL NO.

NAME OF OCCUPANT ____ ROOM/APT. ____

LEFT IN CHARGE Sec. Officer Marrero BADGE N

EQUIPMENT INVOLVED IN IGNITION

YR.	MAKE	MODEL	SERIAL NUMBER	POWER

ITEM

	UNIT	PTS											
W R I	5 E 010	20	5 E 007	12	5	004	15	5	006	12	5	003	05

OPERATIONS

Upon arrival at Box 84 found fire in construction shanties on the 3rd floor. Transmitted 10:76 and 10:41-1.Units operated VIZ:

E10-Stretched line from standpipe and extinguished fire.

L10-Found sprinklers ~~and~~ in operation.Shut down sprinklers and made necessary search,ventilation and overhauled as required.

E7-assisted E10.

E4-Stretched line to ~~X~~ 4th floor.Stood fast.

E6-assisted E4.

B.C. Rudolph S. Quatrone (signature) Bn-1 BN. 0043

Rudolph S Quatrone

TYPE FULL NAME TIME OF ARRIVAL

D.C. ____ DIV. ____

TYPE FULL NAME TIME OF ARRIVAL

PAGE 1 OF

WHITE—FIRE RECORD YELLOW—BATTALION PINK—REFERRAL GOLD—ADMINISTRATIVE COMPANY

BF-24A (12/86) 151128-86153l-R028-N

REPORT — ADDITIONAL DATA

STRUCTURAL FIRE, TRANSPORTATION FIRE, NON-STRUCTURAL FIRE OR EMERGENCY

TYPE REPORT	MONTH	DAY	YEAR	HOUR	MINUTE	BOROUGH	BOX NUMBER	INCIDENT NUMBER	BUILDING NO.	COND. ON ARRIVAL	EQUIP. INVOLVED	HEAT FORM	MATERIAL TYPE	MATERIAL FORM	CONSTRUCTION	USE	STATUS	FLOOR CODE	OCCUPANCY	AREA CODE	MANNER	OCCUPANCY	REVISED REPORT
6	05	20	88	00	36	1	0084	0008		3	9B	97	97	97	1	591	1	03	50	35	00	99	

ADDRESS

BORO	NUMBER (INCL. HYPHEN)	SUFFIX	PREFIX DIRECT	STREET NAME	TYPE	SUFFIX DIRECTION
1	7			WTC		

SPK. PERFORMANCE	% DAMAGED	FLAME	SMOKE	WATER	PRESENT	TYPE	POWER	PERFORMANCE
1	1	1	6	6	1	1	2	1

OCCUPANT (Driver) | ROOM/APT.

BUILDING

STORIES	AREA WIDTH	AREA LENGTH
48	0400	0400

IF MOBILE PROPERTY: TYPE YR. MAKE MODEL V.I.N. LICENSE STATE

189	195	201	207	213	219	225	231
7 010 20	7 001 20	7 008 20					

237 UNIT PTS. 243 249 255 261 261 273 279

OPERATIONS

Operations as follows;
L8-Assisted L10 Made primary and secendory search of 5-10,11,12.Then up.
L1-Made search and examination of the 4th floor.Then assisted L8 in the examination of upper floors.Then up.
Batt 4-Supervised E10 and L10 on fire floor.
Batt 2-Supervised E4 and E6 and L1 on floors above fire.
FCU- SOP
MSU- SOP
E3- Stood fast.
Batt 6- Stood fast.

B.C. Rudolph S. Quatrone Bn-1 B.N. D.C. DIV.

Rudolph S Quatrone — TYPE FULL NAME; 0043 — TIME ARRIVED; TYPE FULL NAME; TIME ARRIVED

PAGE 2 OF

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Attachment G-B

PACO 2002 Report: World Trade Center General Description of All Building Systems and the Capital Program. Extracted page.

Miscellaneous Life Safety Improvements and Sprinklerization Program

12. Miscellaneous Life Safety Improvements and Sprinklerization Program

12.1 Description of the Program

The initial base building provided for Fire Standpipe (FSP) protection in the Towers and Plaza Buildings and no sprinkler system installation (except in the sub-grade Levels). In response to the enactment of Local Law 5 and other NYC Building Code Local Law enactments related to the fire protection of high rise office towers built after 1973 the Port Authority voluntarily retrofitted the WTC complex to comply with the new NYC Building Code Requirements supplemented by PA imposed life safety requirements unique to the WTC. These requirements provided tenants the option of achieving fire protection compliance by a compartmentalization or a sprinklerization option. The Scope of miscellaneous life safety and sprinkler system upgrades/improvements, other than certain fire alarm, concourse circulation and blast recovery improvements discussed elsewhere, included but was not limited to:

12.2 Scope of Fire Life Safety Improvements

Architectural Modifications - On all floors:

- Upgraded core wall construction to provide for a 2 hour rated fire separation from one side of a tower floor to the opposite side (Where core wall were already 2 hour rated, such as in elevator shafts, upgraded where not previously required).
- Installed a double acting set of rated HM fire doors in the core corridors where the above 2 hour rated fire separation crossed the corridor (This enabled one half of a typical tower floor to serve as a horizontal fire refuge for the other.) Doors included special magnetic hold open hardware and closures linked to smoke detectors in floor return air ducts.
- Restored all ceilings in corridors and lobbies affected by fire protection installations.
- Provided slab-to-slab 1hour fire rated walls to enable a maximum of 10,000 gross square foot compartmentalization on each side of the 2 hour rated fire separations.

Sprinkler System Installation

- The entire WTC complex was fully sprinklered. The Sprinkler System was installed in three basic phases. Phase 1: Sub-grade areas for the initial building construction; Phase 2: Sprinkler riser/main installation throughout 1 & 2 WTC including the sprinklerization of corridors, storage rooms, lobbies and certain tenant/PA spaces in 1976 in compliance with Local Law 5 to provide tenants with a compartmentation or sprinkler fire protection option; Phase 3: The full sprinklerization of the entire complex for all remaining places from 1983 to the 2001(including 1993 blast recovery and ongoing up-grades/improvements and replacements).
- The Tower sprinkler systems were fed in the various zones from the gravity feed fire reserve tanks located on the 20th, 41st,75th and 110th Floors through a 4" vertical sprinkler riser located in the Janitor Closet on each floor of each building in the WTC complex.

EXP 013001

CONFIDENTIAL

CONFIDENTIAL SILV 55 000020

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Appendix H
Interim Report on Evolution of WTC Fires, Smoke, and Damage Based on Image Analysis

H.1 COLLECTION AND ANALYSIS OF VISUAL MATERIAL

Photographic and video images of damage and fires in the World Trade Center (WTC) towers and WTC 7 are critical for guiding the investigation led by the National Institute of Standards and Technology (NIST). The conditions of the towers immediately following the plane strikes, the rates of fire development and spread through the buildings, and indications as to the floors on which the structural collapses may have begun and their causes are examples of issues that are being addressed using imagery. Observations discussed below demonstrate the importance of such visual evidence.

This appendix is designed to provide an update on NIST efforts to collect and analyze visual material available for the WTC disaster. This effort is part of Project 5, Reconstruction of Thermal and Tenability Environment, and this is the focus of the material presented. It is important to recognize that the effort is coordinated with the other projects that form the NIST-led WTC Investigation, and the visual material is being used as the basis for additional analysis in these projects.

The amount of visual material recorded on September 11, 2001, was extraordinary. The terrorist attacks occurred in an area that is the national home base of several news organizations, has several major newspapers, and is the center of the fashion industry. As a result, there were likely hundreds of professional photographers and videographers equipped with excellent equipment and the knowledge to use it in the immediate area. New York City is also a major tourist destination, and visitors often carry cameras to record their visits.

The WTC towers (WTC 1 and WTC 2) were immense, and they dominated the New York City skyline. When WTC 1 was struck by American Airlines Flight 11 around 8:46 a.m., the approach of the plane was captured by at least two videographers who were coincidentally filming nearby. Other photographers and videographers in the vicinity began recording within a few seconds of the impact. As fires grew in the tower, smoke pouring from the building formed a plume that could be seen for miles in all directions in the clear air of September 11, 2001. People in Manhattan, Brooklyn, Queens, and New Jersey began to turn their cameras toward the WTC complex. The major news organizations began coverage almost immediately and began moving professionals into position to cover the event. Numerous other videographers and photographers, both professional and amateur, started moving toward the WTC in order to create their own visual records.

At the time United Airlines Flight 175 struck WTC 2, around 9:03 a.m., the approach and collision of the aircraft were recorded by numerous cameras from a variety of directions. Many people continued to record images until WTC 2 collapsed, around 9:59 a.m. Following this collapse, the amount of visual material decreased markedly as people rushed to escape the area and the huge dust clouds generated by the collapse obscured the site. This situation was only exacerbated by the collapse of WTC 1, around

10:28 a.m. The visual record between the period following the collapse of WTC 1 and the collapse of WTC 7, around 5:21 p.m., is much less complete, but there is still a substantial amount of material.

Even as the disaster unfolded, it was clear that a large amount of visual material was being recorded that was being used to inform the public, demonstrate the immensity of the disaster, and to chronicle the associated human suffering. It is now clear that the imagery of September 11, 2001, is the most extensive ever recorded of such a single tragic event. The resulting visual record offers an unparalleled opportunity to contribute to the technical understanding of the tragedy of September 11. Even though it was clear that an extensive visual record of the events of September 11 existed, approaches for obtaining access to photographs and videos and cataloging the material had to be developed. These critical aspects of the task have required a great deal of time and effort.

H.1.1 Sources

Potential sources of visual material have been identified in a number of ways. Recordings of newscasts from September 11, 2001, and afterwards, documentaries and other remembrances, provided information directly, but also pointed toward other potential sources of material. The major photo clearinghouses, such as AP, Reuters, and Corbis, have World Wide Web sites that were reviewed for material related to September 11. Several members of the media suggested sources. Several collections of visual material have been assembled for charitable or historical purposes. Collections from the *Here is New York City* exhibition and the *September 11 Digital Archive* were reviewed. Many photographs and videos began appearing on the Web as early as September 11. These could often be identified by Web searches, and in many cases contact information was provided. Public appeals for visual material were made during Investigation news conferences and updates. News accounts of these events led many to contact NIST using the toll-free number or the Investigation Web site. Frequently, a new source would provide information about other potential sources.

NIST hired a visual media consultant, Mr. Valentine Junker, to act as its representative in the New York City area. In addition to interacting with a number of individuals, his efforts were particularly valuable in interfacing with the major television networks and local New York City stations as well as the major photographic news services.

H.1.2 Procedures

The identification of sources was only the first step in the collection process. It was then necessary to contact the source, request the material, and make arrangements for its transfer. Special considerations such as copyright and privacy issues often needed to be addressed. Once an agreement was reached, arrangements were made to review and transfer copies of the material to NIST.

In the collection process, emphasis has been placed on obtaining material in a form that is as close as possible to the original in order to maintain as much spatial and timing information as possible. In the case of digital photographs and videos this implies a direct digital copy. For film or slide photographs, it would be a high-resolution digitized version of the original media, and for analog video, a direct copy from the original source. While it was not always possible to maintain this standard, the majority of material ultimately collected was handled in this manner.

H.1.3 Contents

Significant progress has been made in collecting visual material related to September 11, 2001. Thus far, in excess of 150 hours of video have been assembled. At the time of preparation of this update, video footage has been provided by NBC, CBS, ABC, CNN and local New York City stations WABC, WCBS, WNBC, WPIX, WNYW and New York City One. In many cases, the videos provided not only include material broadcast (known as air checks), but also material that was recorded but not broadcast (known as outtakes). Additionally, videotapes recorded by more than 20 individuals have been received.

Photographs have been provided by a number of sources dominated by commercial photo services, the New York City Police Department (NYPD), the New York City Fire Department (FDNY), and individuals. Well in excess of 6,000 photographs, representing more than 185 photographers, have been received. Professional news organizations that have provided material include AP, Corbis, Reuters, the *New York City Times*, the *Daily News*, and the *Star Ledger*. As for the videos, many of these organizations have provided access to unpublished photographs. The majority of photographs have come from individual photographers, both professional and amateur.

It is difficult to estimate the actual amount of relevant visual material recorded on September 11, 2001, and thus to estimate how complete the collection efforts have been. There is certainly material that has not been identified and collected. However, NIST believes that the extraordinarily large collection of video material that it possesses is sufficient for the Investigation.

H.2 DATABASING AND CATALOGING

It would be impossible to effectively use the vast amount of visual material collected for the Investigation without some means of organizing and cataloging the material.

H.2.1 Digital Storage

Very early in the task, the decision was made to save all material in digital format on large digital data storage devices. This approach has several advantages. Because the material is in digital form, it can be assessed quickly. It is not necessary to search for a particular photographic collection or videotape, and no special equipment is required to display it. Because most material is received in other forms, the digital storage is in effect a backup system for the original. Additional redundancy is provided by backing up the entire digital storage system at regular intervals. Because videos are saved digitally, they can be analyzed using a variety of commercially available editing software.

Various storage solutions were considered. An approach was finally adopted in which a central server along with two 325 gigabyte and one 160 gigabyte external hard drives were connected with eight personal computers equipped with 70 gigabyte hard drives. The personal computers not only provide additional disk storage, but also serve as workstations for data entry and analysis. All of the systems are connected by high-speed ethernet to form a single network configured such that the entire system becomes, in effect, a single mass storage device. The total amount of storage available is roughly 1.4 terabytes.

Due to security concerns related to the sensitive nature of some of the visual material and copyright issues, the computer network has been set up with its own dedicated connections and is isolated from the internet backbone of NIST. Policies have been adopted that require all viewing and analysis of the material to be done in secured rooms using secured networks.

H.2.2 Digitizing Techniques

When new visual material is received at NIST, it is stored digitally on the dedicated system. If the material is already in digital form this simply means copying and saving it on the system. Analog material must be first digitized in some manner. For instance, a photograph might be scanned and digitized, or a video might be converted to a digital video format (typically mini-DV) and then copied to a hard disk.

Each arriving video is logged into VideoList, a Microsoft Access database written specifically for this application. There it is assigned a unique identification number. Pertinent information concerning the tape is recorded, including its duration, the network and broadcast date if applicable, its physical format (e.g., VHS, Hi-8, or mini-DV), where the tape is stored, whether the tape is an original or a copy, its source, whether it has been digitized, whether it contains embedded timecode, and general notes on its content. Figure H–1 shows an example of the entry sheet for the VideoList database. Videos to be stored digitally are copied onto mini-DV media, and each copy is also logged into the database. VideoList also contains a calculator for assisting in the calculation of clip timing that is described in H.3.1. Selected video material is then transferred to hard disk for storage. Video material is often found to have natural breaks, such as when the camera is turned off and on (e.g., by an individual videographer) or when multiple cameras are used (e.g., during a newscast). It is advantageous to treat each of these breaks as the end of an individual video. This is accomplished by a process known as “clipping.” By using Adobe Premiere software and a personal computer to control the video player, it is possible to identify and note such breaks in a “clip file.” The clip file can also contain notes related to the material. Once a clip file has been generated for an entire tape, the software goes through and automatically generates multiple data files containing the video material. The material is stored in “avi” format, which maintains all of the digital information. The maximum video file size that can be handled by this system is 1 gigabyte. This corresponds to slightly more than 4 1/2 min of avi video. Longer continuous video segments are broken into lengths having roughly this period. Breaking longer videos up in this manner also makes them easier to search and catalog.

H.2.3 Searchable Database

As noted earlier, a vast amount of visual material has been collected and saved digitally as part of the investigation. Without some organization, it would be impossible to use this material effectively. A commercial database program written specially for organizing visual material, Cumulus, was chosen for this purpose. This software is designed to collect individual “assets” in specified catalogs and to allow the assets to be characterized with a variety of attributes. It is possible to generate specific attributes and include these in specially designed forms for data entry. Once a catalog has been assembled, it is possible to search for assets having a specific attribute or combinations of attributes. Quite sophisticated searches can be created. It is also possible to order assets based on a particular attribute. As an example, when dates and times are assigned, the assets can be ordered in chronological order.

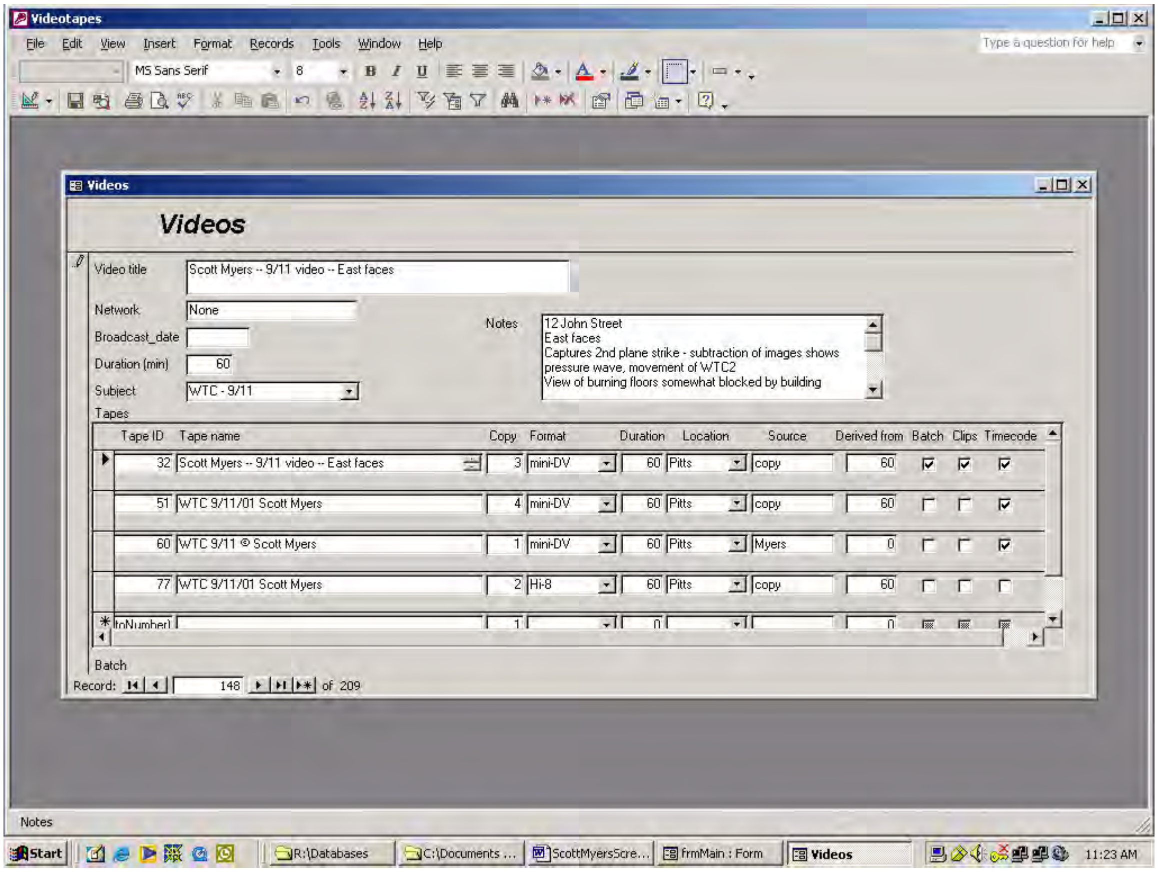


Figure H–1. An example of the VideoList data entry sheet for video assets.

Two separate catalogs, one for photographs and one for video clips, have been created for visual materials collected as part of the Investigation. Each catalog has a similar set of attributes that is used to characterize the assets that are included. These attributes were chosen based on the needs of this task dealing with fire conditions within WTC 1, 2, and 7 and by consultation with members of other project teams. Tables H–1 and Table H–2 list the attributes used for photographic and video catalogs, respectively. A description of each attribute is provided along with details on how information concerning the attribute is input into the worksheet. Figure H–2 shows an example of the first screen for the photographic data entry form.

Cumulus allows thumbnails of entire catalogs or selected subsets to be displayed. This makes it possible to review large numbers of photographs and video clips quickly and to decide which are most likely to be useful for a particular purpose. A variety of asset characteristics can also be shown simultaneously. Typically, the asset name and the time the asset was recorded are displayed. Figure H–3 shows an example of thumbnails taken from the video database.

Table H–1. Attributes for photographic assets.

Attribute	Definition	Entry Choice
Asset Reference	Location of photograph in file system	Set by Cumulus
Categories	List of categories under which the photograph is listed, typically the photographer's name or source	Set by Cumulus
Record Name	File name of photograph	Set by Cumulus
Photographer	Photographer's name	Text
Received from	Where photograph was obtained (“Other” may refer to a third party, for example)	Photographer WWW Other
Original Source	How photograph was added to the collection	Digital Copy of Original Digital Copy from Program Digitized Slide or Negative Digitized Photograph Uploaded from Web
Use Limited	Photographer has requested that use of the photograph be limited	Checkbox
Copyright	A copyright exists	Checkbox
Copyright Agreement	Usage agreement with NIST	Text
Shot From	Location of photographer	Text
Date Recorded	Date and time of shot	Date and time
Time Uncertainty (s)	Number of seconds uncertainty in the time assigned	Integer
View Direction	Location of photographer with respect to the WTC	North Northeast East Southeast South Southwest West Northwest
WTC Faces WTC 1 North Face WTC 1 East Face WTC 1 South Face WTC 1 West Face WTC 2 North Face WTC 2 East Face WTC 2 South Face WTC 2 West Face WTC 7 North Face WTC 7 East Face WTC 7 South Face WTC 7 West Face	Building face(s) visible in the photograph	Checkbox for each choice

Attribute	Definition	Entry Choice
Distance Near Medium Far	Clarity of the photograph Near = Can make out details in windows Medium = Can count windows Far = Unable to count windows	Checkbox for each choice
Building WTC 1 WTC 2 WTC 7 Other Building	Building(s) visible in photograph	Checkbox for each choice
1st Plane Strike	Photograph shows the plane strike on WTC 1	Checkbox
2nd Plane Strike	Photograph shows the plane strike on WTC 2	Checkbox
WTC 1 Collapse	Photograph shows the collapse of WTC 1	Checkbox
WTC 2 Collapse	Photograph shows the collapse of WTC 2	Checkbox
WTC 7 Collapse	Photograph shows the collapse of WTC 7	Checkbox
Street	Street scene, or a street is visible in the photograph	Checkbox
Debris Aircraft Debris Collapse Debris Debris Inside Building Street Debris	Debris is visible in the photograph Type of debris: Aircraft = Can be identified as plane debris (e.g., tires, engines) Collapse = Resulting from collapse Inside Building = Visible through windows or openings Street = On street	Checkbox for each choice
Fireball	Initial fireball from plane strike is visible	Checkbox
Thermal	The thermal is a tall region of the smoke plume that results from the lift caused by the hot gases of the initial fireball	Checkbox
Plume	Smoke plume generated by the fires within the towers and blown downwind. This marker is checked if the smoke plume in the photograph extends farther than a single tower width.	Checkbox
Flames Visible	Flames are visible in the photograph	Checkbox
People Inside Falling Outside	The photograph includes people Inside = People inside the buildings, at the windows or climbing down Outside = People on the street	Checkbox for each choice
Falling building component	The photograph shows a building component falling (e.g., aluminum cladding)	Checkbox
Streamers Falling	The photograph shows a streamer, an object that emits smoke as it falls and leaves a trail	Checkbox
Dripping	Molten material dripping from WTC 2 is visible	Checkbox
Hanging Floor	A sagging or hanging object suggesting a floor is visible within the windows	Checkbox

Attribute	Definition	Entry Choice
Building Core	Photograph shows the core of WTC 1 or WTC 2 — both remained standing briefly during collapse before falling	Checkbox
FDNY FDNY Apparatus FDNY Personnel	FDNY personnel or vehicles are visible, including EMTs, fire trucks, and ambulances	Checkbox for each choice
NYPD NYPD Apparatus NYPD Personnel	NYPD personnel or vehicles are visible, also includes FBI and other police officials	Checkbox for each choice
Impact Aircraft	Photograph shows aircraft approaching WTC 1 or WTC 2 before or during the strike	Checkbox
Other Aircraft	Aircraft other than the impact aircraft are included in the photograph, such as helicopters or fighter jets	Checkbox
Good for Analysis	Mark photograph for possible window-by-window analysis	Checkbox
Analyzed	The photograph has been used for window-by-window analysis	Checkbox
Notes	Notes, including a description of how the photograph was timed	Checkbox

Not all collected visual material is incorporated into the two catalogs. Photographs and videos judged not to contain information directly relevant to the Investigation are not included. Even so, the number of photographs and clips included in the catalogs is huge. At the time of writing, the photographic catalog includes 6,759 assets and the video catalog includes 6,911 assets.

H.3 TIMING OF PHOTOGRAPHS AND VIDEO CLIPS

Since one of the major goals of this task is the development of time lines for fire growth and spread in WTC 1, 2, and 7, it is important to assign times of known accuracy to assets included in the two catalogs. This task is complicated by the absence of accurate times for the majority of visual materials collected.

H.3.1 Digital Timestamps

Modern photographic and video digital cameras often record camera clock times as part of their output. For photographs, this information is usually stored as an integral part of the image in a header known as an Exif file. Similarly, digital video cameras often embed a variety of information, including the camera clock time, as part of what is known as meta data. Software is available for reading these clock times from Exif and other meta data media file formats. While a great help, these times usually still require some adjustment because people do not generally set their camera clocks accurately. In some cases, camera clocks were off by days or even years. Even so, the relative times over the short time period of the events of September 11, 2001, are quite accurate.

Table H–2. Attributes for video assets.

Attribute	Definition	Entry Choice
Asset Reference	Location of video clip in the file system	Set by Cumulus
Categories	List of categories under which the video clip is listed, typically the photographer's name or source	Set by Cumulus
Record Name	File name of video clip	Set by Cumulus
Photographer	Photographer's name	Text
Content	Content of video clip WTC 9/11 Footage = Events before collapse of WTC 7 Street Scene (no timing) Debris field = Ground Zero after WTC 7 collapse Construction = Construction of WTC towers from documentary Normal Operation = Normal operation of building, usually from documentary Animation = Animation of 9/11 events from documentary Still(s) = Photographs contained within documentary Interview = Clip only shows interview	WTC 9/11 Footage Street scene (no timing) Debris field Construction Normal operation Animation Still(s) Interview
Use Limited	Videographer has requested that use of the videotape be limited	Checkbox
Copyright	A copyright exists	Checkbox
Copyright Agreement	Usage agreement arrangements with NIST	Text
Shot From	Location of videographer	Text
Date Recorded	Date and time of beginning of video clip	Date and time
End Recording	Date and time of end of video clip	Date and time
Duration	Number of minutes:seconds contained in clip	Real number
Time Uncertainty (s)	Number of seconds uncertainty in the time recorded / end recording	Integer
View Direction	Location of videographer with respect to the WTC	North Northeast East Southeast South Southwest West Northwest

Attribute	Definition	Entry Choice
WTC Faces WTC 1 North Face WTC 1 East Face WTC 1 South Face WTC 1 West Face WTC 2 North Face WTC 2 East Face WTC 2 South Face WTC 2 West Face WTC 7 North Face WTC 7 East Face WTC 7 South Face WTC 7 West Face	Building face(s) visible in the video clip	Checkbox for each choice
Distance Near Medium Far	Clarity of the video clip Near = Can make out details in windows Medium = Can count windows Far = Unable to count windows	Checkbox for each choice
Building WTC 1 WTC 2 WTC 7 Other Building	Building(s) visible in video clip	Checkbox for each choice
1st Plane Strike	Clip shows the plane strike on WTC 1	Checkbox
2nd Plane Strike	Clip shows the plane strike on WTC 2	Checkbox
WTC 1 Collapse	Clip shows the collapse of WTC 1	Checkbox
WTC 2 Collapse	Clip shows the collapse of WTC 2	Checkbox
WTC 7 Collapse	Clip shows the collapse of WTC 7	Checkbox
Street	Street scene, or a street is visible in the video clip	Checkbox
Debris Aircraft Debris Collapse Debris Debris Inside Building Street Debris	Debris is visible in the video clip Type of debris: Aircraft = Can be identified as plane debris (e.g., tires, engines) Collapse = Resulting from collapse Inside Building = Visible through windows Street = On street	Checkbox for each choice
Fireball	Initial fireball from plane strike is visible	Checkbox
Thermal	The thermal is a tall region of the smoke plume that results from the lift caused by the hot gases of the initial fireball	Checkbox
Plume	Smoke plume generated by the fires within the towers and blown downwind. This marker is checked if the smoke plume in the video clip extends farther than a single tower width.	Checkbox
Flames Visible	Flames are visible in the video clip	Checkbox

Attribute	Definition	Entry Choice
People Inside Falling Outside	The video clip includes people Inside = People inside the buildings, at the windows, or climbing down Outside = People on the street	Checkbox for each choice
Falling building component	The video clip shows a building component falling (e.g., aluminum cladding)	Checkbox
Streamers Falling	The video clip shows a streamer, an object that emits smoke as it falls and leaves a trail	Checkbox
Dripping	Molten material dripping from WTC 2 is visible	Checkbox
Hanging Floor	A sagging object suggesting a floor is visible within the windows	Checkbox
Building Core	Video clip shows the core of WTC 1 or WTC 2 – both remained standing briefly during collapse before falling	Checkbox
FDNY FDNY Apparatus FDNY Personnel	FDNY personnel or vehicles are visible, including EMTs, fire trucks, and ambulances	Checkbox for each choice
NYPD NYPD Apparatus NYPD Personnel	NYPD personnel or vehicles are visible, also includes FBI and other police officials	Checkbox for each choice
Aircraft Impact Aircraft Other Aircraft	Aircraft are visible in the video clip Impact: Shows aircraft approaching WTC 1 or WTC 2 before or during the strike Other: Helicopters or fighter jets	Checkbox for each choice
Major Change Major Fire Change Major Smoke Change Windows Opened	One of the following events takes place in the video clip: Major Fire Change: Fire flares up, dies down, or spreads to a new region Major Smoke Change: Smoke bursts, dies down, or spreads to a new region Windows Opened: Window breaks open, either due to fire or to people	Checkbox for each choice
Good for Analysis	Mark video clip for possible window-by-window analysis	Checkbox
Analyzed	The video clip has been used for window-by-window analysis	Checkbox
Notes	Notes, including a description of how the video clip was timed	Text

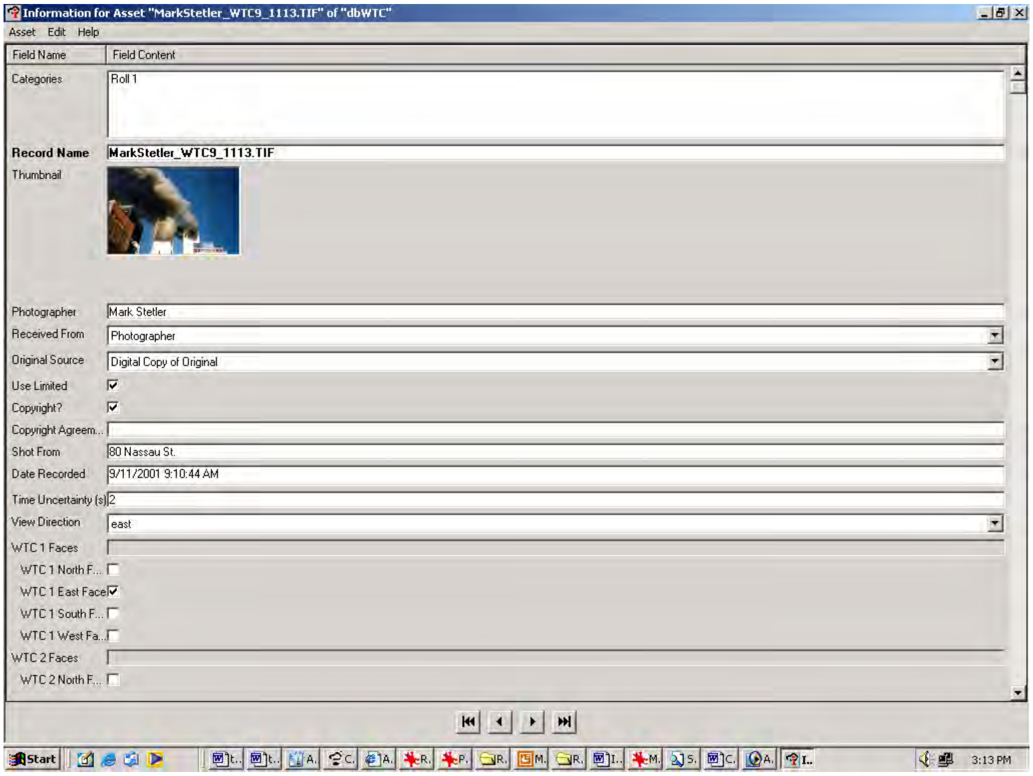


Figure H–2. An example of the first page of the Cumulus data entry sheet for photographic assets. Thumbnail © 2001 Mark Stetler.

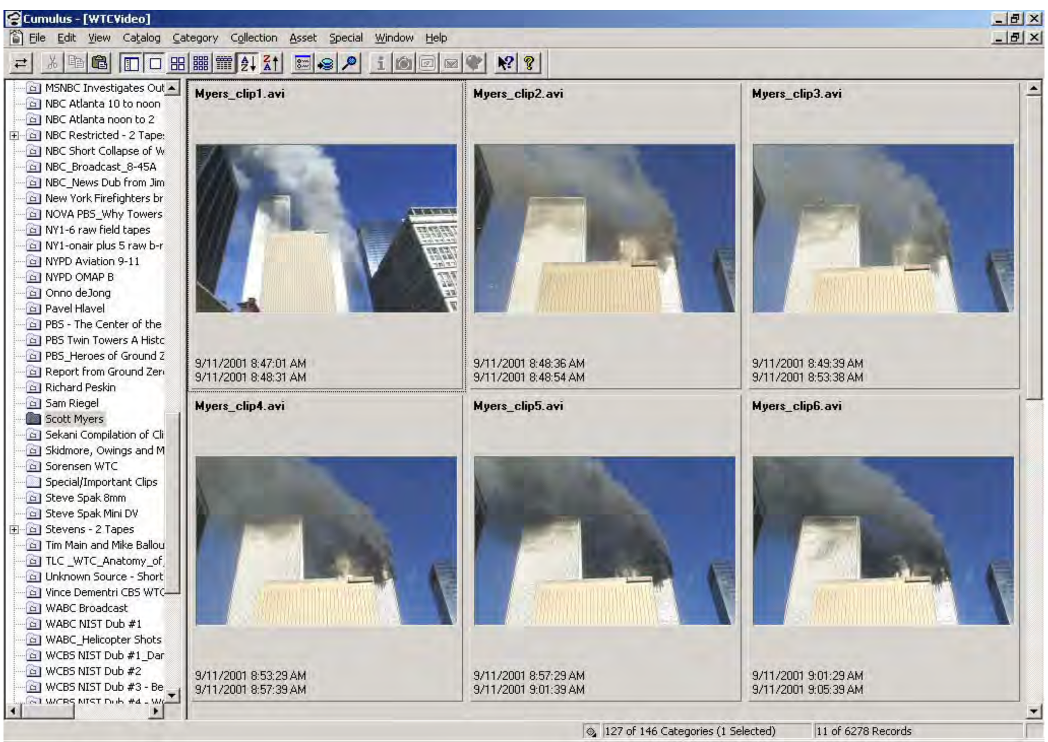


Figure H–3. An example of a Cumulus asset screen display for the video database. Thumbnails are shown along with the time and dates when the recording started and ended. Thumbnails © 2001 Scott Meyers.

Occasionally analog photo and video cameras imprint a time stamp on their outputs that can provide relative times similar to Exif or meta data, but generally there is no time information available, and such material must be timed in some other way. Some of the approaches used are described later in this section.

Photograph Tools

In order to make the best use of the information embedded in digital photographs, software was required to retrieve the Exif file information and software to adjust the recorded clock times. The commercial software package CatDV is able to retrieve meta data embedded in a variety of media formats, including digital photographs and mini-DVs. The Access database PhotoTiming was written for the purpose of determining the actual times for a set of photographs given the Exif time for each and an accurate time reference. For a set of photographs sharing a common clock from the same digital camera, an accurate time for a single photograph is sufficient to set the times for the entire set. Figure H–4 shows a PhotoTiming data sheet for a selected photographer. A file generated by CatDV containing the Exif data for each photograph, if available, is read into PhotoTiming. The equivalent Exif and actual times are entered into the appropriate fields at the upper right of the data sheet. Selection of the Calculate Photo Times button fills the Actual Time column with the appropriate value for each Exif time. In this example, the Exif time was found to be off by 62 s.

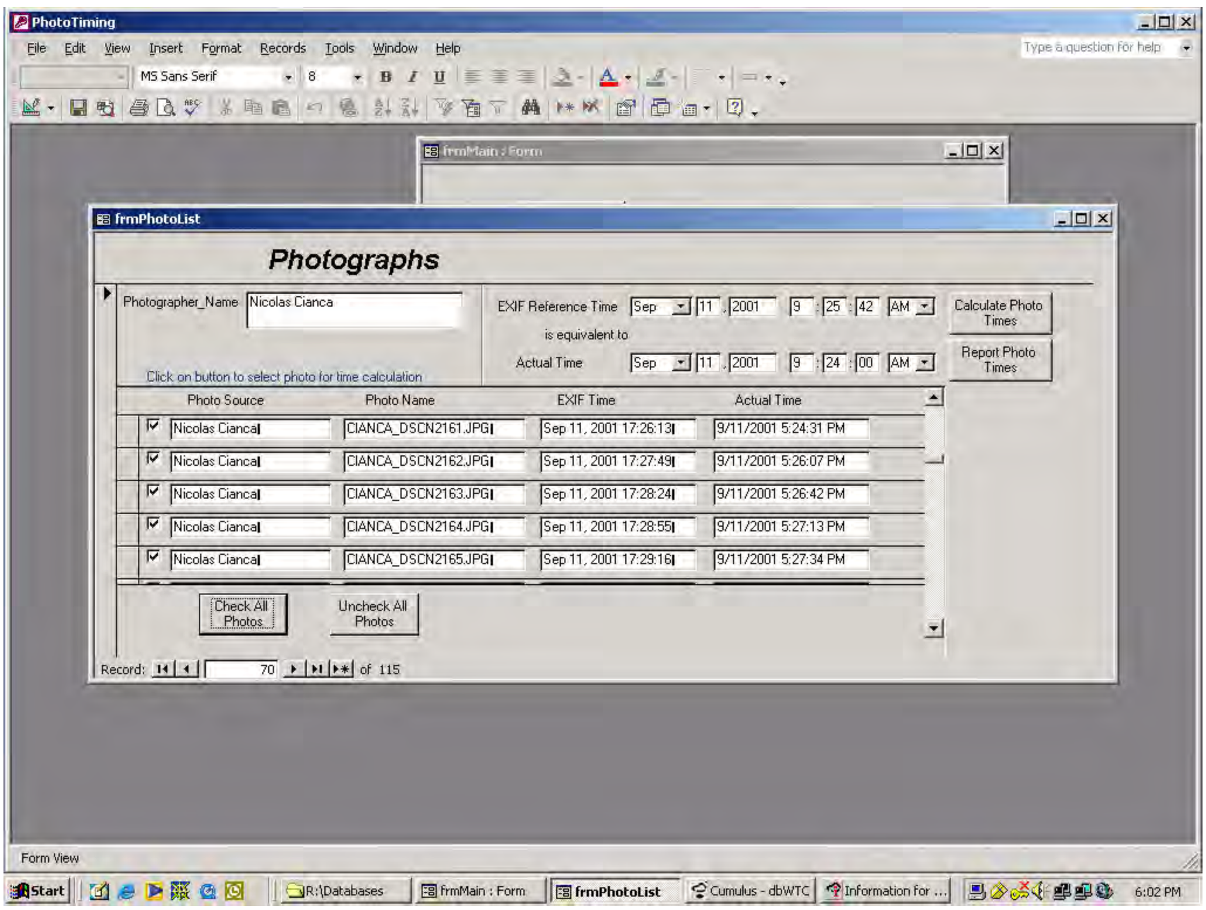


Figure H–4. An example of the PhotoTiming sheet for calculating times for photographs containing Exif meta data.

Video Tools

In addition to containing the video database described in Section H.2.3, VideoList also assists with timing the clips from a videotape. This function is similar to that in the PhotoTiming tool. For a broadcast video that was filmed in real time, the timing of every clip in the video, except for replays, can be set from knowing the time at a single point. An example is shown in Figure H–5. A clip file generated in Adobe Premiere for a specified video is read into VideoList. The mini-DV time of an event in the video whose timing is known, such as the moment of the second plane strike, is identified. Both times are entered into the fields at the upper right of the data sheet. Clips to be timed (excluding replays) are identified by a check mark, and the requested calculation results in the actual times in and out for each clip as shown in Figure H–5. This tool is also useful in calculating timings for continuous video segments broken into multiple clips.

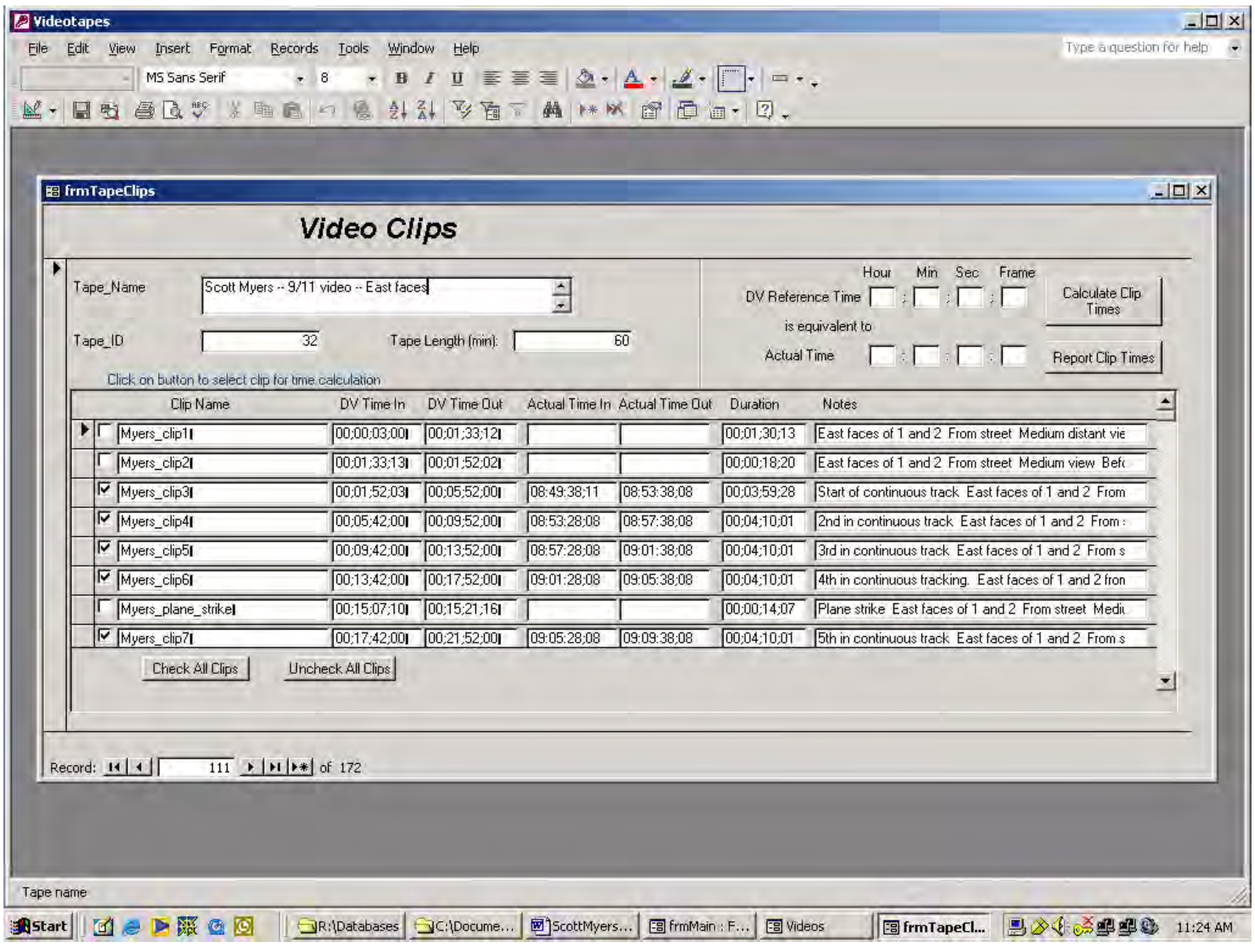


Figure H–5. An example of the VideoList sheet for calculating clip times for video assets.

For each mini-DV video that contains meta data, CatDV is used to extract the clock times for the In and Out point for each clip. These values enable the timing of every clip in the video from a single reference time.

H.3.2 Reference Time

Faced with the timing considerations above, a timing scheme was developed in which all of the times in the databases are placed on a relative time scale tied to a single well-defined event. Due to the large number of different views available, the moment the second plane struck WTC 2 was chosen to be this time. This event was defined to have occurred at 9:02:54 a.m. based on times for major events included in the earlier Federal Emergency Management Agency (FEMA) report (McAllister 2002) describing the events of September 11, 2001.

H.3.3 Timing Techniques

Once the reference time was chosen, it was possible to place times on videos that showed the second plane strike. By matching other photographs and videos to these initially assigned videos, the assignments were extended to visual materials that did not include the primary event. By such a bootstrap process, it was possible to place photographs and videos extending over the entire period of the event on a single time line. Sets of photographs containing Exif times and video clips that either contained meta data or were continuous over relatively long periods were particularly useful for this purpose because a single time assignment would allow the entire series to be timed. Sets of photographs recorded on film or analog videos that were frequently turned on and off caused the most difficulty in timing, and individual matches were required for each photo or video clip.

Matching visual images and assigning times has turned out to be a demanding task requiring unique approaches. A variety of characteristics have been employed to match times in different photographs and videos. These include distinct shadows cast on the buildings by the smoke plumes, the appearance and locations of smoke and fire plumes, the occurrence of well-defined events such as a falling object or the sudden appearance of smoke, and a variety of other unlikely clues such as a clock being recorded in an image.

To assist in the timing process, relative times for the five major events of September 11, 2001: first plane strike, second plane strike, collapse of WTC 2, collapse of WTC 1, and collapse of WTC 7 have been determined with 1 second accuracy. These times are summarized in Table H–3. Note that the building collapse times are defined to be when the entire building is first observed to start to collapse. In the case of WTC 7, a penthouse on the roof sank into the building before the main collapse started.

Table H–3. Times for major events of September 11, 2001.

Event	Relative Time from Visual Analysis	Adjusted Time from Television Broadcasts	Time Reported in the FEMA Study
First plane strike	8:46:25 a.m.	8:46:30 a.m.	8:46:26 a.m.
Second plane strike	9:02:54 a.m.	9:02:59 a.m.	9:02:54 a.m.
Collapse of WTC 2	9:58:54 a.m.	9:58:59 a.m.	9:59:04 a.m.
Collapse of WTC 1	10:28:20 a.m.	10:28:25 a.m.	10:28:31 a.m.
Collapse of WTC 7	5:20:47 p.m.	5:20:52 p.m.	5:20:33 p.m.

It is not only important to assign relative times for photographs and videos, but also to estimate how accurately they are known. For this reason, timing uncertainties are estimated for each determination and are included in the databases.

The bootstrap timing process was initially quite difficult. However, team members' timing skills improved with practice at the same time as more visual material became available and the number of timed assets increased. At the present time, 3,032 of the 6,759 catalogued photographs and 2,673 of the 6,911 video clips in the databases are timed with assigned relative accuracies of 3 seconds or better.

H.3.4 Absolute Time Accuracy

Many of the news broadcasts on September 11, 2001, included small clocks, known in the industry as "bugs," imprinted on the screen. As such broadcasts were timed, it became apparent that there were small differences between times for the second plane strike based on these bugs and the time used as the basis for the database. Checks with several broadcasters indicated that the bugs should be quite close to the actual time because their clocks are regularly updated from highly accurate sources such as geopositioning satellites or the precise atomic-clock-based timing signals provided by NIST as a public service. Careful checks showed small time differences between different video recordings, but these were generally less than 1 second. These small discrepancies are likely due to variations in transmission times resulting from the different pathways that the video signals take to the sites where they are recorded. Based on four such video recordings, the time of the second plane impact is estimated as 9:02:59 a.m., or 5 seconds later than the time assumed in developing the database. The estimated uncertainty is 1 second. Table H–3 compares times for the major events taken from the database, adjusted to television time, and reported in the FEMA report (McAllister 2002). Possible explanations for the observed differences are still under investigation. Because times based on the television broadcasts appear to be accurate (i.e., those in column 3 of Table H–3), 5 seconds will be added to times included in the databases when precise times are reported for the Investigation.

H.4 ANALYSIS OF VISUAL IMAGES

Once the two visual databases became available, it was possible to use the images to begin characterizing the events of September 11, 2001. Some of the images are quite close up and can be used to learn specific details concerning the towers. As an example, Fig. H–6 shows an image of the east face of WTC 2 recorded at 9:26:20 a.m., and Fig. H–7 shows an enlarged portion of the same photograph. The photograph has been enhanced using Adobe Photoshop, and lettering has been added to indicate the floors and the numbering system used to identify specific windows in the tower. The amount of detail available is evident. For instance, large piles of debris are present on the north side of the tower on floors 80 and 81, and locations with fires visible or with windows missing are easily identified.

H.4.1 Window Numbering

The system used to describe window locations in the two towers and WTC 7 requires some elaboration. It is based on the outer-wall column numbering system used in plans for the buildings. First, consider the towers. In these structures individual windows were placed between two exterior columns. In order to refer to a particular window the designation for the column to the right as viewed from the outside is assigned to that window. These columns are numbered from 1 to 59 from right to left across a tower face, and windows are numbered from 1 to 58. Faces for the towers are also assigned numbers as follows; WTC 1—north: 1, east: 2, south: 3, and west: 4, and WTC 2—west: 1, north: 2, east: 3, and south: 4. By combining the floor number, the face number and a column number, a specific window on one of the

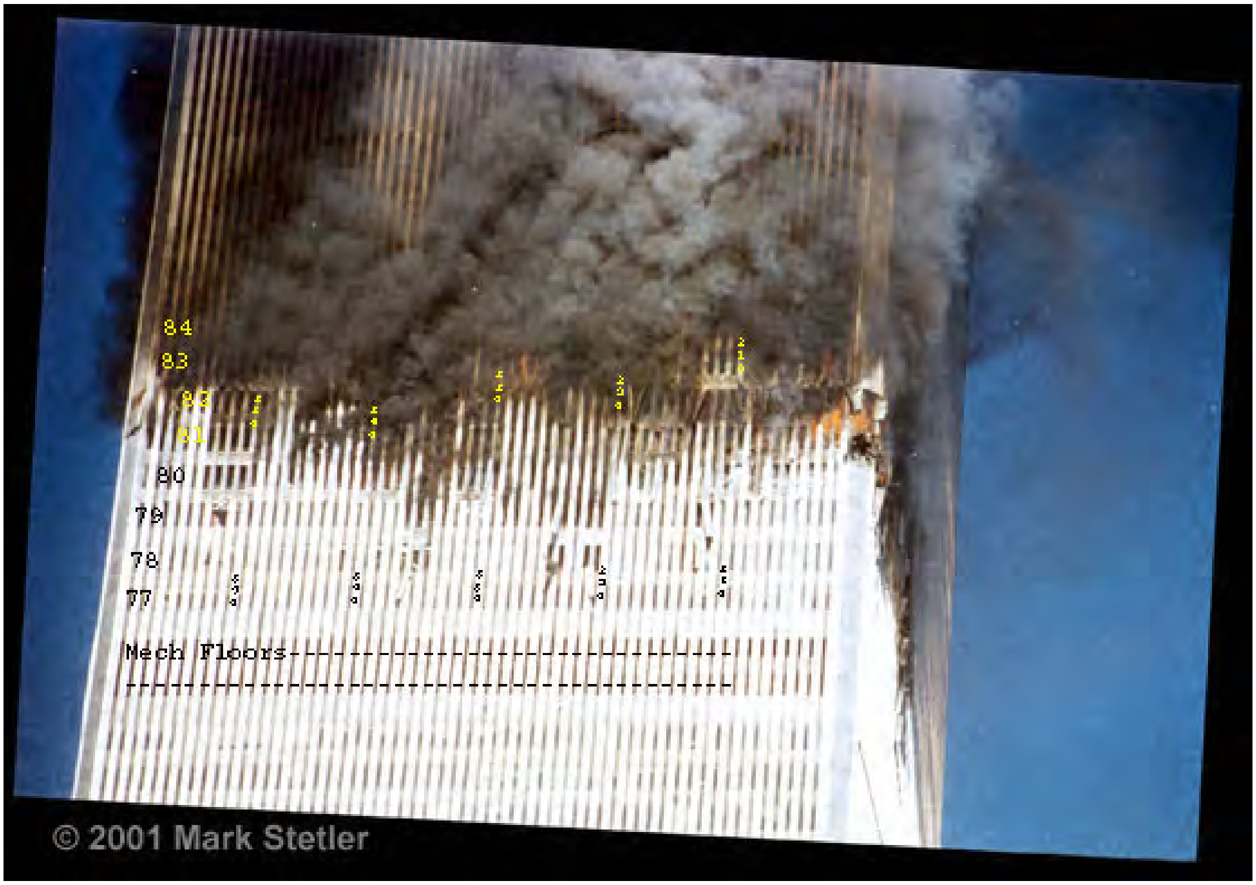


Figure H–6. Photograph taken at 9:26:20 a.m. on September 11, 2001, showing the east face of WTC 2. It has been enhanced, and lettering indicating floors and columns has been added.

towers can be identified. As an example, for WTC 1, the number 94-214 refers to the fourteenth window from the right on the east face of floor 94.

The window numbering system is somewhat different for WTC 7. It is also based on the outside column numbers, but in this building the numbering of columns was continuous around the structure and ranged from 1 to 57. Column 1 was located at the northwest corner of the building, and the numbering proceeded counter clockwise around the building faces with columns 15, 28, and 42 located at the southwest, southeast, and northeast corners, respectively. Note that the total number of perimeter columns is actually 58. An extra column, numbered 14A, was included on the west face between columns 14 and 15. Unlike the towers, the number of windows to the right of a given column varied from one to five depending on location. In some cases, the windows are located in front of the column. Individual windows to the right of a column are assigned letters increasing from left to right as seen from the outside. As an example, 12-45c refers to a window on the north face of WTC 7 that is the third window to the right of column 45 on floor 12.

Figure H–7. This photograph is cropped from the image shown in Figure H–6. It was taken on September 11 and shows the east face of WTC 2 at the northeast corner from floor 77 to floor 82. Note the large piles of debris evident on floor 80 and floor 81.

H.4.2 Fire Properties

Photographs and video images have been used to characterize a number of properties relevant to fire growth and spread in the towers as a function of time. Specific properties addressed include whether or not fire and smoke are present and whether windows are still in place. When smoke and/or fire are present, additional details concerning their appearances are documented. A numbered coding system is used to describe these characteristics. The key for this numbering system is shown in Fig. H–8.

H.4.3 Window-by-Window Assessment

The key in Fig. H–8 is used as the basis for a window-by-window assessment of the towers. The results are coded in three separate data sheets using Microsoft Excel. The floor and window locations are identified using the numbering system described in the last section. Separate files containing the three data sheets are generated for each face of a tower and time analyzed. Figure H–9 shows a portion of such a data sheet describing fires (i.e., sheet one) on the east face of WTC 1 around 9:42 a.m.

KEY FOR ANALYSIS

Sheet #1: Fire Visible

0	No fire
1	Spot fire
2	Fire visible inside
3	External flaming
9	Not visible

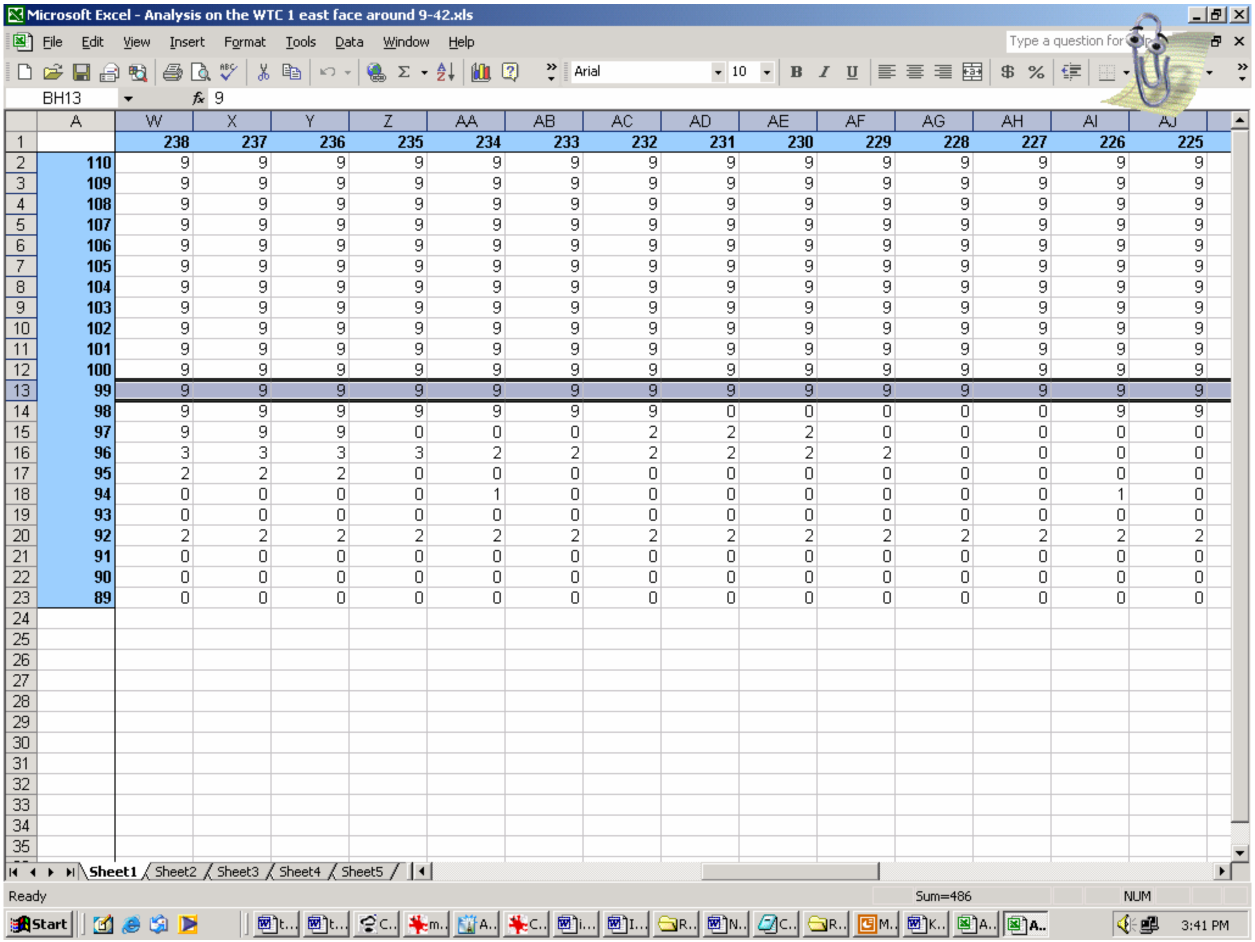
Sheet #2: Smoke

0	No smoke evident
1	"Light smoke"
2	"Heavy smoke"
9	Not visible

Sheet #3: Windows

0	Window open
1	Window in place
9	Not visible

Figure H–8. The key used to describe observations with regard to fire, smoke, and window breakage in Excel data files for individual windows in the two towers.



	A	W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG	AH	AI	AJ
1		238	237	236	235	234	233	232	231	230	229	228	227	226	225
2	110	9	9	9	9	9	9	9	9	9	9	9	9	9	9
3	109	9	9	9	9	9	9	9	9	9	9	9	9	9	9
4	108	9	9	9	9	9	9	9	9	9	9	9	9	9	9
5	107	9	9	9	9	9	9	9	9	9	9	9	9	9	9
6	106	9	9	9	9	9	9	9	9	9	9	9	9	9	9
7	105	9	9	9	9	9	9	9	9	9	9	9	9	9	9
8	104	9	9	9	9	9	9	9	9	9	9	9	9	9	9
9	103	9	9	9	9	9	9	9	9	9	9	9	9	9	9
10	102	9	9	9	9	9	9	9	9	9	9	9	9	9	9
11	101	9	9	9	9	9	9	9	9	9	9	9	9	9	9
12	100	9	9	9	9	9	9	9	9	9	9	9	9	9	9
13	99	9	9	9	9	9	9	9	9	9	9	9	9	9	9
14	98	9	9	9	9	9	9	9	0	0	0	0	0	9	9
15	97	9	9	9	0	0	0	2	2	2	0	0	0	0	0
16	96	3	3	3	3	2	2	2	2	2	2	0	0	0	0
17	95	2	2	2	0	0	0	0	0	0	0	0	0	0	0
18	94	0	0	0	0	1	0	0	0	0	0	0	0	1	0
19	93	0	0	0	0	0	0	0	0	0	0	0	0	0	0
20	92	2	2	2	2	2	2	2	2	2	2	2	2	2	2
21	91	0	0	0	0	0	0	0	0	0	0	0	0	0	0
22	90	0	0	0	0	0	0	0	0	0	0	0	0	0	0
23	89	0	0	0	0	0	0	0	0	0	0	0	0	0	0

Figure H–9. A portion of the Excel spreadsheet describing fires on the east face of WTC 1 around 9:42 a.m. is shown. The numbers at the left refer to floors, and those at the top are the window numbers.

While the data sheets capture the desired behaviors, it is very difficult to use them to track changes without visualizing the results in some way. Two approaches have been developed for this purpose. The first employs a Web-based system that generates color-coded maps of the results contained in the data sheets. Figure H–10 shows such a map for the fire data included in the data sheet shown in Fig. H–9. The second approach uses the program Smokeview (Forney and McGrattan 2003; Forney, Madrzykowski, and McGrattan 2003) to generate a time-dependent visualization of the results. Smokeview was developed at NIST in order to display the results of fire dynamics calculations. In the current application, it is used to visualize the properties of interest on a three-dimensional representation of a tower façade as a function of time. Because Smokeview allows the point of view to be varied at will, this approach is a powerful means for investigating the temporal behavior of the fires on different faces of the tower. Figure H–11 shows a frame taken from a visualization in which results from the fire and windows data sheets for WTC 2 have been combined.

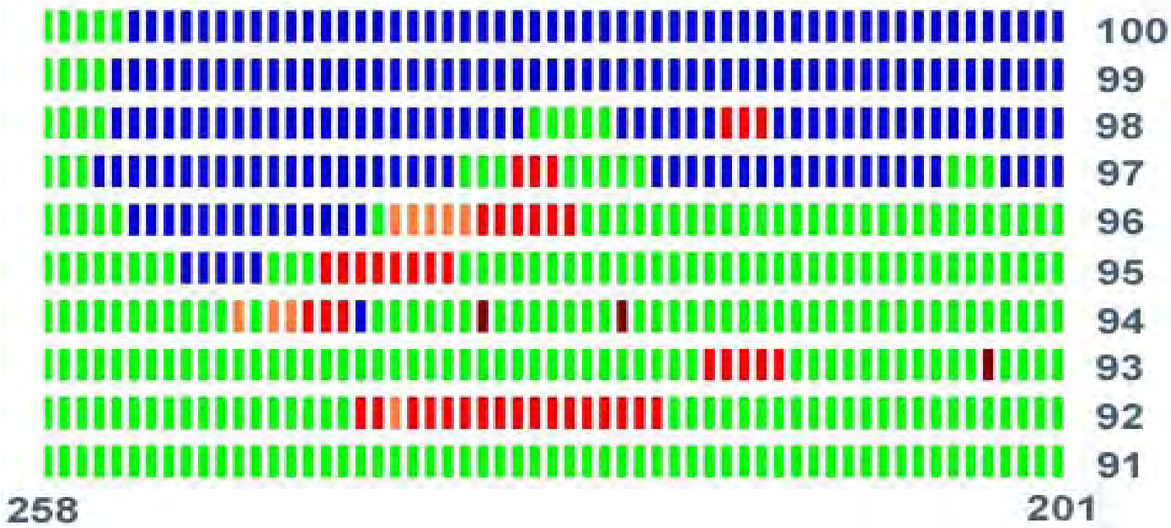


Figure H–10. A representation of fires for floors 91 to 100 on the east face of WTC 1 around 9:42 a.m. is shown. Results are taken from the Excel spreadsheet shown in Figure H–13. The color coding is based on the key shown. The color assignments are: 0-No fire, 1-Spot fire, 2-Fire visible inside, 3-External Flaming, and 9-Can't see.

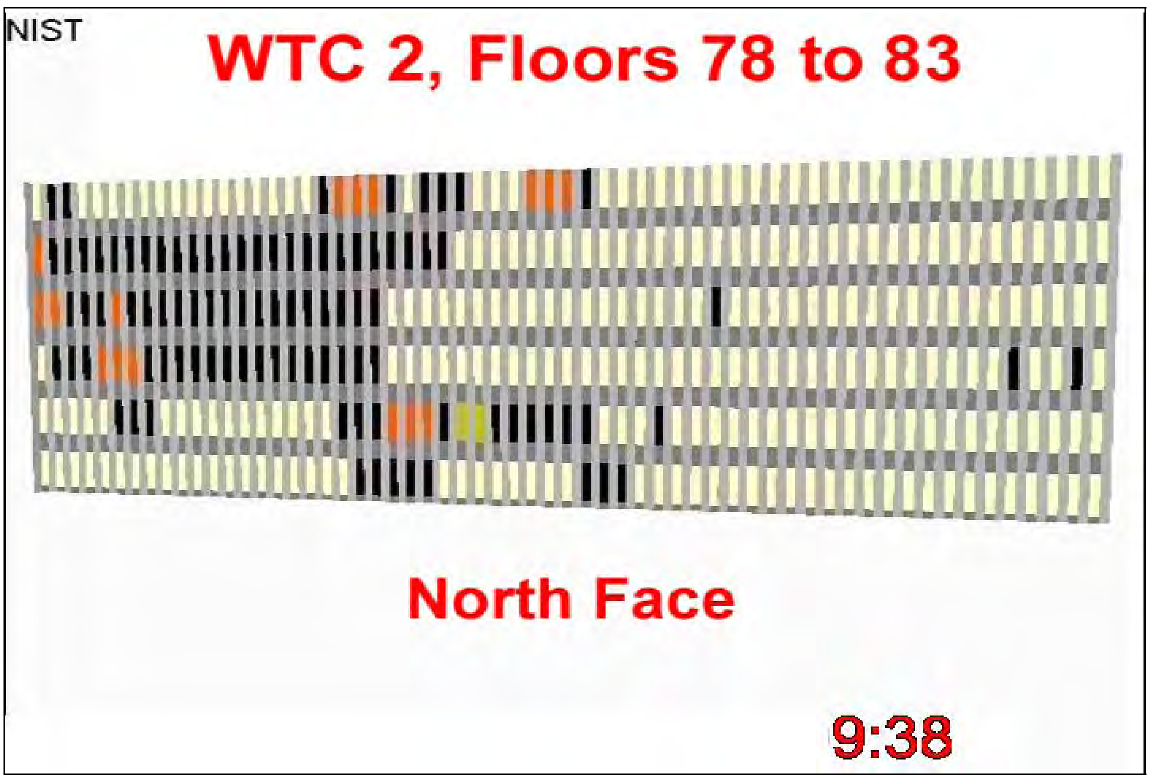


Figure H–11. A single frame from a time-dependent visualization generated by Smokeview is reproduced here. The frame is a three-dimensional representation of the condition of windows and fires on WTC 2 from the time the second tower was struck at 9:02:59 a.m. until it collapsed at 9:58:59 a.m. The color assignments are: – window in place, – missing window, – external flaming, – fire inside, and – spot fire.

H.5 INITIAL DAMAGE PATTERNS ON WTC 1 AND WTC 2 DUE TO THE PLANE STRIKES

Close-up photographs and videos have been used to characterize the initial damage to the façades of the towers struck by the two planes along with precise determinations of the locations of the plane strikes. For WTC 2, analysis of videos has also been employed to estimate the speed of the airplane that struck the tower and to show that the tower swung back and forth for several minutes after it was struck. The period of the swinging has also been determined.

H.5.1 WTC 1

Damage Resulting from Plane Strike

A detailed drawing of the damage to the steel façade of WTC 1 was included in the FEMA *World Trade Center Building Performance Study* (McAllister 2002). A careful inspection using photographs and videos in the database confirmed the accuracy of this analysis. Figure H–12 shows a drawing that represents this damage. It is similar to that included in the FEMA report, but it incorporates several minor changes that better reflect the geometry of the north face of WTC 1 in the vicinity of the plane strike.

It was observed that the wing tips and the end of the vertical stabilizer at the plane's tail section damaged the aluminum column covers on the steel façade without cutting through the steel below or completely removing the covers. By inspection it was possible to map out locations on columns where the wingtips and the vertical stabilizer struck the tower. These locations were then transferred to the representation of the damaged steel façade shown in Fig. H–12 and are represented by dashed lines, with wings to the right and left and the vertical stabilizer in the center. The good agreement between the damage pattern and the wing tip locations is evident. It is reported in the FEMA report (McAllister 2002) and widely in the media that American Airlines Flight 11 struck floors 94 to 98 of WTC 1. The dotted horizontal lines on the left side of Fig. H–12 indicate the locations of concrete floors. It can be seen that while the tip of the left wing of the aircraft struck very close to the base of floor 94, the wing end marked column 153 at the very top of floor 93. It is evident from the figure that the right wing actually struck well up on floor 99 on column 109. The impacted floors therefore range from floor 93 to floor 99.

Fireballs and Missing Windows

Additional insights into the initial damage inflicted on the towers by the plane strikes can be obtained by considering locations where fireballs are observed immediately following the plane strikes as well as locations where windows are missing. Videos and photographs recorded during and immediately following the plane strike on WTC 1 show that significant fireballs formed at the plane strike location on the north face, as well as near the center of the east face and on the western side of the south face. Figure H–13 compares window damage for the four sides of WTC 1 immediately after the plane strike. The floors shown extend from 91 to 100. The missing windows on the north face are consistent with the plane strike location and strike angle. The plane struck very close to the center of the face. Interestingly, the damage on the east and west faces appears to be asymmetric with a much higher number of windows missing on the east face than on the west. This observation is consistent with the formation of a fireball on the east side of the tower and not on the west side. Areas obscured by smoke are also much larger on

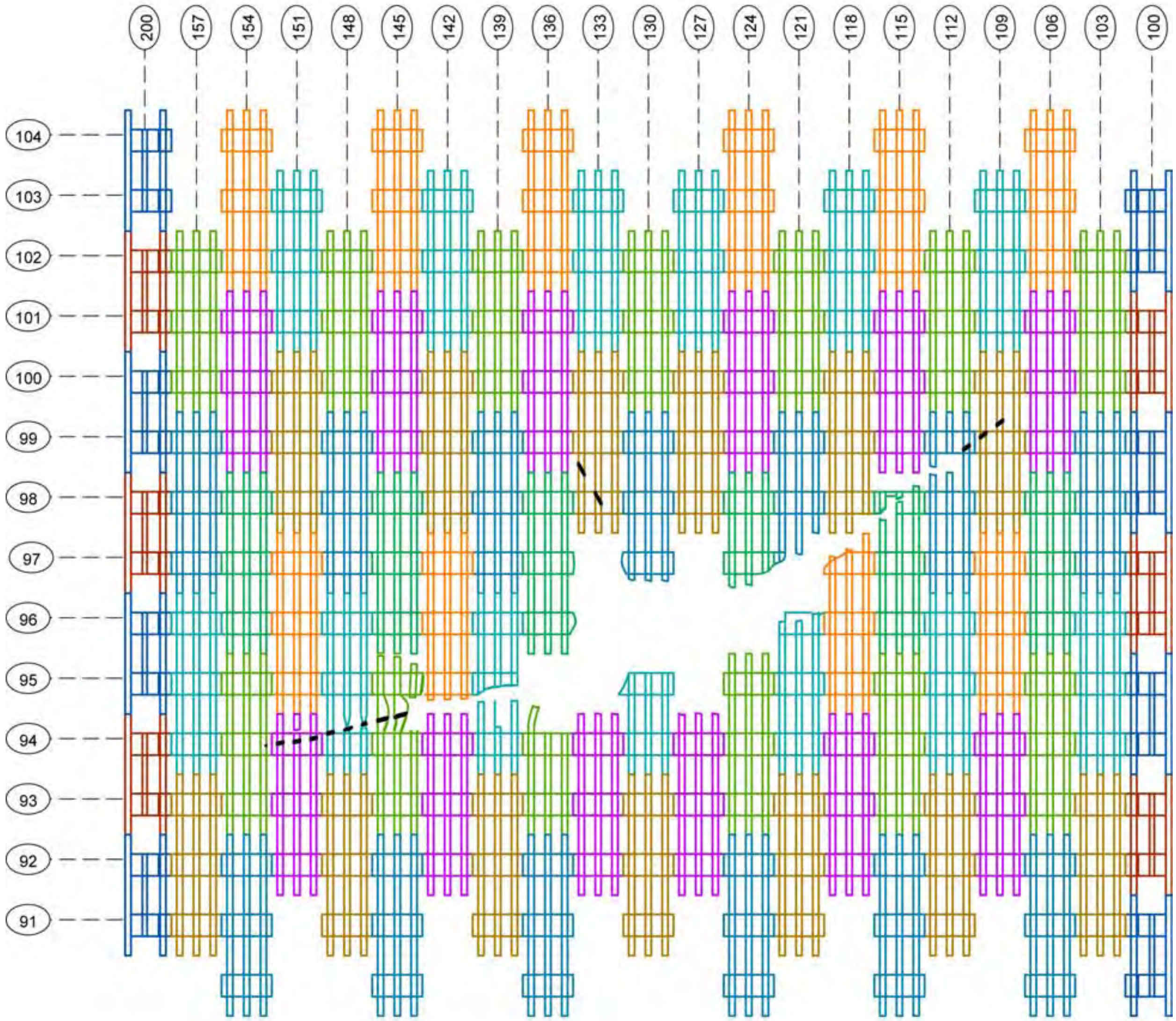


Figure H–12. A drawing of the damage to the steel façade of WTC 1. The dark dotted lines show locations where the airplane wings and vertical stabilizer marked the aluminum cladding on columns.

the east side suggesting that more fire is present on this face as well. An asymmetry is also apparent on the south face where only a single window is missing on the east side while numerous windows are missing and significant smoke is present on the west side. Taken together, these observations suggest that debris and fuel from the airplane as well as any building materials and contents tended to pass straight across the building on the west side, while material on the east side was somehow reflected and more heavily damaged the east face.

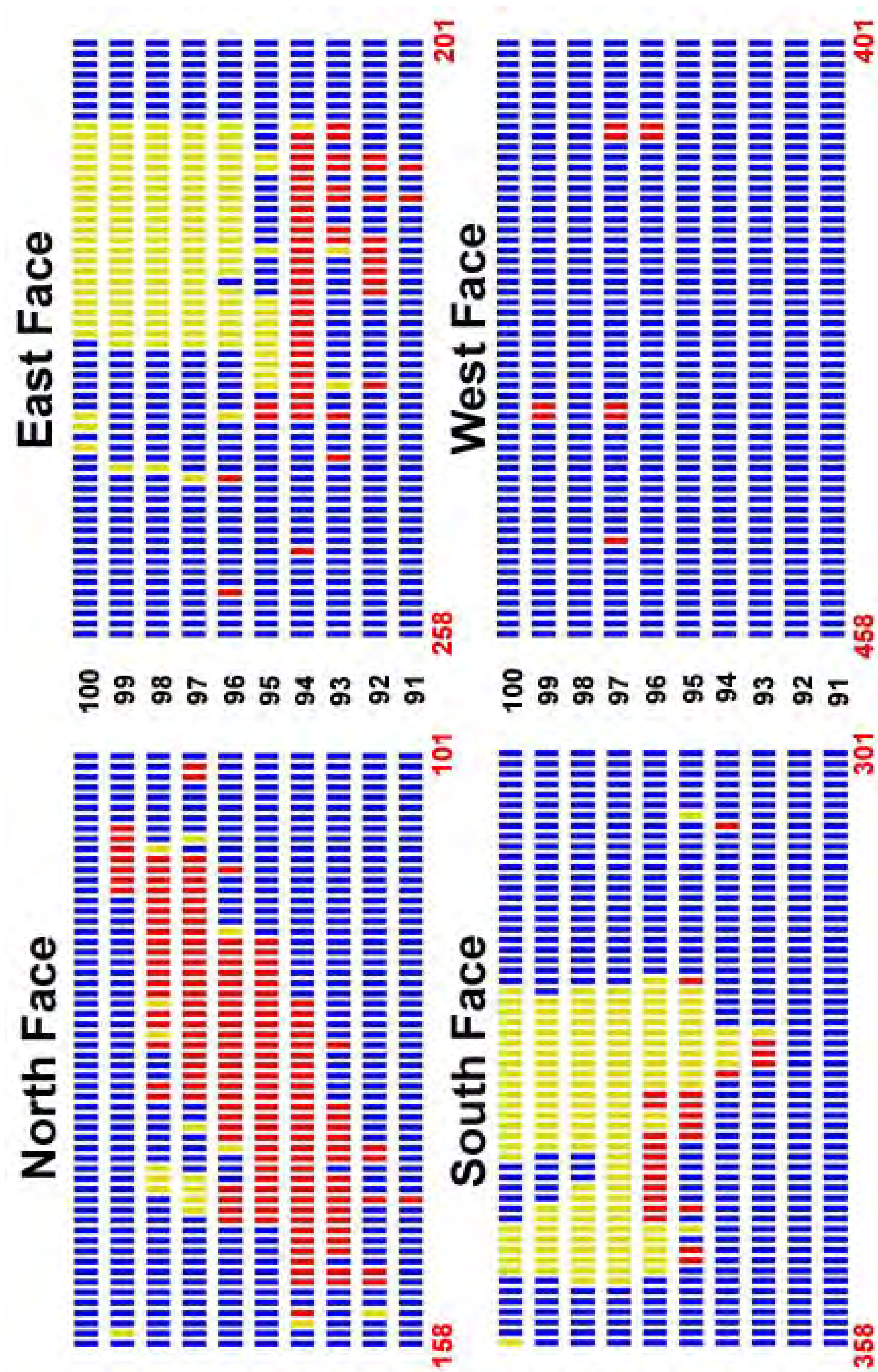


Figure H–13. The condition of windows is shown for the four faces of WTC 1 around 8:47 a.m. shortly after it was struck by American Airlines Flight 11. The colors represent ▌- window open, ▌- window in place, and ▌- not visible.

Panel Section in Street

A photograph supplied by the NYPD provided additional details with regard to the initial damage suffered by WTC 1. Figure H–14 shows a full three-story three-column-wide steel panel section lying on the corner of Cedar Street near its intersection with West Street. This location is to the south of and roughly 210 m from the south face of WTC 1. The photograph was taken prior to the collapse of either tower. Closer inspection shows that there is an aircraft wheel embedded in one of the windows. The most likely source location for this panel section has been identified as being near the center of the south face of WTC 1 (i.e., columns 329 to 331) and extending from the middle of floor 93 to the middle of floor 96. This conclusion remains tentative since, as indicated in Fig. H–13, the area is obscured by smoke in all of the close-up photographs of the area in NIST's possession. If the location is identified correctly, the wheel is stuck in window 95-329.

H.5.2 WTC 2

Calculation of Plane Speed

One of the videographers who provided material to the Investigation filmed from the top of his apartment building located to the east of the WTC complex. His camera was located on a tripod so that the images are very steady. One of the events he captured was United Airlines Flight 175 as it approached WTC 2. Figure H–15 is a series of cropped frames captured from this video that show the plane approaching the building.

The images included in Fig. H–15 have been used to determine the speed of the plane as it approached the tower. This is done by identifying the locations of the nose and tail of the airplane relative to a fixed point defined to be the point on the frame where the plane passes out of sight behind the corner of the building. The plane is very nearly level relative to this point, so it is appropriate to simply count the number of picture elements, pixels, between this location and the two measurement points on the aircraft.

This analysis, which presumes that the aircraft at this time travels in a straight path such that the nose and tail pass through the same point in space, has the advantage of being independent of the orientation of the flight path with respect to the line of sight of the observer.

Figure H–16 shows the locations of the two points as a function of time. Using linear least squares curve fits, the exact relative times when the nose and tail pass the reference location are estimated. The difference between these two times is the period required for the entire length of the aircraft to pass the reference location. The result is 0.1939 s. Since the length of the plane is known to be 155.0 ft, the speed can be determined simply by dividing this length by the passage time to give 155.0 ft/0.1939 s = 799 ft/s = 545 mph. An uncertainty estimate based solely on the uncertainty in the determined time difference yields a value of ±18 mph with 95 percent confidence.

Figure H–14. Photograph showing a full panel section lying in Cedar Street near its intersection with West Street. An aircraft wheel can be seen imbedded in one of the windows. The building behind the panel is Saint Nicholas Greek Orthodox Church and the lower section of WTC 2 can be seen across Liberty Street.

Note that the airplane speed and uncertainties are slightly different than listed in an earlier report (NIST 2003) due to a correction of the plane length to reflect the actual distance between the nose and the end of the body at the rear stabilizer and a math error in the uncertainty calculator. Uncertainties associated with aircraft motion that are not aligned with the aircraft body are judged to be less than the uncertainties in plane passage time.

Observation of WTC 2 Sway Following the Plane Strike

Close examination of the video revealed a perceptible movement of WTC 2 after it was struck by the aircraft. The building rocked back and forth much as a pendulum for at least 4 minutes. Image

Figure H–15. Series of sequential cropped frames taken from a video shot on September 11, 2001, showing the plane approaching WTC 2. The frames, ordered from left to right and top to bottom, are separated by 33.3 ms.

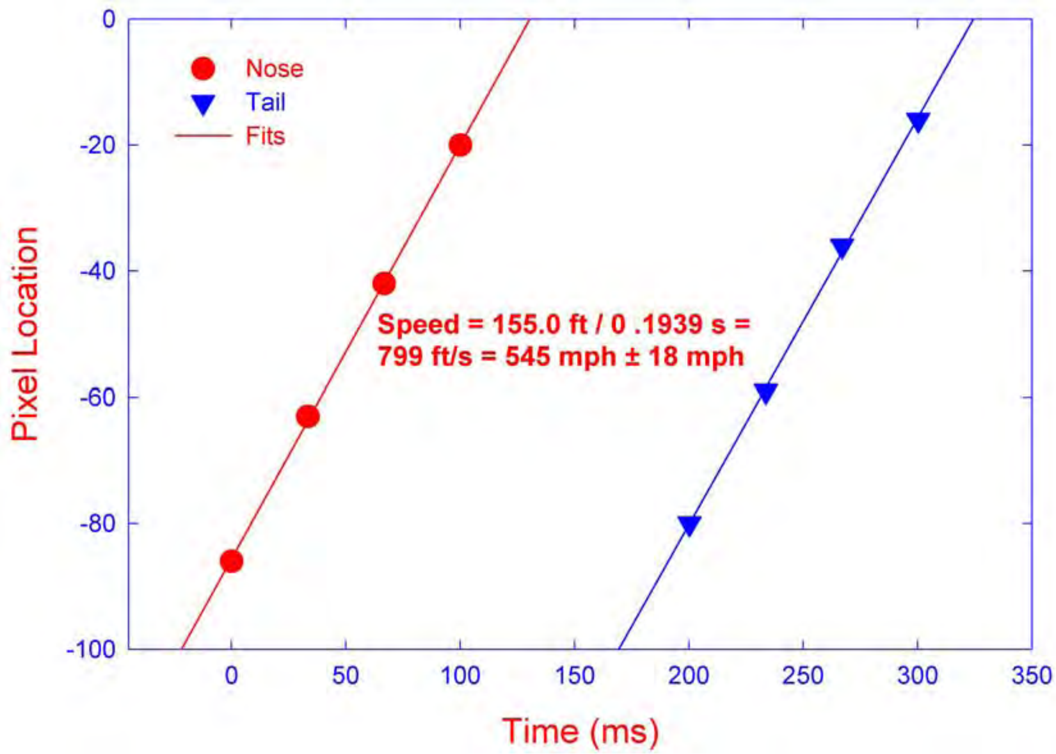


Figure H–16. Plots of pixel locations for the nose and tail of the plane that struck WTC 2 as a function of time taken from the images shown in Fig. H–15. Straight lines are the results of linear least squares curve fits to the data. Extrapolation of the lines to pixel 0 allows the time for the passage of the plane to be calculated.

analysis was used to enhance this motion and estimate the period required for the building to sway through one complete cycle. This was accomplished by creating a new video in which a single frame just prior to the plane strike was subtracted from subsequent frames. In this way, small differences between images can be identified. If the image is unchanged from the initial frame, the result should be a black image, but any changes in location or color will appear in the difference video. When this approach was applied to the video, a region of windows was observed on the building that seemed to appear and disappear. Figure H–17 shows several frames of a composite video formed by overlaying half frames of the original video and the difference video. In the initial frame (time = 0.0 seconds), the plane has not yet appeared and the difference frame is black. In the next frame (time = 10.7 seconds), the plane is approaching the building. The plane is evident in the difference frame since it represents a change in the frame. WTC 2 is still dark except near the top where changes due to smoke movement are apparent. In the third frame (time = 11.3 seconds), the plane has struck the building and dramatic changes in the appearance of the building façade in the difference frame occur. Careful inspection shows what appear to be curved lines running across the face of WTC 2. These curves result from an interaction between the straight lines formed by the windows on the tower and the straight lines of picture elements that make up the detector in the digital video camera. This well-known behavior is called the moiré effect. The moiré effect also provides a sensitive approach for determining the displacement of the building. Such an analysis is in progress and will be reported at a later time.

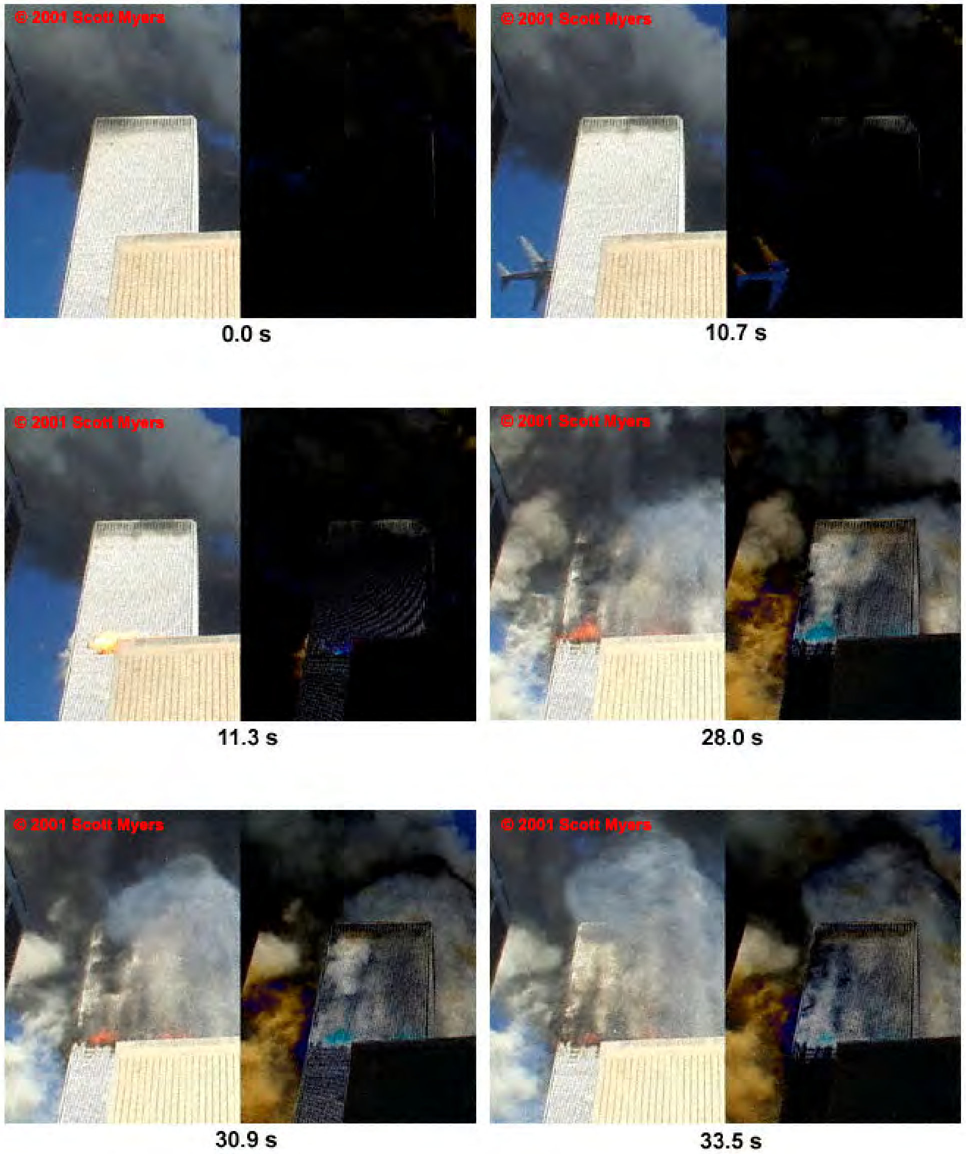


Figure H–17. Frames from a composite video are shown. The half frames on the left are taken from a video showing the plane strike on WTC 2, while the half frames on the right are generated by subtracting a frame recorded prior to the plane strike from all subsequent frames in the original video. Times refer to the period since the start of the difference video.

Following the plane strike, areas of the tower face above the strike floors become hidden by smoke, and it is difficult to see the moiré patterns in the difference frames. However, the area of the tower below the strike floors to the left of the building in the foreground continues to show a distinct difference pattern because it is not obscured by smoke. This pattern is apparent in the fifth frame (time = 30.9 seconds). On the other hand, frames 4 and 6 (times = 28.0 seconds and 33.5 seconds, respectively) have been chosen because they are near null points, and the area appears dark in the difference frame because the location of the building is essentially unchanged from its position before the plane struck. When the video is played, the moiré patterns in this area of the tower face alternately appear and disappear in the difference video.

Because the absence of color is easiest to identify, it is straightforward to determine times when the null points occur in the difference image. Figure H–18 shows a plot of time versus null point number obtained from the difference video. The points fall on a straight line having a slope of 5.647 seconds ± 0.008 seconds (95 percent confidence interval). Because the building passes through a null point twice during a single full oscillation, the period required for a single oscillation is 11.3 seconds.

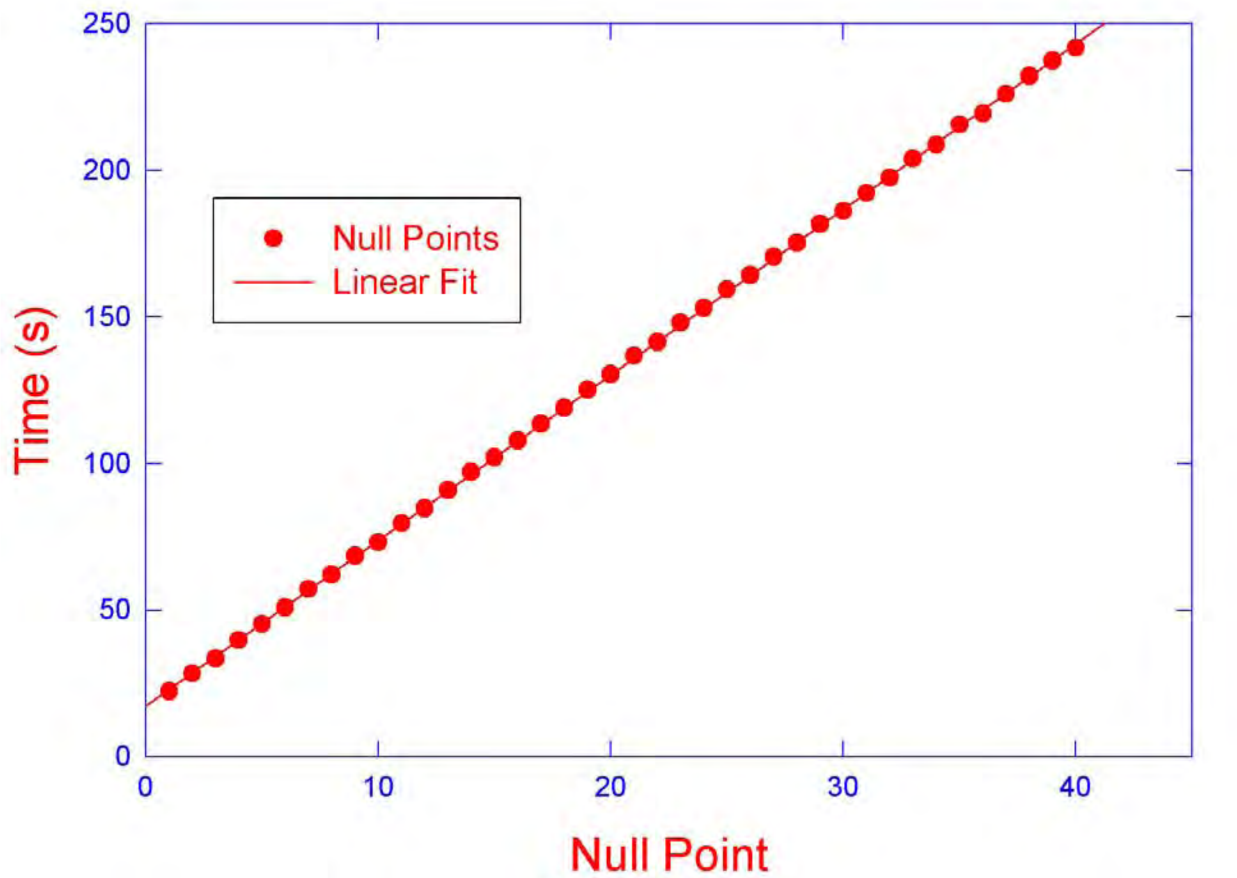


Figure H–18. The time when null points are observed in the difference video following the plane strike on WTC 2 are plotted versus the null point number. The points fall on a very good straight line having a slope of 5.647 seconds ± 0.008 seconds.

The measured oscillation period is consistent with measurements that are available from WTC 1 that yielded periods of 10.9 seconds in the east-west direction (averaged over a 9-year period that ended in 1993) and 11.6 seconds in the north-south direction (averaged over a 14-year period that also ended in 1993). The cores of the two towers were oriented perpendicular to each other so the motion monitored here should be comparable to the east-west direction of WTC 1.

Damage Resulting from the Plane Strike

The results of an analysis of the damage to the steel façade of the south face of WTC 2 are provided in Fig. H–19. Much of the steel damage pattern is revealed, but it should be noted, as indicated, that a portion of this face on the east side of the plane strike location was constantly obscured by smoke, and the detailed pattern could not be discerned. The FEMA report (McAllister 2002) also includes a figure describing the damage to the steel façade inflicted by United Airlines Flight 175. The pattern in Fig. H–19 differs somewhat from that provided in this earlier study. Some inconsistencies in façade dimensions have also been corrected in the current version.

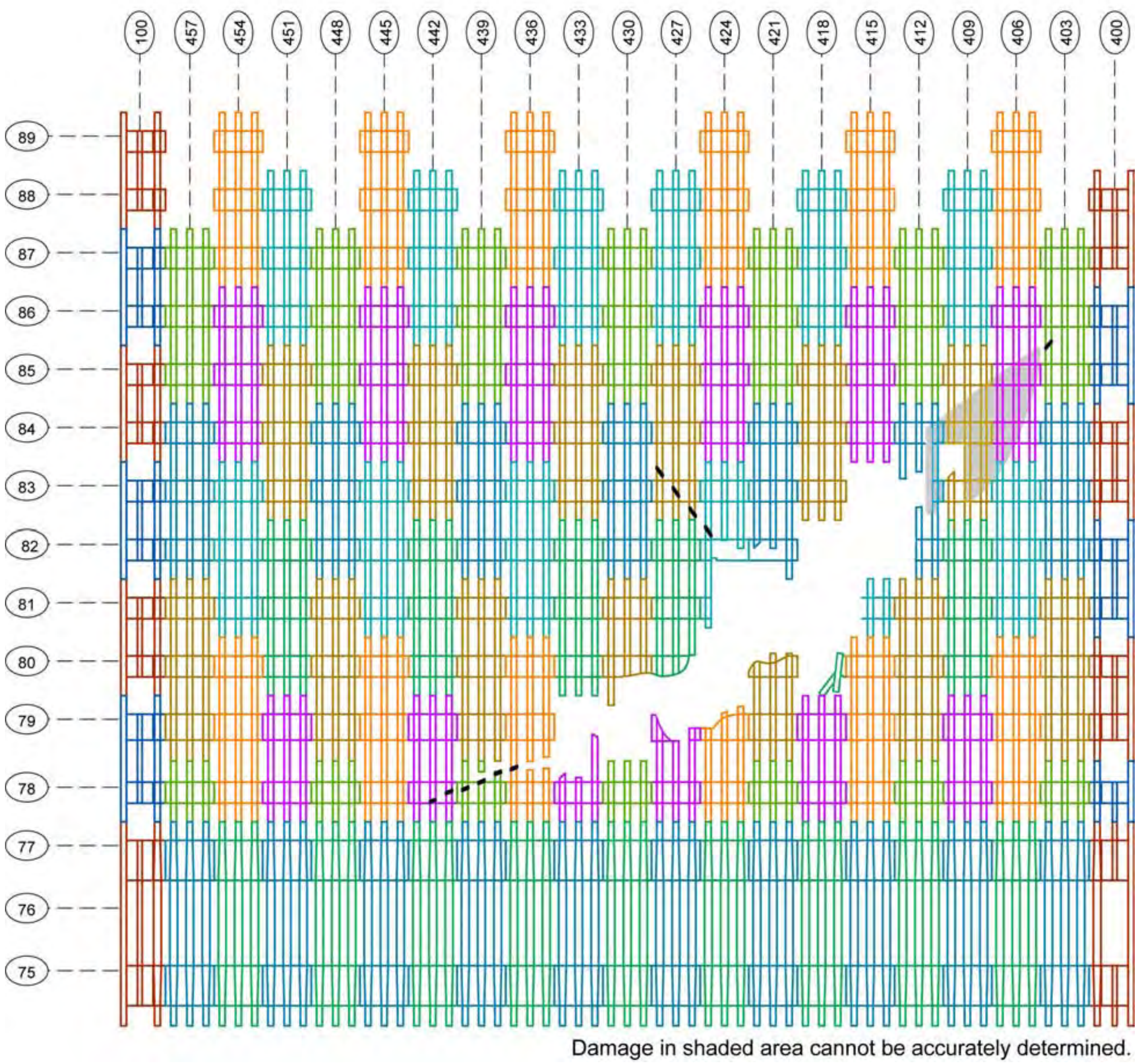


Figure H–19. A drawing of the damage to the steel facade of WTC 2. The dark dotted lines show locations where the airplane wings and tail marked the aluminum cladding on the columns. The area shaded in gray was hidden by smoke and could not be observed.

As was true for WTC 1, in areas of the façade struck by the wing tips and the upper portion of the vertical stabilizer the aluminum covering was marked, but the aluminum covers were not removed and the steel was not cut through. Measurements for the location of the left wing tip were mapped out as shown in Fig. H–19. As already noted, the area at the end of the right wing was obscured by heavy smoke. However, there were brief periods when the location of the last column struck by the wing tip could be observed. This location is indicated on column 404 of floor 95 in Fig. H–19. The center of the plane strike is clearly located towards the east side of the face. The left wing mark extends to the bottom of the spandrel located below floor 78. The actual location of the concrete floor is well above this point, which means the lowest point struck lies on floor 77. Thus, the plane strike location on WTC 2 extends from floor 77 to floor 85. This can be contrasted to the FEMA study (McAllister 2002) and most media sources that report the floors struck extended from floor 78 to floor 84.

Fireballs and Missing Windows

Intense fireballs were observed on the south, east, and north faces of WTC 2 following the plane strike. Figure H–20 compares missing windows on floor 77 to floor 86 for the four faces of WTC 2 shortly after the plane struck at 9:02:59 a.m.

The distribution of missing windows on the south face traces roughly the outline of the plane strike, with missing windows increasing in height from left to right. Recall that a portion of the east side of this face could not be observed due to smoke obscuration. The analysis indicates that a very large number of windows were removed on the east face by the collision and subsequent fireball. This is particularly true on floor 80 to floor 82. Photographs and videos show that extensive areas of the aluminum covering the façade and holding windows were removed, exposing the steel panels, as a result of the plane strike and fireballs. This damage is much more extensive than observed on the east face of WTC 1, consistent with the plane strike occurring closer to this face. In contrast to the extensive damage on the east face of the tower, no missing windows were found on the west face.

A large number of windows are also missing on the north face of WTC 2. A substantial area of the aluminum façade was also removed during the plane strike and subsequent fireball. The missing windows on this face almost appear to be a mirror image of the south face with damage towards the center being on lower floors than on the eastern edge. This suggests that a great deal of debris passed through the entire length of the building. This hypothesis is supported by close up images that show large piles of debris on the east side of the north face on floor 80 and floor 81, and on floor 79 near the center of the face. Figure H–21 includes a photograph showing these debris piles. Recall that piles of debris were also evident on floor 80 and floor 81 on the north side of the east face (see Fig. H–7).

H.6 FIRE BEHAVIORS IN THE TWO TOWERS

Analysis of the fire spread in the towers is ongoing as this update is being prepared, but sufficient information is available to allow some of the fire characteristics to be described.

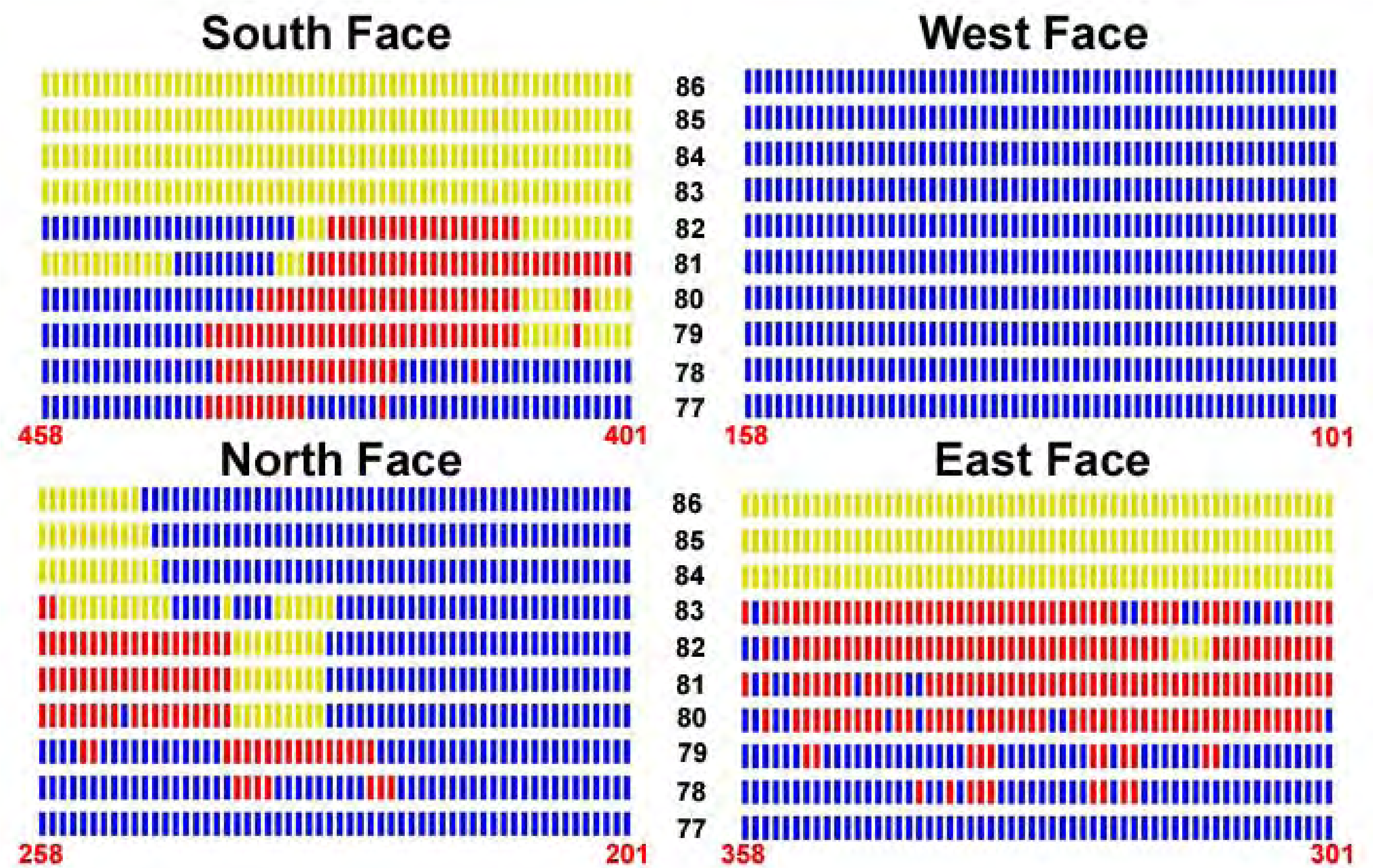


Figure H–20. The condition of windows is shown for the four faces of WTC 2 around 9:03 a.m. shortly after it was struck by United Airlines Flight 175. The colors represent ▌- window open, ▌- window in place, and ▌- not visible.

Figure H–21. An enhanced photograph of the north face of WTC 2 taken at 9:58 a.m. shows the east side of the north face. Piles of debris are evident on floor 80 and floor 81 on the northeast corner and on floor 79 to the right of column 230. The "cold" area extends over floor 80 to floor 82 from roughly column 229 to 240. There is a break in column 253 on floor 81, and the adjacent column, 254, is bent. Floors 80, 81, and 82 appear to be partially collapsed. It appears as if the collapsed floor 83 in the northeast corner has broken into sections.

H.6.1 WTC 1

It has already been mentioned that substantial fireballs formed on the north, east, and south faces immediately following the plane strike. A brief period of intense burning from openings on these faces was observed after the fireballs dissipated, but in a short period (on the order of 60 seconds) the fires seemed to "damp down" and very little flame and only light smoke was evident from the outside. This period of light burning lasted several minutes before fires began to reappear.

Rapid early fire growth was observed on the east side of the north face on floor 96 and floor 97, the center of the east face on floor 94 and floor 97, and the western side of the south face on floor 96. Even though relatively little initial damage was sustained by the west face, heavy smoke followed shortly by flame appeared around window 97-437 at 8:55 a.m. After this time, a very rapid fire spread was observed across the west face on this floor. Within a couple of minutes, over half of the windows were emitting smoke, and flames were visible in many. Even though floor 92 was not directly struck by the airplane, fire appeared on the east side of the tower on this floor shortly after 9:00 a.m.

Following the initial development of large fires, fire spread continued until WTC 1 collapsed around 10:28 a.m. At times the fires displayed the systematic, relatively slow spread expected for fire growth in a typical building. For instance, after the initial rapid growth phase, fires on floors 92, 94, 96, and 97 on the east face began to move deliberately toward the south. As they spread, the fires would burn intensely at a given location for a while before dying down. As a result, these fires developed the appearance of a wave moving slowly across the building.

There were also certain times and locations during which fire appeared to spread quite rapidly. Some of these episodes were clearly connected with rapid fire growth and likely flashover in rooms. During the first half hour, significant fires were observed toward the centers of floors 92, 94, 96, and 97 on the east face that were spreading towards the north. Each of these fires eventually reached a certain point where further fire spread was inhibited for many minutes. A review of building plans showed that walls of offices or meeting rooms were presented at the locations where fire spread was inhibited.

Apparently, these walls served as effective fire breaks that protected against further fire spread. However, for each of these floors fire and smoke eventually appeared at one of the windows beyond the walls, and after one of these windows was broken fire growth was extremely rapid and robust across the remaining windows. These observations are consistent with the occurrence of flashover within an enclosed space.

At other times, unusually rapid fire growth apparently occurred in areas that are believed to have been relatively open and not constrained by walls. One of these episodes occurred around 9:54 a.m. on the north face. Fire suddenly appeared on floor 96, a location to the west of the damage inflicted by the airplane. Within a very short period of time, fire could be seen in roughly 10 windows covering a distance of more than 30 ft.

Another example of very rapid fire growth appeared to take place on floor 98. In the early period of the fire, this floor did not appear to be heavily involved, and this remained true for quite a while. However, after 9:30 a.m., fire began to appear on this floor and by 10:00 a.m., fires were observed over significant lengths on all four faces of the tower.

One of the more unusual fire spread episodes in WTC 1 occurred just after the collapse of WTC 2 around 9:59 a.m. Within a couple of minutes, a large intense fire suddenly appeared on the south side of the west face on floor 104 in an area well above any other apparent fire. This unusual jump in fire location is difficult to explain, but is likely associated with vertical shafts located in the core of the tower.

For most of the time following the plane strike, no fire was observed on any of the floors on the south face over lengths extending from the eastern edge of the tower to near the center of the face. Fires were not observed in this region of the building until around 10:00 a.m. By the time this tower collapsed roughly 25 minutes later, intense fires extending over significant lengths of the originally uninvolved area were burning on floor 94 to floor 98 in this area.

A final example of rapid fire spread and growth in WTC 1 was described previously in the *May 2003 Progress Report* for the Investigation (NIST 2003). In this case, a line of smoke appeared suddenly over a significant length of floor 92 on the north face of WTC 1 at 10:18:48 a.m., or roughly 9 minutes before the collapse of the tower. Puffs of smoke were observed simultaneously on the north face from floors 94, 95, and 97. More isolated puffs were seen at the same time from floor 92 and floor 95 on the west face and from floor 92 on the south face. Very shortly (seconds) after the appearance of the smoke, a localized fire on floor 95 to the west of the plane strike location grew very rapidly and flames erupted from windows. Following the smoke release, a large fire began to spread rapidly across the western side of floor 92 on the north face. Previous to the appearance of the smoke, only small fires were evident on this floor. By the time the tower collapsed, this fire had spread across most of the floor and had reached the western wall. This fire was responsible for the large burst of flame from the north face observed when this tower collapsed.

H.6.2 WTC 2

The fire behavior observed in WTC 2 was qualitatively different than occurred in WTC 1. Intense fireballs were created by the released jet fuel on the south, east, and north faces immediately after the airplane struck the building. As observed for WTC 1, the fireballs were followed by a brief period (on the order of a minute) of intense flaming from windows over a large area of the building. Most of these flames then “damped down” as observed in WTC 1, but two regions of intense burning remained. One of these areas was located on floor 81 and floor 82 at the northeast corner of the tower. Flames were evident from windows on either side of the corner as well as the corner itself, which had become exposed by removal of the corner facing during the plane strike. This area is in the vicinity of large piles of debris formed during the plane strike. The second fire was located primarily on floor 79 just to the left of the center (roughly from windows 79-231 to 79-238) of the north face. This is in the area of the second debris pile described earlier. Both of these fires died slowly with time when compared to fires at other locations in WTC 1 and WTC 2. Both were still burning lightly when the tower fell 56 minutes after the plane strike.

A curious aspect of the fire behavior is the existence of an area of the building façade between these two fire locations on the north face where very little fire and/or smoke was observed before the tower collapsed. This area is roughly rectangular in shape, covering floor 80 to floor 82 and extending across windows 249 to 239. Infrared images recorded shortly following the plane strike showed that this region was quite cool relative to other sections close to the fires. This area will be referred to as the “cold spot.” Spreading fires seemed to move around this cold spot.

In general, the fires in WTC 2 appeared to be less active than those observed in WTC 1. The fires covered a smaller area of the façade and did not spread as quickly. This is true even when the shorter time between the plane strike and collapse for WTC 2 (1 hour 42 minutes for WTC 1 and 56 minutes for WTC 2) is taken into account. Nevertheless, there was significant fire spread, and instances of rapid fire growth similar to those seen in WTC 1 did take place.

Around 9:29 a.m., large flames and heavy smoke erupted from an area on the north face just to the right of the cold spot (around window 83-236) on floor 83. Four minutes and forty-one seconds later, flames suddenly appeared at a separate location on the same floor further to the right near window 83-226. Another area of fire formed just to the right of the cold spot on floor 82 around 9:54 a.m. or 5 minutes before the collapse. The fires on floor 79 of the north face also spread towards the west, approaching the western edge of the tower just prior to the collapse.

Initial fire growth on the east face was on floor 82. Around 9:12 a.m., flames could be seen in nearly half of the windows on this floor, and heavy smoke was pouring from additional windows. Only limited fire was evident on lower floors at this time. The fires on floor 82 grew smaller after this time, and most were no longer visible when the tower collapsed. Around 9:35 a.m., heavy flames and smoke appeared over large areas of floor 79 and floor 80. These fires abruptly died down 45 seconds later, before growing back slowly during the remainder of the time before the tower collapsed.

In the early period following the plane strike, fire growth on the south face was seen primarily on floor 81 with active fires present on both sides of the airplane strike location. Smaller isolated fires were present on other floors around the area damaged by the airplane. These fires were relatively quiet and stationary until just prior to the collapse. At 9:56 a.m., there was a sudden release of smoke along much of floor 80 extending from the area of the plane strike to near the western edge. During the next 2 minutes, an intense fire developed covering approximately windows 81-441 to 81-454.

No smoke or fire was observed near the floors struck by the airplane on the west face of WTC 2. Some smoke was apparent at windows higher on the face. This was most likely coming from windows broken by occupants located on these floors.

H.7 EVIDENCE FOR COLLAPSED FLOORS IN WTC 2

H.7.1 Hanging Objects

In the *May 2003 Investigation Progress Report* (NIST 2003), a photograph was shown in which there appeared to be a floor draped across a number of windows extending roughly from 310 to 342 across the east face of floor 82 of WTC 2. Figure H–22 compares an image taken shortly after the plane strike at 9:03 a.m. and one taken at 9:55 a.m. shortly before the tower collapsed. At the earlier time, the hanging object is already present, but is seen through the windows draped much higher on the floor 82. An interpretation consistent with these observations is that floor 83 along the east side of WTC 2 was partially collapsed over a significant fraction of its length by the passage of the plane through the building. At the later time the floor has sagged further. By reviewing a number of photographs and videos, it has been determined that the change in floor position occurred between 9:34 a.m. and 9:38 a.m.

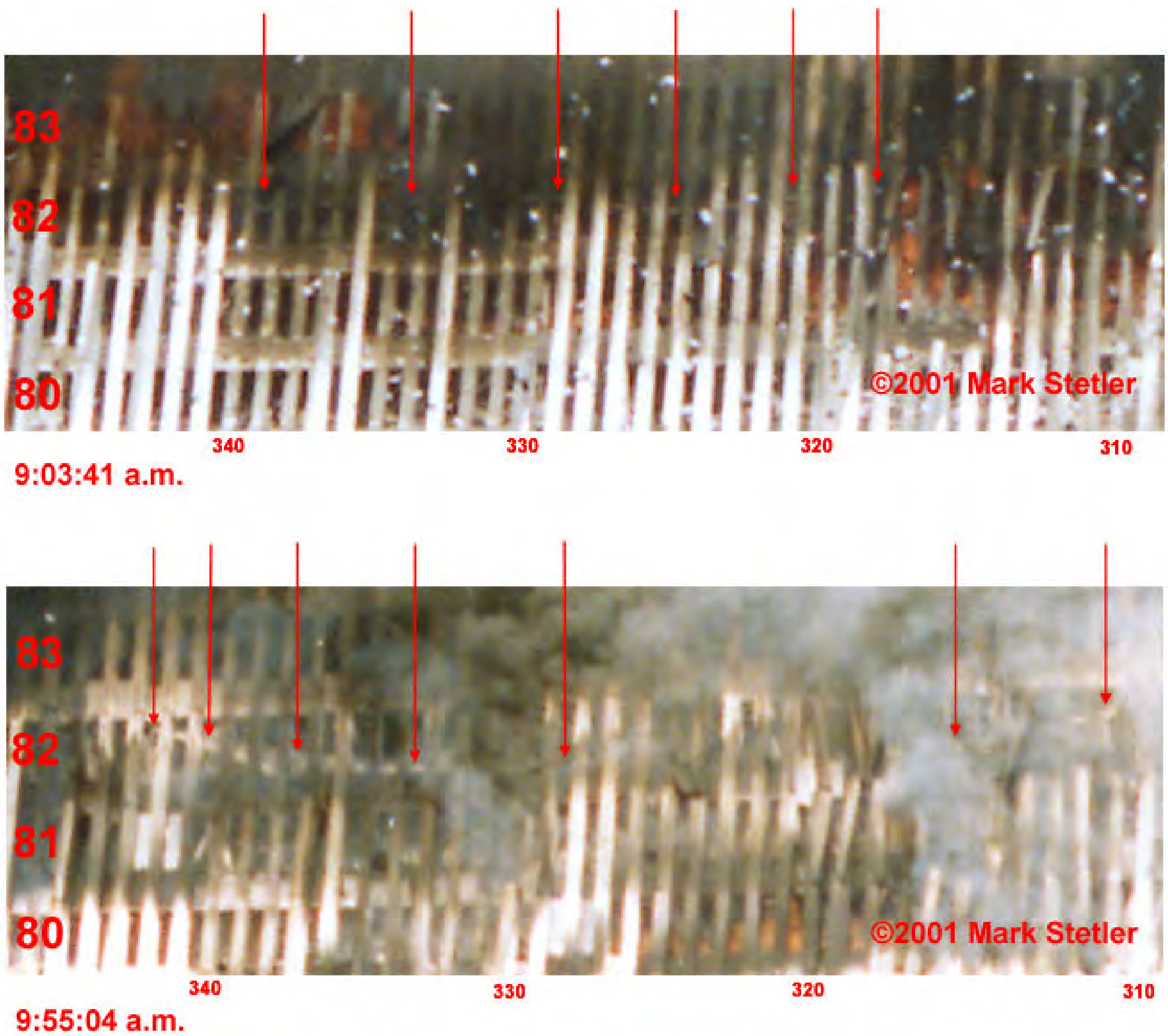


Figure H–22. Images of the east face of WTC 2 taken shortly after the plane struck and shortly before the tower collapse are shown. An object that is most likely a floor can be seen through windows on floor 82 as indicated by the arrows. The object has dropped lower between the times the two photographs were taken.

Very similar objects, albeit of shorter length, are seen hanging in windows in images taken from the north. These objects are apparent in Fig. H–21 hanging below floors 81, 82, and 83. As seen through windows on floor 82 (corresponding to the floor 83), the floor appears to have split into at least two sections.

H.7.2 Molten Material

It has been reported in the FEMA report (McAllister 2002) as well as in the media that what appeared to be molten metal was observed pouring from the north face near the northeast corner. This is the area where the sustained fires were seen. Video records and photographs indicate that the material first appeared at 9:51:52 a.m. and continued to pour intermittently from the building until the time of collapse. Some of the material can be seen falling in Fig. H–21. Close-up video and photographs of the area where the material is pouring from have been examined and show that it is falling from near the top of window 80-256. The most likely explanation for this observation is that the material had originally pooled on the floor above, that is, floor 81, and that it was allowed to pour out of the building when this floor either pulled away from the outer spandrel or sank down to the point where the window was exposed. The fact that the material appears intermittently over a several minute period suggests that the floor was giving way bit by bit.

The composition of the flowing material can only be the subject of speculation, but its behavior is consistent with it being molten aluminum. Visual evidence suggests that significant wreckage from the plane passed through the building and came to rest in the northeast corner of the tower in the vicinity of the location where the material is observed. Much of the structure of the Boeing 767 is formed from two aluminum alloys that have been identified as 2024 and 7075 and closely related alloys. These alloys do not melt at a single temperature, but melt over a temperature range from the lower end of the range to the upper as the fraction of liquid increases. The Aluminum Association handbook (Aluminum Association 2003) lists the melting point ranges for the alloys as roughly 500 °C to 638 °C and 475 °C to 635 °C for alloys 2024 and 7075, respectively. These temperatures are well below those characteristic of fully developed fires (ca. 1,000 °C), and any aluminum present is likely to be at least partially melted by the intense fires in the area.

H.8 PROGRESS ON COLLECTION OF IMAGES AND ANALYSIS FOR WTC 7

Visual material is also required to characterize the initial damage to, fire spread in, and collapse behavior of WTC 7. Considerable useful material has been collected, but the visual record for times between the collapses of WTC 1 and WTC 7 is much less complete than those for the two towers. The reasons for this are easy to understand. Following the collapses of the towers, most people were focused on escape or rescue. A large dust cloud was formed by the collapses, and fires developed that generated large amounts of smoke. Both tended to obscure views of WTC 7, particularly from the south due to the northwesterly wind direction on September 11, 2001.

Both photographs and videos have been included in the database that show fires and damage to the east, north, and west faces of WTC 7. Some of this material has been timed, but in general the record is insufficient to allow generation of a complete time line of fire behavior for the relevant period. Numerous images show the upper portion of WTC 7 from the south, but the actual face of the building is generally obscured by smoke. No clear images of the lower portion of the south face have been obtained despite a careful search and repeated appeals for the public's help. This is particularly unfortunate since most of the damage caused by the collapses of the towers, and particularly WTC 1, should have occurred on this face.

There is considerable interest in images showing the collapse of WTC 7. Currently, there are at least four videos in the database that include the collapse, primarily from northerly directions, as well as several photographs. While not ideal, these are providing adequate information for characterizing the collapse sequence, and some progress along these lines has been made.

An effort has begun to map out the same information concerning fires, smoke, and windows as in the towers using visual material in the database. This effort will continue with a goal of mapping out as much of the fire time line as possible based on the material in the database.

H.9 SUMMARY WITH KEY FINDINGS

This section provides a brief summary of progress on the collection and analysis of visual data along with key findings.

The approaches used to identify and obtain visual material related to the WTC disaster are described along with the approaches employed by NIST to archive and catalog the material. Material is either saved in its original digital format or digitized and saved, and a commercial software package has been used to provide data entry, a searchable database, and ready access to assets for review. The large numbers of attributes used to characterize the photographs and videos are included.

Separate databases are provided for photographic and video materials. A major effort has focused on assigning accurate times to the material, and the approaches used are summarized. In excess of 6,700 photographs and 6,900 video clips have been included in the databases and 45 percent and 39 percent, respectively, of these have assigned times accurate to 3 seconds or better.

Major events timed to an accuracy of 1 second are:

- First plane strike on WTC 1: 8:46:30 a.m.
- Second plane strike on WTC 2: 9:02:59 a.m.
- Collapse of WTC 2: 9:58:59 a.m.
- Collapse of WTC 1: 10:28:25 a.m.
- Collapse of WTC 7: 5:20:52 p.m.

An approach has been developed to characterize the observed fire behaviors at the periphery of the buildings on a window-by-window basis by determining whether windows are open or closed and whether smoke and/or fire are observed. If smoke is present, it is characterized as “light” or “heavy”, and fires are characterized as “spot” (a small local fire), “fire inside,” and “external flaming.” The observations are coded in separate electronic spreadsheets for each building, façade, and time.

Two approaches are used to visualize the fire-related parameters. The first is a Web-based application that displays single sides of the towers at a single time. The second is a time-dependent three-dimensional representation based on Smokeview (Forney and McGrattan 2003; Forney, Madrzykowski, McGrattan, and Sheppard 2003).

Photographs and videos have been used to characterize several aspects related to the plane strikes on the towers and the distribution of damage on the external faces. For WTC 1, locations where the ends of the wings and vertical stabilizer of the tail section struck the north face and the damage to the steel façade are mapped. The behavior of fireballs generated by the release of fuel as a result of the collision of the aircraft with the tower and initial tower damage as reflected in broken windows is used to characterize the distribution of damage to the facades of the tower. In addition, it has been shown that an exterior panel section from the south face was dislodged and landed on the ground. It contained an aircraft wheel that passed through the tower.

The following conclusions are reached concerning the immediate effects of the plane strike on WTC 1:

- The airplane struck columns on the north face ranging from 109 to 152 and covering floor 93 to floor 99.
- Damage and initial fire growth were greater on the east face of the tower than on the west. Significant damage and early fire growth occurred on the west side of the south face, but not on the east side.
- A three-story panel section was knocked from the south side of the tower and had an aircraft wheel lodged in window 95-329.

Visual evidence related to the plane strike on the south face of WTC 2 is more extensive than for WTC 1. This has allowed additional analyses beyond the mapping of damage on the plane strike face and façade damage to the remaining faces. The following conclusions have been reached concerning the immediate effects of the plane strike on WTC 2:

- The aircraft struck the tower with a measured speed of 545 miles per hour ± 18 miles per hour.
- The collision of the aircraft caused a measurable sway of the tower that lasted more than 4 minutes. The period of oscillation was 11.3 seconds.
- The airplane struck columns on the south face ranging from 404 to 443 and covering floor 77 to floor 85.
- Large areas of the façade were removed and/or damaged along the east face of the tower and on the eastern side of the north face. No façade damage or window breakage was evident on the west face.
- Debris piles are observed in the northeast corner of the tower primarily on floors 80 and 81. Debris is also evident towards the center of the north face on the floor 79.
- Column 253 on the north face is broken on the floor 81 and the column 254 is severely distorted.

Detailed maps for fire behavior are currently being made. This update characterizes general fire behaviors for the two towers and notes some particularly interesting observations. For WTC 1, the following observations are highlighted.

- Extensive fires observed immediately following the plane strikes and which are most likely associated with released jet fuel damped down after roughly 60 seconds.

- In the period following the plane strikes fires tended to reappear over a period of many minutes. Initial fire growth was principally observed on floor 96 and floor 97 on the north face, floor 94 and floor 97 on the east face, floor 96 on the south face, and floor 97 on the west face.

- Observed fire spread rates were quite variable. Examples of both relatively slow and very rapid apparent fire spread are described.

- Interior walls at several locations were inferred to protect areas of the towers for a period of many minutes, though they were typically eventually breached by nearby fires.

- Following the collapse of WTC 2, a large fire appeared and grew rapidly on the west face at floor 104.

- There was an extensive area of the façade on the eastern side of the south face for which no fire was observed until at least 1 hour following the plane strike. When fires finally did appear in this area their growth was rapid over multiple floors.

- A large amount of smoke was suddenly released from floor 92 on the north face at 10:18:40 a.m. Smoke was expelled simultaneously from other floors and faces. Immediately after the smoke release rapid fire growth was observed at an isolated location on floor 95 and across much of the west side of the north face on floor 92.

Observed fire behaviors for WTC 2 were somewhat different than for WTC 1. This is true even when the differences in times between plane strikes and collapses (1 hour and 42 minutes for WTC 1 and 56 minutes for WTC 2) are considered. The following observations concerning fire behavior in WTC 2 are emphasized:

- Extensive fires observed immediately following the plane strikes and most likely associated with released jet fuel damped down after roughly 60 seconds.

- Two regions of intense fire remained following the initial fire period due to jet fuel burning. These fires were located on floor 81 and floor 82 in the northeast corner and towards the center of the north face on floor 79. These fires burned for longer periods than observed elsewhere in WTC 1 and WTC 2. They are located in regions of the tower where debris piles are observed.

- No large fires were observed in a multi-floor region on the north face located between the two fire areas described in the last bullet.

- Initial fire growth in areas away from the sustained fires was along the east face of floor 82. Large fires did not appear on lower floors of this face until later and were sporadic in space and time.

- Prior to the tower collapse, fire spread primarily from east to west was observed on floors 79, 82, and 83 of the north face.

- A sudden release of smoke from windows on the west side of the south face on floor 80 occurred at 9:56:37 a.m. This was followed very shortly by the appearance of heavy fire.

A number of photographs and videos show what appears to be floor 83 hanging across window openings over a large fraction of floor 82 on the east face of WTC 2. This object is observed very shortly after the plane strike and is found to drop lower prior to the tower collapse. On the north face, shorter lengths of what appear to be floors 81, 82, and 83 are seen hanging through the windows below.

Starting around 9:52 a.m., a molten material began to pour from the top of window 80-256 on the north face of WTC 2. The material appears intermittently until the tower collapses at 9:58:59 a.m. The observation of piles of debris in this area combined with the melting point behaviors of the primary aluminum alloys used in the Boeing 767 suggest that the material is molten aluminum derived from aircraft debris located on floor 81.

The visual record for the period following the collapses of the two towers is much less complete than prior to this time. In addition to the general chaos caused by the collapses, significant dust and smoke from fires started by the collapses obscured the site. As a result, it has not been possible to identify clear visual images showing the damage to the south face of WTC 7 caused by the collapses of WTC 1 and WTC 2. The number of videos and photographs showing fires on the east, north, and west faces of WTC 7 is limited and sporadic. The images that are available are being used to generate an approximate time line for fire growth and spread.

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Appendix I
INTERIM REPORT ON ASSESSMENT OF SPRAYED FIREPROOFING IN THE WTC TOWERS––METHODOLOGY

I.1 INTRODUCTION

The structural steel in the World Trade Center (WTC) towers was "fireproofed" with sprayed fire resistive materials (SFRMs). These materials are packaged as dry ingredients, and water is added by a pressurized system as the materials are sprayed onto the steel. The water mixes with the cementitious materials and provides "stickiness" that allows the SFRM to adhere weakly to the steel. With time, the cementitious materials harden, and excess water evaporates. When dry, SFRMs provide an insulation barrier to reduce the vulnerability of the steel to excessive temperature rise during a fire.

Analysis of the effects of the fires on the structural capacity of the damaged WTC towers as a function of time requires knowledge about the condition of fireproofing on the various structural components, namely, the exterior columns, the spandrel beams, the floor trusses, and the core columns. Because of the method of application, sprayed fireproofing will have variable thickness, especially when applied to long, thin elements such as the diagonals and chords of the floor trusses. In addition, fireproofing was dislodged during the impact, either from direct impact by debris or from vibrations of the members. The thermal properties of the fireproofing also need to be known as a function of temperature.

The thermal-structural analysis of the WTC towers focused on two objectives: (1) analysis of the undamaged buildings exposed to postulated fires, and (2) analysis of damaged buildings exposed to the fires that occurred after impact. In order to reduce the uncertainties in the calculated thermal histories of various structural elements, the condition of the sprayed fireproofing as it existed on September 11, 2001, needs to be estimated as accurately as possible. In addition, reasonable estimates of the extent of fireproofing dislodged by the aircraft are needed. This appendix discusses the approach that will be used for this purpose.

To gain an understanding of the effect of fireproofing thickness and its variability on the steel temperature during exposure to fire, a simple finite-element model was used for a sensitivity study. The information gained from that study is reviewed first. A brief summary of the construction history of the sprayed fireproofing in WTC 1 and WTC 2 is presented. This is followed by a quantitative assessment of in-place thickness and its variability based on available data. The rationale for the thickness of fireproofing to be used in the structural fire endurance analyses is presented. The tests conducted to determine the thermal properties of fireproofing materials similar to those used in the WTC towers are reviewed. The approach used to gain an understanding of the inherent fragility of sprayed fireproofing is discussed, and the scheme for estimating the extent of damage during impact is summarized.

I.2 SENSITIVITY OF THERMAL RESPONSE TO FIREPROOFING GEOMETERY

The fireproofing thickness has a great effect on the thermal response of the structural elements for a given fire condition. While others have considered the effect of thickness of fireproofing, the effect of the variation of thickness along the length of a member is not well known. A sensitivity study using finite element modeling of heat transfer was conducted to investigate the sensitivity of steel temperature to the variability in fireproofing thickness.

The simplified model that was used is shown in Fig. I–1. A 1 in. thick, 60 in. long steel plate (cyan color) was coated with fireproofing material (purple color) and subjected to the uniform radiative flux arising from a 1,100 °C fire. As shown in Fig. I–1 (b), the fireproofing is modeled with a layer of finite elements (0.125 in. thick and 0.6 in. long) having the thermal properties of fireproofing (purple). A parametric study was conducted with average thickness of fireproofing varying from 0 in. to 2 in. in increments of 1/4 in. The effect of variability in thickness was modeled by imposing a normal probability distribution on the fireproofing thickness along the length of the steel plate. The assumed standard deviation varied from 0 in. (uniform thickness) to 1 in. A psuedo-random number generator was employed to determine the thickness at each cross section based on the assumed average thickness and standard deviation. The layer representing fireproofing was taken to be twice the average thickness, and the thickness of fireproofing at any cross section was modeled by assigning a low heat capacity and a high thermal conductivity to those elements that do not provide fireproofing. Figure I–1 (c) shows an example of variable thickness fireproofing; in this case, the average thickness is 1 in. and the standard deviation is 3/8 in.

When the model in Fig. I–1 is exposed to the thermal flux representing an 1,100 °C fire, the surface of the insulation heats up quickly to the gas temperature (1,100 + 273 = 1,373 K). Numerical simulation was performed over a 2-h period, and the steel temperature at five locations was recorded at 30 min, 60 min, 90 min, and 120 min of exposure. The temperature recording locations are 6 in. from each end and at 12 in. intervals, which are shown as numbers 1 to 5 in Fig. I–1 (a). The initial temperature of the model is 300 K.

Figure I–2 shows temperature contours (in K) through the fireproofing and steel at 60 min after initial exposure for the model shown in Fig. I–1 (a). The fireproofing surface temperature is close to the gas temperature of 1,373 K, while the steel temperature is 311 K. If the fireproofing were of uniform thickness, the isotherms would be a series of lines parallel to the plate. It is seen that, when the thickness of fireproofing is variable, the isotherms follow the shape of the fireproofing surface contour. Thus, the temperature history at any point in the steel depends on the loc al thickness of the fireproofing.

Figure I–3 shows the steel temperature at the far sensor #1 (6 in. from the end) as a function of time for various insulation thicknesses ranging from 0 in. to 2 in. (the thickness is indicated by the numbers on the curves). For the case in Fig. I–3 (a), the fireproofing is of uniform thickness, and for the cases in Fig. I–3 (b), the thickness varies with a standard deviation of 1 in. The time to reach a temperature of 600 °C is used as a measure of relative performance. It is seen that the presence of high variability in thickness has a detrimental effect of the protection provided by the fireproofing. For example, for a uniform thickness of 0.5 in., it takes about 60 min for the steel at point #1 to reach 600 °C; but when the standard deviation of the thickness is 1 in., the average thickness has to be 1.75 for the same level of thermal protection.

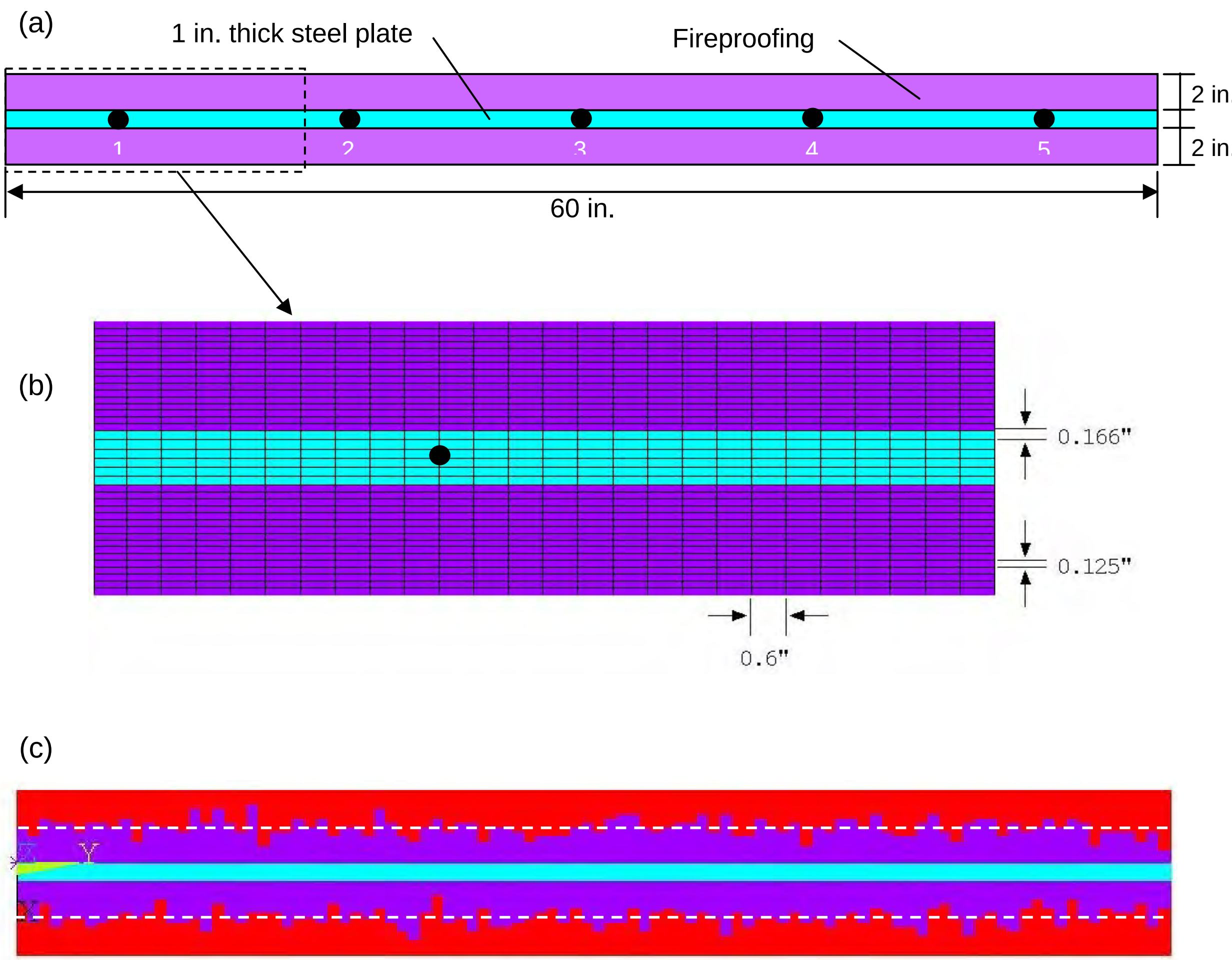

Figure I–1. Model used to study effects of fireproofing thickness and variability of thickness on steel temperature: (a) physical model used in analyses (points 1 to 5 are locations where temperatures are monitored), (b) finite element mesh used to represent physical model, and (c) finite element model to represent variable thicknessof fireproofing (purple) (the elements in red represent material of high thermal conductivity).

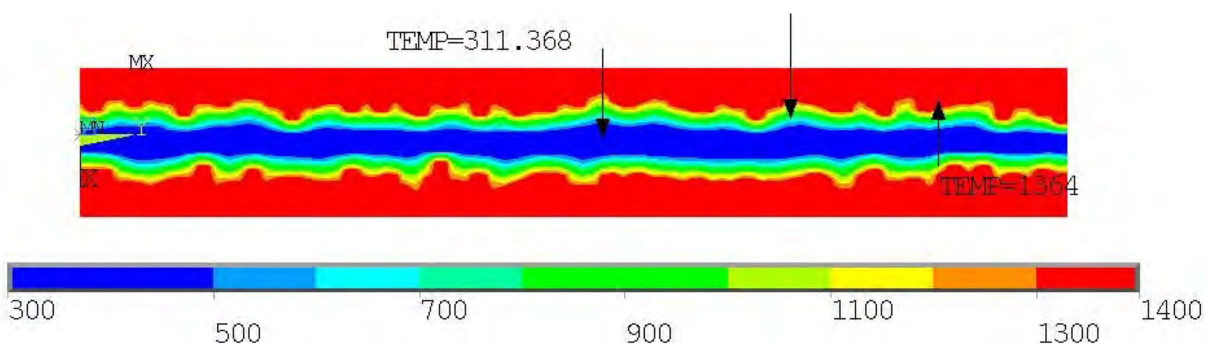

Figure I–2. Temperature distribution after 1 h of exposure to gas temperature of 1,100 °C (1,373 K).

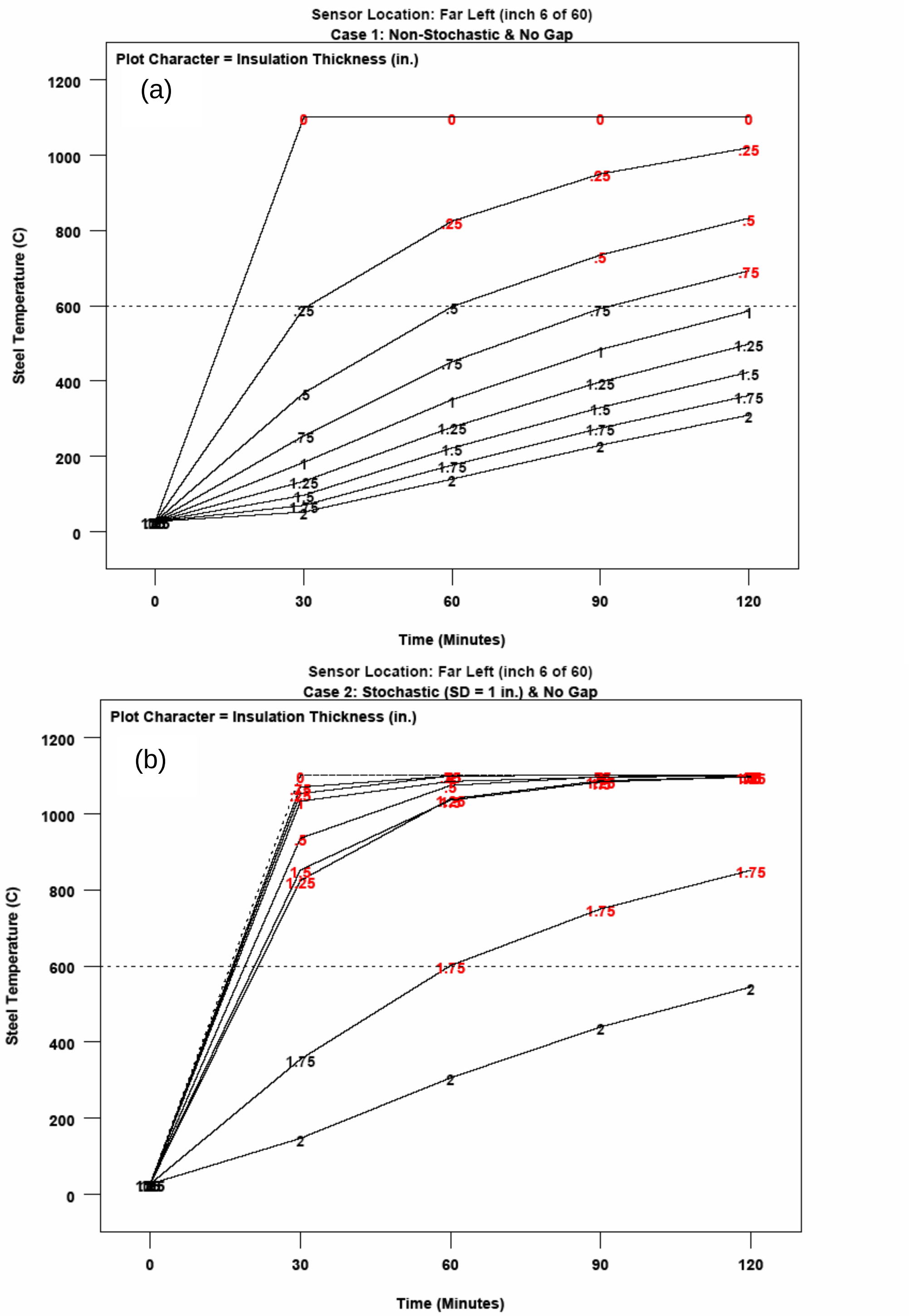

Figure I–3. Variation of steel temperature (at a point 6 in. from end of plate) with time for different average thicknesses of fireproofing (shown as numbers on the curves): (a) uniform thickness, and (b) variable thickness with a standard deviation of 1 in.

In addition to the effect of variation in thickness, it is important to understand the effect of missing fireproofing over a portion of a member. As an example, Fig. I–4 shows missing fireproofing from a diagonal of a bridging truss of the WTC towers floor system. Figure I–5 (a) shows an example of a numerical model with missing fireproofing. In this case, there is 12 in. of missing fireproofing on the steel plate, which is otherwise protected by 2 in. of uniform thickness fireproofing. Figure I–5 (b) shows the temperature contours (isotherms) at time 50 min. For comparison, Fig. I–5 (c) shows isotherms at the same time in a plate with no gap in the fireproofing. As expected, the bare steel at the missing fireproofing is at the gas temperature, but more importantly the "gap" in fireproofing leads to a "leakage" of heat into the interior steel.

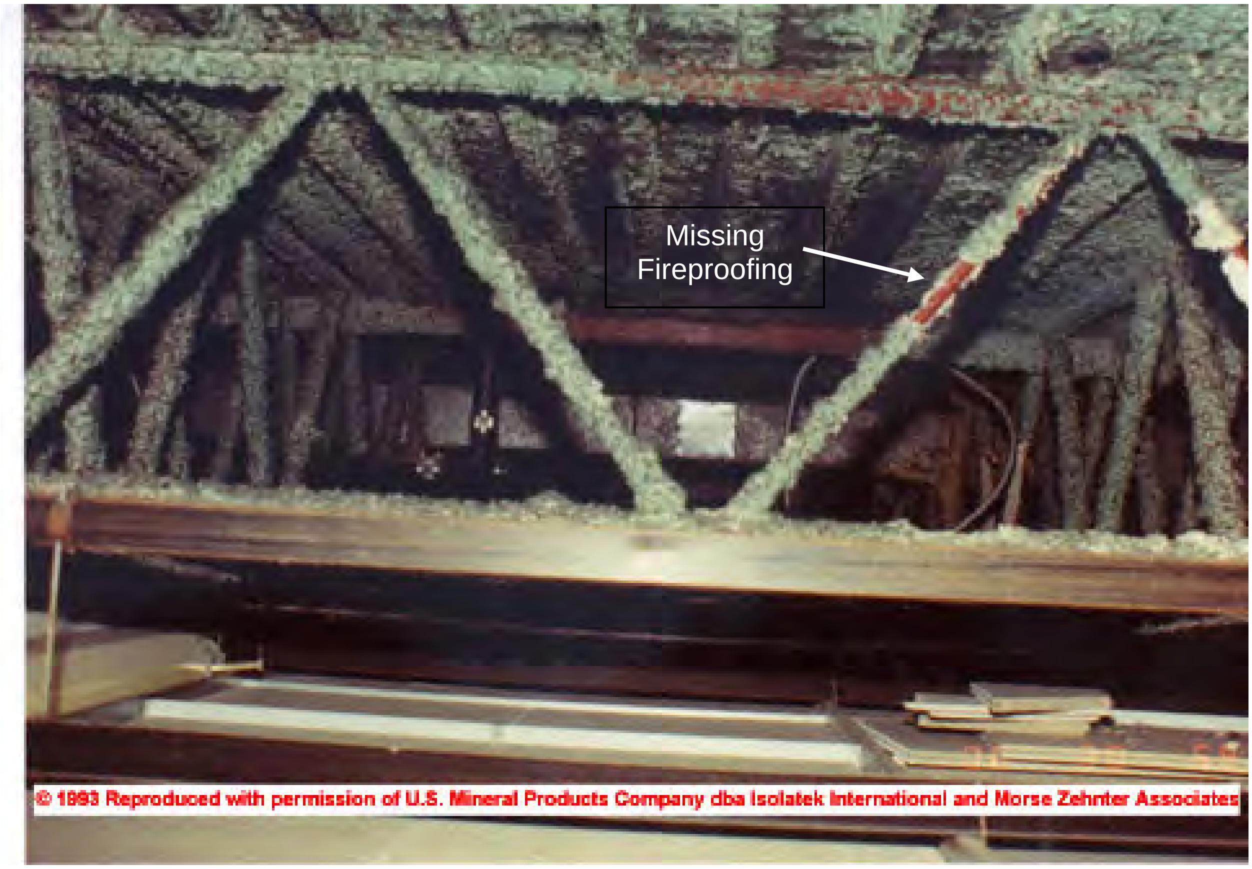

Figure I–4. Example of "gap" in fireproofing on diagonal member of a bridging floor truss.

The combined effects of variation in thickness of the fireproofing and length of missing fireproofing were examined by a factorial study with the following factors:

- Average thickness of fireproofing varying from 0 in. to 2.0 in. in 1/4 in. increments;
- Standard deviation of fireproofing thickness of 0 in., 0.25 in., 0.5 in., 0.75 in. and 1.0 in.; and
- Length of missing fireproofing varying from 0 in. to 30 in., in 6 in. increments.

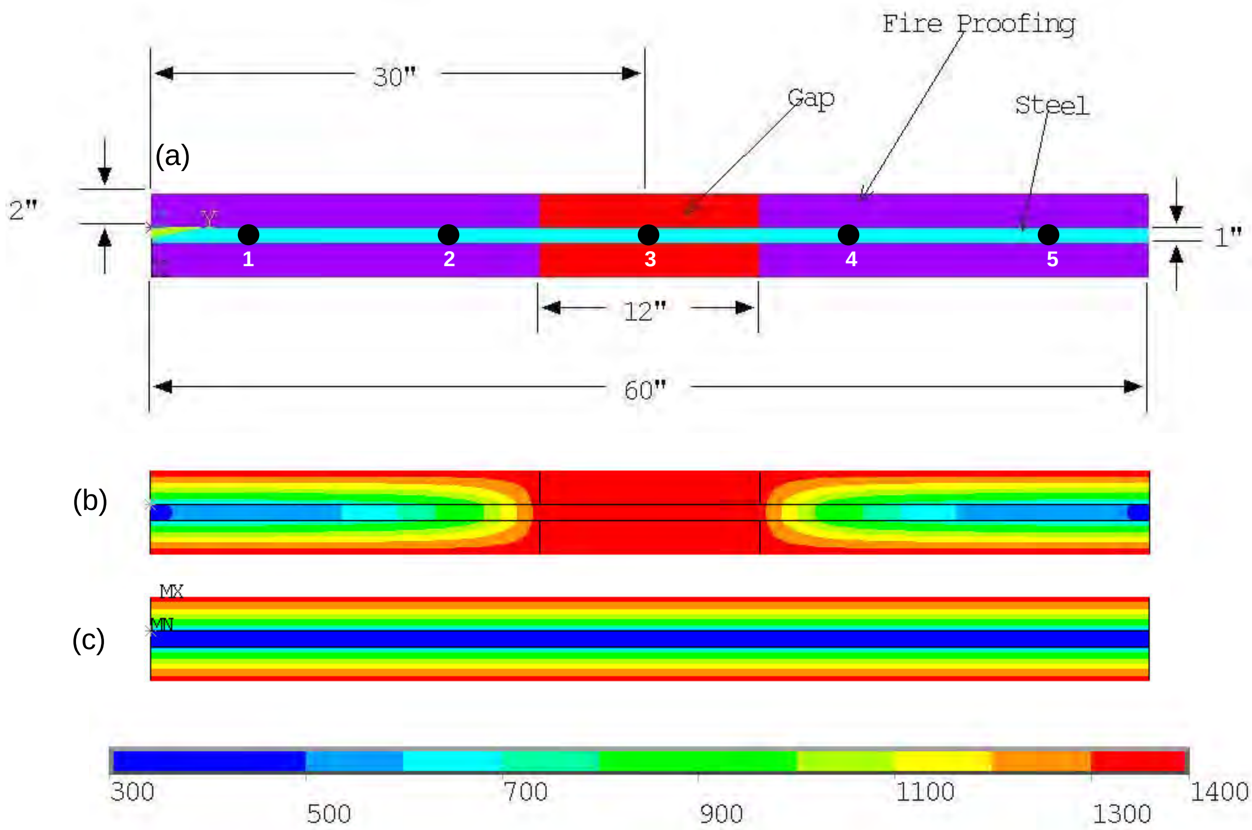

Figure I–5. Effects of gap in fireproofing: (a) model of plate with fireproofing having 2 in. uniform thickness and 12 in. gap, (b) isotherms (K) at time = 50 min with 12 in. gap, and (c) isotherms without gap.

The results of the sensitivity study can be summarized in a series of plot matrices, which show the time histories of the steel temperature for different combinations of gap length and variability in fireproofing thickness. For example, Fig. I–6 shows the plot matrix for the temperature history at point #2 (18 in. from the end of the plate). Each plot contains a series of curves representing different average thickness of fireproofing, as in Fig. I–3. Each column of plots represents a constant value of thickness variability (standard deviation), and each row represents a constant gap length. The plot in the upper left corner represents the case of uniform thickness of fireproofing and no gap, which is the same plot as in Fig. I–3(a). (Note that for the case of uniform thickness and no gap, the steel temperature at any point in a cross section is the same along the length of the plate, as shown in Fig. I–5(c).) For gaps of 24 in. and 30 in., the temperature at point #2 rises rapidly because there is no fireproofing on the plate at that location. This explains the shapes of the curves in the two lower rows. In going from left to right in one of the top four rows it is seen that as variability of thickness increases, the time histories shift upward, thereby reducing the time to reach 600°C. This is the same observation as shown in Fig. I–3. Moving from the top to the bottom in any column shows the effects of increasing gap length. The effect of gap length depends, of course, on where the steel temperature is measured. At a point within the portion of steel that is bare, the temperature rises quickly. At points within the steel that are surrounded with fireproofing, the gap provides a path for heat flow, as shown in Fig. I–5 (b). As a result, points in the steel within the vicinity of the missing fireproofing will experience higher temperatures, as indicated by the rising trend of the curves in going downward from the top of a column in Fig. I–6. The National

Institute of Standards and Technology (NIST) does not have sufficient information to determine the frequency of occurrence of these gaps or their typical locations. Therefore, gaps in fireproofing will not be considered in the thermal modeling.

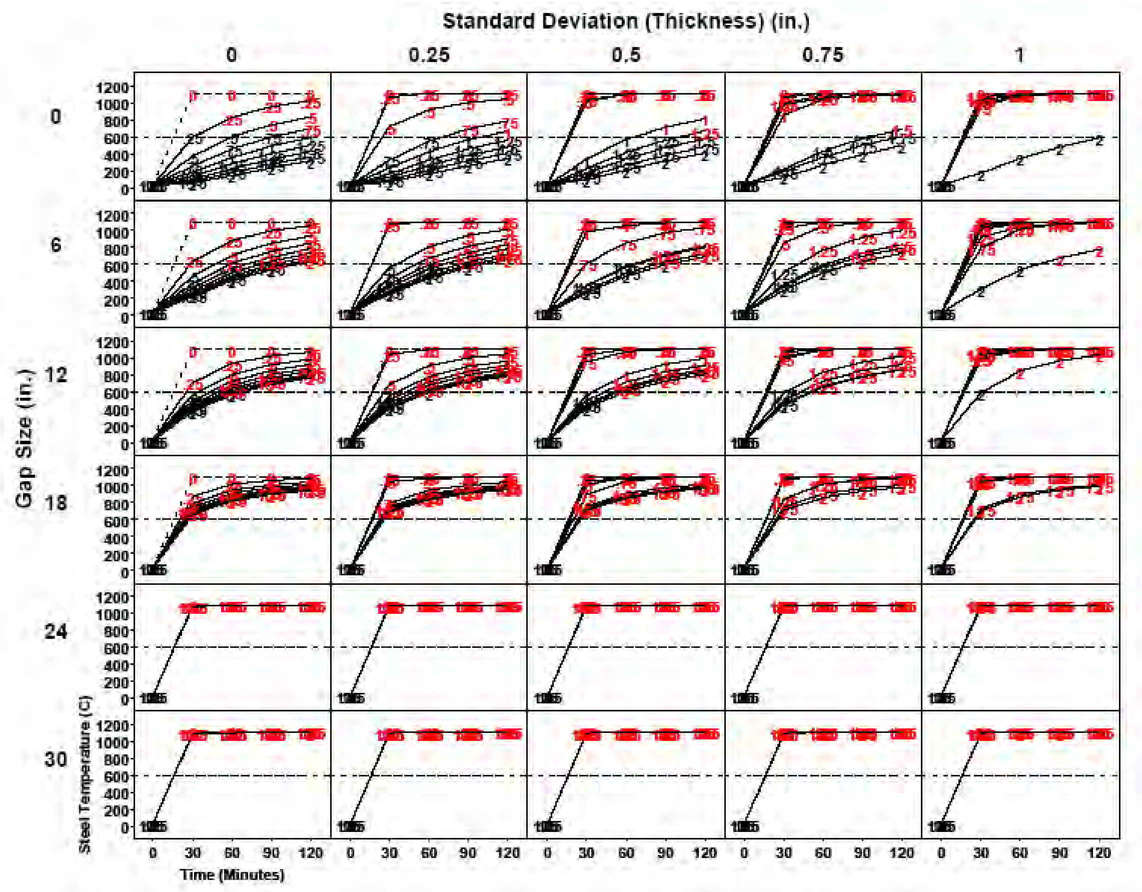

Figure I–6. Example of plot matrix from sensitivity study of the effects of missing fireproofing and variability of fireproofing thickness on steel temperature. Each graph is a temperature history of the steel at point #2 (see Fig. I–5) for different thicknesses of fireproofing.

I.3 IN-PLACE CONDITIONS OF FIREPROOFING BEFORE IMPACT

I.3.1 History of WTC Fireproofing

In Appendix 4 of the *May 2003 Progress Report* (NIST SP 1000-3), the history of the sprayed fireproofing, as reconstructed from available documentation, was reviewed. Basically, the following significant activities took place:

- 1969: Decision made to use 1/2 in. of CAFCO BLAZE-SHIELD Type D (asbestos fibers) sprayed fireproofing.

- 1970: Use of CAFCO BLAZE-SHIELD Type D was discontinued at the 38th floor of WTC 1; remaining fireproofing to use CAFCO BLAZE-SHIELD Type DC/F (mineral wool fibers).
- 1994: Thickness measurements of fireproofing on trusses of floors 23 and 24 of WTC 1.
- 1995: Port Authority performed study to establish sprayed fireproofing thickness for tenant alterations.
- 1999: Port Authority established guidelines for fireproofing repairs, replacement, and upgrades.
- Late 1990s: Floor truss fire protection upgraded to 1 1/2 in. during tenant alterations using CAFCO BLAZE-SHIELD Type II. In-place measurements of thickness, density, and bond strength recorded.

I.3.2 Specified Thickness of Fireproofing

WTC project specifications for spray-applied fireproofing do not provide required material thickness or hourly ratings. However, a letter dated October 30, 1969, from Robert J. Linn (manager, Project Planning, WTC) to Mr. Louis DiBono (Mario & DiBono Plastering Co., Inc.) states, in part:

> …Tower “A” columns that are less than 14WF228 will require 2 3/16″ thick of ‘Cafco [B]laze-Shield Type D’ spray-on Fireproofing. All Tower columns equal to or greater than 14WF228 will require 1 3/16″ of fireproofing…
>
> All Tower beams, spandrels and bar joists requiring spray-on fireproofing are to have a 1/2″ covering of “Cafco.”

No reference is made in this letter to the required thickness of fireproofing of core box columns or exterior built-up columns.

Alcoa was the supplier of the aluminum cladding on the exterior columns (Contract WTC 400.00), and the following “Note 11” was included among the “General Notes” of their drawings:

> 11. Exterior column and spandrel fireproofing–CAFCO BLAZE SHIELD Type D Fireproofing. Interior column and spandrel fireproofing–Vermiculite plaster aggregate fireproofing with finished plaster coat on exposed areas of columns. (3 hr on spandrels, 4 hr on columns)

Fireproofing Thickness

Rating	**Cafco**	**Vermiculite Aggregate**
4 hr (heavy column)	1 3/16″	7/8″
3 hr (spandrels)	1/2″	1/2″

In 1995, the Port Authority performed a study to establish the requirements for applying spray-on fireproofing to the floor trusses in the case of new construction (alterations conducted when tenants vacated the space) in the towers. The study estimated the fireproofing requirements for the floor trusses of the towers based on "the fireproofing requirements for Design No. G805 contained in the Fire Resistance Directory" of Underwriters' Laboratories. The study concluded that 1 1/2 in. of spray-on mineral fiber fireproofing, "when applied directly to the chords and web members," was sufficient to achieve the required 2 h rating for the floor trusses. In the years between 1995 and 2001, fireproofing was upgraded in a number of the floors affected by the fires on September 11, 2001.

The specified fire protection is summarized in Table I–1.

Table I–1. Specified passive fire protection.

Structural Component	Member Size	Location	Material	Thickness (in.)
Floor trusses	All	NA	CAFCO DC/F	1/2
Interior columns[a]	< 14WF228	NA	CAFCO DC/F	2 3/16
	≥ 14WF228	NA	CAFCO DC/F	1 3/16
Exterior columns	"heavy"	Exterior faces	CAFCO DC/F	1 3/16
	"heavy"	Interior faces	Vermiculite aggregate	7/8
Spandrel beams	All	Exterior face	CAFCO DC/F	1/2
	All	Interior face	Vermiculite aggregate	1/2

a. No thicknesses specified for core beams and box columns.
Key: NA, not applicable.

In a letter dated July 25, 1966, from Emery Roth and Sons to the Port of New York Authority, it is stated "Since the deck is non-structural it will not be fire proofed." Photographs show that in some areas the underside of the metal deck was indeed not fireproofed, while in other areas fireproofing appears to be present but of undetermined thickness and possibly resulting from overspray. Photographs reveal that the dampers and damper saddles were not fireproofed. Additionally, it is unclear whether the bridging trusses were required to be fireproofed in all areas. Subsequent to the design and construction of the WTC towers, some information has been found that further describes the elements of the structural systems that were indeed fireproofed.

I.3.3 As-Applied Thickness and Variability

The actual thickness of a spray-applied fire protection material generally exceeds the specified thickness by some amount. Since both towers collapsed on September 11, 2001, and most of the fireproofing was either dislodged or abraded (or scraped) off in the collapse, no examples remain of the "as installed" condition of the fireproofing. To make an estimate of the as-applied thickness and variability in thickness, several sources of information have been employed, including measurements taken by the Port Authority, condition surveys and anecdotal information, and photographs and video clips showing the condition of the fireproofing in selected areas. Each of the structural components or systems is considered here separately.

Steel Truss-Supported Floor System

Qualitative information on the "as installed" fireproofing thickness for the floor system first appears in Sample Area Data Sheets from 1990, in which comments on the state of the in-place fireproofing were recorded. As an example, the data sheet for floor 29 of WTC 1 states the following for the South West quadrant of the floor:

> Fluffy spray-on fireproofing coating the support beams, joists, and deck above the ceiling. The thickness of the material on the beams and joists was consistently about 1/2″ Regarding the deck it ranged from very sparce [sic] in areas to 1/4″ other areas.

Similar statements were recorded for the remaining quadrants of the floor.

Information regarding quantitative inspection of existing fireproofing appears in documentation from 1994. That year, the Port Authority performed a series of thickness measurements of the existing fireproofing on floors 23 and 24 of WTC 1. Six measurements were taken from "both flanges and web" of each of 16 randomly chosen trusses on each floor at those locations where the fireproofing was not damaged or absent.

The averages of six measurements per joist that were recorded on the two floors are presented in Table I–2. Measured average thickness varied between 0.52 in. and 1.17 in. For the 32 measurements (16 on each floor), the overall average was 0.74 in. and the standard deviation of these averages was 0.16 in. Four of the 32 floor trusses, had an average thicknesses between 0.52 in. and 0.56 in. These measurements suggest that the minimum average thickness exceeded 1/2 in.

This same report stated that, on floor 23,

> ... truss members located adjacent to the outside walls (within 3 ft) are devoid of fireproofing material. Visual inspection on floor 24 was not possible, as this area still has a lowered ceiling in place.

The data in Table I–2 can be examined further to understand the variability of the fireproofing thickness in the non-upgraded locations. Figure I–7 (a) shows the average thicknesses measured on the floor trusses of floors 23 and 24. The values appear to be similar for the two locations in terms of overall average thicknesses and the variation in average thickness. A formal analysis of variance indeed indicated no statistically significant differences between the overall mean thicknesses for the two floors. Thus, the two groups of data can be combined into one. A question to be answered is whether the values of average thickness follow a normal distribution. To answer this question, histograms and normal probability plots are used. Figure I–7 (b) shows a histogram of the average thicknesses, and it appears to be non-symmetrical and skewed to the right, which is characteristic of a lognormal distribution.[1] Figure I–7(c) is the normal probability plot of the average thicknesses for the combined data. If the points fall approximately on a straight line, it indicates that the data are normally distributed. It is seen that there are systematic deviations of the data from the best-fit line. To examine whether the data are represented better by a lognormal distribution, the average thicknesses, in Table I–2 were transformed by taking their

[1] In a lognormal distribution, the natural logarithms of the values of a variate have a normal distribution.

natural logarithm. Figure I–7 (d) is a histogram of the natural logarithms of thickness, and Fig. I–7(e) is the corresponding normal probability plot. It is seen that the data are less dispersed about the straighter line, and the correlation coefficient has increased form 0.97 to 0.99. Thus, there is some indication that the distribution of fireproofing thickness is lognormal in the non-upgraded floor trusses.

Table I–2. Average fireproofing thickness from six measurements taken in 1994 on each of 16 random floor trusses on floors 23 and 24 of WTC 1.

Fireproofing Thickness (in.)	
Floor 23	**Floor 24**
0.60	0.76
0.53	0.60
0.70	0.90
0.76	0.72
0.88	0.64
0.89	0.80
0.83	0.68
1.17	0.65
0.88	0.67
0.71	0.77
0.82	0.96
0.52	0.66
0.69	0.65
0.52	1.11
0.64	0.95
0.52	0.56

Source: Data provided by Port Authority of New York and New Jersey.

A lognormal distribution for the average thickness of the fireproofing on the non-upgraded floor trusses is explained as follows. It is expected that the thickness of fireproofing will be highly variable due to the difficulty in spraying the material on the relatively thin members. If the overall thickness is low and the variability is high, a normal distribution would require a fraction of the surfaces to have negative values of fireproofing. If the thickness distribution is lognormal, the thickness cannot be zero, and there is a low likelihood of having thickness close to zero. If the underlying distribution of fireproofing thickness is lognormal, the average thickness overestimates the thickness expected to be exceeded with 50 percent probability, and the median is the appropriate statistic for the 50 percentile value.

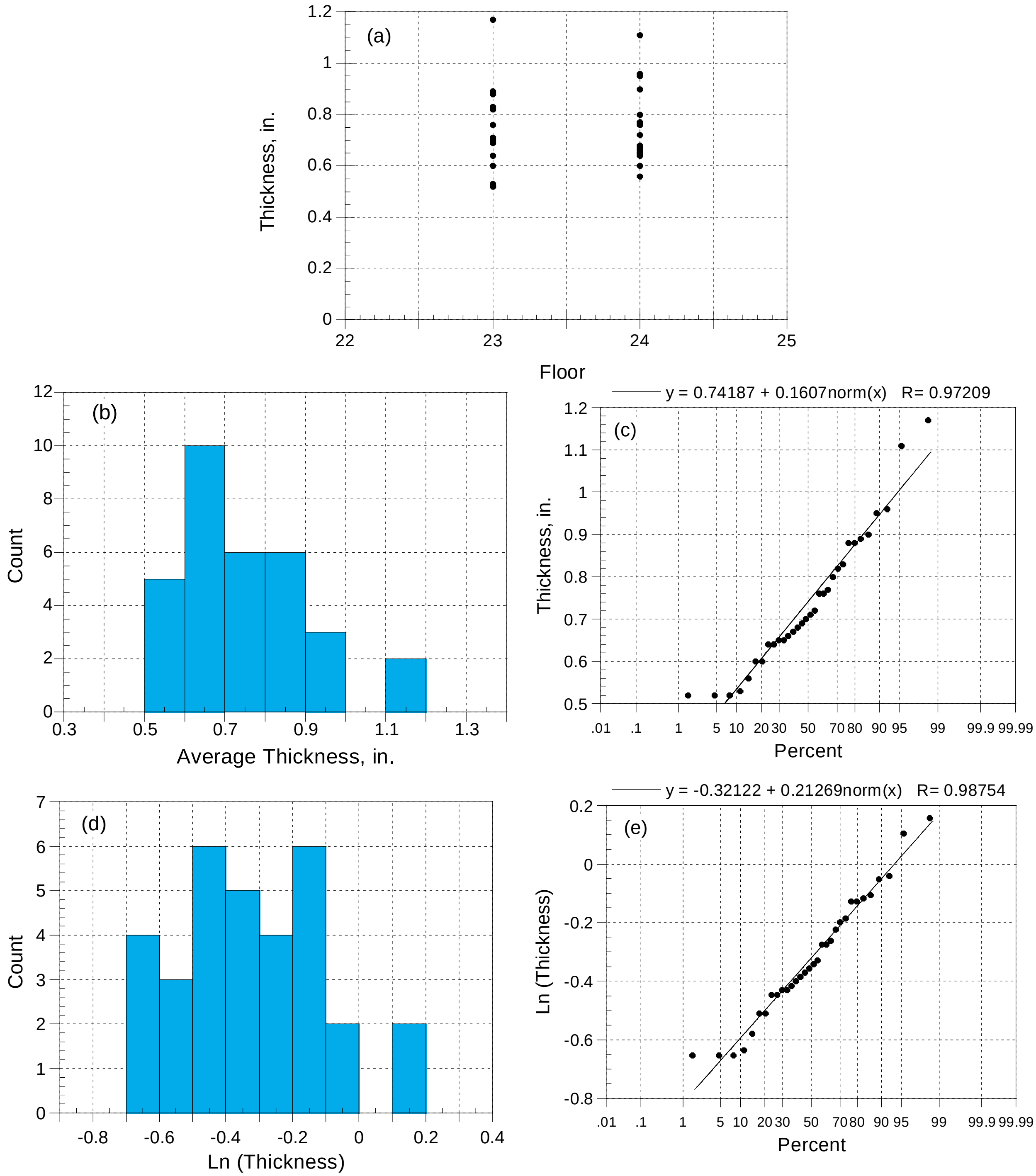

Figure I–7. (a) Dotplot of average thickness from floor trusses for floors 23 and 24, (b) histogram of average thickness, (c) normal probability plot of average thickness, (d) histogram of natural logarithm of average thickness, and (e) probability plot of natural logarithm of average thickness.

As stated, the standard deviation of the average thicknesses in Table I–2 is 0.16 in. Since each of the averages is based on six individual measurements, the variability in average thickness is less than the variability of the fireproofing thickness on a given element. If it is assumed that the true average thicknesses of fireproofing at the truss locations represented in Table I–2 are the same, it is possible to estimate the variability of individual measurements from the following well-known relationship:

$$S_{\overline{X}} = \frac{S}{\sqrt{n}} \tag{I.1}$$

where:

$S_{\overline{X}}$ = standard deviation of the average thicknesses

S = standard deviation of the individual thickness measurements

n = number of measurements to obtain the average thickness

Thus, an estimate of the standard deviation of the individual measurements is $0.16\sqrt{6} \approx 0.4$ in. Since it is unlikely that there is no difference in average fireproofing thickness at different cross sections, the standard deviation of 0.4 in. is an upper limit for the variability of fireproofing thicknesses in the non-upgraded floor trusses on the basis of the information provide in Table I–2.

Analysis of Photographs

Additional data regarding the thickness of fireproofing has been gathered by evaluating photographic evidence. Although photographic evidence of the state of the fireproofing is limited, two groups of photographs have been located and used for estimating fireproofing thickness.

The first group of photographs was provided to NIST by Morse Zehnter Associates and includes images of floor trusses from WTC 1 (floors 12, 22, 23, and 27) and WTC 2 (floor 26). From this group, only photographs from floors 22, 23, and 27 of WTC 1 were analyzed. Photographs provided by Morse Zehnter Associates were taken in the mid-1990s and illustrate the fireproofing conditions prior to the upgrade carried out by the Port Authority. Thus, fireproofing thickness on the photographed trusses should be at least 1/2 in. as specified by the Port Authority on October 1969.

The second group of photographs, taken in 1998, was provided by Gilsanz Murray Steficek (consulting engineers). This group illustrates the state of fireproofing after the upgrade program that was initiated in 1995. The photographs were of trusses for floor 31 and below in WTC 1.

Selection of which photographed trusses were used to estimate thickness of fireproofing was based on clarity of fireproofing edges and whether a feature of known dimensions was present. Thus, only photographs where reference measurements could be performed were used. The general approach to the analysis involved the estimation of distances based on the computed reference length per pixel. The procedure is summarized as follows:

- A feature of known dimension (based on construction drawings) that could be used as reference was located in the photograph. For example, the dimension of the bare vertical leg of a damper saddle was a dimension that could be obtained from shop drawings.

- In the photograph, the length of the reference dimension was measured in pixels.
- The scaling factor of length per pixel was computed by dividing the known dimension in inches by the number of pixels. For example, if the vertical leg of the damper saddle was measured as 48.2 pixels in the photograph, and it is known that the actual size of the leg was 3.13 in., the scaling factor would be 3.13 in./48.2 pixels = 0.065 in./pixel.
- Only truss webs or struts (diagonal bar at end of truss) located near and in the same plane as the reference object were selected for analysis. This selection was made to minimize error due to perspective.
- It was assumed that the fireproofing on web bars was applied evenly around the perimeter of the bar. Based on this assumption, a "virtual" centerline along the length of the bar was drawn in the photograph.
- Lines were drawn perpendicular to the "virtual" centerline. The number of pixels along the lines from the "virtual" centerline to the edge of the fireproofing was determined from the cursor positions indicated by the software. Measurements were made at regularly spaced intervals to avoid bias. Figure I–8 is an example of a series of measurements made on a strut.

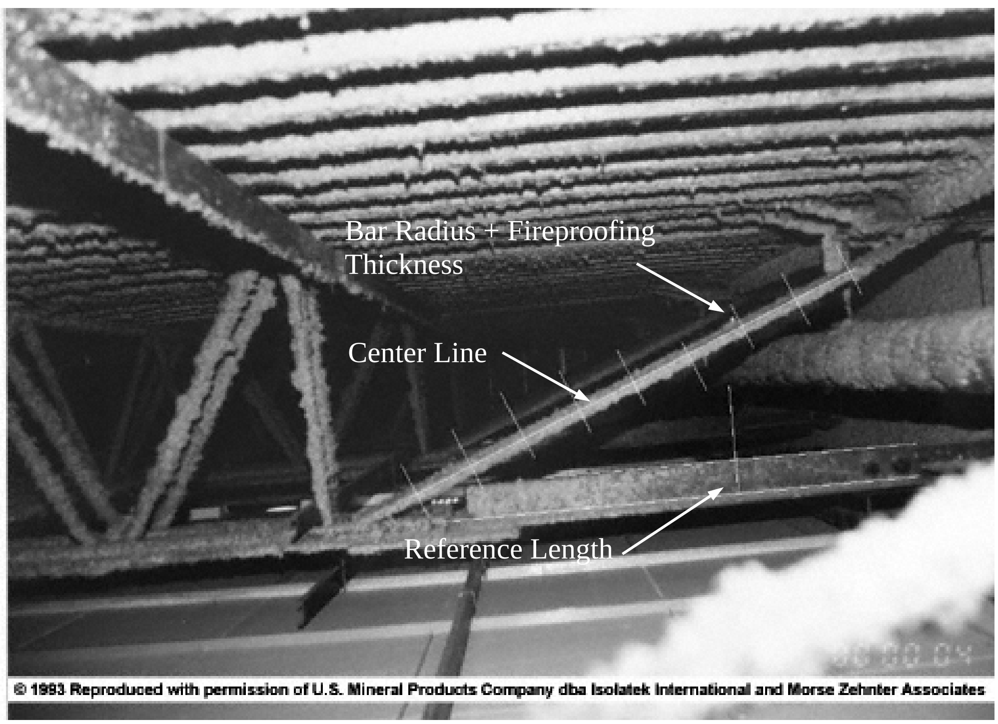

Figure I–8. Example of measurement procedure used to estimate fireproofing thickness from photographs.

- Each measurement in pixels was multiplied by the scaling factor (in./pixel) to estimate the bar radius plus fireproofing thickness.

- The radius of the bar was subtracted to provide the estimate of the fireproofing thickness.

It was observed that the estimated thickness of fireproofing in the non-upgraded floors tended to be larger for the webs of the main trusses. Hence estimates of fireproofing thickness were divided into three groups:

- Webs of main trusses,
- Webs of bridging trusses, and
- Diagonal strut at the exterior wall end of the truss.

No estimates of fireproofing thickness on top and bottom chords were possible using photographs. For the upgraded floors in WTC 1 that were included in the second group of photographs, only estimates of the thickness on the web bars of the main trusses were made. Figure I–9 (a) shows normal probability plots of the fireproofing thickness estimated from the photographs. It is seen that the points for the "upgraded" main trusses follow a generally linear trend, which indicates that the estimated thicknesses for the upgraded main trusses are approximately normally distributed. The estimated thicknesses from the non-upgraded floors, however, do not follow linear trends on the normal probability plot. Figure I–9 (b) shows normal probability plots of the natural logarithms of the thicknesses. The transformed values for the non-upgraded fireproofing now follow generally linear trends, which means that a lognormal distribution is more appropriate for the non-upgraded floors. This reinforces the observation noted in the previous section. Thus there is strong evidence that the original fireproofing thickness on the floor trusses follows a log normal distribution.

The average, standard deviation, and coefficient of variation were computed for the total number of measurements in each of these groups. The results are summarized as follows:

- Main trusses before upgrade: Average thickness 0.6 in., standard deviation = 0.3 in., and coefficient of variation = 0.5.
- Bridging trusses before upgrade: Average thickness 0.4 in., standard deviation = 0.25 in., and coefficient of variation = 0.6.
- Diagonal struts before upgrade: Average thickness 0.4 in., standard deviation = 0.2 in., and coefficient of variation = 0.5.
- Main trusses after upgrade: Average thickness 1.7 in., standard deviation = 0.4 in., and coefficient of variation = 0.2.

Port Authority Data on Upgraded Fireproofing on Trusses

As discussed in the *May 2003 Progress Report* (NIST SP 1000-3), the Port Authority provided information on fireproofing thickness from tenant alteration Construction Audit Reports prepared in 1997 to 1999. Those reports included average thicknesses of fireproofing at the "bottom of truss." In 2004, the Port Authority provided NIST reports of the individual measurements for many of the average thicknesses in the Construction Audit Reports. With the individual measurements, it is possible to investigate the

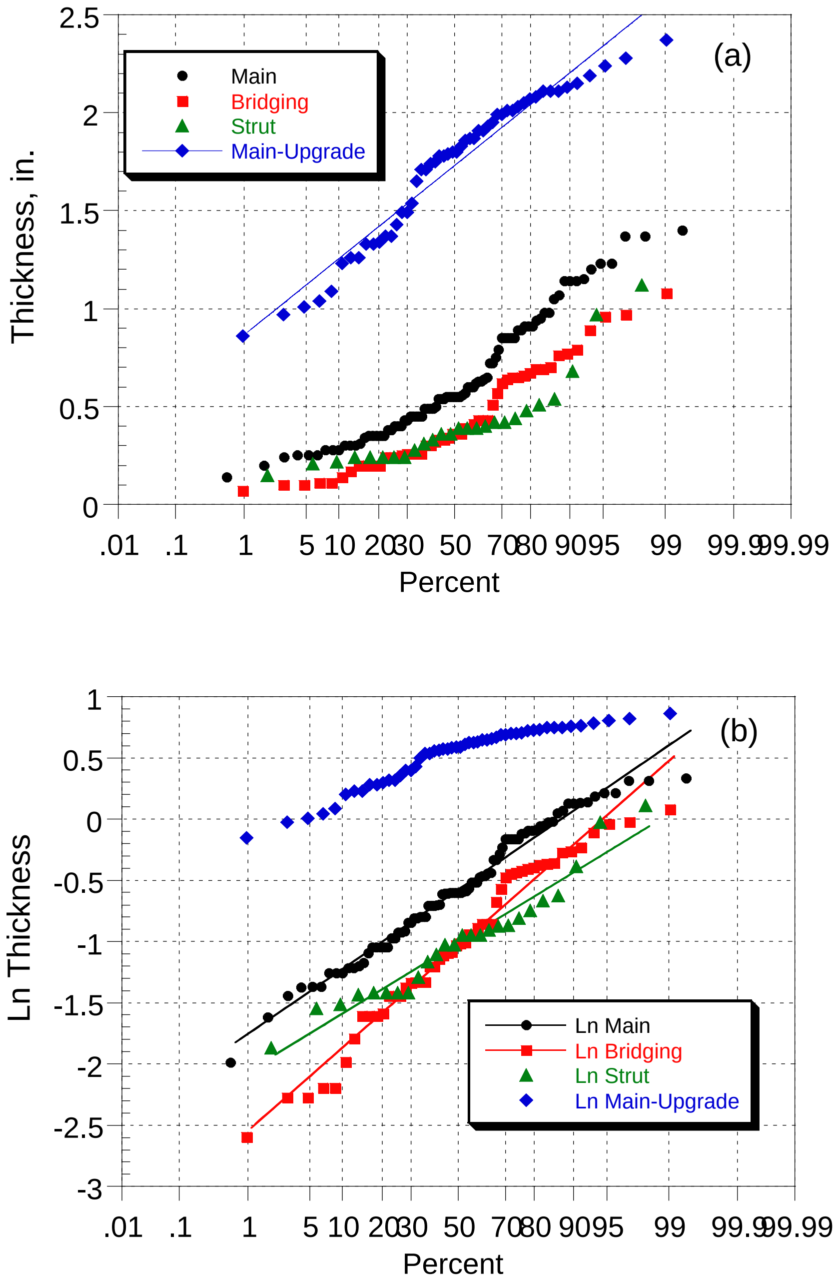

Figure I–9. (a) Normal probability plot of estimated fireproofing thickness based on photographs, and (b) normal probability plot of natural logarithms of thickness.

variation of thickness at a cross section of a truss member and the variation in average thickness from truss to truss. To permit such analyses, only those data having the same number of individual measurements at each cross section were used. This resulted in 18 data sets for WTC 1 (including floors 93, 95, 98, 99, and 100) and 14 data sets for WTC 2 (including floors 77, 78, 88, 89, and 92).

An analysis of the individual measurements was carried out to determine the underlying distribution for the measured thicknesses. Figure I–10 (a) is a dotplot of the individual measurements in WTC 1 (144 measurements) and in WTC 2 (112 measurements). It is observed that the central values and ranges are similar for the two towers, and the two groups of measurements were combined into one group. Figure I–10 (b) is the histogram of the individual measurements, and Fig. I–10 (c) is the corresponding normal probability plot. A straight line fit to the normal probability plot shows a tendency of the points to deviate from the line. Figure I–10 (d) is a histogram of the natural logarithms of the individual thickness values, and Fig. I–10 (e) is the corresponding lognormal probability plot. A comparison of the probability plots shows that natural logarithms fall closer to a straight line. Thus, it appears that the thickness of the upgraded fireproofing on the floor trusses is described by a lognormal distribution. This contradicts the observation based on analysis of photographs from lower floors discussed in the previous section. The overall average thickness of the 256 individual measurements is 2.5 in. with a standard deviation of 0.6 in. Thus, the average thickness on the upgraded upper floors appears to be greater than that estimated from photographs taken on upgraded lower floors.

The overall standard deviation of 0.6 in. includes two contributions: (1) the variation of thickness at the cross section (within-truss variability), and (2) the variation of average thickness between trusses (between-truss variability). Figure I–11 shows these two components of the thickness variability for the two towers. Figures I–11 (a) and (c) show the within-truss variability, and Figs. I–11 (b) and (d) show the variation of average thickness of each truss. From analysis of variance, it was found that the within-truss standard deviation is 0.4 in., and the between-truss standard deviation is also 0.4 in. The within-truss standard deviation of 0.4 in. is similar to the standard deviation of the estimated individual thickness obtained from analysis of the photographs of upgraded main trusses.

Column Fireproofing Thickness

NIST requested that the Port Authority provide available information on the thickness of fireproofing for the exterior and interior columns of the WTC towers. Specifically, the request included the following:

- The fireproofing material used and the thickness on the various plates comprising the exterior columns and spandrels.
- The fireproofing material used and the thickness on core columns.
- Confirmation that the wide flange column sections were protected with CAFCO BLAZESHIELD Type DC/F with specified thickness of 2 3/16 in. for sections smaller than 14WF228 and 1 3/16 in. for 14WF228 and larger.
- Information on in-place fireproofing thickness.

The Port Authority replied that, due to inaccessibility of exterior columns and core columns, there were no recent records of fireproofing thickness for these elements. The only available measurements of fireproofing thickness were for beams and columns accessible within elevator shafts. The most complete data set included measurements on beams and columns taken within shaft 14/15 in WTC 1. These measurements were taken in April 1999 and included measurements from floor 1 to floor 45. The thicknesses were recorded to the nearest 1/8 in., with a few thicknesses recorded to the nearest 1/16 in. The columns included 10 to 18 replicate measurements, and the beams included 11 to 16 replicate measurements.

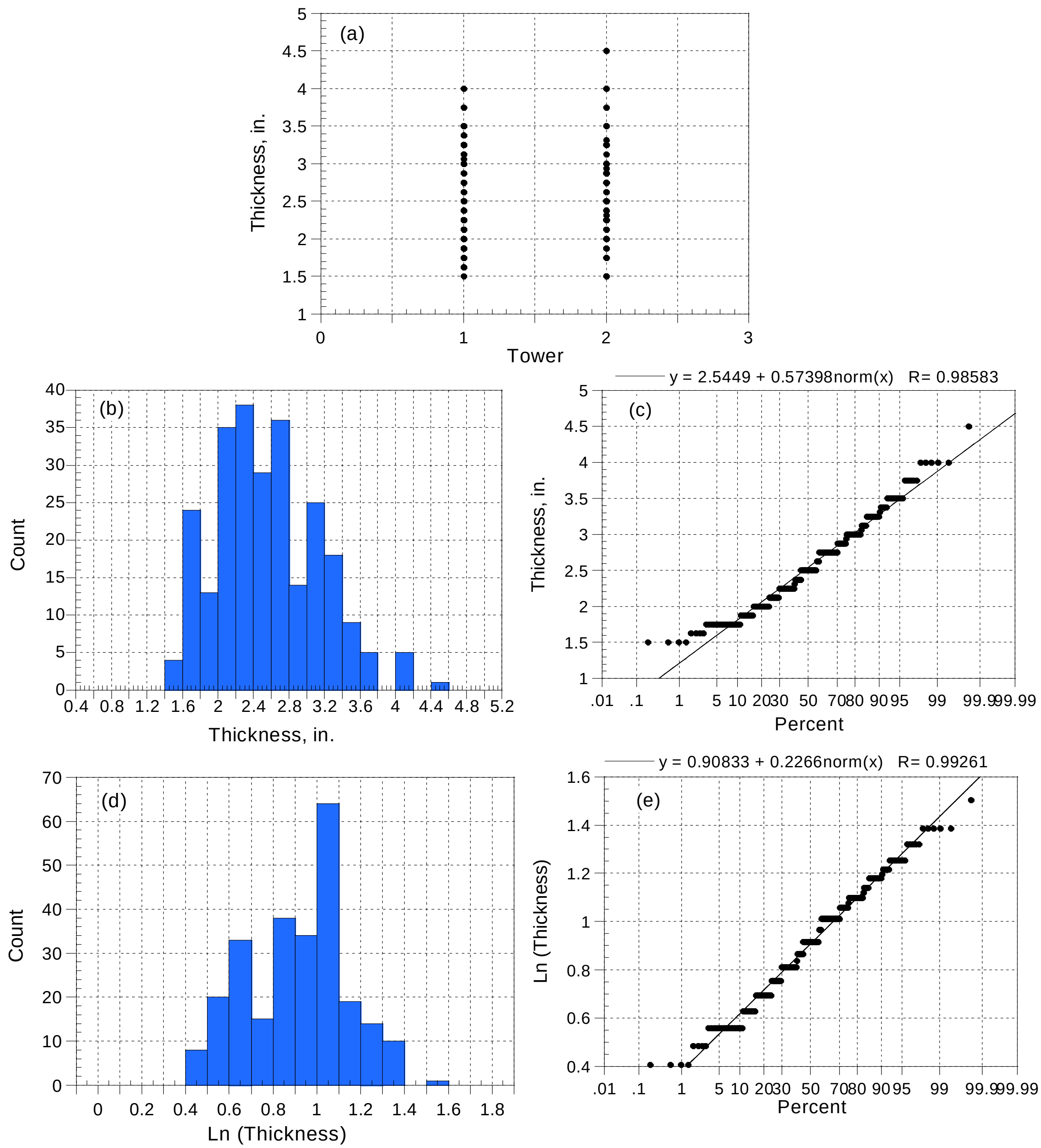

Figure I–10. (a) Dotplot of individual thickness measurements on floor trusses from Port Authority Construction Audit Reports, (b) histogram of thickness measurements, (c) normal probability plot of thickness measurements, (d) histogram of natural logarithms of thickness measurements, and (e) normal probability plot of natural logarithm of thickness measurements.

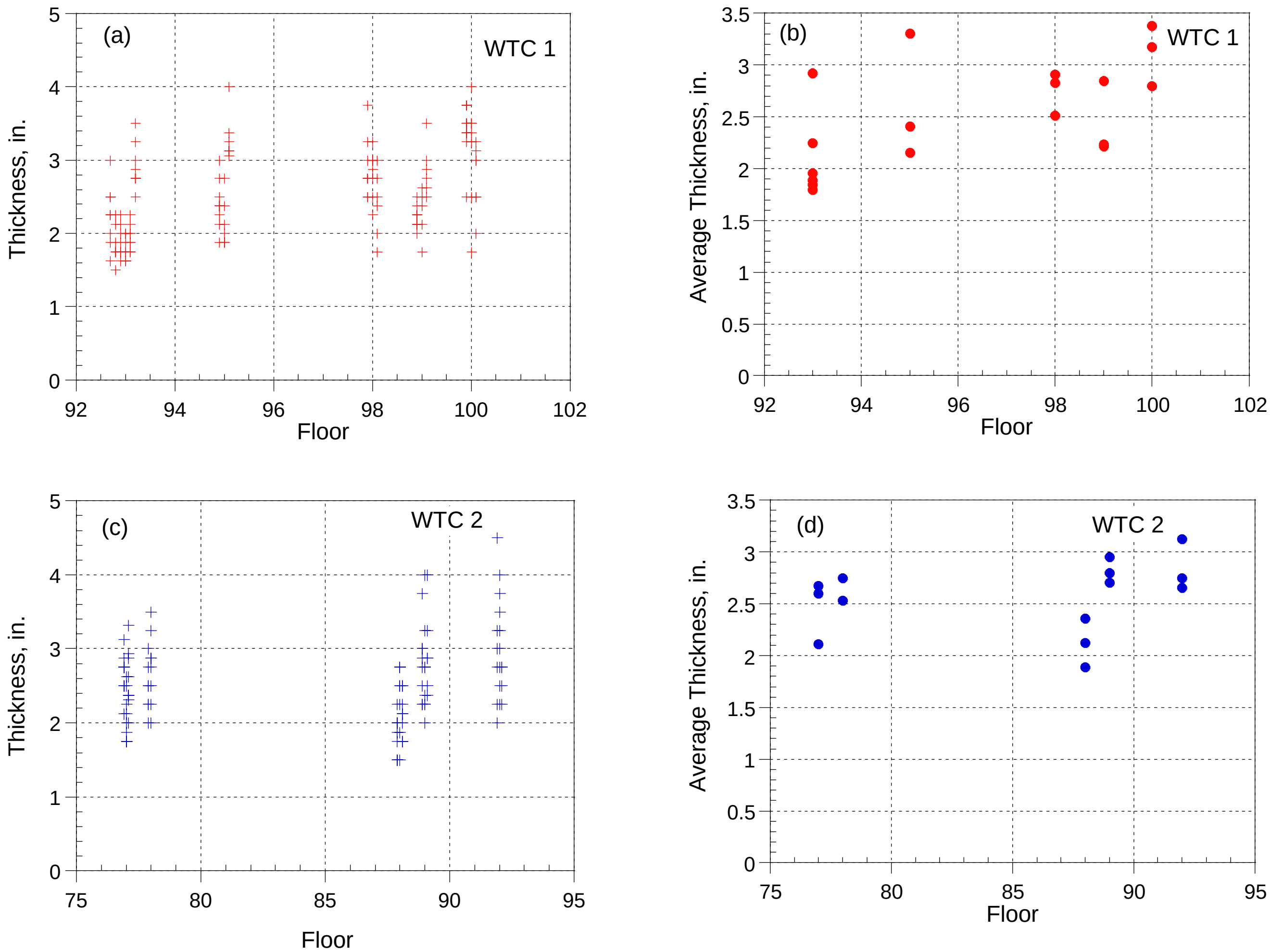

Figure I–11. Fireproofing thickness on floor trusses in upgraded portions of WTC towers: (a) individual measurements in WTC 1, (b) average thickness in WTC 1, (c) individual measurements in WTC 2, and (d) average thickness in WTC 2.

Figure I–12 (a) shows the individual and average fireproofing thickness on the core columns. Analysis of variance indicated no statistically significant differences among the average values and all data were pooled together. The average thickness for the columns is 0.82 in., the standard deviation is 0.20, and the coefficient of variation is 0.24. The information from the Port Authority indicated that the "minimum thickness required" for the columns was 0.5 in. Figure I–12 (b) is the normal probability plot of the individual thickness measurements. Because most of the thicknesses were reported to the nearest 1/8 in., the points are staggered instead of uniformly distributed. The plot, however, shows that the points follow a linear trend, and it appears that the thickness of the fireproofing on the core columns could be described by a normal distribution. Figures I–12 (c) and (d) shows the corresponding plots for the thickness of fireproofing on the beams. The average thickness for the beams is 0.97 in., the standard deviation is 0.21 in. and the coefficient of variation is 0.21. The information from the Port Authority indicated that the "minimum thickness required" for the beams was 0.75 in.

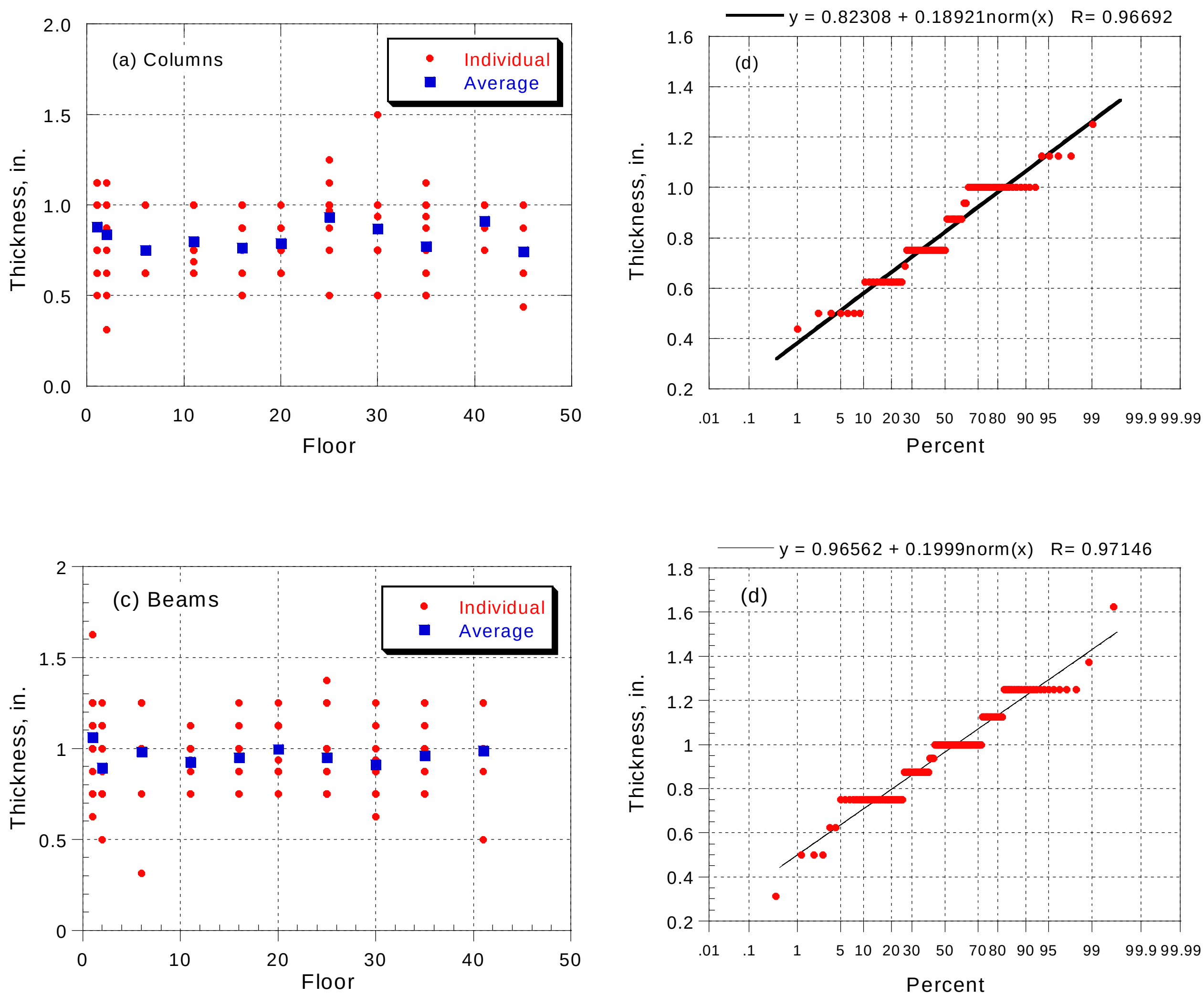

Figure I–12. (a) Individual and average thickness for core columns, (b) normal probability plot of individual measurements on columns, (c) individual and average thickness for core beams, and (d) normal probability plot of individual measurements on beams.

As might be expected, the variation in thickness of fireproofing for the beams and columns is lower than the variation observed in the floor trusses. The planar surfaces of the beams and columns result in more uniform application of the sprayed fireproofing than for the slender truss members. This results in reduced differences in the average thickness of fireproofing on different members and less variability within a member.

I.3.4 Equivalent Thickness

The sensitivity study summarized in Section I.2 indicated that variation in the thickness of fireproofing reduced the "effective thickness" of the fireproofing. It would be impractical to attempt to account for the variation in fireproofing thickness in the thermal modeling by introducing variable thickness fireproofing in the finite-element models. As an alternative, it was decided to attempt to determine the "equivalent uniform thickness" of fireproofing that would result in the same thermo-mechanical response of a member as variable thickness fireproofing. An approach similar to the methodology described in Section I.2 was used to model a 1 in. diameter by 60 in. long bar with fireproofing and subjected to the heat flux arising from a 1,100 °C fire. The bar was subdivided into 0.6 in. long elements, so that there were 100 elements along the length of the bar. The thermal history of the bar was calculated, and that history was used to calculate the length change of the unrestrained bar under a tensile stress of 12,500 psi. The bar was assumed to be similar to the steel used in the floor trusses, and the temperature dependence of the coefficient of thermal expansion and the modulus of elasticity were based on NIST measurements.

The fireproofing thickness in the models was based on the measurements summarized in the previous section for the web bars of main trusses in the original condition and after the upgrade. Specifically, the following target values were investigated:

- Original: average thickness = 0.75 in., standard deviation = 0.3 in., lognormal distribution.

- Upgrade: average thickness = 2.5 in., standard deviation = 0.6 in., lognormal distribution.

The variation of fireproofing thickness along the length of the bar was established by using a psuedo random number generator to select values from a lognormal distribution with central value and dispersion consistent with the above average values and standard deviation. Three sets of random data were generated for each condition.

When the randomly selected thicknesses of each element were applied to the bar, it resulted in sudden changes in fireproofing thickness along the length of the bar. This resulted in a "rough" surface texture as shown by the dotted thickness profile in Fig. I–13 (a). It was felt that this rough texture (see also Fig. I–1 (c) might not be representative of actual conditions, so an alternative approach was to use 5-point averaging to reduce the roughness of the fireproofing profile. The solid line in Fig. I–13 (b) shows such a "smooth" profile. The two profiles in Fig. I–13 (a) have approximately the same average value and standard deviation and have similar cumulative distribution of fireproofing thickness as shown in Fig. I–13 (b).

As stated, the calculated thermal histories of the bar elements were used to calculate the unrestrained length change of the bar due to thermal expansion and an applied stress of 12,500 psi. Work is currently underway to examine the performance of the bar under fully restrained conditions in which the induced stress history is computed. For comparison, the deformation of the bar with different but uniform thickness of fireproofing was calculated. The "equivalent thickness" was taken as the uniform thickness that resulted in similar deformation as under the variable thickness conditions. Figure I–13 (c) shows the results of these calculations for the original fireproofing. The three continuous curves are the deformation-time relationships for uniform thickness of 0.4 in., 0.5 in., and 0.6 in. The solid symbols represent the results for three cases with "rough" texture, and the open symbols are for the "smooth" texture. The following values summarize the six variable thickness profiles:

- Rough 1: average = 0.79 in., standard deviation = 0.29 in.

- Rough 2: average = 0.77 in., standard deviation = 0.27 in.

- Rough 3: average = 0.79 in., standard deviation = 0.31 in.

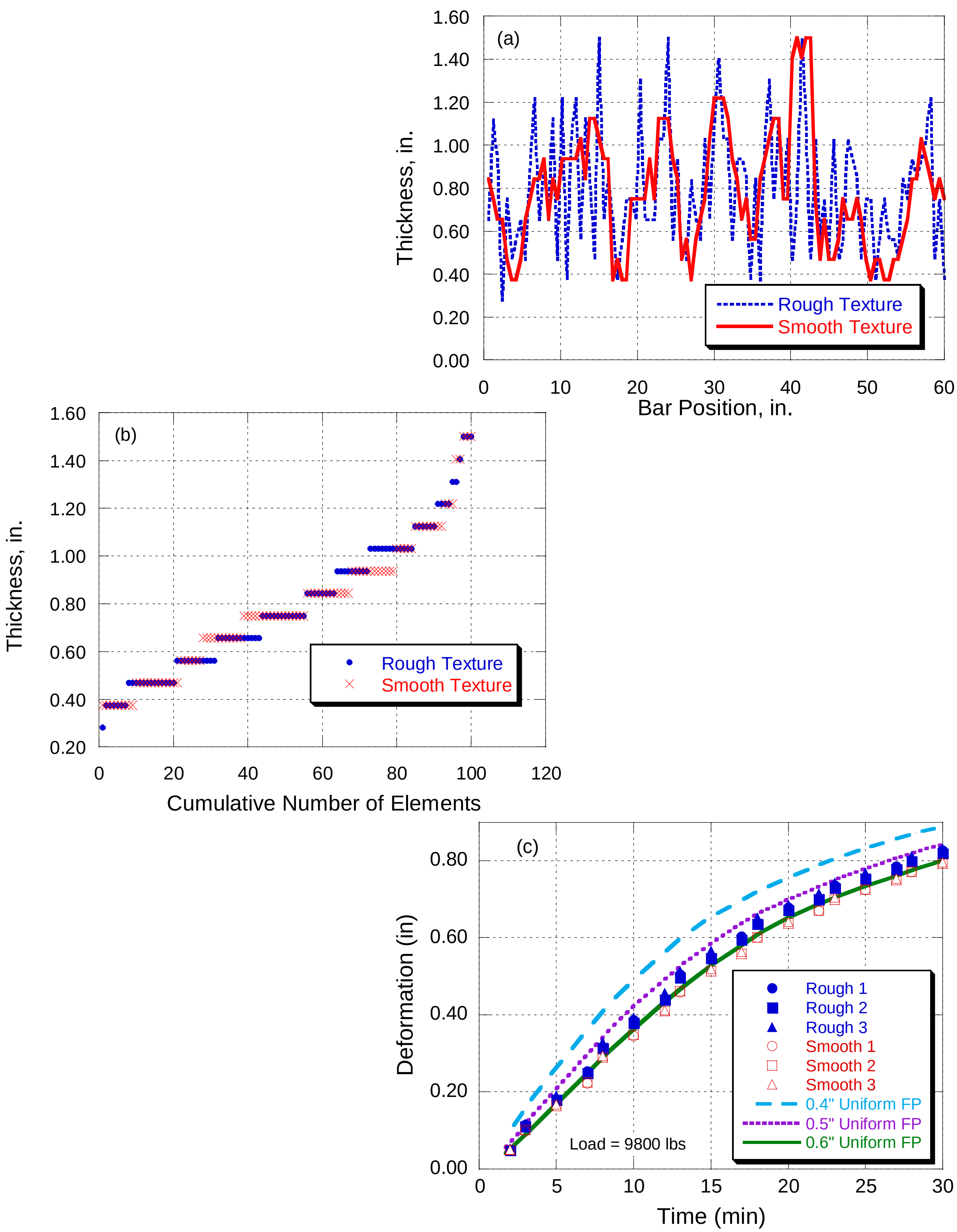

Figure I–13. (a) Randomly generated thickness profiles with average thickness of 0.75 in. and standard deviation of 0.3 in., (b) cumulative element size, and (c) deformation of 1 in. bar compared with deformation for uniform thickness of fireproofing.

- Smooth 1: average = 0.79 in., standard deviation = 0.28 in.
- Smooth 2: average = 0.78 in., standard deviation = 0.31 in.
- Smooth 3: average = 0.78 in., standard deviation = 0.32 in.

Figure I–13 (c) shows that the "rough" texture reduces the effectiveness of the fireproofing by a small amount compared with the "smooth" texture. As noted above, it is believed that the "smooth" texture is more representative of the actual conditions. On the basis of these analyses, it is concluded that fireproofing with an average thickness of 0.75 in. and a standard deviation of 0.3 in. provides equivalent protection to 0.6 in. of uniform thickness.

The results for the upgraded fireproofing are shown in Fig. I–14. Only the "smooth" texture was used, and the values for the three cases are as follows:

- Case 1: average = 2.50 in., standard deviation = 0.71 in.
- Case 2: average = 2.43 in., standard deviation = 0.51 in.
- Case 3: average = 2.55 in., standard deviation = 0.63 in.

Figure I–14 (a) shows the three profiles, and Fig. I–14 (b) shows the normal probability plots of thickness values. Because the three randomly generated profiles do not have the same averages and dispersions, the responses show more scatter than in Fig. I–13 (c). On the basis of these analyses, it is concluded that an average thickness of fireproofing of 2.5 in. with a standard deviation of 0.6 in. is equivalent to 2.2 in. of uniform thickness.

I.3.5 Thickness of SFRM for Use in Analyses

Analyses of available data on fireproofing thickness and thermal modeling revealed the following:

- From measurements of fireproofing thickness, the average values exceeded the specified thickness.
- Fireproofing thickness was variable, and the distribution of thickness in the floor trusses appears to be described best by a lognormal distribution.
- The standard deviation of fireproofing thickness on the trusses varied between about 0.3 in. to 0.6 in.
- The standard deviation of fireproofing on columns and beams from the core tended to be lower, with a value of 0.2 in. for the available data.
- No information is available on the fireproofing thickness on the exterior columns and spandrel beams.
- Variation in thickness reduces the effectiveness of fireproofing, and the equivalent uniform thickness is less than the average thickness.

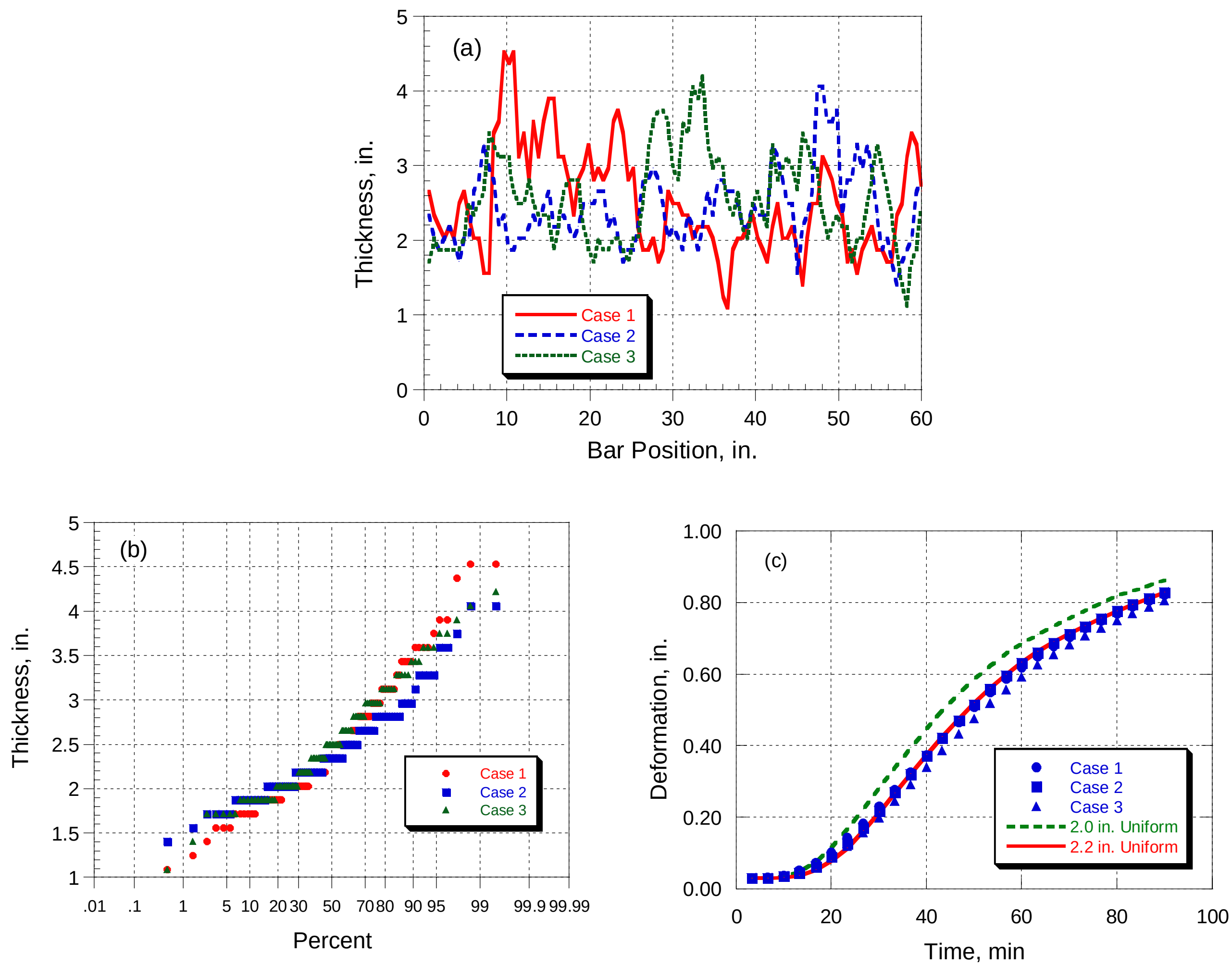

Figure I–14. (a) Randomly generated thickness profiles with average thickness of 2.5 in. and standard deviation of 0.6 in., (b) normal probability plots of thickness values, and (c) deformation of 1 in. bar compared with deformation for uniform thickness of fireproofing.

Based on the above findings, the following uniform thickness for the undamaged fireproofing will be used in calculating thermal response under various fire scenarios:

- Original fireproofing on floor trusses: 0.6 in.
- Upgraded fireproofing on floor trusses: 2.2 in.
- Fireproofing on other elements: the specified thickness.

The choice of specified thickness for those members lacking data is justified by offsetting factors as follows: (1) measured average thicknesses exceed specified values, and (2) variation in thickness reduces the effectiveness of fireproofing.

I.4 THERMAL PROPERTIES

Based on the information provided by the manufacturers, three SFRMs have been identified in WTC 1, 2, and 7: (1) CAFCO BLAZE-SHIELD Type DC/F, (2) CAFCO BLAZE-SHIELD Type II, and (3) Monokote MK-5. Of the three SFRMs, only CAFCO BLAZE-SHIELD Type II is currently sold in the U.S., and CAFCO BLAZE-SHIELD Type DC/F is sold in Canada.

CAFCO BLAZE-SHIELD Type DC/F is manufactured by Isolatek International (Stanhope, New Jersey) and was used in the interior columns, floor systems, and the exterior faces of the exterior columns of WTC 1 and WTC 2. CAFCO BLAZE-SHIELD Type II, also from Isolatek, was used in subsequent retrofit of WTC 1 floor systems. CAFCO BLAZE-SHIELD Type DC/F and Type II are portland cement-based products. Monokote MK-5 a gypsum-based SFRM, was manufactured by W.R. Grace and Co. (Cambridge, Massachusetts) and used in WTC 7. W.R. Grace stopped the production of Monokote MK-5 in the 1980s. In addition to these three SFRMs, vermiculite plasters, manufactured by W.R. Grace until the 1970s, were used on the interior faces of the exterior columns of WTC 1 and WTC 2.

To provide thermophysical property data for the modeling effort in fire-structure interaction, the thermal conductivity, specific heat and density of each SFRM were determined as a function of temperature up to 1200 °C. Tests were performed by Anter Laboratories, Inc. in Pittsburgh, PA through an open-bid contract. Anter Laboratories is an ISO 9002 certified company.

Samples of CAFCO BLAZE-SHIELD Type DC/F and Type II were prepared by Isolatek, Inc. in Stanhope, New Jersey, and sample of Monokote MK-5 were prepared by W.R. Grace and Co. in Cambridge, Massachusetts according to their respective application manuals. Since Monokote MK-5 is no longer on the market, it was specially manufactured by W.R. Grace according to the original MK-5 formulation. The samples were made from the same batch of raw material, shipped to NIST for examination and documentation, and sent to Anter Laboratories for testing. The sample is 9 in. long, 4.5 in. wide, and 3 in. thick. Three samples of each material were sent for testing. Two of them were used for the thermal conductivity measurements, and the third was used to prepare specimens for the other measurements involved.

I.4.1 Thermal Conductivity Measurements

The thermal conductivity measurements were performed according to ASTM C 1113 Test Method for Thermal Conductivity of Refractories by Hot Wire (Platinum Resistance Thermometer Technique). This test method is based on heating two specimens with a platinum wire placed between them. The thin platinum wire serves not only as a heater, but also as a temperature sensor, since the variation of its electrical resistance during the test is converted into variation of temperature. Thermal conductivity is calculated based on the rate of temperature increase of the wire and power input. It was reported that substantial shrinkage during the measurements occurred for the three materials. The two MK-5 specimens shrunk, exposing the platinum wire positioned between them. For this reason, no thermal conductivity measurement could be performed for this material at 1,200 °C. Figure I–15 (a) shows preliminary results for thermal conductivity as a function of temperature. The results show similar trends of increased thermal conductivity with increasing temperature; however, the Monokote MK-5 specimens had a different behavior than CAFCO BLAZE-SHIELD Type DC/F and Type II at temperatures above 500 °C.

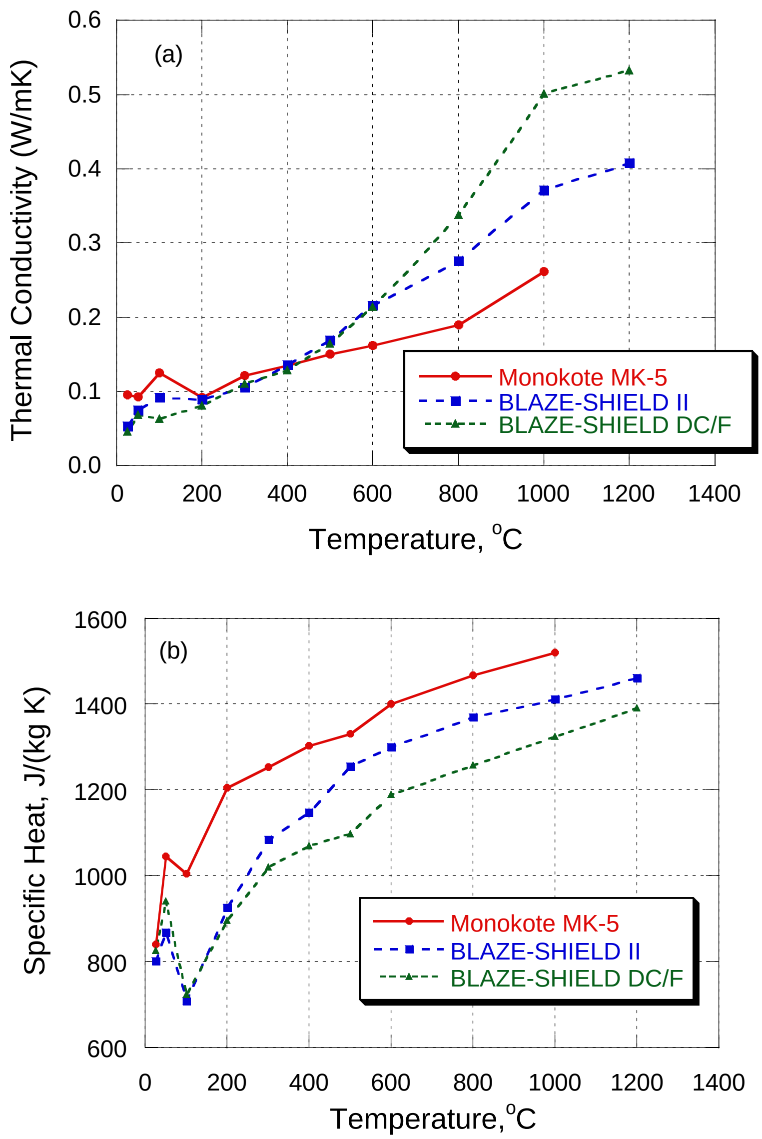

Figure I–15. Preliminary test results: (a) thermal conductivity as a function of temperature, and (b) specific heat as a function of temperature.

I.4.2 Specific Heat Measurements

For the specific heat capacity measurements, the same instrument (Unitherm Model QL-3141) was used with a slight modification. A thermocouple was added to the system and mounted on the specimen, parallel with the platinum wire at a known distance from the thermocouple. The test was performed in a similar manner as the thermal conductivity measurements, but from the thermocouple output the thermal diffusivity of the material was derived. Knowing the thermal conductivity, the thermal diffusivity, and the density calculated from the thermal expansion results and the thermogravimetric analysis, the specific heat of the material was calculated. Figure I–15 (b) shows preliminary results for specific heat as a function of temperature. It is seen that the materials had similar increasing trends with temperature, but the actual values differed.

I.4.3 Density Measurements

Densities of the samples were not measured directly (except at room temperature) but were calculated from TGA (thermal gravimetric analysis) and thermal expansion measurements. The TGA tests were performed according to ASTM Test Method E1131 using an Orton Model ST-736 TGA instrument. The thermal expansion tests were performed according to ASTM Test Method E228 using a Unitherm Model 1161 instrument. Since the materials were not isotropic, separate tests were performed for the X and Z orientations. It was assumed that the X and Y directions had the same thermal expansion. The Z direction was defined as the direction perpendicular to the fibrous strands in the specimens. The specimens were tested from room temperature to 1,200 °C at a heating rate of 2 °C/min. All of the specimens shrank during the tests and, in all cases, lost contact with the pushrod before reaching the maximum test temperature.

From the thermal expansion test results, the change in volume for each material was calculated at each temperature of interest. The density values were calculated from the results of the TGA and thermal expansion.

I.5 RESPONSE TO IMPACT

In order to estimate the extent of damage or loss of SFRM due to aircraft impact, the detailed finite element analysis of aircraft impact into the WTC towers, conducted within the framework of Project 2 of the investigation, will provide the following information:

Debris Field—A database and graphics of the major fragments of the aircraft and destroyed structural components of the towers, including their mass, approximate size, speed, and trajectory will be developed in the global analysis of aircraft impact into WTC 1 and WTC 2. The trajectory of each fragment will consist of the initial point of entry, point of exit or resting place. This debris field database will be used to estimate which areas within the impacted floors would likely have lost their fireproofing due to direct impact by debris.

Deformations and Accelerations—Estimates of accelerations and deformations, including localized effects, as a function of time on steel members in each of the two towers will be developed in the global analysis of the aircraft impact. Accelerations will be determined at representative locations on the floor truss systems and columns in the impact-affected zones of both towers (floors 93 to 98 of WTC 1 and floors 78 to 83 of WTC 2). These accelerations will be compared with the threshold values estimated from the adhesion and cohesion properties of SFRM developed in the experimental and analytical study presented below to estimate the likely extent of damage to the fireproofing on the columns and floor systems.

Preliminary results from the subassembly impact analysis of an aircraft engine into a strip of the towers with a width and height of single exterior panel (three exterior columns width and three floor height) extending all the way through the core indicate that the accelerations on the lower chords of floor trusses will need further analysis to account for high frequency vibrations and the short-duration sharp peaks in the computed acceleration time-histories and their effects on damage to SFRM. One possible approach is to low-pass filter the acceleration records to remove these high-frequency vibrations. Another approach is to develop “shock spectra” for a number of steel members with fireproofing configurations using finite element analysis to determine, for a given frequency, the acceleration amplitude that is needed to dislodge the fireproofing based on its adhesive and cohesive strength. The shock spectra will then be compared with spectra of the calculated acceleration time-histories to estimate the extent of damage to the fireproofing.

I.5.1 Mechanical Properties of SFRM

The purpose of these tests is to develop a rational basis for estimating the extent of loss of SFRM as a result of impact loads on protected members. Tests will (1) determine the mechanical properties of CAFCO BLAZE-SHIELD Type DC/F, and (2) verify models for estimating loss of fireproofing when a protected member is subjected to impact-induced vibration. The mechanical properties to be measured are:

- SFRM cohesive strength, and
- SFRM adhesive strength to steel substrates with and without primer.

The adhesive and cohesive strengths will be measured for static loads, as described below for Phase I tests. The tests will be done on 1/4 in. thick steel plate specimens, with and without primer (Tnemec Series 10 red primer), and for nominal SFRM thickness of 3/4 in. and 1 1/2 in. Specimens are fabricated and testing will be done during June and July, 2004.

From the measured strength properties, estimates will be made of the local accelerations required to damage or dislodge the SFRM, as described below. These estimates will be verified by impact tests of plates and bars covered with SFRM and instrumented with accelerometers, as described in the Phase II tests.

Phase I—Tensile Pull-off Test to Measure Adhesive Bond Strength and Cohesive Strength

Specimen—Steel plates (8 by 16 by 1/4 in.) with CAFCO BLAZE-SHIELD Type DC/F and nominal thickness of 3/4 in. and 1 1/2 in.

Pull-Off Test Procedure (see Fig. I–16)

- Using a fine-tooth saw, cut into SFRM applied to plate to obtain 2 3/4 in. square test specimens to ensure that the area resisting the applied load is well defined.
- Affix aluminum plates with two-component adhesive.
- Allow adhesive to cure.
- Measure force required to pull off the plate.
- Record load and note failure mode (cohesive, adhesive, mixed).

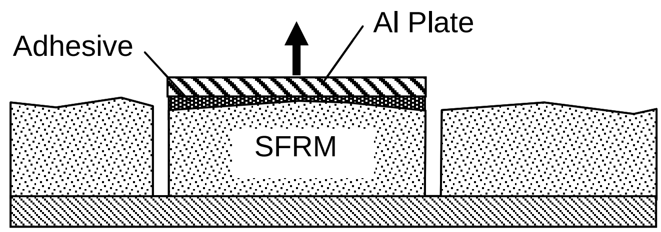

Figure I–16. Pull-off test of SFRM applied to steel plate.

If all failures are adhesive, the cohesive strength will be determined by bonding the SFRM block to a steel plate with adhesive and repeating the test.

Phase II—Verification of Models to Predict Dislodgement of SFRM

Impact tests of plate and bar specimens will be done to determine the impact loads needed to produce different levels of accelerations. Plates and bars with SFRM will be subjected to different levels of impact until the SFRM is dislodged. Two simplified models will be used to estimate the relationships between material strengths and impact required to dislodge the SFRM. Model predictions will be compared with test results.

CASE 1: Planar Element

The simplified model considers the substrate and SFRM as rigid bodies. The SFRM would dislodge when the inertial force exceeds the smaller of the adhesive bond strength or cohesive strength. Figure I–17, shows the free body of the fireproofing being acted upon by its inertial force and the adhesive force. The acceleration to dislodge the SFRM is:

$$a = \frac{f_b}{\rho\, t} \qquad \text{(I.1)}$$

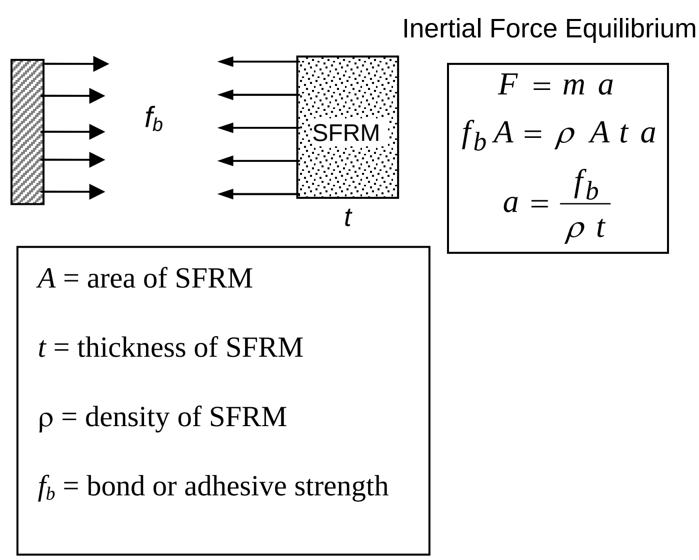

Figure I–17. Derivation of acceleration to dislodge SFRM from planar substrate.

where:

f_b = bond or adhesive strength

t = thickness of SFRM

ρ = density of SFRM

For example, for an SFRM with cohesive and adhesive strength of 150 psf, a density of 15 pcf, and an applied thickness t = 1 in., we would find that a = 119g, where g is the gravitational acceleration. This shows that acceleration on the order of 100g would be required to dislodge this SFRM from a planar surface.

CASE 2: Encased Round Element

Again, a rigid body model is used. In this case, the SFRM would mobilize its cohesive tensile strength, f_t, and adhesive bond strength, f_b. Figure I–18 shows the derivation for the relationship between material strengths and acceleration to dislodge the SFRM from a round bar. The required acceleration is as follows:

$$a = \frac{4 f_t (d_0 + (\alpha - 1) d_i)}{(d_0^2 - d_i^2) \rho \pi} \tag{I.2}$$

where:

f_t = cohesive tensile strength of SFRM

d_0 = outside diameter of SFRM

Inertial Force Equilibrium

$$\text{Mass} = m = \pi \frac{(d_o^2 - d_i^2)}{4} \rho$$

$$F = f_t (d_o - d_i) + f_b d_i$$

$$\text{Let} \quad f_b = \alpha \; f_t$$

$$F = f_t (d_o + (\alpha - 1) d_i) = \pi \frac{(d_o^2 - d_i^2)}{4} \rho \, a$$

$$a = \frac{4 f_t (d_o + (\alpha - 1) d_i)}{(d_o^2 - d_i^2) \rho \pi}$$

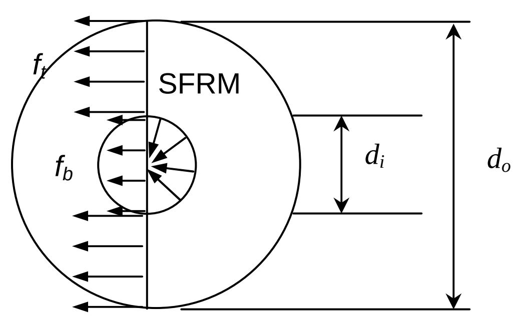

Figure I–18. Derivation of acceleration to dislodge SFRM surrounding a round bar.

d_i = steel bar diameter

α = Rrtio of bond strength to cohesive strength of SFRM

ρ = Ddnsity of SFRM

For example, if the steel bar has a diameter of d_i = 1 in., the SFRM has an outside diameter of d_0 = 2 in., a density ρ = 15 pcf, a cohesive tensile strength of f_t = 300 psf, and a bond strength to cohesive strength ratio of α = 0.5, we would find that an acceleration of a = 152g is required to dislodge the SFRM from the bar.

I.6 SUMMARY

This appendix has focused on conditions of the fireproofing (or SFRM) in the WTC towers before and after aircraft impact. Results of simplified finite-element simulations of heat transfer under fire conditions have shown that variability in thickness of fireproofing reduces the effectiveness of the fireproofing so that protection is less than implied by the average thickness of the fireproofing. As a result, the NIST-led investigation sought available information on the in-place condition of the SFRM used in the WTC towers. Limited information was provided by the Port Authority in the form of thickness measurements taken at various times during the 1990s. Additional information was obtained from photographs of floor trusses provided to NIST. Analysis of the data indicated that fireproofing thickness was variable, as would be expected for application to floor truss members with small cross sections. Based on analyses of the available data, the following values were taken to be representative of the SFRM thickness on the floor trusses:

- Original SFRM: Average thickness of 0.75 in. with a standard deviation of 0.3 in. (coefficient of variation = 0.40)

- Upgraded SFRM: Average thickness of 2.5 in. with a standard deviation of 0.6 in. (coefficient of variation = 0.24)

Based on finite-element simulations of a 1 in. round bar covered with SFRM having lognormal distributions for thickness that are consistent with the above values, it is concluded that the original fireproofing on the floor trusses is equivalent to a uniform thickness of 0.6 in. and the upgraded fireproofing is equivalent to a uniform thickness of 2.2 in.

No information is available on in-place conditions of the fireproofing on the exterior columns and spandrel beams, and little information is available on the conditions of fireproofing on core beams and columns. In subsequent thermal analyses, the fireproofing on these elements will be taken to have uniform thicknesses equal to the specified values. This assumption is believed to be justified by the offsetting factors of measured average thicknesses tending to be greater than specified thicknesses and the reduced effectiveness of a given average thickness of fireproofing due to thickness variability.

Another objective of this appendix is to review the methodology that will be used to estimate how much of the SFRM may have dislodged as result of aircraft impact. Simple static models have been developed for an order of magnitude estimate of the acceleration that would be required to dislodge the SFRM. Based on these models and assumed, but representative, values of density and strength (adhesive and cohesive), it is estimated that acceleration on the order of 100g to 150g (where g is the acceleration due to gravity) would be needed dislodge the fireproofing. Additional analytical studies will be conducted to account for dynamic effects, and tests will be performed to verify these predictions.

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Appendix J
INTERIM REPORT ON EXPERIMENTS TO SUPPORT FIRE DYNAMICS AND THERMAL RESPONSE MODELING

J.1 INTRODUCTION

The reconstruction of the World Trade Center (WTC) fires involves two computation models:

- Fire Dynamics Simulator (FDS): This is the first large-domain CFD fire model that predicts and visualizes the spread, growth and suppression of a fire based on the underlying scientific principles governing fluid motion. The model numerically solves the conservation equations of mass, momentum and energy that govern low-speed, thermally driven flows with an emphasis on smoke and heat transport from fires. The companion software package, called Smokeview, graphically presents the results of the FDS three-dimensional (3D) time-dependent simulation. Smokeview animates in three dimensions the smoke particulates, heat fluxes, temperatures and fluid velocities within a building. Users of the package can view the enclosure from any angle and from inside or outside.

- Fire-Structure Interface (FSI): This code effects the transfer of radiant and convective heat from a CFD fire model, such as FDS, to a coupled, transient, three-dimensional finite element model for the thermal response of structural members, such as ANSYS. The members may be simple (e.g., bare steel) or complex (e.g., insulation-coated steel).

For application to the Investigation, each of these needs experimental data (a) to guide adaptation/development of the models for this specific purpose and (b) with which to ascertain the accuracy of the model predictions. Ideally the uncertainty in the agreement with experiments will be much smaller than the effect of varying the unknowns in the actual conditions on September 11, 2001.

The following text describes three sets of experiments designed to accomplish this. All three sets have been completed, and the analysis of the data is under way. Full reports are in preparation and will be completed in summer 2004. Attachment 1 is a short paper on the modeling and experiments that will be presented at Interflam in July 2004.

J.2 EXPERIMENTS FOR ACCURACY ASSESSMENT OF THERMAL ENVIRONMENT MODELING

The purposes of these experiments, conducted in the National Institute of Standards and Technology (NIST) Large-scale Fire Laboratory from March 10 through 26, 2003, were to:

- Assess the accuracy with which FDS predicts the thermal environment in a burning compartment and;

- Establish a data set to validate the prediction of the temperature rise of structural steel elements using FSI.

Within a large test compartment, assorted steel members were exposed to controlled fires of varying heat release rate (HRR) and radiative intensity. The steel members were bare or coated with spray-applied fireproofing of two thicknesses. The thermal profile of the fire was measured at multiple locations within the compartment. Temperatures were also recorded at multiple locations on the surfaces of the steel, the insulation, and the compartment. Prior to each test, a prediction of the thermal environment in the compartment was determined using FDS. Following the tests, the prediction and experimental results were compared.

J.2.1 Description of Experiments

The test compartment consisted of a steel stud frame lined with calcium silicate board. The internal dimensions of the compartment were 3 m high, 7 m deep, and 4 m wide. There were four openings in the west wall through which air entered the room; they totaled 1.75 m^2 (10.8 ft^2) in area and were located 1 m (3.3 ft) above the floor. There were four openings in the east wall through which heat and combustion products were emitted; they also totaled 1.75 m^2 (10.8 ft^2) in area and were located 2 m above the floor. A photograph is shown in Fig. J–1 and a schematic in Fig. J–2.

In each of the six tests, the four test subjects were a bar, two trusses, and a thin-walled tubular column. These are depicted in Figs. J–3 through J–5. Depending on the test, these specimens were either left unprotected or were coated with spray-applied fire protective insulation material, BlazeShield DC/F. The fibrous insulation was applied by an experienced applicator who took considerable care to apply an even coating of the specified thickness. As such, the insulated test subjects represent a best case in terms of thickness and uniformity.

The fires consisted of liquid hydrocarbon fuels sprayed by a two-nozzle spray burner onto a 1 m × 2 m (3.3 ft × 6.6 ft) pan. The fuels were (a) heptanes and (b) a mixture of nominally 60 percent (by mass) heptanes with 40 percent toluene. The latter fuel produced a significantly sootier flame.

The instrumentation for the tests comprised up to 352 channels of data.

- The combustion products were collected in a 6 m × 6 m (21.5 ft × 21.5 ft) hood. Instrumentation in the exhaust duct enabled calculation of the rate of heat release throughout a test.

- Fourteen heat flux gauges were placed strategically around the compartment to measure the transport of radiant energy; in addition, there were four slug calorimeters measuring the total heat flux parallel to the trusses.

Most of the channels were for thermocouples that measured the temperatures on the surface of the walls and ceiling, within the walls, on the surface of the steel components, and at the surface of the spray-applied insulation. With the large number of measurements, it was possible to go beyond the traditional point-by-point comparison and discover why the model either under- or overpredicted a given measurement. A description of the test series appears in Table J–1. Table J–2 shows the dimensions and variability of the spray-applied insulation. The measurements were taken at numerous locations along the perimeter and length of each specimen using a pin-thickness gauge specifically designed for this type of insulation.

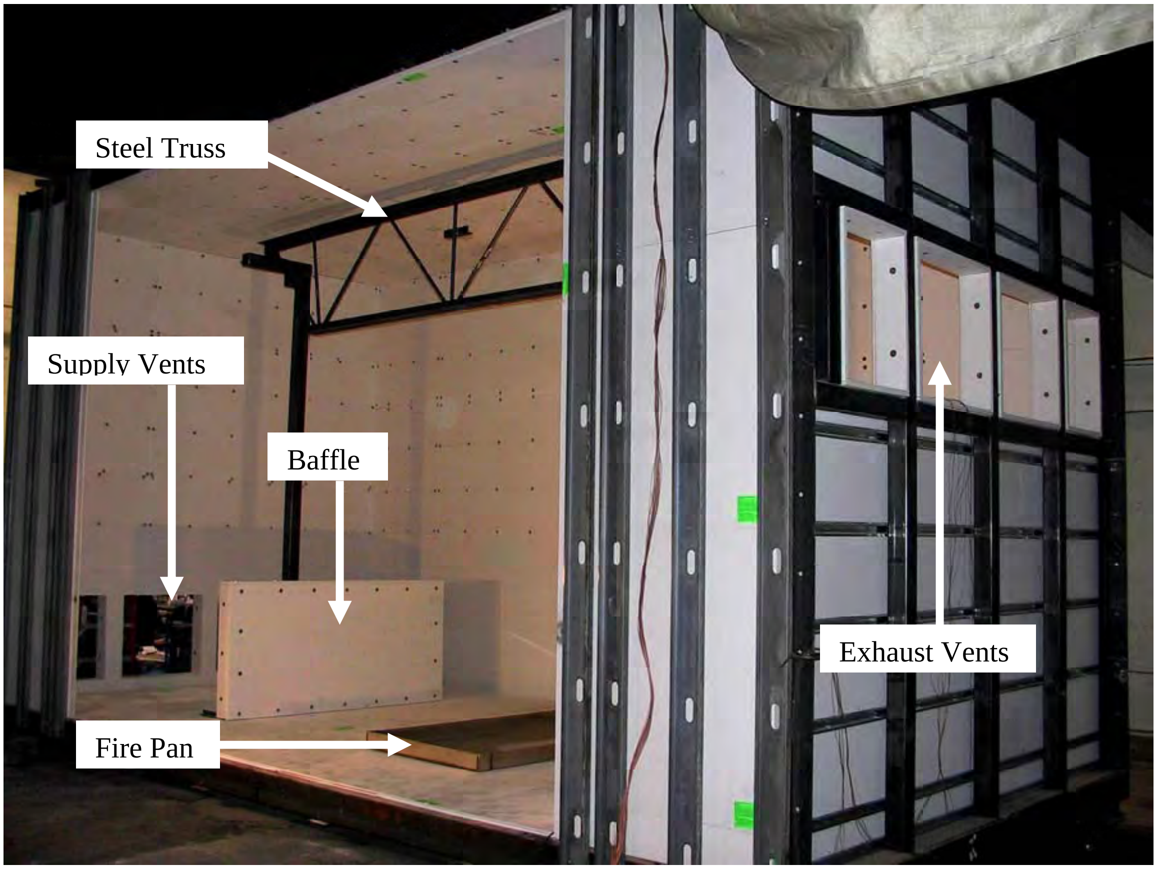

Figure J–1. Experimental enclosure during construction, viewed with access panels removed.

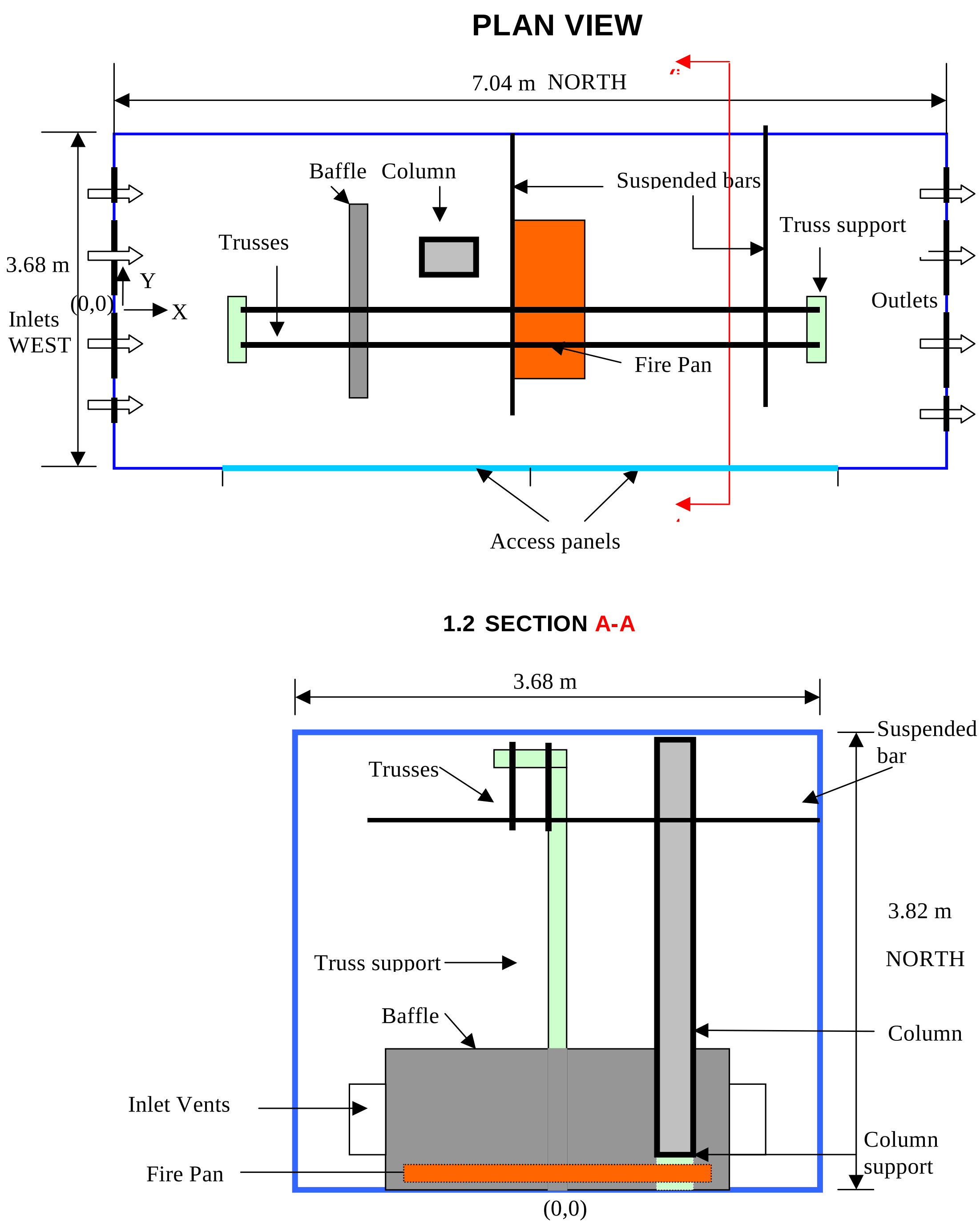

Figure J–2. Compartment content layout (not to scale).

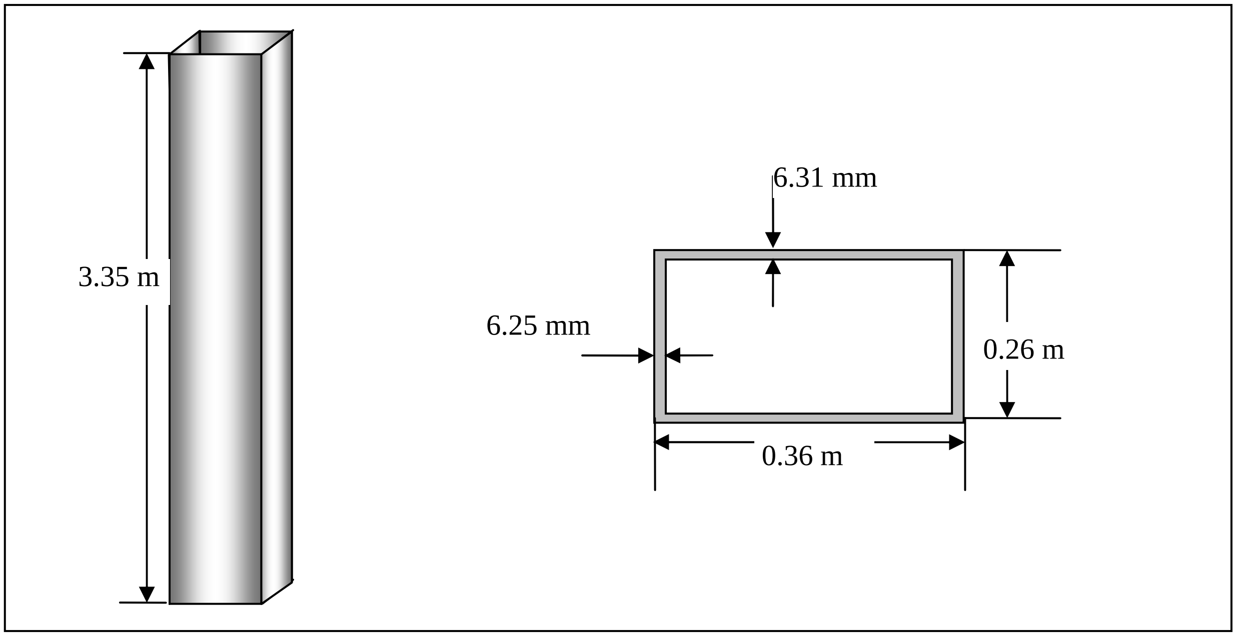

Figure J–3. Tubular column.

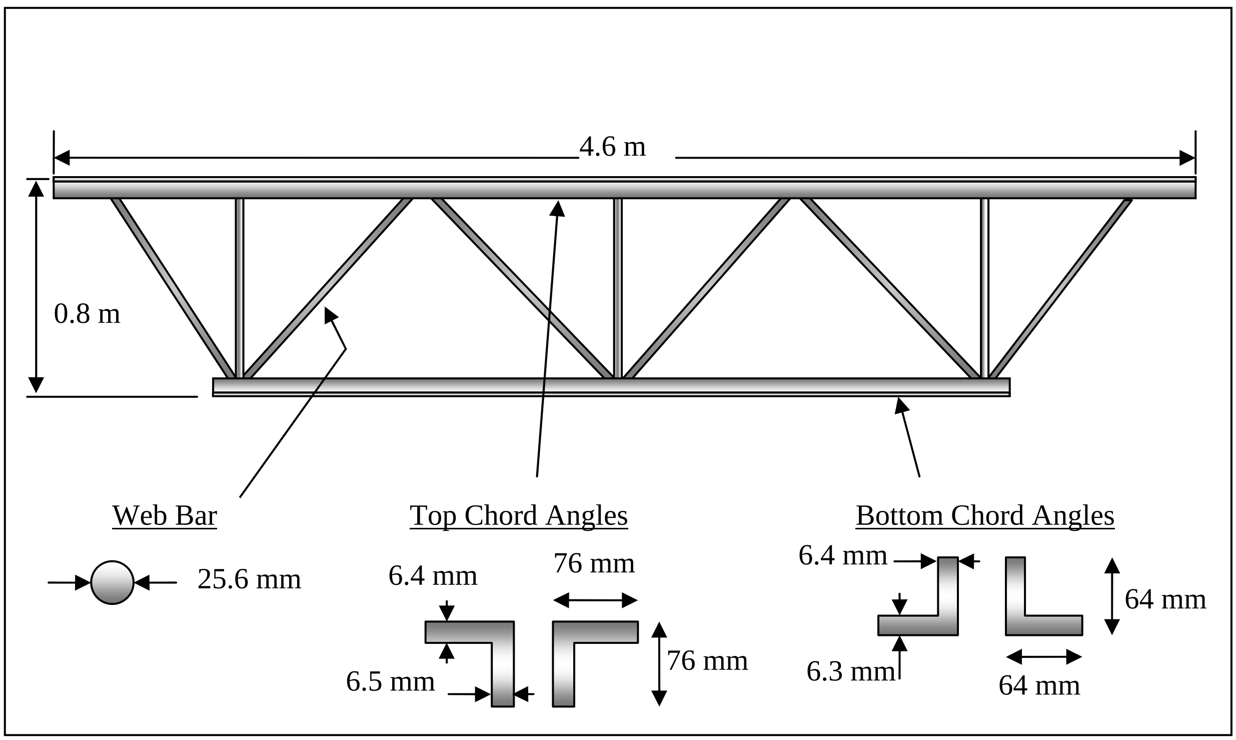

Figure J–4. Bar joist.

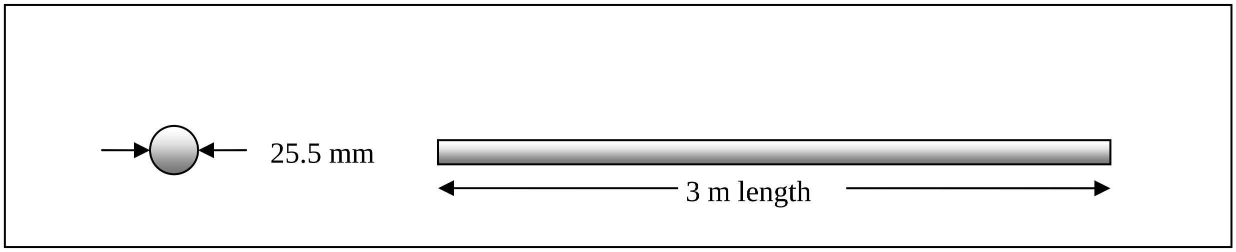

Figure J–5. Simple bar.

Table J–1. Test matrix.

Test	Measured Heat Release Rate (MW)	Fuel	Planned Insulation Thickness (mm)	Planned Test Duration (min)
1	2.0	Heptanes	None	15
2	2.4	Heptanes/toluene	None	15
3	2.0	Heptanes/toluene	None	15
4	3.2	Heptanes	Same as test 5	15
5	3.0	Heptanes	See Table J–2	50
6	3.0	Heptanes	See Table J–2	50

Table J–2. Summary of insulation on steel components.

Test	Item	Specified Thickness (mm)	Applied Thickness (mm)	
			Mean	Std. Deviation
5	Bar	19.1	23.0	5.5
	Column	38.1	41.0	3.0
	Truss A	19.1	26.9	7.3
	Truss B	38.1	40.5	8.2
6	Bar	19.1	25.3	4.6
	Column	19.1	21.4	3.5
	Truss A	19.1	26.0	6.9
	Truss B	19.1	25.6	6.9

The HRR are shown in Fig. J–6. The important features of the HRR curves are:

- The expected (from the calorific value of the fuels) and measured heat release rates agreed within the relative expanded experimental uncertainty of ±11%.
- The heat release rates are steady over the time interval in which the burner was turned on.

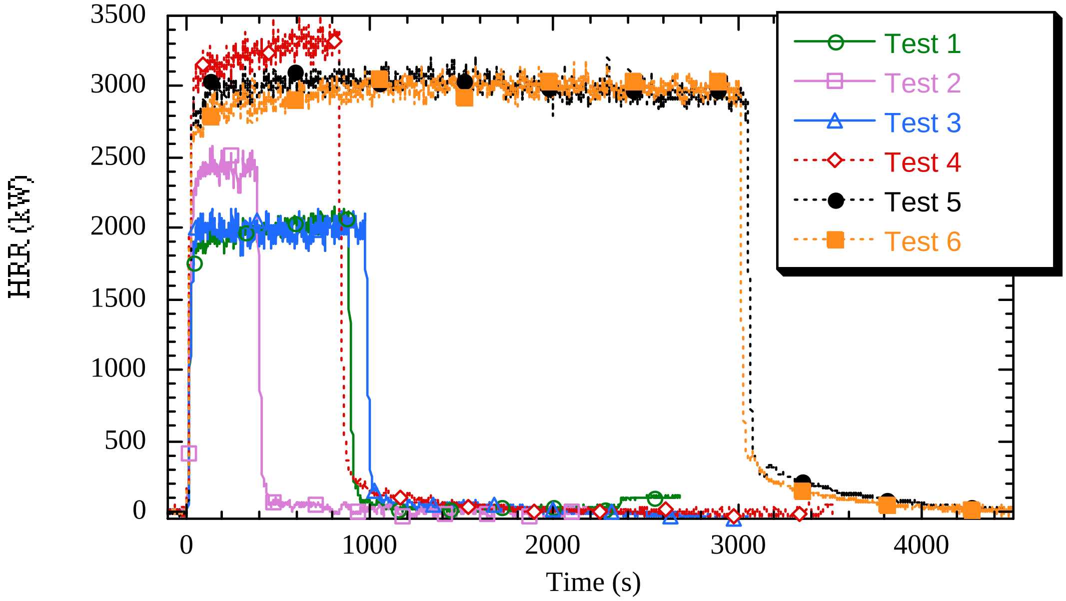

Figure J–6. Measured heat release rate as a function of time during tests 1–6.

In each test, the baseline signals from all the measurement devices were established; then the burner was ignited and continued burning at a steady rate until the temperature at any steel surface approached approximately 600 °C. (Above this temperature, there was concern that loss of strength might lead to collapse and accordant damage to the test facility.) At that point, the burner was turned off. In test 2, the steel reached the target temperature at about 6 min and was terminated at that time. During test 4, the Omega GG glass braid wire thermocouple extension cables failed leading to erroneous thermocouple readings. This was likely due to the opening-up of the Kaowool thermal insulation protective blanket around the thermocouple wires and subsequent shorting. Thus the test was terminated early, and the data have not been processed. To prevent reoccurrence of this problem during tests 5 and 6, the extensions were water cooled and double wrapped with Kaowool insulation. Each layer of Kaowool insulation was secured to the thermocouples using steel wire. Additionally, the thermocouple extensions were rerouted through the cooler west side of the compartment. After tests 5 and 6, the thermocouple extensions were visually inspected and were found to be undamaged.

J.2.2 Preliminary Results

Figure J–7 shows typical temperature data obtained in the tests. These data are for truss A in test 5. The thermocouple location notation is as follows: TU: Truss Upper Chord, TM: Truss Middle (Web), TL: Truss Lower Chord; 1 to 4: locations across the length of the test specimen; S: on the steel surface, I: on the outer surface of the insulation; A: truss A. For the informative nature of this progress report, it is only important to note the following:

- The curves that rise sharply from the beginning of the test are those for temperatures on the outside of the insulation.

- The curves that rise more gently are those for temperatures at the interface between the steel and the insulation.

From the figure, one can see that:

- The 19.1 mm (0.75 in.) insulation delays the rise to a peak steel temperature by almost an hour at all locations.

- The highest temperature reached at the steel surface is approximately 300 °C lower than the temperature at the outside face of the insulation material.

The curve patterns for the other steel specimens in the tests with insulated steel are similar in shape.

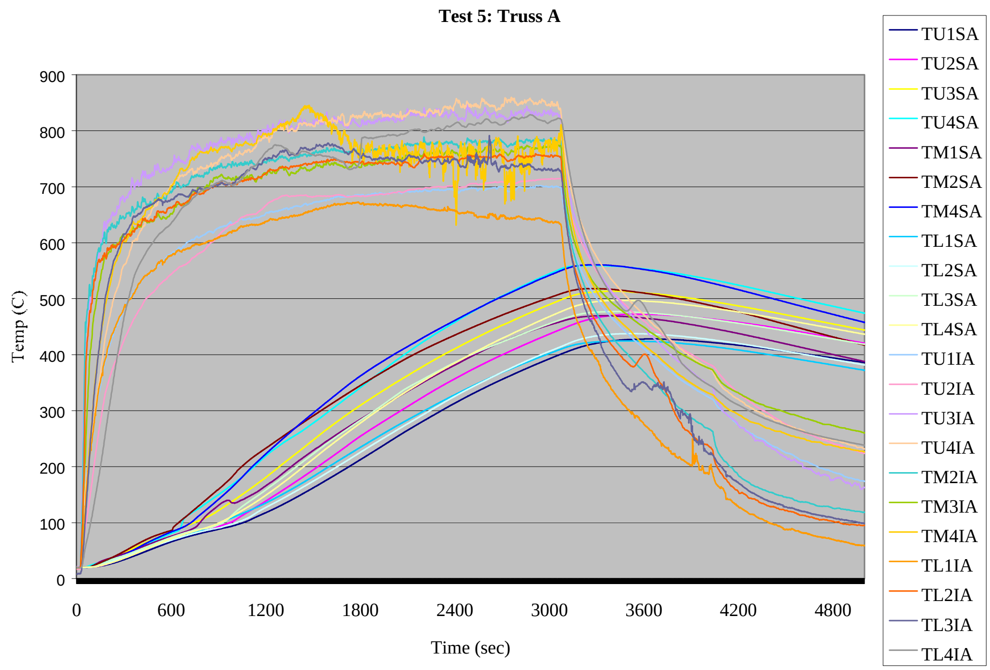

Figure J–7. Temperature-time history for truss A in test 5.

By contrast, Fig. J–8 shows the same plot for truss A from test 3, in which the truss was not insulated and the fire was of shorter duration and lower intensity. Nonetheless, the outer surface of the steel reached the targeted maximum temperature (just short of 600 °C) in about one-third the time. As expected, this result is typical of the fire response of the uninsulated steel specimens in tests 1 through 3.

Figures J–9 through J–12 show comparisons of the modeled and measured temperatures and heat fluxes for tests 3 (2.0 MW heptanes/toluene fire) and test 5 (3.0 MW heptanes fire). The agreement for the highest temperatures is excellent. Analysis of both the model and the thermocouple measurements is under way to determine the source of their differences at the lower temperatures, especially in test 5. The spikes in the heat flux plots are artifacts—the result of the periodic nitrogen blasts to reduce soot accumulation on the gage surface.

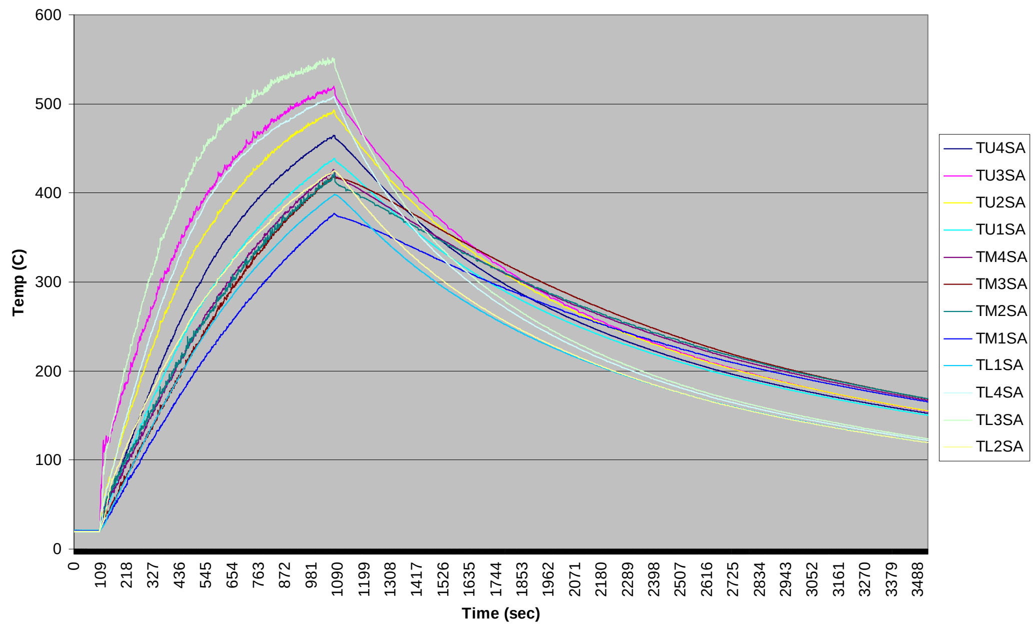

Figure J–8. Temperature-time history for truss A in test 3.

While the data analysis is not yet complete, the following are valuable preliminary observations:

- The prediction of the upper layer temperatures was within experimental uncertainty. Since the heat flux to the walls and objects within the upper layer is highly dependent on the upper layer temperature, these predictions were also accurate.The prediction of the time for the steel surfaces to reach 600 °C was accurate.

- The sootier burning fuel led to similar temperature rise in the ceiling and the steel above the fire plume.

- The model predicted the asymmetric shape of the fire plume, caused by obstructions to uniform flow through the compartment.

Further analysis will use Smokeview for visual comparison of the test results and the model predictions. This will determine how well FDS captures both the fire phenomena and the thermal patterns in the compartment. Quantitative analysis of the data will then determine the numerical accuracy of the predictions. Similar analysis will be performed to determine the accuracy of the finite element modeling of the thermal patterns within the bare and insulated steel components.

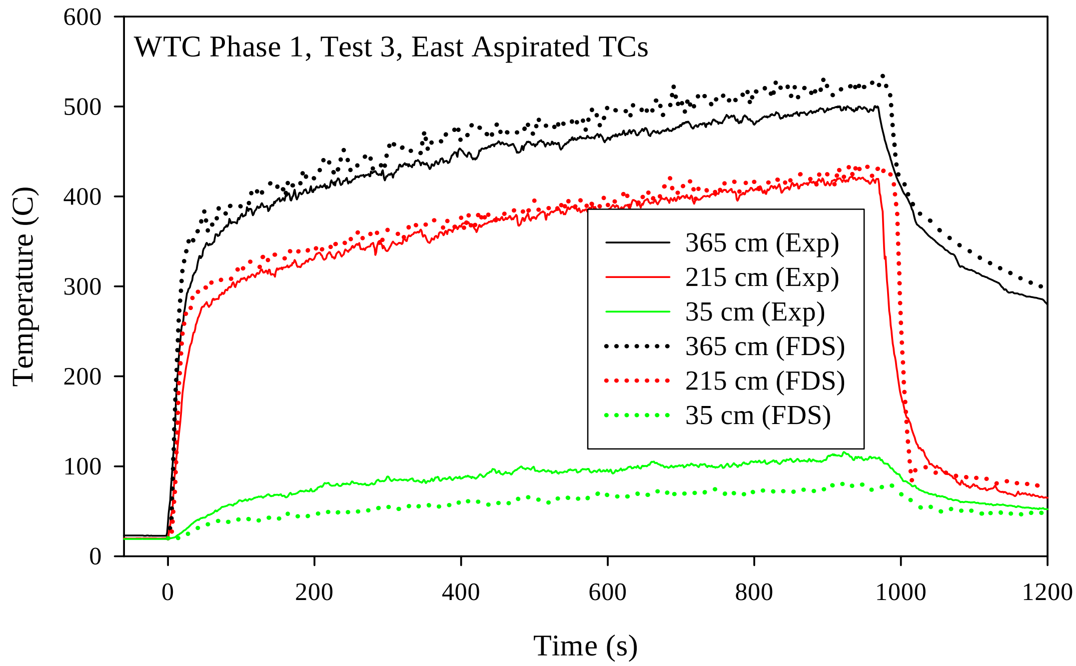

Figure J–9. Comparison of gas temperatures, test 3, exhaust side of compartment.

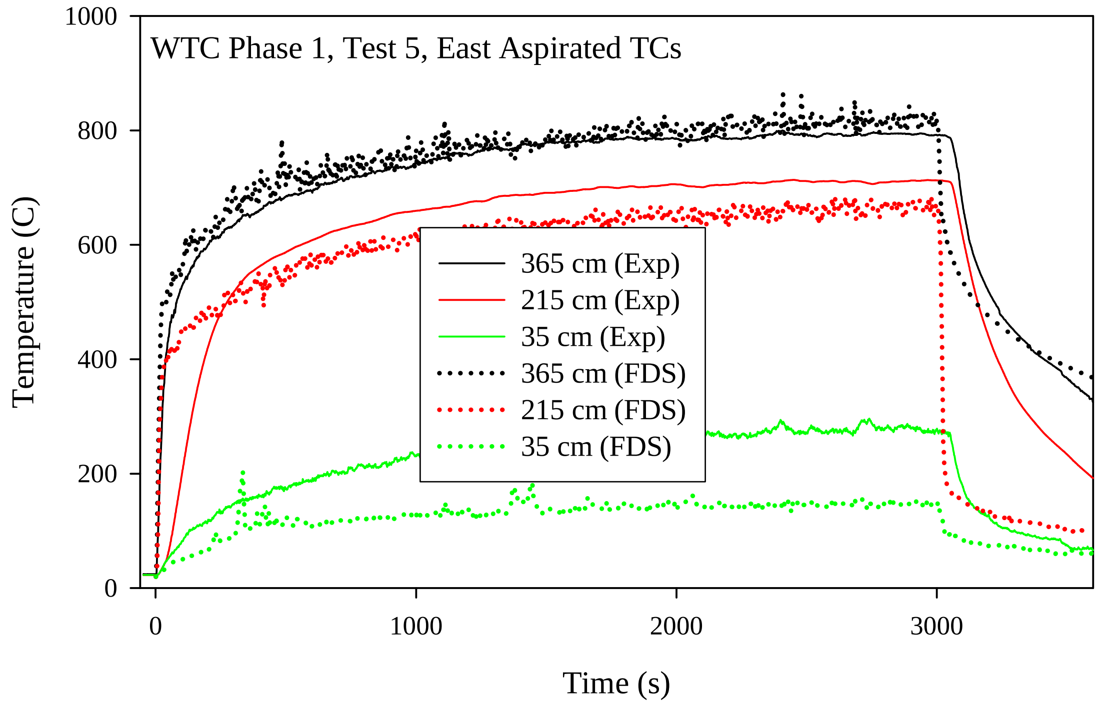

Figure J–10. Comparison of gas temperatures, test 5, exhaust side of compartment.

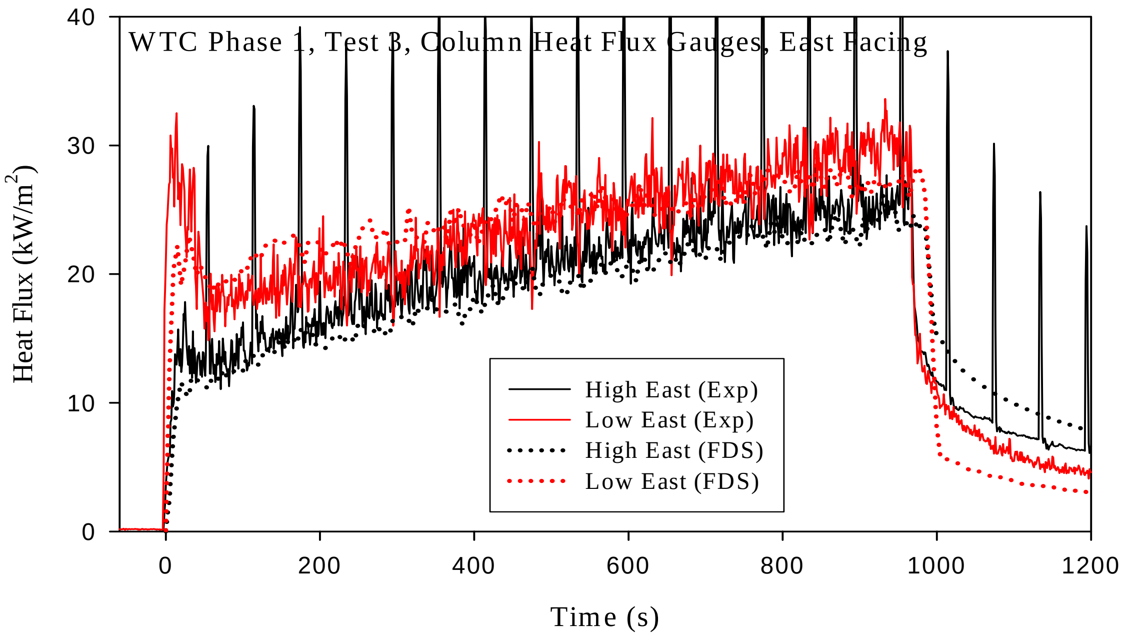

Figure J–11. Comparison of heat fluxes to column, test 3.

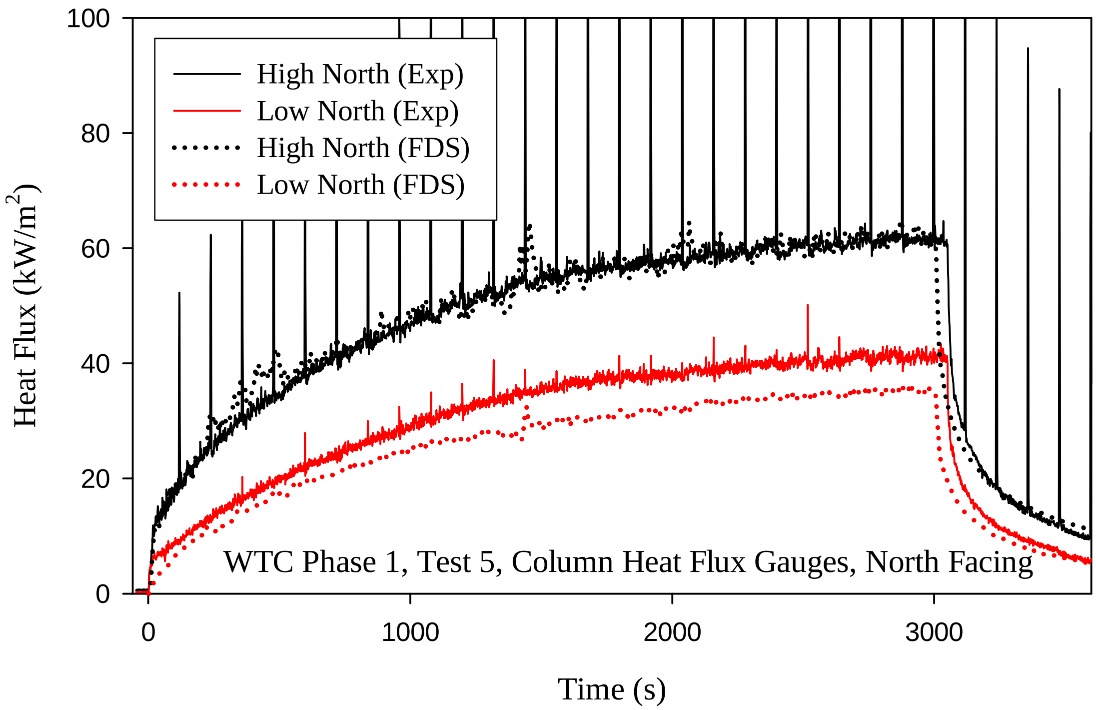

Figure J–12. Comparison of heat fluxes to column, test 5.

J.3 EXPERIMENTS FOR GUIDING THE FDS FIRE GROWTH PREDICTIONS

In the early stages of this Investigation, the FDS combustion module was enhanced to enable the inclusion of charring materials, such as those that comprise much of the office furniture. Thermophysical property data from the combustion of small (100 mm × 100 mm) samples of the furnishings were obtained using the Cone Calorimeter. These data became input to the fire simulation. A set of real-scale experiments was then designed and performed (July and August 2003) to identify any need for further enhancements to the fire model. In each of these experiments, a single workstation (i.e., an office cubical or module), similar to those in the WTC offices, was burned under a hood. A soffitted ceiling allowed for the collection of hot fire effluent and the accordant thermal radiation to the test specimen. Some of the tests examined the effects on the burning rate of jet fuel and/or noncombustible material occluding a fraction of the workstation surfaces.

J.3.1 Description of the Experiments

Materials

Workstations come in a variety of styles and finishes. However, they tend toward similar size and mass. They are also fabricated of materials with similar burning behavior, e.g., the work surfaces are generally laminated particleboard or wood. Most of the workstations burned here were of a single generic type. For comparison, one high-end unit (identical to those in the aircraft impact floors in WTC 1) was also burned in a manner identical to one of the tests of the generic units.

The generic workstation examined is shown photographically in Figs. J–13 and J–14. The layout, including the placement of the various nonstationary items, was suggested by personnel from a company that supplied office furnishings to the occupants of WTC 1. Information on the distribution of papers and other office items was provided by a frequent visitor to these offices. The workstation covered a footprint nominally 2.44 m × 2.44 m (8 ft × 8 ft) and was surrounded by privacy panels.

- The panels were 1.22 m (4 ft) high, except that on one side the panel was 1.52 m (5 ft) tall and supported a bookcase. On the side opposite the bookcase, there was a 1.22 m (4 ft) wide open entrance opening. The panels were made of a steel and softwood frame, covered on both sides with layers of fiberglass padding and perforated steel and a thermoplastic cover fabric. A few sheets of copier paper were tacked to the cubicle walls on three sides.

- The work surfaces were formed from four sections of laminated medium density fiberboard supported by steel brackets from the wall panels. Four document boxes contained a total of four reams of copier paper. Additional paper was stacked horizontally on the desk surface.

- The seat and back of the office chair were a nonthermoplastic fabric over polyurethane foam supported by a one-piece thermoplastic shell; its five-legged base was thermoplastic with steel framing and support elements.

Figure J–13. Photograph of right side of generic workstation.

Figure J–14. Photograph of left side of generic workstation.

- The three file cabinets (0.91 m wide, 0.51 m deep, 0.68 m high [36 in. × 20 in. × 27 in.], with two horizontal drawers) were painted steel; they rested directly on the carpet. Two of the cabinets contained two reams each of copier paper as a rough means of assessing the extent to which paper in file drawers might contribute to a fire.

- The bookcase (1.22 m [48 in.] long) had a steel shelf and top but these were supported only on their ends by combustible end panels; the steel front closure panel was fabric-covered steel and it was open (on top of the bookcase). Ten document boxes held about 13 reams of copier paper.

- The carpet tiles were nylon fiber-faced over a dense foam rubber backing. A square area 2.74 m × 2.74 m (9 ft × 9 ft) was covered with 36 carpet tiles.

- The computer monitor was a nominally 17 in. CRT-based unit. Its front face was taped with fiberglass tape and it was pointed toward the wall panel opposite the cubicle opening for safety in the event of an implosion. The keyboard was placed in its normal location, parallel to the sloped segment of the work surface. The computer processor (tower-type container with plastic only on the front face of the container) was placed on the floor next to a waste paper basket (both on the side opposite the cubicle opening).

- The wastebasket was thermoplastic and contained one ream of copier paper atop five balled-up paper ream wrappers.

Thermophysical characterizations of six of the generic workstation materials (carpet, panels, work surface, chair seat, paper stack, and computer monitor shell) were obtained using the Cone Calorimeter. These data were to serve as input to FDS. Since the physical behavior of at least some of the work station materials (and the objects from which they were taken) was expected to be more complex than was revealed in the Cone tests, it was anticipated that these full-scale tests would provide clues as to necessary empirical adjustments in the FDS predictions.

The high-end unit was similar to the generic workstation, with the principal differences being:

- The wall panel construction was somewhat different having a 3 mm (0.125 in) layer of flame-retarded polyester fiber beneath the outer fabric, a more open steel panel beneath this, a central fiberboard layer (3 mm thick) and an all steel peripheral frame (no wood). The fiberboard roughly doubled the amount of woody fuel within the wall panels of the enclosure and put it into a much higher surface area form in which it could be expected to burn appreciably faster. It should be noted, however, that this increase in woody fuel was only about 10 percent to 15 percent of the total available in the desk surfaces. Also, its enclosure deep within the wall panels delays its burning.

- The file cabinets (four, with a total face length of 2.67 m rather than three with a total face length of 2.44 m) had a flammable, charring plastic surface on the drawer fronts that added 15 percent to 20 percent of flammable area.

The chair was constructed somewhat differently (seat and back as separate pieces) and behaved as if its upholstered surfaces were flame-retarded. The high-end workstation is shown in Fig. J–15.

Figure J–15. Photographs of high-end workstation.

Test Configuration and Instrumentation

The workstation to be tested was placed on top of a double layer of 13 mm (1/2 in) thick calcium silicate sheets. These in turn rested on a set of four weighing cells, one at each corner. Each workstation (including furnishings) weighed approximately 730 kg (1,600 lb) and contained approximately 300 kg (660 lb) of combustible material. The weighing cells are accurate to ±0.1 kg (0.2 lb). The entire assembly was placed beneath the hood of the NIST 10 MW calorimeter hood to allow continuous heat release rate measurement.

A fire typically forms a layer of hot, smoky combustion products near the ceiling of a room. Thermal radiation from this layer plays a significant role in fire spread. Thus the test fixture included partitions to hold the combustion products from these test fires. The ceiling was a 3.66 m × 3.66 m (12 ft × 12 ft) section of 13 mm thick calcium silicate board. It was supported on a water-cooled steel frame 2.74 m (9 ft) above the floor of the workstation. To keep the gases from flowing quickly across the ceiling and thus not forming an appropriate layer, the ceiling was surrounded on all four sides by a steel skirt that draped down 0.61 m (2 ft) from the ceiling.

The test instrumentation included:

- Instrumentation in the hood exhaust duct for measurement of rate of heat release.
- Four video cameras placed to record the progression of flame spread over the objects in the cubicle. Their view of the combustibles became observed by the wall panels and the flames themselves as the fire grew in intensity. An observer narrated the fire growth to supplement what the cameras recorded directly.
- In two tests, an upward-facing, water-cooled Schmidt-Boelter total heat flux gage 127 mm (5 in) above the cubicle floor, near the cubicle center.
- An external total heat flux gage mounted so as to view the entire fire from one side. This provided a signal that was proportional to the instantaneous HRR of the fire and was useful for certain timing issues.
- In two tests, six 24-gage chromel/alumel thermocouples to follow the progress of the fire on the underside of the desk surfaces.

Ignition Scenario

The initiation of the tests was in keeping with the workstation being one of a large array on a given floor of a large office space and a fire propagating through the array. Thus a large ignition source (a 2 MW, four-nozzle spray burner over a 2 m × 1 m pan) was placed immediately adjacent to the exterior of one wall panel of the test station, simulating the burning of the adjacent workstation. The size and placement (pan bottom 0.81 m above the floor) of this ignition source were guided by preliminary FDS predictions.

The igniter fire was supported by a flow of commercial-grade liquid heptanes (mixed isomers) sprayed at a nominally steady rate from the four nozzles pointing downward toward the pan. The heptane mix was supplied from a reservoir by a variable-speed pump. The desired fuel flow was preset before the test and measured in triplicate by catching the flow from each nozzle in a volumetric cylinder for a typical period of 20 s. The nominally 2 MW fire supported by the heptane flow typically impinged almost continually on the ceiling above the igniter. There was thus an essentially continuous wall of flames radiating toward the workstation along the central three-fourths of the length of one panel. In addition, the workstation was subject to radiation from the hot ceiling and the hot smoke captured below the ceiling.

Test Variables

There are numerous variables that might influence the burning of a workstation such as that examined here, starting with the nature of the materials and their spatial arrangement, the ignition conditions, etc. There are also variables unique to the WTC fires: the amount of paper-based clutter in a workstation, the possible presence of jet fuel (and its amount), the possible presence and amount of inert rubble generated by the airplane impact (fallen ceiling tiles, inert dust from pulverized concrete and/or wallboard), varying degrees of impact-induced break-up and compaction of the work station itself, and presence of combustible solids from the airplane.

The tests were designed keeping the purposes in mind: (a) identify the current capability of FDS to predict complex burning behavior using combustion data from small-scale specimens and (b) obtain clues to improving the combustion algorithm should the predictions be of insufficient accuracy. Thus the two selected variables were those that could affect the workstation fires in manners that test FDS and are important in the WTC context: the presence or absence of both jet fuel and of inert rubble. The inert rubble was taken to be representative of fallen ceiling tiles.

The primary test set thus focused on two levels of two variables examined in a full factorial design. This calls for four tests: (1) no inert rubble and no jet fuel, (2) rubble but no jet fuel, (3) jet fuel but no rubble and (4) rubble and jet fuel. The levels of the two variables were estimated to produce differences in burning behavior that would be clearly observable. Thus the inert rubble was chosen to cover approximately 30 percent of the horizontal surfaces facing the hot ceiling. As performed, 24 of the 40 ceiling tiles were on the horizontal desk surfaces; 14 were on the central, open floor area; and 2 were on the chair seat. The fraction of the plan view horizontal area covered by these tiles was approximately 31 percent. A total of 4 L (approximately 1 gal) of Jet A was spread over these same horizontal surfaces.[1]

Two additional tests were conducted. The first test (in the entire series) utilized what was nominally one-half of a generic workstation, though it included both a full chair and full computer. This was done to gage the burning behavior of what was an entirely unknown system. The fourth test was of the high-end workstation. Both of these were conducted with no jet fuel and no inert rubble present.

Test Procedure

The workstation was assembled a few hours before a test. Since the ambient humidity was high (approximately 70 percent) for most of the tests, the paper was covered with plastic sheeting if it was to be exposed for more than 2 hours, although this probably did not preclude significant moisture pick-up in the outer portions of the paper piles. That moisture would be expected to somewhat slow the ignition process relative to a more normal humidity of 50 percent.

At the beginning of each test, all instruments were calibrated. [The heat release rate was also measured for a few minutes after the test ended in order to verify the calorimeter baseline.] The heptanes flow was measured in triplicate. The test was initiated by starting the heptanes flow and immediately igniting it with a torch. This defined time zero. The heptanes flow was left on until late in the test unless there was

[1] Areas close to the impact could have been drenched with higher quantities. The workstations would also have had to be extensively fragmented, which is not a situation being examined here. Another pragmatic factor that kept the jet fuel loading down was the probability that higher levels would have pushed the calorimeter beyond its maximum allowable capacity.

an indication that the fire was going to significantly exceed the 10 MW calorimeter capacity; this happened only once (test 5). The full test was video taped by all four cameras and a narration was fed to one camera, describing the sequence of ignition events that spread the fire over the accessible flammable surfaces. In two cases (tests 3 and 4) the residual weight of the paper piles in the two file drawers was obtained as a measure of the participation of this paper in the overall fire.

In the two tests in which jet fuel was placed on the horizontal surfaces prior to the start of a test, this was done just before ignition of the 2 MW spray burner. A sprinkling can was used in two separate operations, each involving 2 L of the liquid fuel. First, the liquid was sprinkled on the horizontal desk surfaces and the objects on them (i.e., the various paper stacks or document boxes, the computer monitor and the keyboard) using a timed movement that attempted to allot an equal amount of liquid to each one-third section of the total work surface. The desk surface had been leveled after installation so that the liquid would not run preferentially in one direction. Next, an equal amount of the liquid was sprinkled in a similarly timed manner on the central open section of the carpet (not under the desk surfaces). Since the chair occupied a portion of this space, the allotment for that portion went onto the chair seat and back surfaces. For the objects on or in contact with the desk surface, there was some tendency for the jet fuel to wick into them if they were porous. This was true of the paper, the inert tiles and the wall panel fabric just above the desk surface.

J.3.2 Preliminary Results

Figures J–16 and J–17 show the workstation early in a test and at a time just before the peak heat release was reached. Figures J–18 through J–23 show the heat release rate curves from the six fire tests.

Figure J–16. Photograph of workstation soon after ignition in test 2.

Figure J–17. Photograph of workstation near the peak heat release rate in test 2.

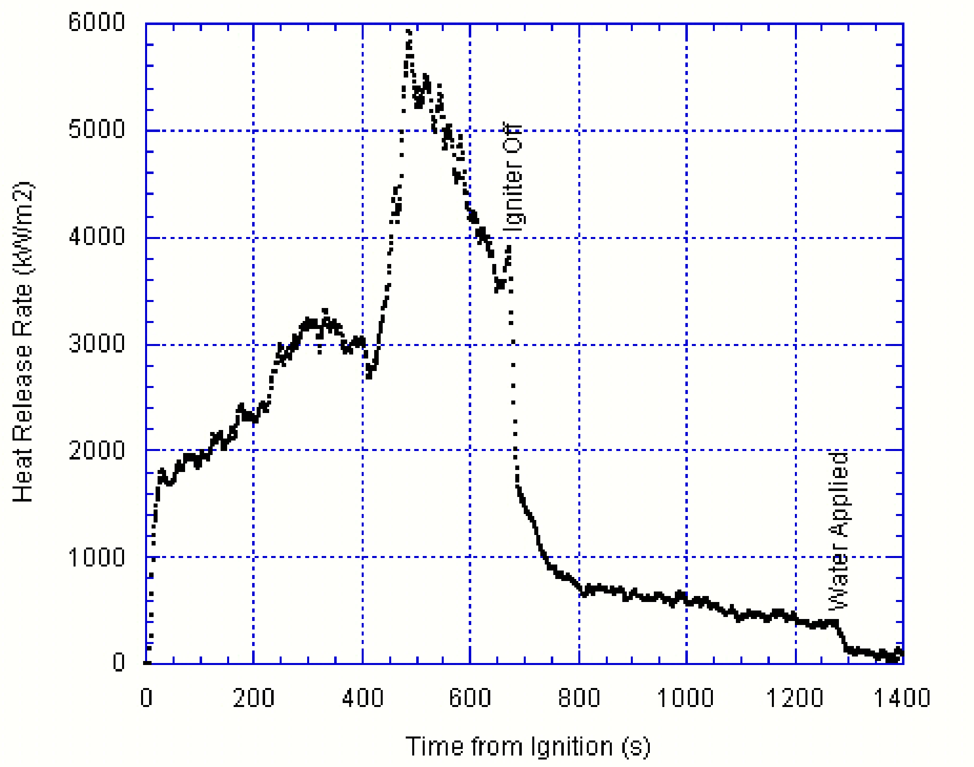

Figure J–18. Heat release rate versus time for single workstation test 1.

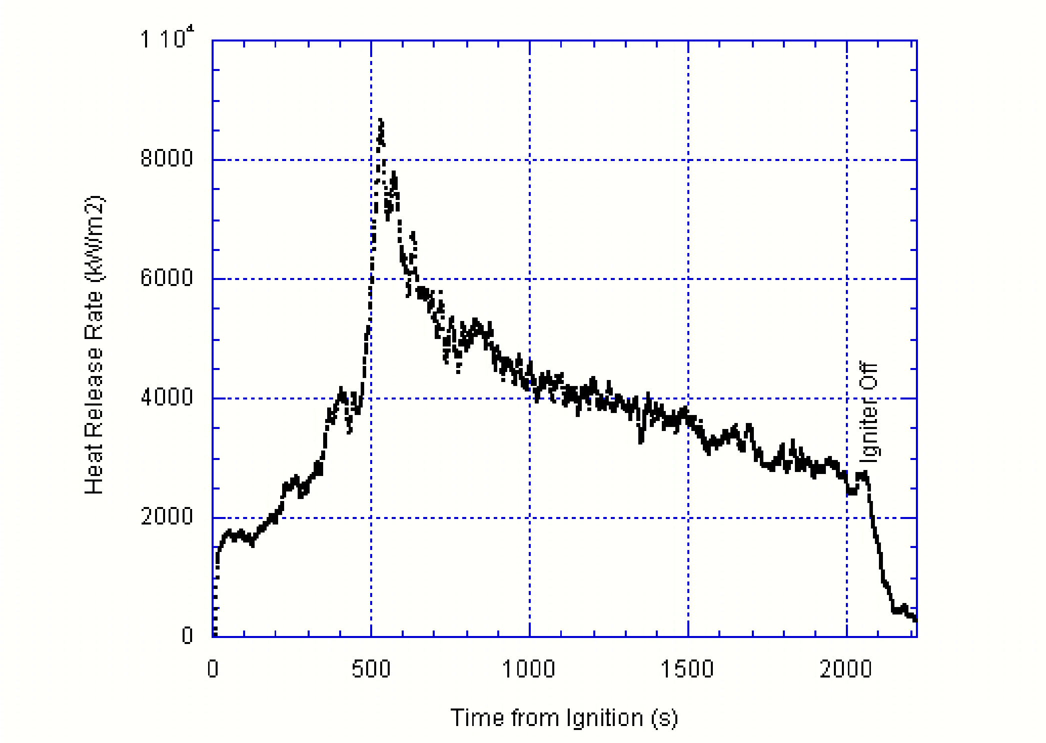

Figure J–19. Heat release rate versus time for single workstation test 2.

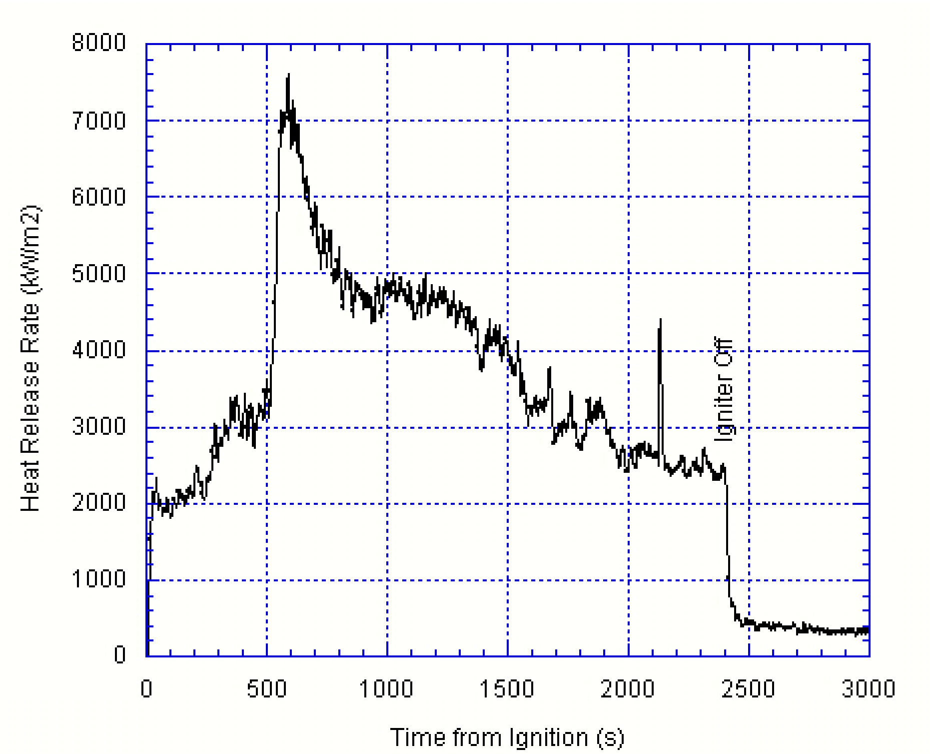

Figure J–20. Heat release rate versus time for single workstation test 3.

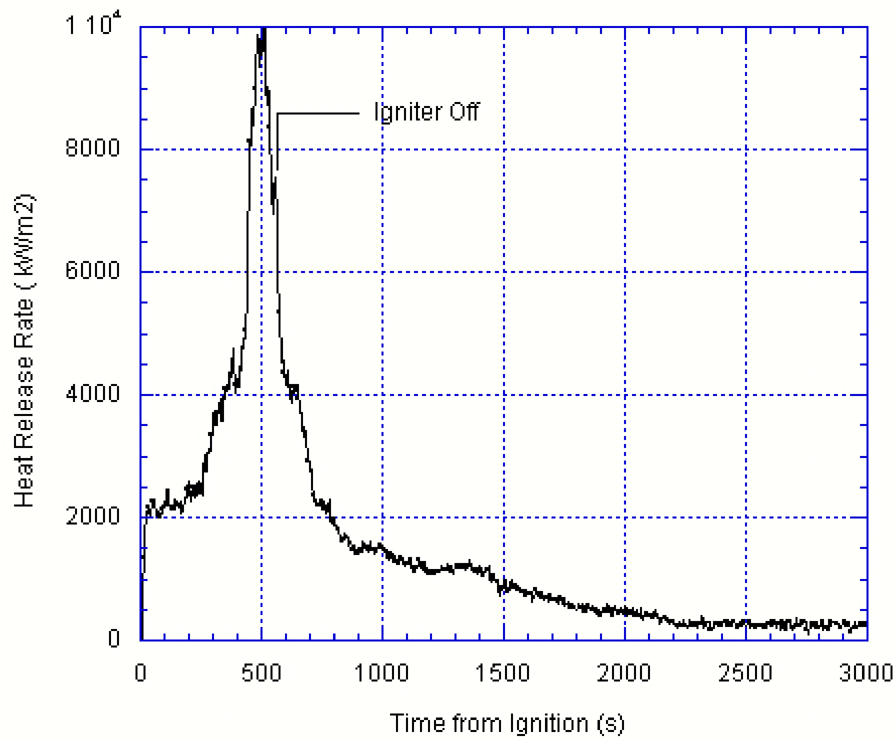

Figure J–21. Heat release rate versus time for single workstation test 4.

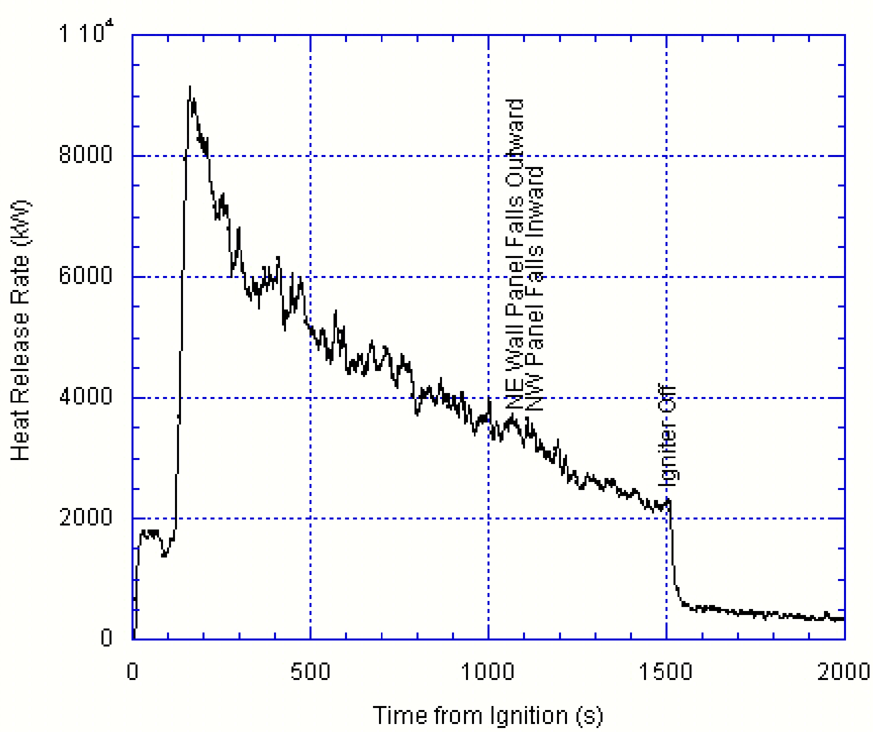

Figure J–22. Heat release rate versus time for single workstation test 5.

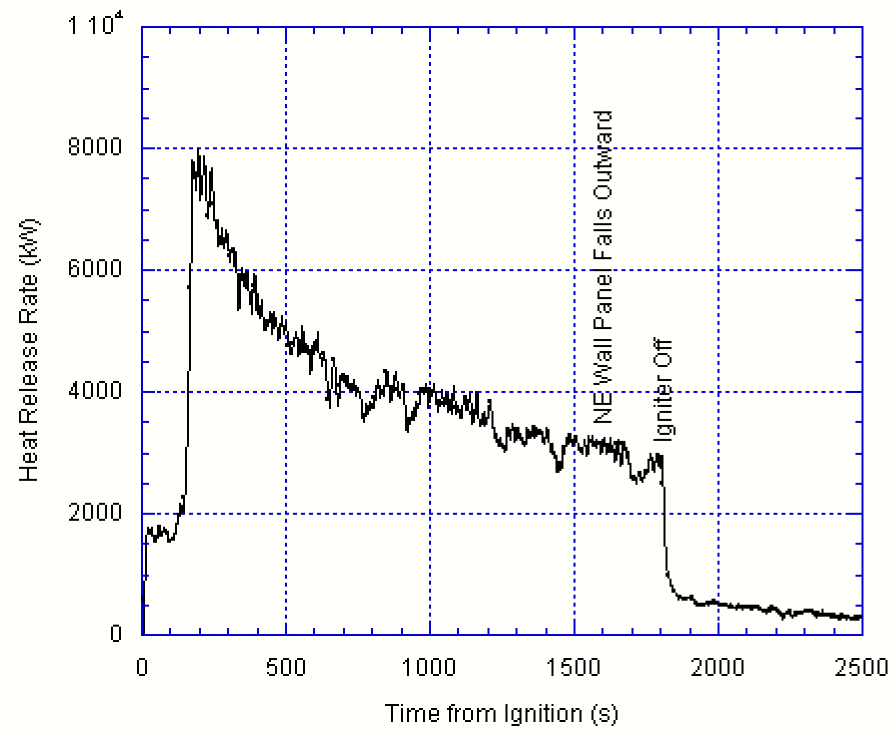

Figure J–23. Heat release rate versus time for single workstation test 6.

Table J–3 lists the peak HRR values from each test plus the time to that peak, rounded to the nearest 10 s. The HRR peak is given in two ways: first, as the absolute highest single reading recorded (for a 1 s interval) and second, as the average of the value at this peak plus values up to 5 s to either side of this peak. The latter compensates for both noise in the calorimeter system and in the fire itself; it is doubtful that objects in a real room can respond in any meaningful way to small HRR fluctuations in a fire that is under 10 s in duration. The averaged values are about 3 percent lower than the absolute peak values, a result of the sharpness of the peaks in every case.

Table J–3. Peak heat release rates and times to peak.

Test	Test Specimen	Jet Fuel (Yes/No)	Inert Tiles (Yes/No)	Peak HRR (kW)	Time to Peak HRR (s)
1	Half of generic workstation	N	N	5920/5770	490
2	Generic work station	N	N	8700/8480	530
3	Generic work station	N	Y	7560/7300	590
5	Generic work station	Y	N	9120/8910	160
4	High-end work station	N	N	9890/9660	510
6	Generic work station	Y	Y	7960/7690	200

The videotapes of the fires show that the peak HRR corresponds closely to simultaneous burning of all the “accessible” combustible surfaces in the workstation interior. This included the top of the desk surface, the objects on it, the exposed area of the materials in the bookcase, the full chair area (but see below), the exposed area of the carpet (i.e., not that under the three steel file cabinets), objects on the carpet (computer processor case and wastebasket) and the underside of the desk, except above the steel file cabinets where the air access was limited. The interior surfaces of the wall panels contributed negligibly because the thermoplastic fabric melted and rolled downward into a mass having much less surface area than the original fabric prior to igniting. This type of behavior is not captured in Cone Calorimeter tests.

The following observations emerge from comparison of the results in Table J–3:

- The peak fire intensity from the half workstation is about two-thirds that of the full workstation. This is probably primarily the result of two factors:
  - The same chair was present in both cases; this chair has an estimated HRR peak in the neighborhood of 1/2 MW by itself.
  - The inert steel file cabinets cover twice as much of the carpet in the second half of the workstation, precluding its participation in the HRR peak; this lowers the HRR contribution from the second half of the workstation.
- The computer was also totally combusted within the first half of the experiments, though this was a substantially lesser heat source than was the chair.
- The ceiling tiles reduced the peak HRR in proportion to their coverage of the burning surfaces; both just under 15 percent. While there was 30 percent coverage of the upward facing horizontal surfaces, there was no coverage of the underside of the desk, the carpeting below the desk or the underside and back of the chair. As noted above, all of these were burning at the peak. (The reduction might become more than linear if nearly all of the upward facing horizontal surfaces were covered since this could preclude the progressive flame spread that gets all accessible surfaces involved)
- The principal effect of the presence of 4 L of Jet A on the horizontal surfaces was in shortening the time to involvement of all accessible combustible surfaces, and thus the time to the peak HRR. The peak itself was boosted upward only about 5 percent, presumably because the Jet A helped boost the overall fuel gasification rate somewhat while adding its high heat of combustion. When the inert tiles were also present, the Jet A was poured across their top surfaces temporarily rendering these surfaces flammable. Since the tiles were porous, the Jet A burning rate on them was reduced, however, and the tiles still managed to produce a 13 percent to 14 percent reduction in the peak HRR relative to the case with Jet A and no tiles.

From examination of the videotapes and the commentary, NIST determined that, when there was no Jet A present, there was a fairly reproducible progression of ignition events leading up to the HRR peak:

- The onset of the igniter fire bathed the entire workstation in radiant heat.
- The igniter fire was not the pilot flame that ignited other objects, although FDS simulations suggest that the peak fluxes on surfaces facing the igniter were of the order of 30 kW/m^2 or more, well above the minimum flux for piloted ignition of the various exposed surfaces. Even the top of the computer monitor shell, which gasified extensively only 20 cm from the spray burner

flames, did not appear to ever have directly been ignited by those flames. Instead the progression of flaming was initiated by the top sheets on the paper stacks. Its presence in many places made it highly instrumental in spreading the flames to the computer monitor shell, the desktop and the top of the chair back. Flaming material from the chair fell onto the carpet, igniting it. Subsequent spread on the carpet was delayed, however, until the chair was fully involved, along with the remainder of the upward-facing surfaces of the desktop. Later, paper ignition brought flames to the two other sides of the desk area in the workstation.

- Once the entire area at and above the top of the desk surface was burning, the "compartment" under the desk flashed over from the radiant heat from the upper compartment and especially the heat from the burning chair. With all flammable surfaces ignited, the HRR quickly peaked. The specific construction of the chair and its location were critical to this fire growth process:

  – One corner of the seat was deliberately placed to extend approximately 15 cm under the desk surface on which the computer keyboard rested. This assured that some heat would reach this space early on, even though the chair flames would contact the relatively ignition resistant underside of the desk.

  – When only the upholstered surfaces of its seat and back were burning, the chair retained its original shape, and little of its heat reached the lower compartment area.

  – When the thermoplastic support shell of the chair began to melt and flow to the floor, extensive heat flowed directly into the "compartment" under the desk.

  – The partial steel skeleton kept the chair from collapsing and maintained burning from desk level to floor level. The flame radiation to the cavity under the desk quickly ignited the materials located there, while the upper part of the chair flames played on the desk, assisting ignition of its underside.

Since chairs of different designs (and thus different burning behavior) could be fabricated from the same materials, the detailed fire behavior of the chair cannot be inferred simply from the Cone Calorimeter data for the component materials. Thus, empirical treatment of its HRR process is necessary.

As expected, the progression of ignition events in the presence of Jet A was different. Ignition of the materials in the cubicle occurred more quickly. However, the manner of the acceleration was not what might have been expected.

- Since the flash point of Jet A is at least 46 °C (approximately 20 °C above the ambient temperature), initially there would be no flammable mixture of vapors above the liquid fuel surface. As the strong radiant flux from the spray burner bathes the workstation, the videos show an increasingly dense aerosol rising from the various wetted surfaces near the spray burner. These did not ignite, as evidently turbulent mixing of the vapor plume above the surfaces quickly diluted the fuel vapor below the lean flammable limit. Instead, in the first Jet A test, a random piece of flaming material rose from the burner area and drifted down on top of the desk, immediately igniting the Jet A. In the second test, a flaming piece of debris from the burner landed on top of the bookcase where there was no jet fuel and it had no effect. The paper stacks on the work surface next to the spray burner finally dried out, began to char, and then transitioned into flaming, igniting the jet fuel.

- The Jet A flames then spread rapidly, but did not sweep continuously around the desktop. Presumably the initial evaporation period left some dry spots that stopped the spread.

- The carpet was ignited by flaming matter dropping from the chair. Apparently the turbulent vapor plume dilution process mentioned above prevented the flames from jumping downward to the carpet from the flaming desk surface, even though the carpet was emitting an aerosol. At any rate, the subsequent flame spread on the carpet was rapid.

- None of these steps is resolved in the HRR curves of Figs. J–21 and J–22 due to the rapidity with which the curves jumped rapidly to their peak values soon after the Jet A ignited. As in the "dry" tests, the peak corresponds to all accessible combustible surfaces burning simultaneously.

The rapid decay in HRR after the peaks in all tests presumably reflects several factors that should be captured in the combustion algorithm in FDS:

- The various paper piles developed a thick ash layer that would drive down their burning rate.

- Char formation on the desk surfaces drove down its burning rate.

- The carpet began to burn out.

However, a number of geometric changes occurred that are beyond the capability of FDS to reproduce:

- The chair fire rapidly collapsed to a pool fire on the floor whose reduced burning area meant a reduced HRR.

- The front of the bookcase, resting on top of that unit, typically fell, changing the location of its 13 reams of paper.

- The desk surface bowed progressively as it charred through, and then it collapsed, with separate sections doing so at differing times. The initial desk collapse probably did not greatly affect its burning rate, but ultimately what was left was a complex rubble pile whose burning would not be predictable from any knowledge of the original configuration coupled with Cone Calorimeter data.

- The wall panels collapsed at random times, both inward and outward, typically rather late in the fire.

Thus, the further one goes out on the HRR curve, past the peak, the less it is predictable by an FDS calculation that retains the original geometry. Fortunately, the major effects appear to occur well after the desk surfaces collapse and the time when contiguous workstations would become ignited and dominate the heat release.

Numerical grids of 10, 20, and 40 cm were used to model the fires and ensure that the model was not sensitive to grid cell size. Figure J–24 shows a preliminary comparison of the FDS HRR prediction with the measured values for test 2 (no jet fuel, no inert tiles). The quality of fit is typical of the test series.

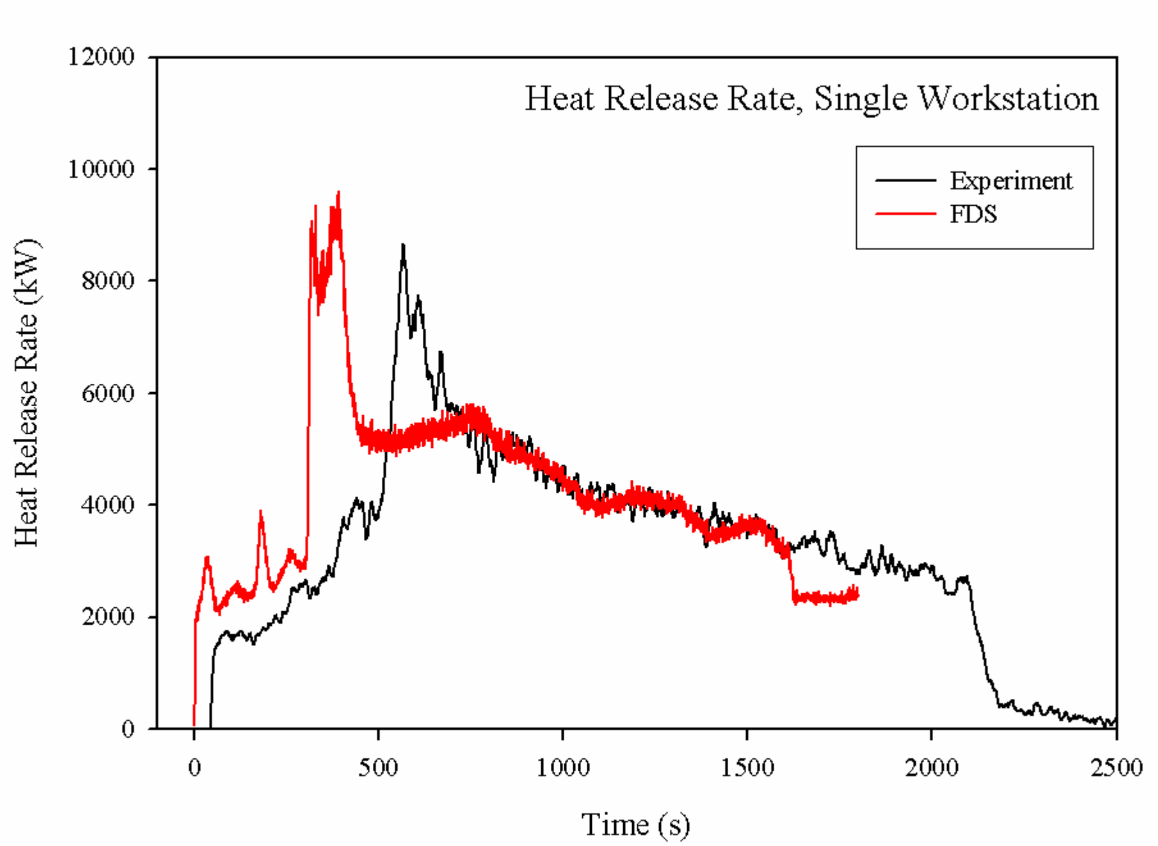

Figure J–24. Preliminary comparison of predicted and measured HRR values.

For these simulations, the thermal properties of the major materials making up the workstations were derived from Cone Calorimeter experiments. The carpet and privacy panel were modeled as thermoplastics, that is, the burning rate was assumed to be proportional to the heat flux from the surrounding gases. The desk was modeled as a charring solid, in which a pyrolysis front propagates through the material leaving a layer of char behind that insulates the material and reduces the burning rate. Details of the pyrolysis models can be found in the *FDS Technical Reference Guide* (McGrattan et al. 2002). Each feature of the experimental curve was related (using annotations made during the tests and from the video tapes) to specific aspects of the workstation combustion.

There are similarities and discrepancies between the experimental data and this prediction.

- The shape and magnitude of the two curves is encouragingly similar, as is the total heat release (area under the curves).
- The peak HRR occurred sooner in the simulation. In the experiment, the time to peak HRR was strongly influenced by the melting of the chair plastic onto the carpet. As noted above, this level of detail is not captured in the numerical model.

- At long times, the simulation drops to the residual HRR of the 2 MW burner somewhat more abruptly than does the experimental curve, indicating that it has run out of combustible mass sooner. The importance of this effect is modest, given the geometric changes during the test (listed above) and the similarity of the total heat released.

A more complete analysis will be detailed in the forthcoming documentation report.

J.4 EXPERIMENTS FOR FIRE MODEL VALIDATION

Following the experiments described above and the accordant improvements in FDS, a series of large-scale experiments was conducted in the NIST Large Fire Laboratory between November 4 and December 10, 2003. The six experiments were designed to assess the accuracy with which FDS predicts the fire spread, heat release rate, and thermal environment in a compartment burning multiple workstations in a configuration characteristic of that found in the WTC buildings. In each of these experiments, sets of three workstations, identical to the generic ones tested in Section J.3, were burned in a large compartment (see Fig. J-25). The challenges to the model included varying the location of the ignition burner (and thus the fire ventilation), adding jet fuel and/or noncombustible material occluding a fraction of the workstations' surfaces, and "rubblizing" the workstations. FDS simulation of each test was carried out before the test was conducted.

The steel-frame experimental enclosure was 10.8 m long × 7.0 m wide × 3.4 m tall (35.5 ft × 23 ft × 11 ft) and was lined with three layers of 13 mm (0.5 in) calcium silicate board (see Fig. J-26). There was a subfloor (not included in the above dimensions) to house instrumentation. The enclosure had openings on the front mimicking window openings through which fresh air entered and heat and combustion products were emitted. The narrowed openings limited the amount of fresh air that entered the burning enclosure.

Each of three workstations was placed on an isolated platform made of calcium silicate board. The top surface of each platform was flush with the floor of the compartment. Each platform was supported on water-cooled load cells, located in the subfloor, to monitor the mass of the workstation throughout the test. The load cells were the same as those described in Section J.3.

Two of the workstations were contiguous, exemplifying a part of the type of cluster that exists in many large office spaces. The third workstation was separated from the other two by an aisle, representing a part of a second cluster. This array was to enable assessment of FDS's ability to replicate two different modes of cubicle-to-cubicle fire spread: direct flame impingement and radiative ignition from the hot ceiling layer.

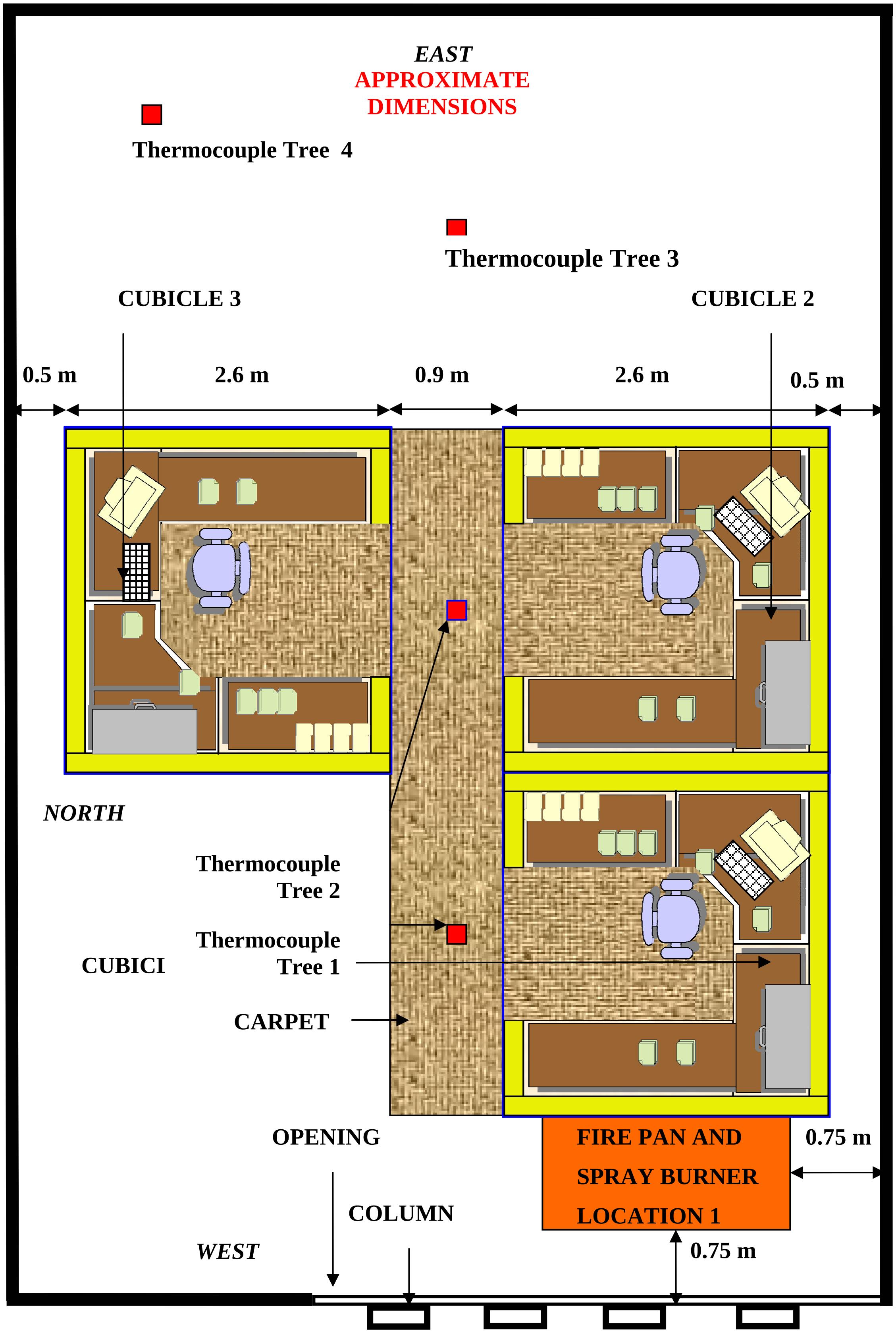

Figure J–25. Plan view of test configuration.

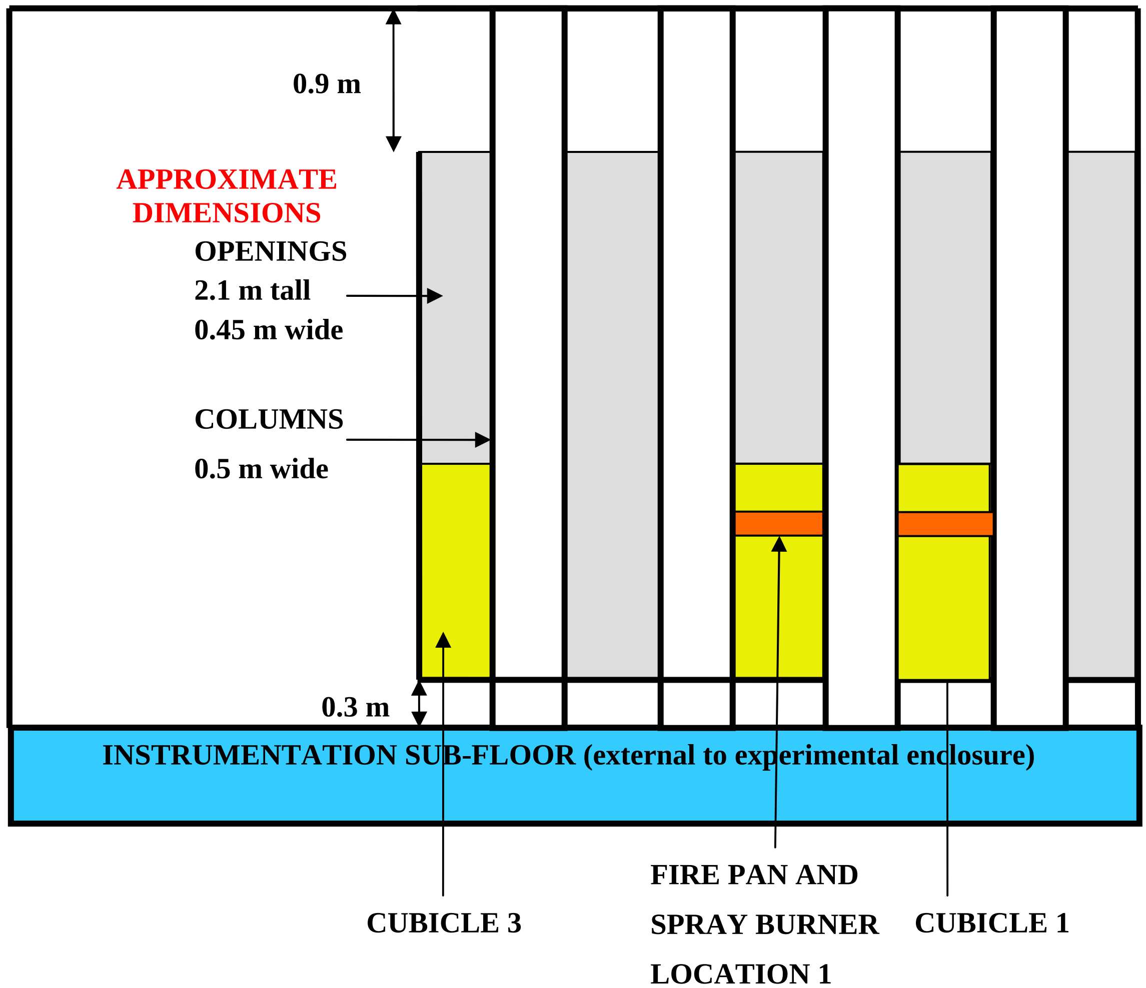

Figure J–26. Elevation view of test configuration.

The west end of the enclosure was located under a 10 m × 12 m hood for collection of the effluent and measurement of the heat release rate.

Other instrumentation included:

- Four floor-to-ceiling trees of thermocouples to measure vertical profiles of temperature(see Fig. J-27);
- Thermocouples on the desk surfaces to track flame spread;
- Two downward-facing, water-cooled Schmidt-Boelter total heat flux gages in the ceiling to measure radiative heat flux;
- Two water-cooled Schmidt-Boelter total heat flux gages mounted on the west wall; and
- Four video cameras placed to record the progression of flame spread over the objects in the cubicle.

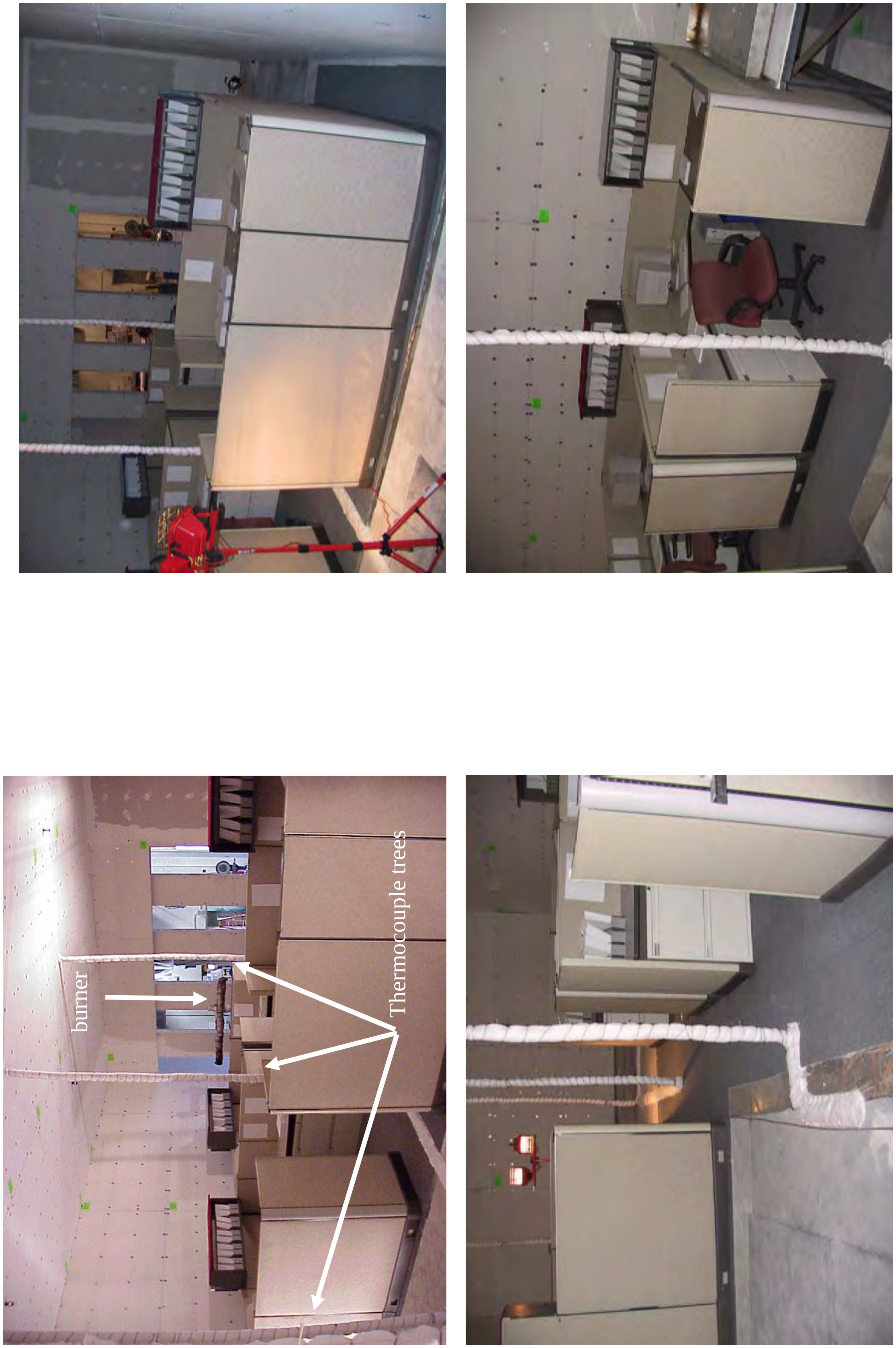

Figure J–27. Views of interior of test fixture.

The liquid spray burner, pan, and fuel (mixture of heptanes) were the same as used in the Section J.3 experiments. Depending on the test, the burner was located abutting the top of a workstation partition at the east end of cubicle 1 or the west end of cubicle 2. The ignition fire intensity was a nominal 2 MW fire. The spray burner was operated for the first few minutes of the tests, for either 2 min or 10 min depending on the test scenario.

Materials

The workstations were of the same type as the generic units used in the experiments reported in Section J.3. The carpet tiles, which covered the floor of the cubicles and the aisle, were also the same type used in experiments reported in Section J.3.

Test Variables

The experimental matrix is shown in Table J–4. The experiments investigated the impact of several parameters on the fire behavior:

Table J–4. Test matrix.

Test	Ceiling Tiles	Jet Fuel	Burner Location	Workstations	Windows
1	None	None	Front	Intact	No
2	None	None	Front	Intact	No
3	Present	Present	Front	Intact	No
4	Present	None	Rear	Intact	No
5	Present	Present	Rear	“Rubble”	No
6	None	Present	Rear	Intact	Yes

- Location of the burner. This was placed either abutting the west end of cubicle 1 or abutting the east end of cubicle 2. These two sites resulted in significantly different access to the air needed for combustion. In the former (“front”) location, much of the oxygen in the air initially entering the enclosure was consumed by the burner and the burning cubicle 1, with the result that limited oxygen was available for combustion in the middle and rear of the compartment. With the burner in the latter (“rear”) location, the fresh air passed directly to the rear of the compartment.

- The application of 12 L of jet fuel evenly distributed about each workstation. The procedure was the same as used in the experiments reported in Section J.3.The presence of fallen ceiling tiles. Having seen the effect of coverage of 30 percent of the top surfaces in the previous test series, NIST covered approximately 70 percent of the top surfaces here.

- Fractured furniture. In one experiment (test 5), NIST investigated the effect of different degrees of “rubblizing” the furniture.

  – In cubicle 1, the workstation pieces were placed unassembled on top of each other, occupying the same footprint as the assembled workstation. The same mass of combustibles was present as in the fully assembled cubicle tests. No steel filling cabinets were used. Ceiling tiles and broken up drywall were intermixed with the rubble.

- Cubicle 2 was the same as cubicle 1, except without the drywall.
- For cubicle 3, the workstation was partially assembled. The same mass of ceiling tile and drywall as in cubicle 1 were intermixed with the cubicle components.
- Window breakage. Test 6 had four glass windows mounted on the north end of the west wall. During the course of the fires in the WTC towers, a number of windows were broken, presumably by the heat from the fires. These result in a change in both the degree and pattern of ventilation.

Test Procedure

This was similar to that followed in the experiments reported in Section J.3, except that the ignition burner was turned off after approximately 2 min for tests 3, 5, and 6 (when jet fuel was present) and for 10 min for Tests 1, 2, and 4. The tests continued until the HRR fell below 0.5 MW, which was typically 60 min after ignition.

J.4.1 Preliminary Results

Figures J–28 and J–29 show the east view of the compartment before and during a test, respectively. A few observations about the tests were:

- The peak HRR was approximately 11 MW for four of the tests. In test 5, the peak value was only approximately 6 MW. In test 6, the peak HRR reached almost 16 MW.
- As in the single workstation tests, the peak value was reached earlier when jet fuel was present.

Figure J–30 shows how the enclosure was represented in FDS. The computational grid size was 0.4 m (1.3 ft) on a side. Note that the chair, computer monitor, and paper have been collected together into "boxes" with comparable mass to the various items that were spread throughout the workstations. To the right are five windows that are similar in size to those of WTC 1 and WTC 2.

The preliminary plot in Fig. J–31 depicts the degree to which the heat release rate measurements agreed with those predicted by FDS.

The overall degree of agreement between the model and the experimental data is quite good, despite some modest local differences. In the analysis leading to candidate improvements in the modeling, it is important to maintain perspective of the effect of these differences on the accuracy needed in reconstructing the actual WTC fires. For fires that are sufficiently severe that they threaten the structural integrity of the building, many such workstations will be burning concurrently. These workstations will be at various stages of their combustion. Thus, for example, features occurring at long times in Fig. J–31 may not merit closer replication, while those features occurring at short times (and thus have a bearing on the ease of fire spread among workstations) may merit attention.

Figure J–28. View of fire compartment before the start of test 6.

Figure J–29. View of fire compartment 2 minutes after the start of test 6.

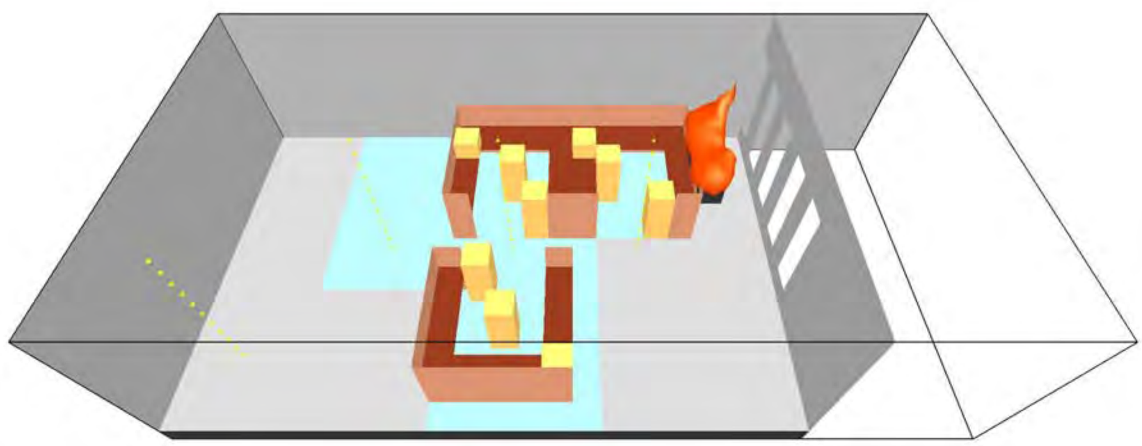

Figure J–30. Fire Dynamics Simulator 3D rendition of experimental enclosure.

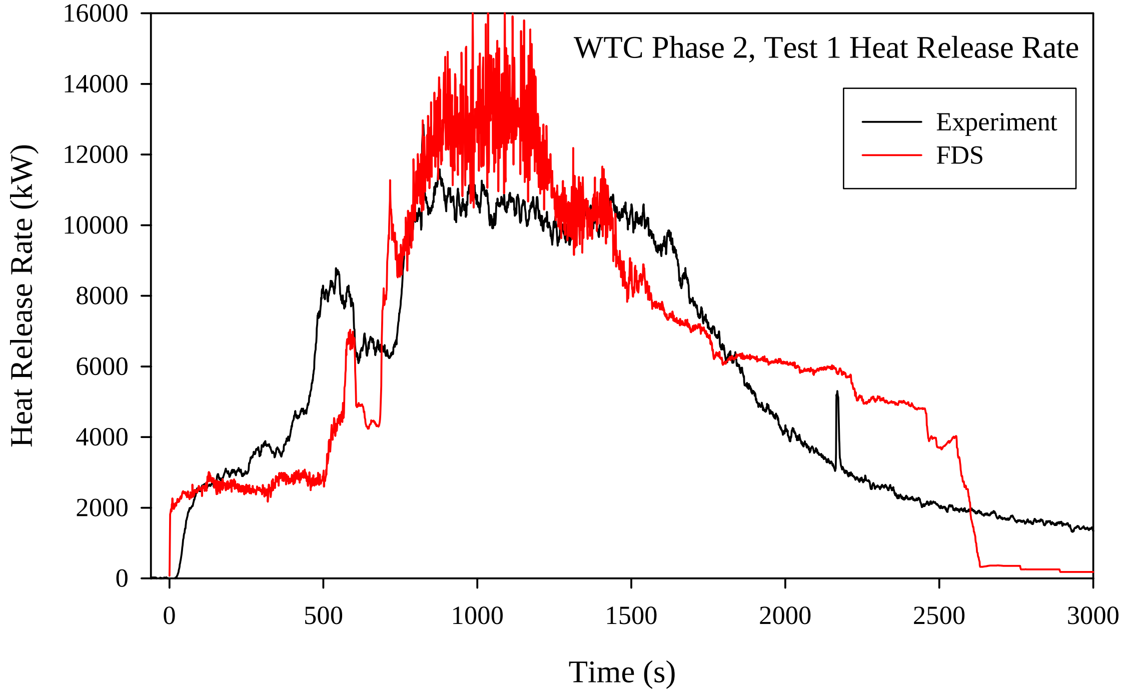

Figure J–31. Comparison of measured and predicted heat release rate for test 1.

J.5 REFERENCES

McGrattan, K.B., H.R. Baum, R.G. Rehm, G.P. Forney, J.E. Floyd, K. Prasad and S. Hostikka. 2002. Fire Dynamics Simulator (Version 3), Technical Reference Guide. NISTIR 6783. National Institute of Standards and Technology, Gaithersburg, MD.

Attachment 1
Simulating the Fires in the World Trade Center

1.1 INTRODUCTION

In the months following the attacks on the World Trade Center (WTC) and the Pentagon, there was an active debate in the fire protection engineering community about the fires that erupted following the impact of the aircraft on the buildings. Because fires of this magnitude in these types of buildings are rare, there is a wide spectrum of opinion about the fire temperatures and their effect on the structural steel. Much of the fire literature consists of empirical correlations derived from experiments ranging from bench scale to room scale. Extrapolating these well-known correlations to the WTC requires a reexamination of the underlying assumptions. Many of these correlations are appropriate for a narrow range of fire sizes and building geometries, and cannot be directly applied to the WTC fire scenarios. As a result, computer fire models that have been developed over the past decade are being applied to the analysis.

As part of the investigation, the National Institute of Standards and Technology (NIST) has conducted simulations of the fires in each building using a computational fluid dynamics (CFD) model known as the Fire Dynamics Simulator (FDS). This attachment will describe the experiments conducted at NIST to calibrate and validate the FDS model for use in the WTC project, and it will describe the techniques developed to simulate the very extensive fires that spread over 6 to 12 floors in the different buildings.

1.1.1 Experimental Program

Two large-scale test series were conducted to provide validation for the FDS, plus various small-scale experiments were conducted to provide the model with input data for different materials. The large-scale test programs are referred to as Phase 1 and Phase 2. Both test series involved fires in compartments with the same ceiling height as a floor in WTC 1 or WTC 2. Phase 1 was a series of fire tests with a liquid fuel spray burner generating a fixed amount of energy in a compartment with various targets and obstructions, like columns, trusses and other steel objects. These tests were designed to test the accuracy of the model, and its sensitivity to changes in various input parameters. Phase 2 was a series of fire tests in which office workstations similar to those used in WTC 1, WTC 2, and WTC 7 were burned in a compartment with limited openings to simulate the under-ventilated conditions of the WTC fires. These tests were designed to test the model's ability to characterize the burning behavior of real furnishings under conditions typical of the WTC fires. Only the Phase 2 work will be discussed in this attachment.

1.1.2 NIST Fire Dynamics Simulator

FDS is a CFD model of fire-driven fluid flow. It solves numerically a form of the Navier-Stokes equations appropriate for low-speed, thermally-driven flow with an emphasis on smoke and heat transport from fires (McGrattan et al. 2002). Version 1 was publicly released in February 2000. The core algorithm is an explicit predictor-corrector scheme, second order accurate in space and time. Turbulence is treated by means of the Smagorinsky form of Large Eddy Simulation (LES). For most applications, FDS uses a mixture fraction combustion model. The mixture fraction is a conserved scalar quantity that is defined as the fraction of gas at a given point in the flow field that originated as fuel. The model assumes that combustion is mixing-controlled, and that the reaction of fuel and oxygen is infinitely fast. The mass fractions of all of the major reactants and products can be derived from the mixture fraction by means of "state relations," empirical expressions arrived at by a combination of simplified analysis and measurement.

Radiative heat transfer is included in the model via the solution of the radiation transport equation for a non-scattering gray gas, and in some limited cases using a wide band model. The equation is solved using a technique similar to finite volume methods for convective transport; thus the name given to it is the Finite Volume Method (FVM). Using approximately 100 discrete angles, the finite volume solver requires about 15 percent of the total CPU time of a calculation, a modest cost given the complexity of radiation heat transfer. FDS approximates the governing equations on a rectilinear grid. The user prescribes rectangular obstructions that are forced to conform with the underlying grid.

All solid surfaces are assigned thermal boundary conditions, plus information about the burning behavior of the material. Usually, material properties are stored in a database and invoked by name by the user. An extensive effort was undertaken to characterize the thermal properties of common items found in an office setting, like privacy panels, stacks of paper, computer monitors, office chairs, pressboard tables, desks, and carpeting. These materials will be described next.

1.2 CALIBRATION AND VALIDATION EXPERIMENTS

The experimental program concentrated on the thermal properties of the office furnishings that constituted the bulk of the combustible fuel within the WTC buildings under study. Several types of office workstations typical of those used in WTC 1 and WTC 2 were purchased at area office supply stores. The thermal properties of the major materials making up the workstations were derived from cone calorimeter experiments. These properties were input into FDS, which was used to simulate the burning behavior of a single workstation burning under a 2.5 m ceiling with baffles to contain a hot layer of smoke above the burning workstation. Other than the baffled ceiling, no walls surrounded the workstation other than its own privacy panels. The thermal properties of the workstation components were adjusted slightly so that the FDS prediction of the heat release rate would match the experiment. Then the model was used to predict the heat release rate of 3 workstations burning within a large enclosure. The purpose of this exercise was to determine if FDS could simulate the dynamics of a fire in a setting similar to WTC 1, WTC 2, and WTC 7.

1.2.1 Description of the Workstation Components

Cone calorimeter experiments at three different heat fluxes were performed for the carpet, desk (wood), computer monitor, chair, privacy panel, and stacked paper. For the simulations of the WTC fires, only the carpet, desk and privacy panel data were used directly. The carpet and privacy panel were modeled as thermoplastics, that is, the burning rate is assumed to be proportional to the heat flux from the surrounding gases. The desk was modeled as a charring solid, in which a pyrolysis front propagates through the material leaving a layer of char behind that insulates the material and reduces the burning rate. Details of the pyrolysis models can be found in the FDS Technical Reference Guide (McGrattan et al. 2002).

The desk was modeled as a charring solid. The thermal properties of the wood and its char were taken from both the calorimeter experiments and the work of Ritchie et al. (1997). It is 2.8 cm thick with density 450 kg/m^3, specific heat 1.2 kJ/kg/K at 20 °C and 1.6 kJ/kg/K at 900 °C, conductivity 0.13 W/m/K at 20 °C and 0.16 W/m/K at 900 °C. The ignition temperature is 360 °C and the heat of combustion is 14,000 kJ/kg ± 800 kJ/kg. Its total available energy content is 210 MJ/m^2 ± 50 MJ/m^2.

The carpet was modeled as a thermoplastic with density 750 kg/m^3, specific heat 4.5 kJ/kg/K, conductivity 0.16 W/m/K, ignition temperature 290 °C, thickness 6 mm, heat of vaporization 2,000 kJ/kg, heat of combustion 22,300 kJ/kg ± 600 kJ/kg, and total available energy content 61 MJ/m^2 ± 2 MJ/m^2.

The privacy panel was modeled as a thermally-thin thermoplastic. The product of specific heat, thickness and density is 0.73 kJ/m^2/K. Its surface density is 0.25 kg/m^2, ignition temperature 380 °C, heat of vaporization 6,000 kJ/kg, heat of combustion 30,000 kJ/kg ± 500 kJ/kg. Its total available energy content is 6.0 MJ/m^2 ± 1.3 MJ/ m^2.

The test compartment walls and ceiling were made of three layers of 1.27 cm (0.5 in) thick Marinite I, a product of BNZ Materials, Inc. (http://www.bnzmaterials.com).[2] The manufacturer provided the thermal properties of the material used in the calculation. The density is 737 kg/m^3, conductivity 0.12 W/m/K. The specific heat ranged from 1.2 kJ/kg/K at 93 °C to 1.4 kJ/kg/K at 425 °C.

In the simulations of the fires within the WTC, the chair, computer, paper, and other miscellaneous items within the workstation were modeled as a single item by lumping the mass together into large “boxes” and distributing them throughout the workstation. It is common practice in fire protection engineering to use surrogate materials for fire experiments, and this practice has been extended to numerical modeling. Over the years, various items have been developed that are representative of various types of commodities. For example, wood cribs are often used to represent ordinary combustibles found in residential or light industrial settings. Paper cartons with various amounts of plastic within are also used as surrogates for a wide range of retail commodities. One in particular is called the FMRC (Factory Mutual Research Corp.) Standard Plastic Commodity, or more commonly, Group A Plastic. This test fuel is often used in sprinkler approval testing at Factory Mutual and Underwriters Laboratories in the US, and similar test fuels have been developed in Europe. In the late 1990s, FDS was used to simulate large scale rack storage fires to determine the effectiveness of the combined use of sprinklers, roof vents and draft curtains (curtain boards). As part of this effort, a considerable amount of work was done to characterize the thermal properties of Group A Plastic (Hamins and McGrattan 2003). Because Group A Plastic has been shown to be fairly representative of fires fueled by a mixture of paper (cellulosic materials) and plastic, and because it has been used in numerous FDS simulations, it was decided to model the contents of the office workstations with a fuel similar to Group A Plastic. Blind predictions of the single open workstation burns were made using the material properties obtained during the sprinkler/roof vent study, and then these properties were adjusted to match the results of the experiments. Thus, the single workstation burns served to calibrate the model. They were not intended to be validation experiments. The validation experiments consisted of burning 3 workstations at a time in an under-ventilated compartment.

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[2] Certain commercial equipment, instruments, or materials are identified in this document. Such identification does not imply recommendation or endorsement by the National Institute of Standards and Technology, nor does it imply that the products identified are necessarily the best available for the purpose.

The surrogate fuel is modeled as a homogenous solid whose density is 172 kg/m^3. The paper carton is treated as a thermally-thin material whose density × specific heat × thickness is 1.0 kJ/m^2/K. Its ignition temperature is 370 °C and the heat of combustion is 30,000 kJ/kg. The heat release rate of the boxes ramps up to 450 kW/m^2 in about 1 min. Note that this fuel package is similar, but not the same, as Group A Plastic. The density has been increased to account for all the miscellaneous items within the workstation. Also note that unlike the desk, partition and carpet, the boxes are simply given a burning rate rather than a heat of vaporization, meaning that the boxes will burn at the given rate regardless of the heat flux upon them as long as the surface temperature remains above its ignition temperature. The reason for this is that it is not possible to predict the burning rate using the heat feedback approach because the geometry of the scattered fuel is too complex to directly predict the response of the material to the thermal insult. By collecting all the scattered items into boxes, the geometry of the combustibles is greatly simplified, and as a result the burning behavior must be simplified as well.

1.2.2 Description of the Simulations

The geometry of the compartment is relatively simple. The overall enclosure is rectangular, as are the vents and most of the obstructions. Numerical grids of 20 and 40 cm were used to model the fires. The purpose of the grid variation was to ensure that the model was not sensitive to the change in grid cell size. Typically, enclosures of this size are modeled using 10 cm grid cells. However, for the simulations of WTC 1, WTC 2, and WTC 7, a 40 cm grid was used. By simulating the experiments at 20 cm and 40 cm, NIST can test if the model produces significantly different results with grid cells of different sizes. Figure 1–1 is a snapshot of a simulation showing the fire and the major geometric features of the compartment for the simulations. Note that the surrogate fuel packages are placed roughly where the computer monitor, chair and paper were located. Six tests were performed, with various ignitor locations and fuel arrangements. A 2 MW burner was pl aced either near the windows of the compartment overlooking the workstation nearest the openings in Tests 1, 2, and 3. The burner was placed towards the rear of the compartment overlooking the workstation in the rear of the compartment in Tests 4, 5, and 6. In Tests 3, 5, and 6 Jet A fuel was poured over the workstations and surrounding carpet. To simulate this in the model, spray nozzles were positioned over the center of each workstation, 2 m above the floor. These nozzles are normally used to simulate water sprinklers, but in this case, the water was replaced by a liquid having similar properties to Jet A. The nozzles were activated for 2 s, in which time the equivalent amount of liquid as in the tests was ejected and spread over the furnishings.

In Test 5, Workstations 1 and 2 were disassembled prior to the burn and the contents were piled on top of the respective load cells. To model this scenario, the burning rate of the collective fuel packages was reduced by one half to account for the decrease in burning area of the fuel pile. The choice of one half was somewhat ad hoc. No free burns of workstation parts had been performed. This was the only test in which the simulated fuel packages had to be modified from their free-burn values. In this regard, Test 5 was used to calibrate, not validate, the model.

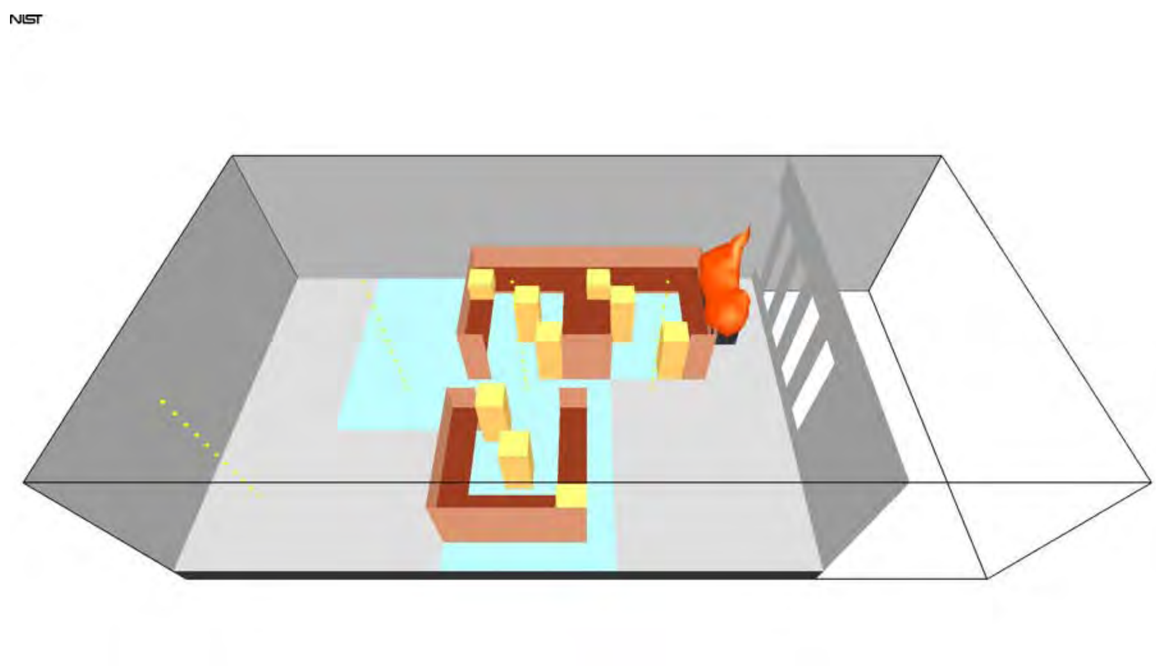

Figure 1–1. Geometry of the Phase 2 simulations.

From a modeling perspective, the objective of the simulations of the Phase 2 experiments was to demonstrate that a simplified model of an office workstation can be used to predict the burning behavior of a group of workstations in an enclosure with features similar to WTC 1, WTC 2, and WTC 7. Because of the magnitude of the simulations of the building fires, the model of the workstation had to be fairly crude. However, because of the many uncertainties in the initial conditions of the fire simulations, the lack of detail in the model is not considered to be a problem. The model fires had similar growth patterns, peak heat release rates, decay patterns, and compartment temperatures.

The model also captured the major features of the individual tests. For example, Tests 1 and 4 were similar in design except for the burner location. In Test 1 the burner was near the windows; in Test 4 it was near the rear of the compartment. The peak heat release rate was reached in about 15 min in Test 1, whereas it was reached in about 10 min in Test 4. The model shows a similar trend. The faster growth of Test 4 is probably due to the fact that the compartment heated up more quickly with the fire deep inside rather than near the windows, leading to more rapid spread of the fire across the pre-heated furnishings. Even though ceiling tiles were distributed over the desk and carpet in Test 4, this did not seem to have a noticeable effect on the growth, or at the very least the burner position seemed to have a far greater role in explaining the difference between Tests 1 and 4. The comparison of HRR between model and experiment is shown for Test 1 in Figs. 1–2. The upper layer temperature in the rear of the compartment for this same test is shown in Figs.1–3. The results for the other tests are comparable. The peak HRR and temperature are predicted well, as well as the duration of the fires. Both the peak values and the duration of the burning are important for the WTC simulations because it is not only important to predict the temperatures that the structural steel was exposed to, but also the duration of the exposure.

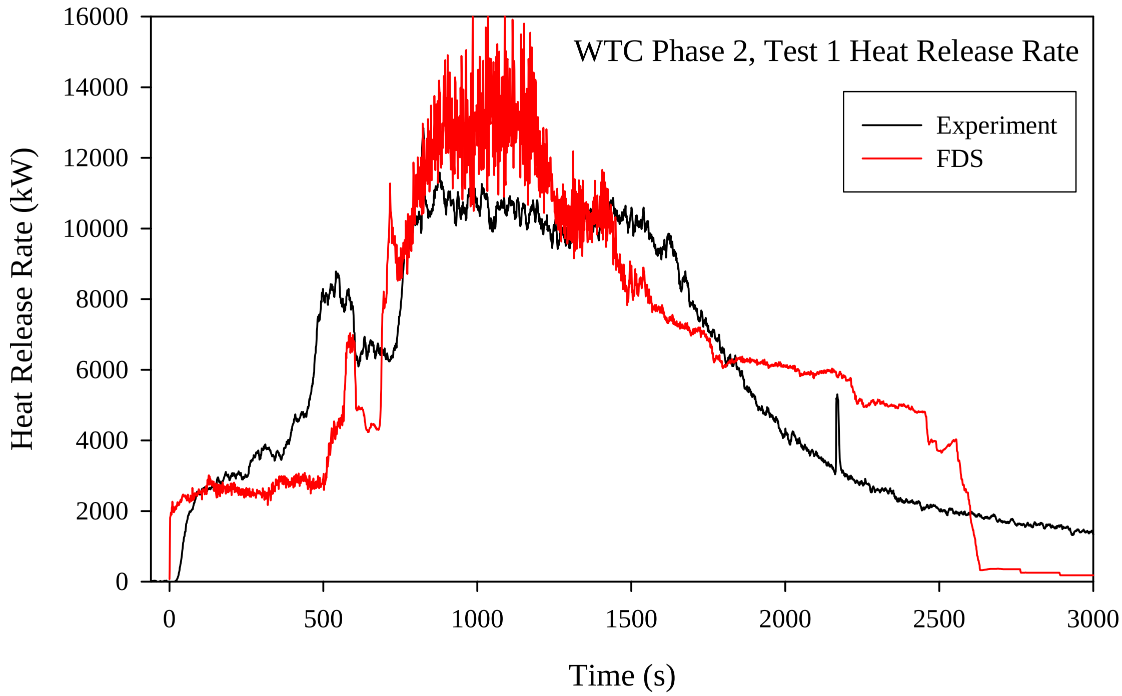

Figure 1–2. Comparison of HRR for Phase 2, Test 1.

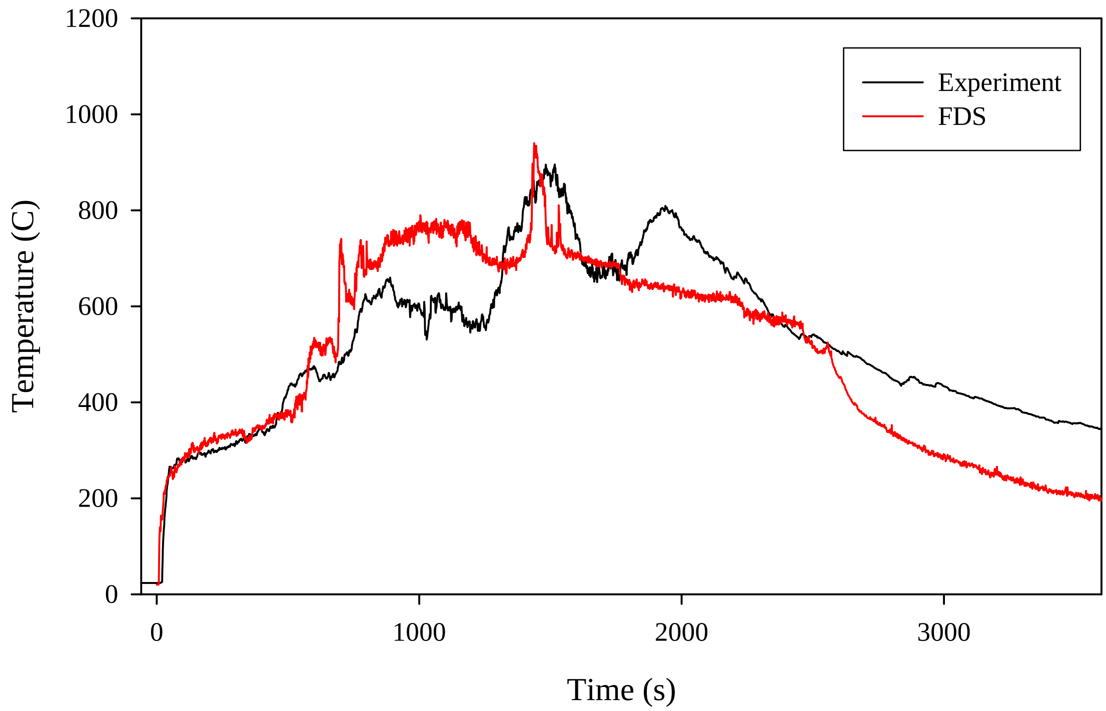

Figure 1–3. Upper layer gas temperature in the rear of the compartment, Phase 2, Test 1.

1.3 SIMULATIONS OF THE FIRES IN WTC 1 AND WTC 2

This section describes how the physical geometry of the buildings was described in the numerical model. Information about the layout of the relevant floors was obtained from architectural drawings provided by the occupants. For floors where information was not available, the geometry of a nearby floor or a floor of similar use was substituted. Information about exterior damage and window breakage was obtained by studying thousands of photographs and videos. There was no attempt made to predict the window breakage in the simulations. This information was provided as a boundary condition.

1.3.1 Numerical Grid

The windows in WTC 1 and WTC 2 were nominally spaced 1 m apart. In addition, the external columns plus their aluminum cladding were assumed to be 0.5 m wide. The slab-to-slab floor spacing was assumed to be 3.6 m. Because of these approximations, a uniform numerical mesh consisting of cells whose dimensions were 0.5 m × 0.5 m × 0.4 m was used. In the model, each tower face consisted of 58 windows, 61 columns, and two 0.5 m spacers next to each corner column. In the real tower geometry, these spacers formed the bevel. Figure 1–4 shows a single floor of the WTC 1 as it is approximated by the numerical model.

The numerical grid for each floor of WTC 1 and WTC 2 was of dimension 128 × 128 × 9 cells. The 128 cells in the horizontal directions allow for several meters of simulation outside of the external walls. The calculations were run in parallel, thus each floor was assigned to a different processor. The floor slabs, core walls, and workstations were approximated as thin obstructions. As described in the previous section, the contents of each workstation were collected into boxes and distributed throughout.

Penetrations in the floor slabs representing elevator shafts and HVAC ducts were created in the model by defining rectangular plates on top of the floor slab that were removed at the start of the calculation. This served to carve out holes in the floor. Window breakage was modeled by removing thin obstructions serving as windows at times obtained from the analysis of photographs and videos. Broken external columns were removed the same way.

1.3.2 Parallel Processing

Modeling the fires on multiple floors of WTC 1 and WTC 2 is computationally intensive, both in terms of CPU time and memory. Up to this point in its development, FDS has been limited to calculations small enough to run on a single CPU and fit into the memory of a desktop personal computer. The WTC study is an example of a large-scale fire modeling problem that is impossible to analyze without the use of parallel processing. In terms of parallelization, the exact details of FDS are not important. The approach taken to run the code on a cluster of machines can be applied to virtually any CFD code, in particular those that involve three spatial dimensions and time. In such cases, the computational demand is fairly well represented by the product of the number of computational grid cells and the number of time steps taken to advance the solution of the governing equations in time. For example, if the computational grid consists of 1 million cells and the simulation requires ten thousand time steps, the demand is 10^{10} cell-cycles. The overall demand can be broken down into memory requirement and CPU time. The memory requirement is a function of the number of grid cells; the CPU time is a function of the number of time steps.

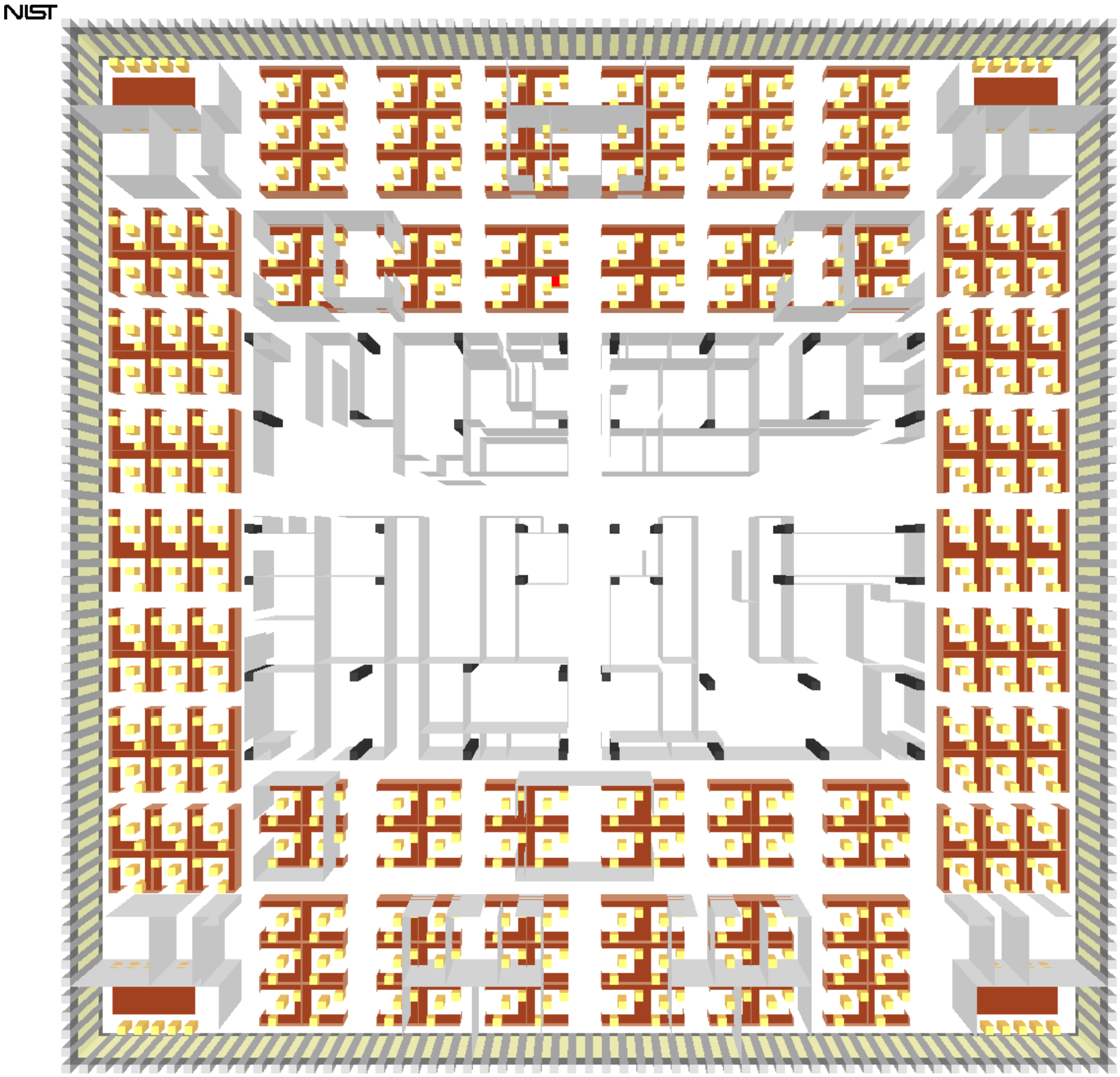

Figure 1–4. Plan view of a typical floor in WTC 1.

Roughly speaking, state of the art 32 bit processors can complete roughly 100,000 FDS cell-cycles per second. Realistic simulations of fires such as those in the WTC require on the order of 500,000,000,000 cell-cycles, or about two months of calculation on a single 2 GHz processor. Plus, the calculation would require 6 to 12 gigabytes of memory (RAM), well over the 4 gigabyte address space of 32 bit processors. Because of this, the WTC calculations are not only impractical on single processor systems, they are impossible on any 32 bit processor. A 64 bit processor system may theoretically handle the static memory requirements of a large simulation time, but the run times for large calculations remain prohibitive.

Because of the computational and memory issues of large fire simulations, a parallel version of the fire model must meet the two fundamental requirements discussed above, as well as satisfy a number of practical implementation concerns. Both the computational and the memory requirements must be distributed across multiple processors. The simulation must be done so that each processor uses less than

4 gigabytes of memory, while enough processors must be used to reduce the simulation to a practical length of time, of the order of one week.

Because the computational load is distributed throughout much of the source code, NIST has chosen to break up the calculation into multiple spatial blocks, with each block essentially doing the same type of calculation. A feature common to most CFD codes is multi-block or multi-mesh structure in which more than one structured grid is used in the calculation. This feature is exploited by simply putting the data and computation for each block on a different processor. This has advantages and some limitations. The advantages are (1) a natural and scalable extension of the existing code, (2) the amount of data communication will be kept to a minimum, since only overlap information needs to be communicated, rather than the data for full blocks, (3) source code changes are localized in small communication routines, (4) development is fairly fast. The disadvantages to the multi-block approach are (1) equal distribution of work across processors (load balancing) depends on spatial symmetry in the simulation, such as the translationally symmetric geometry of the WTC floors, (2) the level of parallelism and the speed up of the calculation is limited to the number of spatial blocks that can be used in the calculation. These limitations are not severe in many cases, including the WTC.

Because NIST is interested in a scalable, portable code, Message Passing Interface (MPI) is used. This is a standard, well-documented system of implementing parallel processing, that can work with shared memory, distributed memory, or combinations of those architectures (Gropp, 1999). Our goal in using MPI was to produce a code that, except for the requirement of the MPI library, would be as portable and standardized as the sequential version. The parallel code runs on most computer platforms, including networked Windows-based PCs. NIST opted for a cluster of commodity personal computers running Linux, connected by a gigabit ethernet network. The individual processors are in the range of 2.0 to 2.8 GHz, and dual processor machines were chosen to save space and to allow the addition of OpenMP code as a future extension to the MPI-based code. For production work in the NIST laboratory, two clusters are used: a smaller, development cluster to develop and debug the code, and a larger cluster with 128 processors. Using both clusters provides the capability to run six to eight large parallel processing jobs simultaneously.

1.3.3 Sample Simulation

Shown in Fig. 1–5 is a sequence of snapshots showing the predicted upper layer temperatures on a floor of WTC 1 at time increments of 15 min. The first image is a cut-away showing the damage to the north face of the tower and the layout of the walls and furnishings. The subsequent images are color coded by temperature, with the red (or dark) patches representing temperatures in the vicinity of 1,000 °C. Initially, these hot areas of active burning are near the impact zone at the north end (foreground of picture), but migrate towards the south as the combustible furnishings are exhausted. Driving the progress of the fires is the breaking of windows that provide air to the oxygen starved fire. The window breakage is not predicted by the model; it is an imposed boundary condition resulting from the analysis of thousands of photographs and videos recorded that day by eye witnesses. The uncertainty in the window break times is on the order of 5 min in areas not obscured by smoke.

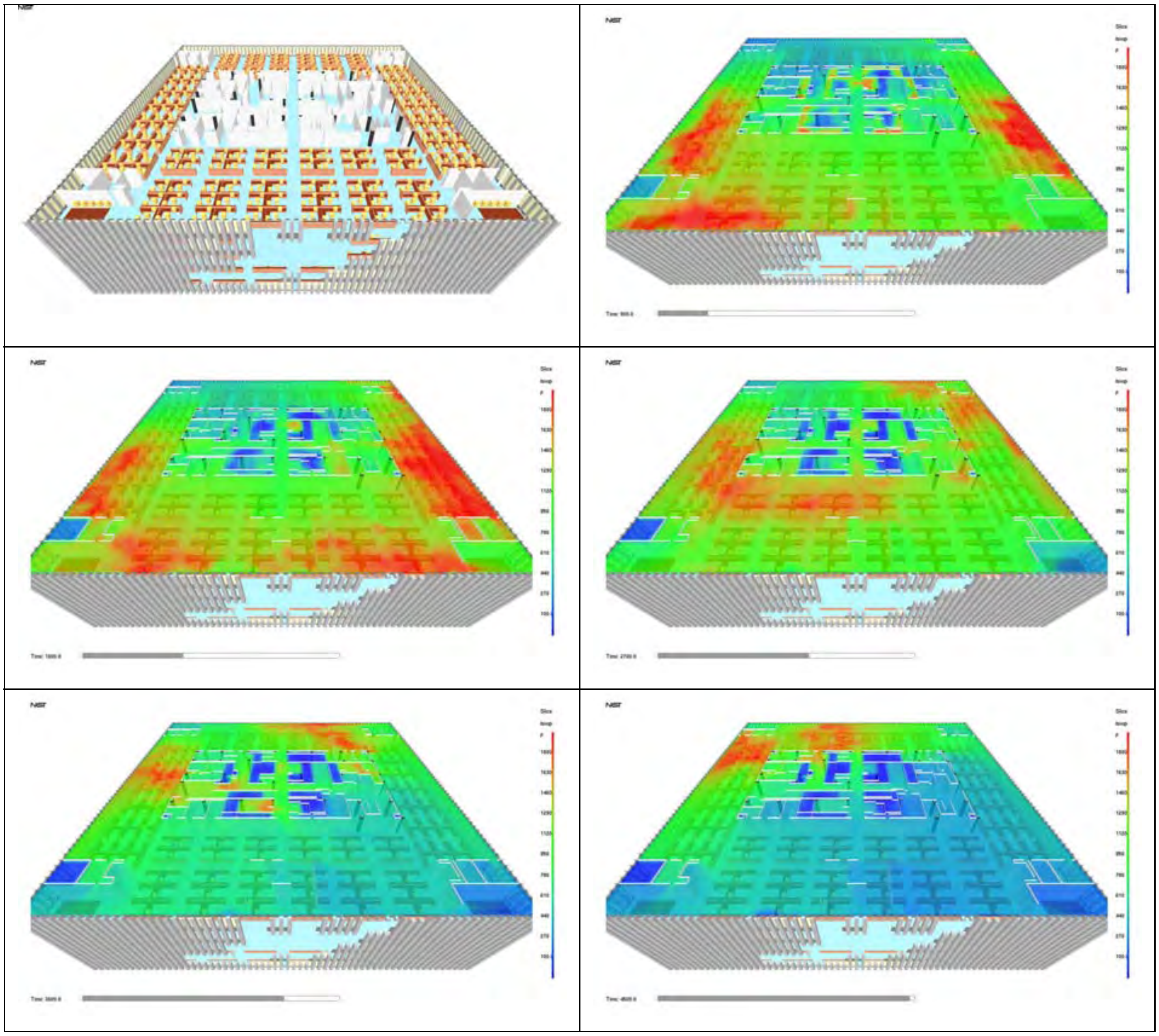

Figure 1–5. Predicted upper layer temperatures of a floor of WTC 1.

The burning behavior shown by the simulation is similar to that of the fires in experiments conducted by Ian Thomas and Ian Bennetts (1999). They looked at fire spread in long and wide enclosures with a single ventilation opening, where the fires were ignited at various points deep within the bench-scale compartments used. The fires would rapidly spread across the liquid or solid fuels covering the floor without consuming much of the fuel. The fires would then surround the compartment opening and burn back into the compartment as the fuel near the opening was exhausted. In the WTC simulations, fires are ignited over a wide area by simulated spray nozzles ejecting a liquid with properties of aircraft fuel. Much of the available oxygen is consumed rapidly, driving the fires to the openings created by the aircraft. The fires move away from the initial impact area as the nearby furnishings are exhausted, and as windows are broken out away in other parts of the building.

1.4 SUMMARY

The investigation into the cause of the collapse of WTC 1, WTC 2, and WTC 7 by NIST will not be completed until the fall of 2004. Work is on-going to simulate the weakening of the structural steel due to the aircraft impacts and the fires. Nevertheless, the fire experiments and simulations performed to date have improved our ability to analyze the response of any large building or structure to fire. In the years ahead, these techniques will become increasingly widespread due to faster computers and the ability to harness an entire set of off-the-shelf personal computers to perform very large calculations. Effective modeling is a combination of fast computers, efficient algorithms, and well-planned small and large scale experiments to provide both input to the model and a validation of results. Projects as complicated as the WTC study are rarely conducted using modeling alone. There is and will always be a need to coordinate computation and experiment to reconstruct the dynamics of large fires.

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Appendix K
Interim Report on Subsystem Structural Analysis of the WTC Towers

K.1 PURPOSE

Project 6 addresses the first primary objective of the technical investigation led by the National Institute of Standards and Technology (NIST) of the World Trade Center (WTC) disaster: to determine why and how the WTC towers (WTC 1 and WTC 2) collapsed following the initial impacts of the aircraft. Specifically, the objectives of this project are to determine the response of the structural components and systems to the fire environment in WTC 1 and WTC 2 and to identify probable structural collapse mechanisms. This appendix documents the progress achieved to date on Project 6 in thermal/structural modeling of WTC 1 and WTC 2.

Project 6 seeks to determine the response of structural components and systems to the fire environment in the WTC 1 and WTC 2 and to identify probable structural collapse mechanisms by (1) evaluating the response of floor and column systems under fire conditions, (2) evaluating the response of the WTC towers without and with aircraft impact damage under fire conditions, (3) conducting tests of structural components and systems under fire conditions, and (4) evaluating competing failure hypotheses for the WTC towers.

K.2 SCOPE OF WORK

The scope of the work consists of the following three tasks:

- **Task 1, Subsystem Structural Analysis.** The objective of Task 1 includes structural analysis of components and two subsystems, a full-floor subsystem, and an exterior wall subsystem. Task 1 is intended to provide guidance for the development of the global finite element models (FEMs) with respect to element types and sizes, appropriate constitutive models, and failure criteria for any given structural component. The subsystem analyses also will help to validate the accuracy of the global analyses, and correlate the results of the fine mesh component analyses with the coarser mesh global analyses of Task 2 and Task 3.

- **Task 2, Global Analysis of the WTC Towers' Response to Fire without Impact Damage.** The objectives of this task are to determine the general vulnerability of the towers to fire-initiated collapse and the role of fire in the towers with respect to structural stability, sequential failures of components and subsystems, and collapse initiation for the towers without impact damage.

- **Task 3, Global Analysis of the WTC Towers' Response to Fire with Impact Damage.** The objectives of this Task are to determine the relative roles of the impact damage and fires in the towers with respect to structural stability and sequential failures of the components and subsystems and to determine probable structural collapse initiation sequences.

Work under Task 1 includes the following:

- Develop and validate ANSYS models of the full floor and exterior wall subsystems.
- Evaluate structural responses for the following loading conditions.
  - Service loads due to gravity (dead and live loads).
  - Elevated structural temperatures.
- Identify the possible, likely and most likely failure modes and failure sequences, and the associated temperatures at failure and times-to-failure.
- Identify the changes in mechanical properties or geometry at initiation of component and subsystems collapse.
- Identify simplifications for the global structural models and/or analyses of subsystem models to use in Task 2 and Task 3.

The scope of this report is to present the progress made in Task 1 work.

K.3 DESCRIPTION OF SUBSYSTEM STRUCTURES

The full floor subsystem modeled is floor 96 of WTC 1. The model is believed to be typical in the upper floors of both towers. The exterior wall subsystem is a nine-column (three-panel) wide by nine-story (three-panel) high section of the WTC 1 between floor 91 and floor 100 and column 150 and column 158. This area is typical of the exterior walls of the towers and connects to a part of the floor system near the corner with different types of trusses.

K.3.1 Full Floor Subsystem Description

Floor 96 of WTC 1 was identified as an office floor with typical floor construction and loading and, therefore, was selected as the basis for this model. Components of the floor subsystem are examined for performance under loads and elevated temperatures in different possible failure modes. Understanding of these component behaviors is used to define the floor models for global analyses of WTC 1 and WTC 2.

The full floor subsystem of floor 96 of WTC 1 includes both office area and core area horizontal framing, as well as columns immediately above and below this floor.

The floor support in the office area consisted of pairs of steel floor trusses (nominally 60 ft in north-south and 36 ft in east-west directions) that span between exterior walls and the central core at 6 ft 8 in. on center. Each of these primary trusses consisted of top and bottom chords fabricated from steel angles and diagonals fabricated from round bars that extended 3 in. above the top chord at the panel points into the concrete slab in the form of a knuckle. The top chords of the primary trusses were supported at the central core by truss seats connected to a steel channel that ran continuously between the core columns. Each pair of trusses was connected to this channel with a seat that included two 1 3/4 in. long slotted holes and two 5/8 in. bolts (one bolt in each truss) as shown in Fig. K–25. Note that the floor truss was not welded to the seat support.

At the exterior wall, the truss pair was supported by a seat angle and fastened with two 5/8 in. diameter bolts in 2 in. long slotted holes. In addition, a gusset plate welded to the spandrel and to the truss top chord tied the supporting column to the truss, and a pair of straps welded to the top chord and to adjacent columns tied those columns into the primary trusses. Primary trusses were interconnected by a transverse bridging system consisting of bridging trusses and bridging angles. These bridging trusses were of similar construction to the primary trusses, although the knuckles for the diagonals did not project above the top chords. The top chord of the bridging trusses sat 1 1/2 in. below the top chord of the primary trusses and provided support for the 1 1/2 in., 22 gauge steel deck and the 4 in. thick lightweight concrete slab. At each corner of the building core, a 36 ft long transfer truss extended out from the corner core column to the exterior wall and supported the 60 ft long primary trusses. The core area floor consisted of a 5 in. thick normal-weight concrete slab on 1 1/2 in., 22 gauge steel deck, supported by wide flange girders and beams connected to the core columns.

Task 1 analyses use the nominal dimensions and design details shown on the drawings, without modifications resulting from any construction deviations or tenant modifications. Those modifications are considered in subsequent Task 3 analyses, which are based on the reference model developed by Leslie E. Robertson Associates (LERA) under a contract for Project 2. Material properties are based on information provided by Project 3.

K.3.2 Exterior Wall Subsystem Description

Each side of the towers' exterior wall consisted of fifty-nine 14 in. square box columns spaced at 3 ft 4 in. on center, with 52 in. deep spandrel plates at each floor level. The exterior wall was constructed from shop-welded prefabricated panels, each consisting, in general, of three columns and three spandrel beams, 13 ft 4 in. wide by 36 ft high. Except at mechanical floors, the base and top of the structure, vertical splices in prefabricated panels were staggered such that within any story, every third prefabricated panel had a vertical splice. Exterior column splices at the upper stories typically consisted of four 7/8 in. diameter ASTM A325 bolts fastened through the welded butt plates at the tops and bottoms of adjoining columns. Special prefabricated panels existed for the mechanical floors where no stagger existed at floors 7, 41, 75, and 108. At these mechanical floors, the column splice detail included supplemental field welding in addition to the bolted connection. Horizontal (spandrel-to-spandrel) connections between prefabricated panels were all field-bolted using splice plates. Corner panels that connected the orthogonal walls at corners were two-stories tall (24 ft) and consisted of two columns, two spandrel plates, and a third column midway between the two columns on alternate floors.

Various grades of steel, having yield strengths ranging between 42 ksi and 100 ksi, were specified to fabricate the perimeter column and spandrel plates. However, fewer grades were actually used with somewhat coarser gradation in yield strength than specified. Plate thicknesses also varied, both vertically and around the building perimeter. Plate thicknesses in the exterior wall were as thin as 1/4 in. at the upper stories, and increased toward the base of the building. The specified plate thicknesses and material yield strengths differed between the two towers, among NS and EW directions and through the height of the tower.

An exterior wall subsystem model, nine columns wide and nine floors high, was selected to study the structural behavior and failure modes of the exterior wall system. This subsystem model represents the exterior wall of WTC 1 between floors 91 and 100 and includes column lines 150 through 158. This area is located near the corner of the tower (column 159 is at the corner of the north face of WTC 1. See Fig. K–27.) The wall subsystem allows evaluations of the interaction of the wall subsystem with thermal expansion of the floor near the corners. It also connects to various types of trusses with different behaviors.

K.4 LOADS

The subsystems and components are analyzed for Dead (D), Live (L), and thermal (T_a) loads. The dead load consists of structural weights and superimposed dead loads. The superimposed dead loads for floors outside the core consist of the weights of ceiling, mechanical and electrical, fireproofing, and floor finish, estimated at 8 psf. The superimposed dead load and design live load are defined in the World Trade Center Design Criteria (LERA 2001). Twenty five (25) percent of the design live load is selected as a reasonable approximation of the load that likely existed at the time of the collapse. (For example, 25 percent of the design live load results in a load of 13.75 psf for the long-span trusses in the two way zone of floor 96 with 55 psf design live load.) The service dead and live loads are applied first, followed by the thermal loads.

The dead and live loads are defined as weights, so that during the collapse process, the gravity loads remain acting on the structure. The weight of debris from the plane will be considered where provided by Project 2.

The thermal loads, T_a, are temperature time histories for all structural members provided by Project 5 for the standard test fire ASTM E119 and between three and five representative building fire scenarios of different intensities and three fire protection conditions.

For analysis of some of the components, discrete values of temperature or temperature distributions in the form of a ramp from 0 °C to 700 °C (or to a temperature below 700 °C that results in the failure of the component) over 0.5 h followed by a constant temperature of 700 °C for another 0.5 h are used. Failure modes of the components are evaluated at room temperature and at different elevated temperatures, as failure modes and failure loads may change with increasing temperature.

Although wind may have had a minor role in the collapse of the towers, Task 1 analyses do not include wind load effects.

K.5 MATERIALS

The mechanical properties of both steel and concrete are affected significantly by temperature. In the following sections, the material properties used in this project are specified as a function of temperature. A material properties catalog is prepared and made accessible to all analysis models. For use in ANSYS, each material is identified with a number; steels are Material ID 1 through Material ID 29, and concretes are Material ID 51 through Material ID 83.

K.5.1 Concrete

Aggregate Types

Two types of concrete were generally used for the flooring inside the towers; lightweight concrete was used in the office areas, and normal-weight concrete in the core area. Thermal properties of normal-weight concrete depend on the type of aggregate. Petrographic inspection by SGH of several samples of lightweight concrete taken from the debris at NIST showed siliceous sand in the lightweight concrete. Because source of coarse and fine aggregates is usually the same, the available data for normal-weight concrete with siliceous aggregates are used.

Actual Compressive Strength

Specified concrete strength for lightweight concrete is 3,000 psi and for normal-weight concrete either 3,000 psi or 4,000 psi, as shown on Drawing Book 8, Sheet AB1–2.1 (SHCR 1973). The actual strength, f_a, of in-place concrete at room temperature is calculated from the specified strength, f_c', as follows:

$$f_a = f_c' \cdot F_1 \cdot F_2 \cdot F_3 \tag{1}$$

where the factor F_1 is the ratio of the average strength of cylinders to specified strength, F_2 is the ratio of in-situ 28-day strength to 28-day cylinder strength, and F_3 accounts for the change in concrete strength with age.

By using F_1 = 1.25 and F_2 = 0.95 (Bartlett and MacGregor 1996) and F_3 = 1.16 based on the formula specified in Section 2.2.1 of American Concrete Institute (ACI) 209 for change of concrete strength with age of concrete, the mean of the ratio of actual strength of in-place concrete to the specified concrete strength f_a / f_c' = 1.38. Based on this mean value, the actual strength of in-place concretes are f_a = 5,500 psi for the specified 4,000 psi normal-weight concrete, 4,100 psi for the specified 3,000 psi normal-weight concrete, and 4,100 psi for the specified 3,000 psi lightweight concrete.

Concrete Properties

The unit weight of the lightweight concrete is 100 pcf according to the WTC Design Criteria (LERA 2003); however, 110 pcf is used based on the two concrete samples described above. The unit weight of the normal-weight concrete is 150 pcf, according to LERA.

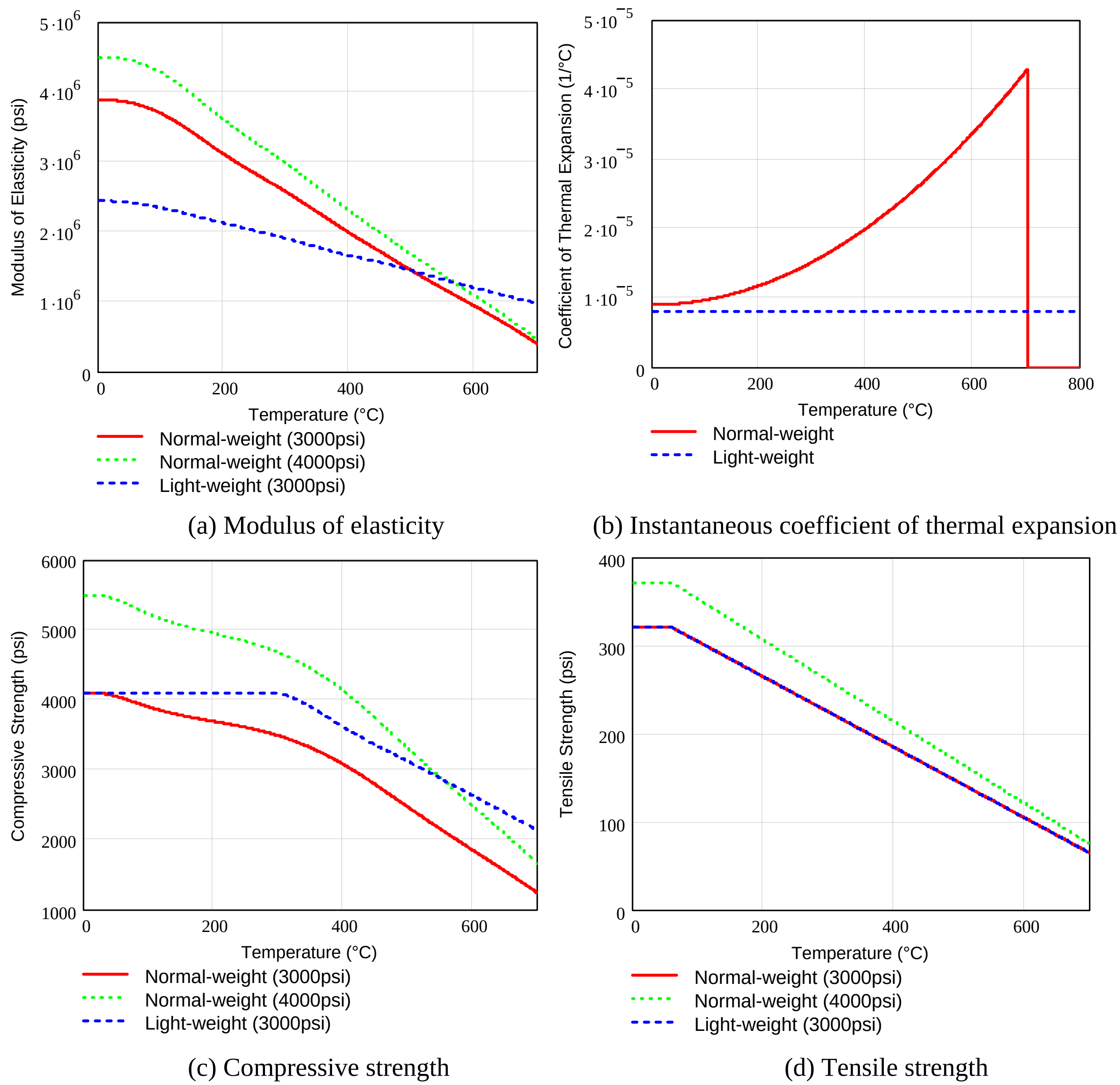
Poisson's ratio, ν_c, of 0.17 is used for both normal-weight and lightweight concrete at all temperatures.

Temperature dependent properties of concrete are modulus of elasticity, instantaneous coefficient of thermal expansion, compressive strength, and tensile strength:

Modulus of elasticity at room temperature is evaluated by the following formula:

$$E_c(RT) = 33\gamma_c^{1.5}\sqrt{f_a} \tag{2}$$

The actual strength, f_a, is used as room temperature compressive strength, and $5\sqrt{f_a}$ is used as room temperature tensile strength. Effects of elevated temperature on the listed properties are based on NIST research (Phan 1996, 2003), and plotted in Fig. K–1.



(a) Modulus of elasticity

(b) Instantaneous coefficient of thermal expansion

(c) Compressive strength

(d) Tensile strength

Figure K–1. Temperature–dependent concrete properties.

Concrete Stress-Strain Relationships

The compressive stress-strain curve, based on the formula by Seanz (1964), is given by:

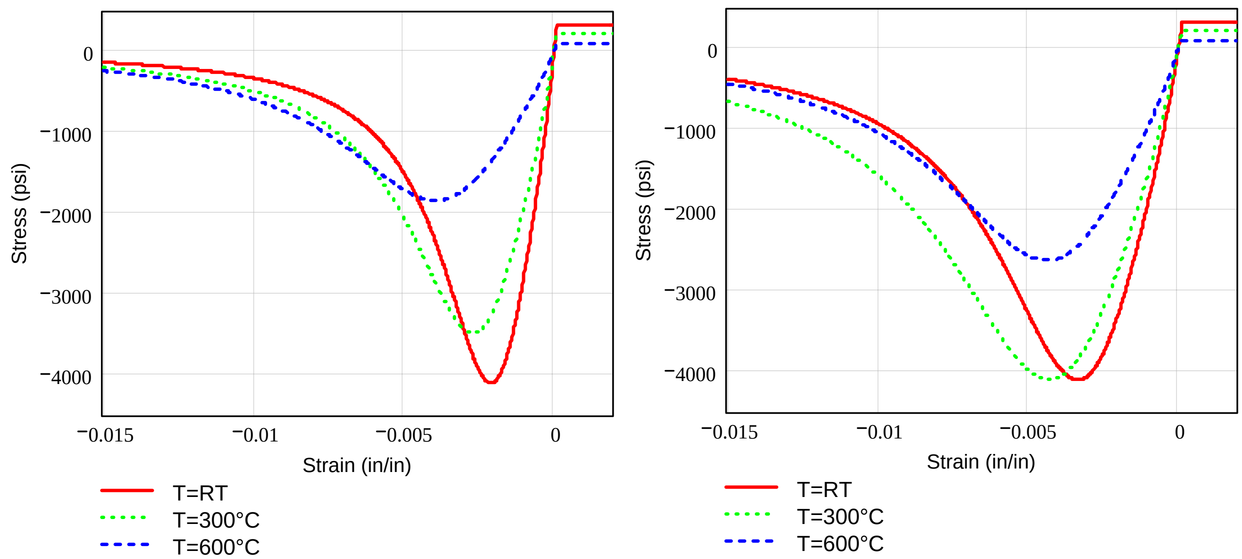
$$\sigma = \frac{K_c f_c \left(\frac{\varepsilon}{\varepsilon_{c1}}\right)}{1 + a\left(\frac{\varepsilon}{\varepsilon_{c1}}\right) + b\left(\frac{\varepsilon}{\varepsilon_{c1}}\right)^2 + c\left(\frac{\varepsilon}{\varepsilon_{c1}}\right)^3} \tag{3}$$

where:

$$c = \frac{K_s - 1}{(K_e - 1)^2} K_c - \frac{1}{K_e} \ , \quad b = 1 - 2c \ , \quad a = c + K_c - 2 \, ,$$

$$K_c = 2 \ , \quad K_s = \frac{1}{0.85} \ , \quad K_e = 1.41 \ , \quad \varepsilon_{c1} = K_c \frac{f_c}{E_c}$$

In tension, stress increases linearly up to the tensile strength. When strained in tension beyond its strength will soften and the stress will drop. However, the descending branch of stress-strain relationship causes significant numerical instability problems which can be avoided by assuming that concrete becomes plastic in tension. Figure K–2 shows a few examples of concrete stress-strain curves at room and elevated temperatures.



(a) Normal-weight concrete (3000psi) (b) Light-weight concrete

Figure K–2. Concrete stress-strain curves.

For the knuckle model in LS-DYNA, solid concrete elements are modeled with Pseudo Tensor material model, where the cap model is used. Since this material model is not temperature dependent, different material types are specified for the lightweight concrete at RT, 150 °C, 300 °C, 450 °C, 600 °C, and 750 °C (Material IDs 51 through 56) with their different stress-strain relationships.

The concrete slab in the truss is modeled with SHELL181 elements with a concrete material model that accounts for different behaviors in tension and compression. One such material model in ANSYS is the cast iron plasticity model which uses the Rankine maximum stress criterion in tension, and the expression for von Mises yield criterion in compression (ANSYS, Inc. 2004). Cast iron plasticity material models for specified 3,000 psi normal-weight concrete, specified 4,000 psi normal-weight concrete, and specified 3,000 psi lightweight concrete are assigned to Material ID 81, 82, and 83, respectively.

K.5.2 Steel

Steels used in WTC 1 and WTC 2 are listed in Table K–1 along with the yield and tensile strengths used in our analysis.

Steel Properties

Figure K–3 shows mechanical properties of steel at high temperatures: (a) modulus of elasticity; (b) Poisson's ratio; (c) yield strength reduction factor; (d) tensile strength reduction factor; and (e) instantaneous coefficient of thermal expansion. All properties, except yield and tensile strength reduction factors for bolt steels, are the same for all steels shown in Table K–1.

Stress–Strain Relationship

Plasticity: Stress-strain relationships at room temperature were provided by Project 3. They were constructed from mill report data, actual test data, and literature information using the Voce hardening law.

Stress–strain relationships at elevated temperatures, without consideration of creep, are obtained by the power law:

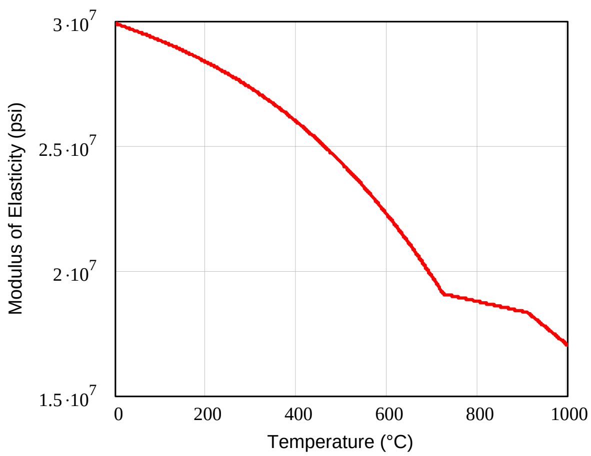
$$\sigma = R_{TS} R_C K(T) \varepsilon_{ep}^{n(T)} \tag{4}$$

where:

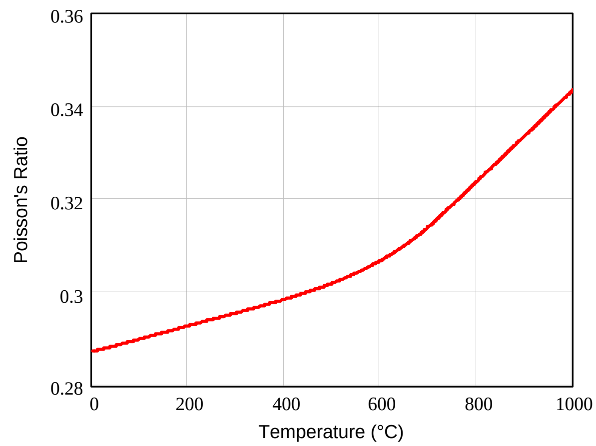
$$K(T) = (k4 - k0)\exp\left\{-0.5\left[\left(\frac{T}{tk1}\right)^{k1} + \left(\frac{T}{tk2}\right)^{k2}\right]\right\} + k0 \tag{5}$$

$$n(T) = (n4 - n0)\exp\left\{-0.5\left[\left(\frac{T}{tn1}\right)^{n1} + \left(\frac{T}{tn2}\right)^{n2}\right]\right\} + n0 \tag{6}$$

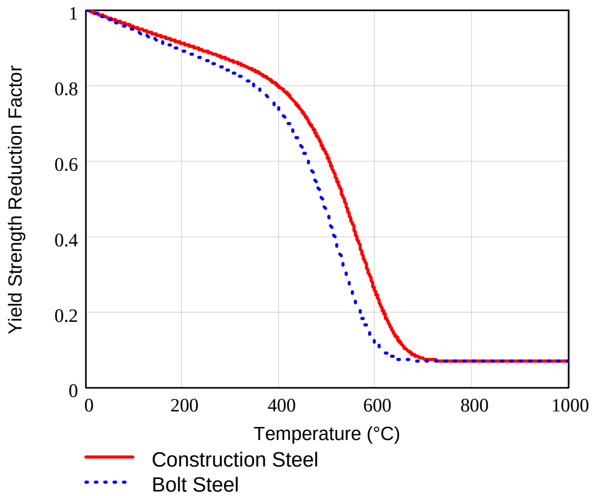
The steel stress-strain relationships at different temperatures vary depending on the type of steel used in the construction of the towers. Values for R_{TS}, R_C, given in Table K–1, and parameters of $K(T)$ and n(T) given in Table K–2, were provided by Project 3. The stress-strain curve is linear with Young's modulus up to the "linearity limit": At the linearity limit, the linear stress-strain curve intersects the power law stress strain curve. (Stress at the linearity limit is not necessarily equal to the yield stress. The linearity limit is required for ANSYS input.)



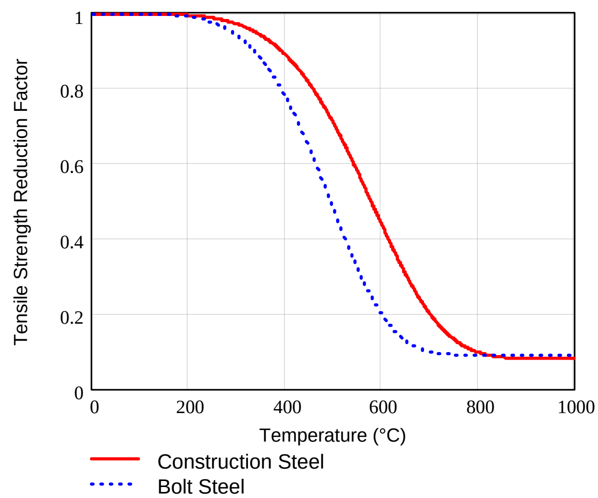
(a) Modulus of elasticity



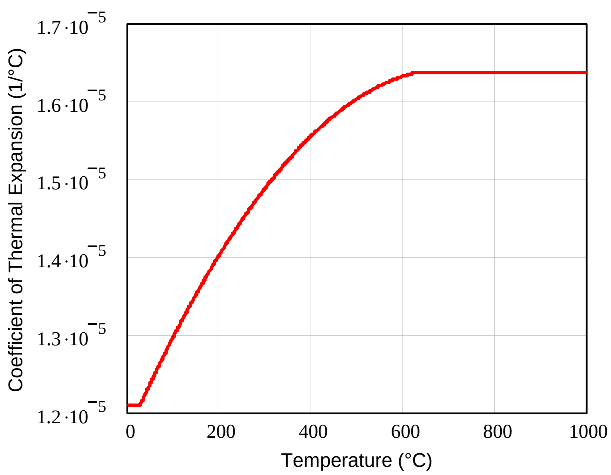
(b) Poisson's ratio



(c) Yield strength reduction factor



(d) Tensile strength reduction factor



(e) Instantaneous coefficient of thermal expansion

Figure K–3. Temperature-dependent properties for all steels.

Table K–1. Steel types used in WTC 1 and WTC 2.

Material ID	Description	σ_{yRT} (psi)	σ_{uRT} (psi)	RTS	RC
1	All 36 ksi core box columns, plates, straps[a]	36,720	64,470	1.086	0.857
2	All 36 ksi core WF, channels, and tubes 36 ksi large area and large inertia “rigid” beams in SAP2000 model[a]	37,000	63,450	1.069	0.954
3	All 42 ksi box columns (1<=0.75 in.)	51,400	79,200	1.070	0.884
4	All 42 ksi box columns (0.75 in. < t <= 1.5 in.)	47,000	74,800	1.010	0.884
5	All 42 ksi box columns (t > 1.5 in.)	42,600	70,400	0.951	0.880
6	42 ksi or 45 ksi Group 3 WF core columns	53,800	74,400	1.005	0.977
7	42 ksi or 45 ksi Group 3 WF core columns	49,000	71,040	0.960	0.954
8	42 ksi Group 4&5 WF core columns	44,200	66,640	0.900	0.948
9	45 ksi Group 4&5 WF core columns	47,800	71,074	0.960	0.939
10	All 36 ksi Plates 1, 2, and 4 in perimeter columns	35,630	61,170	1.031	0.875
11	All (42, 45, or 46) ksi Plates 1, 2, and 4 in. perimeter columns	53,051	74,864	1.011	0.948
12	All 50 ksi Plates 1, 2, and 4 in. perimeter columns. All 50 ksi channels and plates[a]	53,991	75,618	1.021	0.978
13	All 55 ksi Plates 1, 2, and 4 with t<=1.5 in. in perimeter columns	60,817	82,558	1.115	0.903
14	All 60 ksi Plates 1, 2, and 4 with t<=1.25 in. in perimeter columns	62,027	87,250	1.178	0.894
15	All 65 ksi Plates 1, 2, and 4 with t<=0.5 in. in perimeter columns[b]	69,642	90,442	1.221	0.979
16	All 70 ksi Plates 1, 2, and 4 in. perimeter columns	76,735	91,951	1.242	0.955
17	All 75 ksi Plates 1, 2, and 4 in perimeter columns	82,469	96,821	1.308	0.936
18	All 80 ksi perimeter columns steels, regardless of plate	91,517	99,442	1.343	0.987
19	All (85, 90, 100) ksi perimeter column steels, regardless of plate	104,783	115,983	1.566	0.976
20	Laclede truss web bar rounds specified as A36	38,067	59,567	1.004	0.935
21	Laclede truss chord angels (regardless of ASTM Spec) and all rounds specified as A242	55,332	74,050	1.000	0.959
22	A325 bolts[c]	104,783	115,983	1.566	0.976
23	All 42 ksi Plate 3 in perimeter columns	42,600	67,216	0.900	0.912
24	All 45 ksi Plate 3 in perimeter columns	45,900	69,831	0.940	0.921
25	All 50 ksi Plate 3 in perimeter columns	51,400	74,188	1.000	0.935
26	All 55 ksi Plate 3 in perimeter columns	56,900	78,546	1.070	0.906
27	All 60 ksi Plate 3 in perimeter columns	62,400	83,903	1.130	0.949
28	All 65 ksi Plate 3 in perimeter columns	67,900	87,261	1.190	0.975
29	All 70 ksi and 75 ksi Plate 3 in perimeter columns	78,900	95,976	1.310	0.997

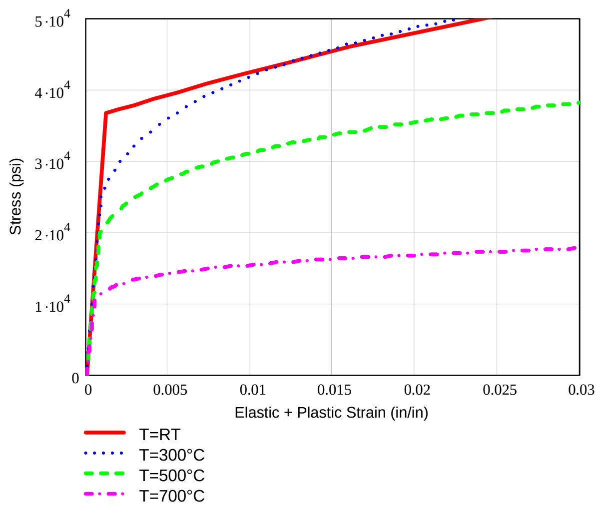
a. Steels in the following members are assumed to have the properties shown in the table:
36 ksi plates and straps (Material 1).
36 ksi channels, tubes, and “rigid” beams (Material 2).
50 ksi channels and plates (Material 12).
b. 65 ksi steels in perimeter columns with t>0.5 in. are assumed to have the same properties as those in Material 15.
c. In the column model, stress-strain relationships of bolts are used.
Note: Bolt properties are assumed to be the same as those in Material 19.

Table K–2. Parameters for *k(T) and n(T).*

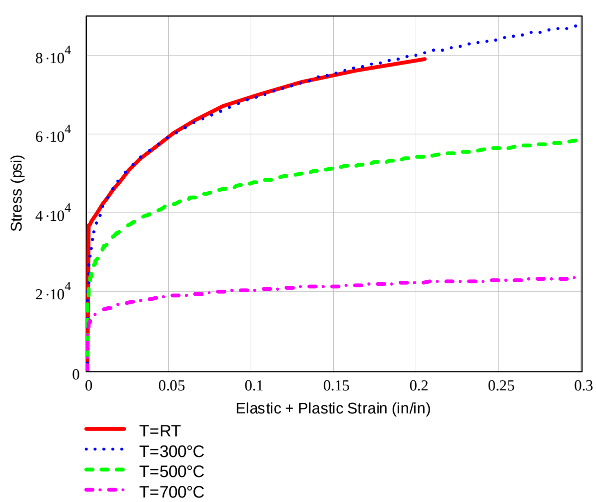
	σ_{yRT} = 36,000 psi	σ_{yRT} > 36,000 psi
tk1, °C	524.1812	511.8266
tk2, °C	523.6799	511.8938
k0, psi	29049.2	26472.1
k1	9.4346	6.5764
k2	9.3532	6.5971
k4, psi	121605.6	122516.7
tn1, °C	524.4304	519.634
tn2, °C	521.241	499.6031
n0, psi	0.1235	0.0342
n1	19.0000	10.0000
n2	19.0000	10.0000
n4, psi	0.2168	0.1511

Figure K–4 shows stress-strain curves of Material ID 1 (see Table K–1 for the material description) at room and elevated temperatures. Figure K–4 (a) is a close-up view of a low strain range, while Fig. K–4 (b) shows strain levels up to 0.3.

The elastic-plastic behavior of steels is modeled with ANSYS material model “Multi–linear isotropic hardening von Mises plasticity.”



(a) Strain < 0.03



(b) Strain < 0.30

Figure K–4. Stress-strain relationships for Material ID 1 steel.

Creep: Steel creeps at elevated temperatures ($T \geq 350^{\circ}C$), and the creep behavior for steels is based on the creep model by Fields and Fields (1991), expressed as:

$$\varepsilon_{cr}(t,T,\sigma) = \frac{1}{100} a(T) \left(\frac{t}{60} \right)^{b(T)} \left(35.5 \frac{\sigma}{\sigma_{yRT}} \right)^{c(T)} \tag{7}$$

where:

$$a(T) = \begin{cases} 0 & \text{for} \quad T < 350^{\circ}C \\ 10^{-(6.1+0.00573T)} & \text{for} \quad 350^{\circ}C \leq T < 500^{\circ}C \\ 10^{-(13.25-0.00851T)} & \text{for} \quad 500^{\circ}C \leq T < 725^{\circ}C \end{cases}$$

$$b(T) = -1.1 + 0.0035T \quad \text{for} \quad T < 725^{\circ}C$$

$$c(T) = 2.1 + 0.0064T \quad \text{for} \quad T < 725^{\circ}C$$

This model is valid for the temperature range of $350^{\circ}C \leq T \leq 725^{\circ}C$. ANSYS uses the "time hardening creep" model, where creep strain rate is given by:

$$\frac{d\varepsilon_{cr}}{dt} = C_1(T)\sigma^{C_2(T)} t^{C_3(T)} \tag{8}$$

and $C_1(T)$, $C_2(T)$, and $C_3(T)$ are temperature-dependent parameters determined by Fields' (1991) creep model given as:

$$C_1(T) = \frac{1}{100} a(T) b(T) \left(\frac{1}{60} \right)^{b(T)} \left(\frac{35.5}{\sigma_{yRT}} \right)^{c(T)}$$

$$C_2(T) = c(T)$$

$$C_3(T) = b(T) - 1$$

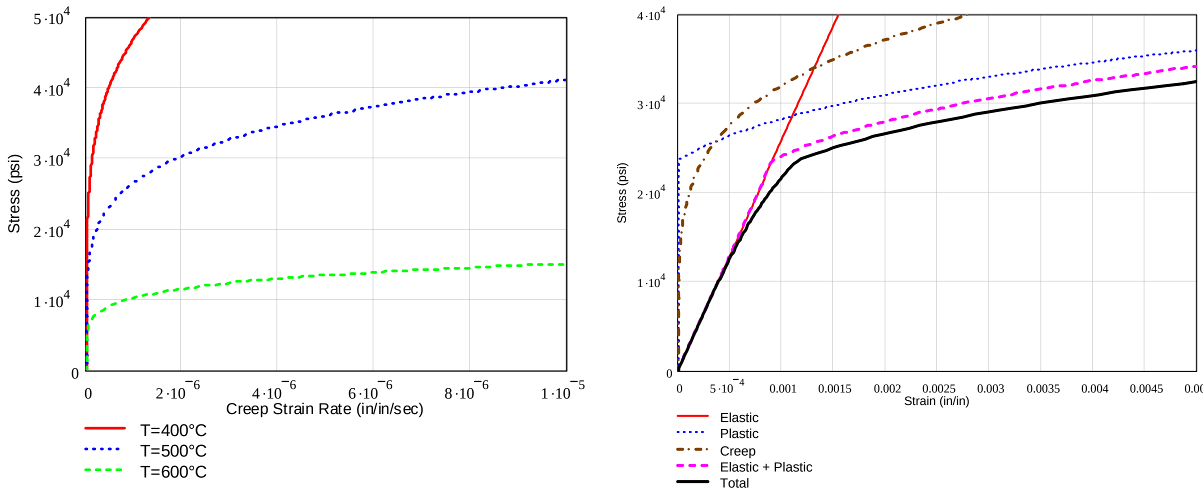
Figure K–5 illustrates creep behavior of steel at elevated temperatures for Material ID 1. Figure K–5 (a) shows creep strain rate at different stress levels and different temperatures, and Fig. K–5 (b) compares elastic, plastic, creep, elastic plus plastic, and total strains at $T = 400^{\circ}C$ and after constant loading for 1,800 s.

Failure Criteria

The failure criteria for steel are defined in terms of plastic strains. The multiaxial fracture strain criterion for different steels and temperatures (Fields 2004) is as follows:

$$\bar{\varepsilon}_f = \alpha(T) \exp\left[-\frac{3}{2} \frac{\sigma_m}{\bar{\sigma}} \right] \tag{9}$$

where stress and strain are true stress and true strain.



(a) Creep strain rate

(b) Comparison of different strains at T=400 °C and t=1,800 s

Figure K–5. Creep behavior at elevated temperatures for Material ID 1 steel.

For the uniaxial stress condition, the plastic strain at fracture reduces to:

$$\bar{\varepsilon}_{f\_uni} = \exp(-0.5)\alpha(T) \tag{10}$$

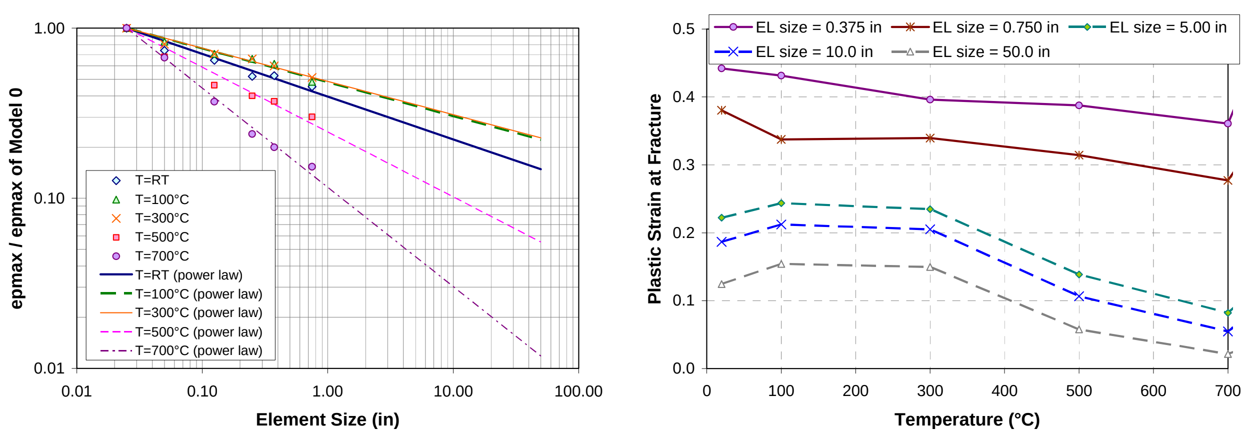
Table K–3 shows the uniaxial plastic strain at fracture, $\bar{\varepsilon}_{f\_uni}$, at different temperatures calculated by the equation above. This criterion is valid for the finite element analysis (FEA) with very fine mesh. For coarse mesh, the equivalent steel fracture criterion was determined numerically as follows. A standard tension test specimen was modeled in ANSYS. The gauge length, width, and thickness of the specimen were 8 in, 1.5 in, and 1 in., respectively, and Material ID 1 steel properties were used. Six different models (Model 0 to 5) were created, each having a different mesh size. Element sizes of Models 0 to 5 were 0.025 in., 0.050 in., 0.0125 in., 0.250 in., 0.375 in., and 0.75 in. It was assumed that Model 0 was able to capture tensile fracture in a uniaxial tension.

Model 0 was subjected to tension until the maximum plastic strain in the direction of applied displacement reached the uniaxial fracture strain determined by Eq. (10) for uniaxial stress condition, and the corresponding elongation of the specimen, Δ_0, was obtained. Models 1 to 5 were then subjected to the same elongation, Δ_0, and the maximum plastic strain in the direction of applied displacement was measured for each model. The maximum plastic strain due to the elongation of Δ_0 is defined as the limiting plastic strain (equivalent fracture plastic strain) for the corresponding element size.

From these six cases, a relationship between element size and equivalent uniaxial fracture plastic strain was established. This process was repeated for temperatures 20 °C, 100 °C, 300 °C, 500 °C, and 700 °C. Figure K–6 (a) shows the ratio of the maximum plastic strain in the direction of applied displacement due to displacement Δ_0 to uniaxial plastic strain by Eq. (10) vs. element size at different temperatures. The FEA results were extrapolated up to the element size of 50 in. Plastic strain shown in Fig. K–6 (b) is used as fracture criterion for the corresponding element size in the FEA.

Table K–3. Uniaxial plastic strain at fracture by Eq. (10).

Material ID	Plastic Strain at Fracture in the Uniaxial Test					
	20	100	300	500	700	1000
1	0.8411	0.6989	0.6610	1.0446	1.8100	3.5862
2	0.8411	0.6989	0.6610	1.0446	1.8100	3.5862
3	0.4908	0.4078	0.3857	0.6095	1.0561	2.0924
4	0.4908	0.4078	0.3857	0.6095	1.0561	2.0924
5	0.4908	0.4078	0.3857	0.6095	1.0561	2.0924
6	0.4908	0.4078	0.3857	0.6095	1.0561	2.0924
7	0.4908	0.4078	0.3857	0.6095	1.0561	2.0924
8	0.4908	0.4078	0.3857	0.6095	1.0561	2.0924
9	0.4908	0.4078	0.3857	0.6095	1.0561	2.0924
10	0.8891	0.7388	0.6987	1.1042	1.9142	3.7907
11	0.4908	0.4078	0.3857	0.6095	1.0561	2.0924
12	0.4908	0.4078	0.3857	0.6095	1.0561	2.0924
13	0.2846	0.2364	0.2236	0.3534	0.6123	1.2132
14	0.3774	0.3136	0.2965	0.4686	0.8120	1.6088
15	0.5338	0.4436	0.4195	0.6629	1.1486	2.2758
16	0.5623	0.4672	0.4418	0.6983	1.2099	2.3972
17	0.7752	0.6442	0.6092	0.9628	1.6681	3.3051
18	0.6545	0.5439	0.5143	0.8129	1.4084	2.7906
19	0.4254	0.3535	0.3343	0.5283	0.9154	1.8137
20	0.8411	0.6989	0.6610	1.0446	1.8100	3.5862
21	0.4908	0.4078	0.3857	0.6095	1.0561	2.0924



(a) FEA Results and Extrapolation (b) Recommended Limiting Plastic Strain

Figure K–6. Maximum plastic strain from the finite element analysis and limiting plastic strain.

K.5.3 Welds

The weld properties at all temperatures are assumed to be the same as those of the base metal of the same ultimate tensile strength. This assumption is validated by the following observations: the exterior column welds are strong enough to fail the base metal; the observed fractures in the exterior columns are mostly through the base metal; and the welds in trusses are resistance welds with no filler added. For the core columns, the area of the welds is significantly less than that of the base metal, and several fractures through the welds have been observed. Fractures in the truss seats and truss connections have also been observed. High temperature properties of the welding metals have not been found in the literature. Susceptibility of existing cracks in the welds to growth (fracture toughness) does not increase with temperature (Stevick 1994).

K.5.4 Bolts

A load-elongation relationship for 7/8 in. A325 bolt with 4 in. length at room temperature was provided by Project 3. Load-elongation relationships at elevated temperatures are constructed by scaling the loads by the yield and ultimate tensile strength reduction factors for bolt steels shown in Fig. K–3 (c) and (d). Figure K–7 shows the load-elongation relationships of a 7/8 in. bolt at different temperatures. Load-elongation relationships of A325 bolts of different size are scaled by proportioning the load by the ratio of the bolt thread area to the bolt body area for a 7/8 in. bolt.

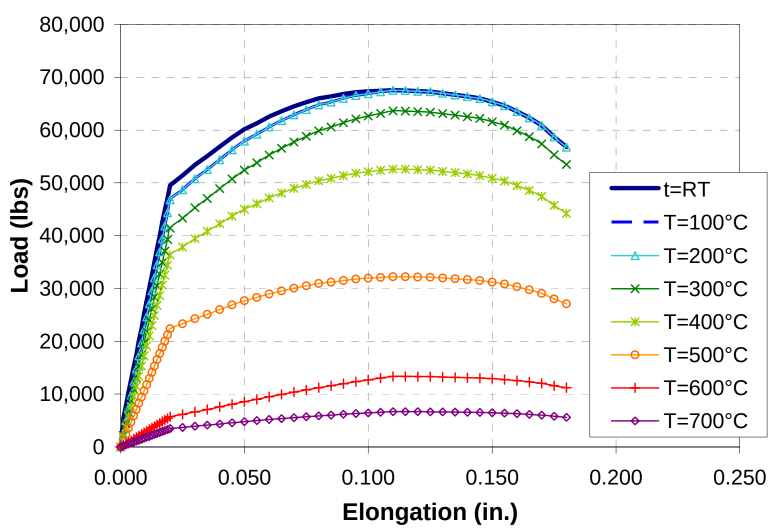


Figure K–7. 7/8 in. A325 bolt load-elongation curves at elevated temperatures.

The load-elongation relationship for bolts with a different length than 4.0 in. is expected to be very similar to the load-elongation relationship of 4.0 in. length as deformations are localized.

Based on the AISC formulas, C-J3–2 to C-J3–4, (AISC 2003), the shear strength for a single shear plane is calculated as 0.67 of the tensile strength given in Fig. K–7 when threads are excluded from the shear plane. When threads are not excluded from the shear plane, the nominal shear strength for a single shear plane is 0.53 of the tensile strength given in Fig. K–7. No shear ductility is assumed at failure.

K.5.5 Coefficient of Friction

The coefficient of friction of 0.33 for calculation of shear in friction-type connections is the AISC LRFD (2003) friction coefficient for uncoated clean mill scale steel surfaces, or surfaces with Class A coatings on blast-cleaned steel surfaces.

K.5.6 Symbols

$\alpha(T)$	=	temperature-dependent material property that defines fracture criterion
$\alpha_c(T)$	=	instantaneous coefficient of thermal expansion of concrete
$\alpha_s(T)$	=	instantaneous coefficient of thermal expansion of steel
$\beta_y(T)$	=	steel yield strength reduction factor due to elevated temperature
$\beta_u(T)$	=	steel ultimate strength reduction factor due to elevated temperature
γ_c	=	unit weight of concrete (110 pcf and 150 pcf for lightweight and normal-weight concrete, respectively)
γ_s	=	Unit weight (490 pcf = 0.284 pci for all steel types at any temperature)
ε_{c1}	=	concrete strain at maximum compressive stress
ε_{cr}	=	creep strain of steel
ε_e	=	elastic strain
ε_{ep}	=	elastic plus plastic strain
$\bar{\varepsilon}_f$	=	effective plastic strain at fracture
$\bar{\varepsilon}_{f\_uni}$	=	uniaxial plastic strain at fracture
ε_p	=	plastic strain
ε_{t1}	=	concrete strain at maximum tensile strength
ε_{tu}	=	concrete strain at full crack formation (separation) in tension
ν_c	=	Poisson's ratio of concrete
ν_s	=	Poisson's ratio of steel
$\bar{\sigma}$	=	effective stress
σ_m	=	mean stress

σ_{yRT} = room temperature yield strength of steel

σ_{uRT} = room temperature tensile strength of steel

$E_s(T)$ = modulus of elasticity of steel

$E_c(T)$ = modulus of elasticity of concrete

F_1 = mix design factor = ratio of the actual 28 day cylinder strength to f_c'

F_2 = in-situ factor = ratio of in-situ 28 day strength to the 28 day cylinder strength

F_3 = aging factor = ratio of mature concrete strength to 28 day concrete strength

f_a = actual strength of in-place concrete

f_c' = specified 28 day strength

$f_c(T)$ = compressive strength of concrete

$f_t(T)$ = tensile strength of concrete

$K(T)$ = sigmoidal function of temperature with six parameters

$n(T)$ = sigmoidal function of temperature with six parameters

R_C = correction factor that has the following two functions: (1) to correct the strain rate effect introduced in the material testing and create the stress-strain curve for zero strain rate, and (2) to match the room temperature stress-strain curve at strain of 0.05

R_{TS} = ratio of the room temperature tensile strength of the steel of interest to the room temperature tensile strength of the steel used to develop the power law model

K.6 MODEL CONVERSION FROM SAP TO ANSYS

The SAP2000 (SAP) floor 96 Model of WTC 1 and the SAP Global Model of WTC 1 are converted into ANSYS. The goal of the conversion is to develop ANSYS models that match the SAP baseline models developed by Project 2 and can be used as a basis of the detailed thermal-structural evaluation. The converted ANSYS models will be modified to incorporate the nonlinear behaviors of the components and simplified for the thermal/structural evaluation of collapse initiation study.

K.6.1 Translation Procedure

Automatic translation software was developed to partially convert the floor model and global model from SAP2000 to ANSYS 8.0:

- The Joints, Frames, and Shells in the SAP model were translated into ANSYS Keypoints, Lines, and Areas. Using geometry definition instead of nodes and elements directly allows for ease in local mesh refinement.

- Lines were meshed with both section and real constants so that a translation between BEAM44 and BEAM188 elements can be achieved by simply changing element types. Areas were meshed with SHELL63 elements in ANSYS to match the Shell elements in SAP. Eventually, Lines and Areas will be changed to nonlinear BEAM188 and SHELL181 elements with a type change.

- Material properties were assigned according to the Criteria Document based on the material definitions and Frame section properties in SAP.

- Frame section properties in SAP were converted into Real Constants for BEAM 44 in ANSYS. Cross section properties in SAP were retained for future conversion into cross section data for BEAM188 elements. Shell thicknesses in SAP were converted into Real Constants for SHELL63 in ANSYS.

- Joint restraints in SAP were translated into DOF constraints in ANSYS.

- Frame distributed loads and area uniform loads were translated into surface loads on Lines and Areas in ANSYS.

- The ANSYS BEAM44 elements support element moment releases, but the ANSYS nonlinear BEAM188 elements do not. Therefore, Frame releases in SAP were modeled by coincident nodes with coupled (CP) degrees of freedom in ANSYS.

- The ANSYS BEAM44 elements allow beam end offsets in three directions, but the ANSYS nonlinear BEAM188 elements only allow beam end offsets perpendicular to the element axis through section offset (SECOFFSET) command. Frame insertion points in SAP were converted in two ways. For offsets along the element axis, additional nodes and rigid MPC184 elements with the proper lengths were used in ANSYS. For offsets perpendicular to the element axis, beam end offsets were defined using Real Constants for BEAM 44, and eventually will be defined using SECOFFSET command for nonlinear BEAM 188.

- Frame offsets and rigid panel factor in SAP were modeled by adding additional nodes and rigid MPC184 elements with the proper lengths in ANSYS.

Those parts of the model that were not converted by the translation software were converted manually.

K.6.2 Challenges

During the conversion of the SAP Floor Model, the following conditions were encountered and were resolved:

- The SAP Floor Model allows automatic division of the frames at joints. This causes problems in the translation software because the frame connectivities in the Graphical User Interface do not show the actual internal element connectivities used in the SAP analysis engine. In order to resolve this problem, the translation software was modified to use the internal element connectivities. The table of internal connectivities was exported from the SAP model after the execution of the analysis.

- Automatic offsets in the SAP model are not available in the ASCII SAP input file prior to the execution of the analysis. The table of element offsets was exported after the execution of the analysis.

- There are both intentional and unintentional duplicate elements in the SAP Floor Model. Each leads to problems in the translator since ANSYS cannot have duplicate lines sharing the same key points. Some duplicate elements are used to model additional steel plates at the ends of the trusses. The duplicate elements were manually deleted and the section properties of the remaining elements were modified to account for the additional steel. Some duplicate elements are from frame elements which have different lengths and are overlapping each other. These were manually corrected.

K.6.3 Status

The automatic translation software developed to convert models from SAP2000 to ANSYS was applied to the floor model and will be applied to the global model shortly. Figures K–8 through K–11 show the converted floor model.

The following analyses were performed to validate the converted ANSYS floor model against the original SAP model.

- One static analysis with gravity loads as defined in SAP as Load Case “DEAD” which include self-weight plus 3.5 psf uniform load in the office area.
- One modal analysis, using structural mass only.

Table K–4 summarizes the comparison of the SAP and ANSYS results for the gravity load case. The total reactions for the SAP and ANSYS models are within 0.1 percent of each other. The maximum slab displacement predicted by the ANSYS model is 3.2 percent smaller than that obtained from the SAP model. This discrepancy is currently under study and is being resolved. The deformed shapes of the gravity load case for the SAP and ANSYS models are shown in Figs. K–12 and K–13.

Table K–4. Comparison of SAP and ANSYS results for gravity load case.

	SAP	ANSYS (BEAM 188)
Total reaction, kip	2,212.81	2,210.85 (-0.09 %)
Maximum slab displacement, in.	0.718	0.695 (-3.2 %)

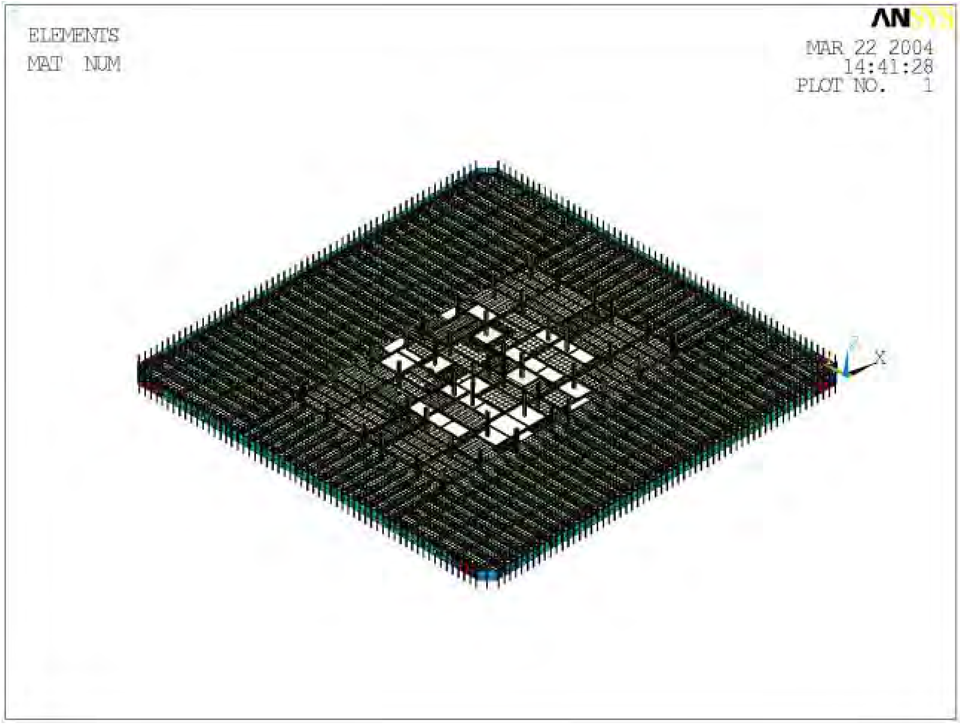


Figure K–8. Converted ANSYS model for floor 96 of WTC 1: overall view.

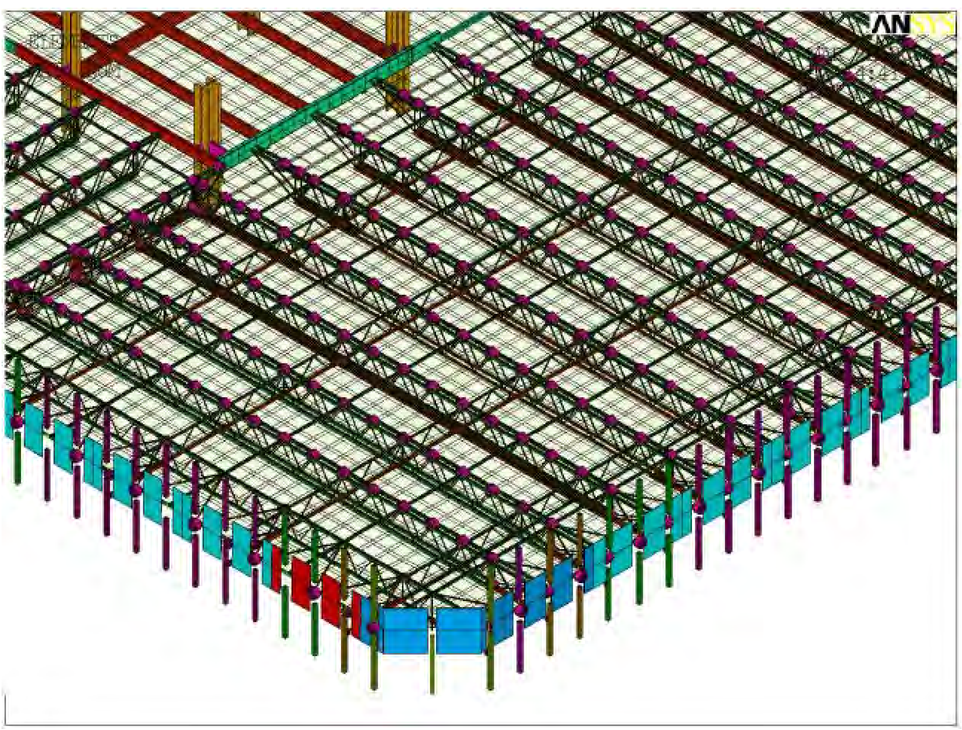


Figure K–9. Converted ANSYS model for floor 96 of WTC 1: partial view near corner of building.

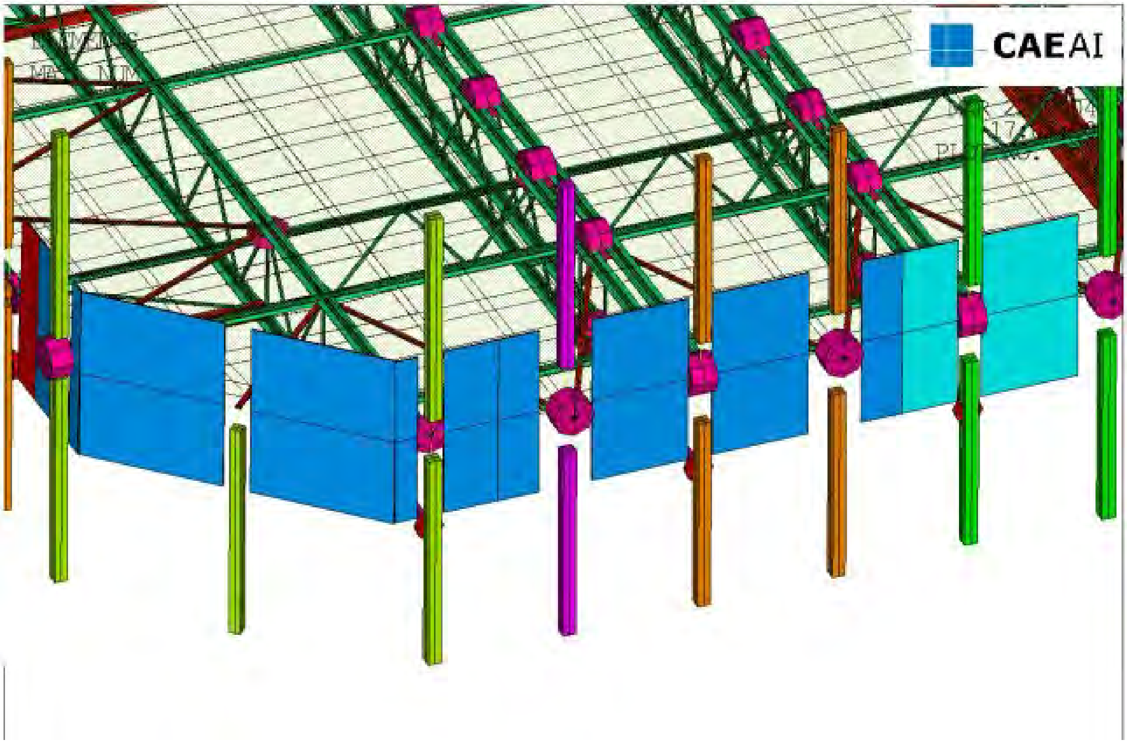


Figure K–10. Converted ANSYS model for floor 96 of WTC 1: close-up view at corner of building.

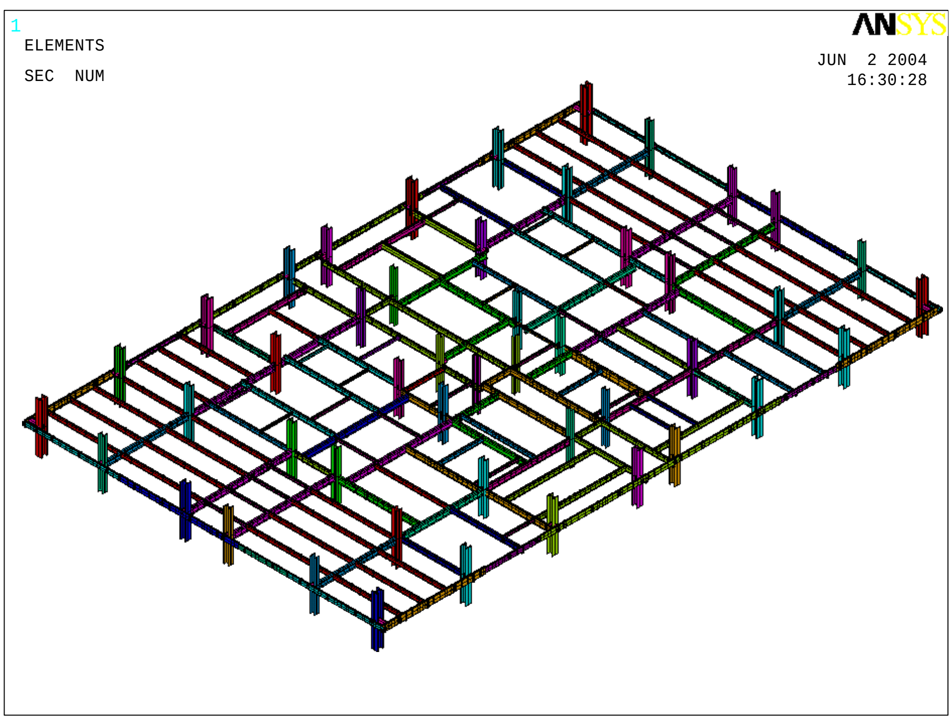


Figure K–11. Converted ANSYS model for floor 96 of WTC 1: view of floor beams and columns.

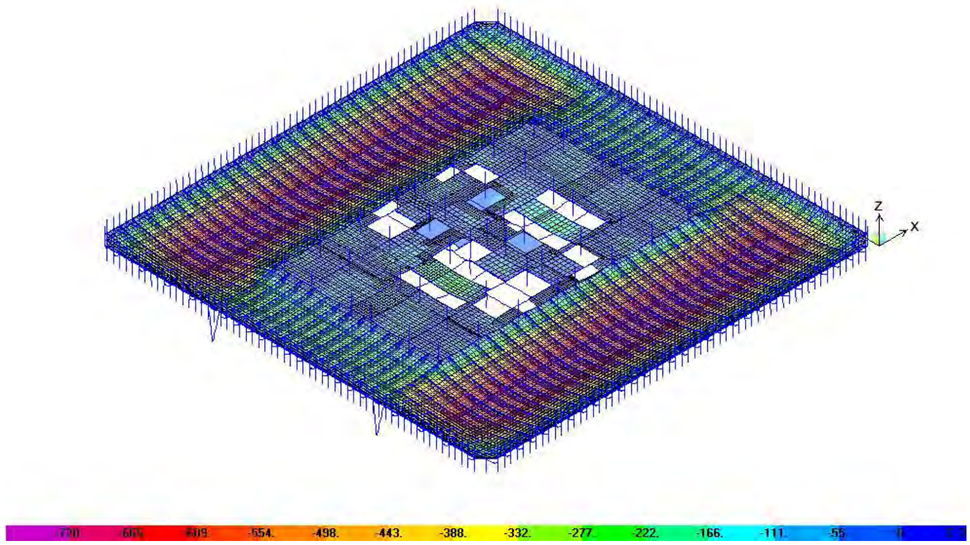


Figure K–12. Deformed shape (x100) of gravity load case for SAP floor model.

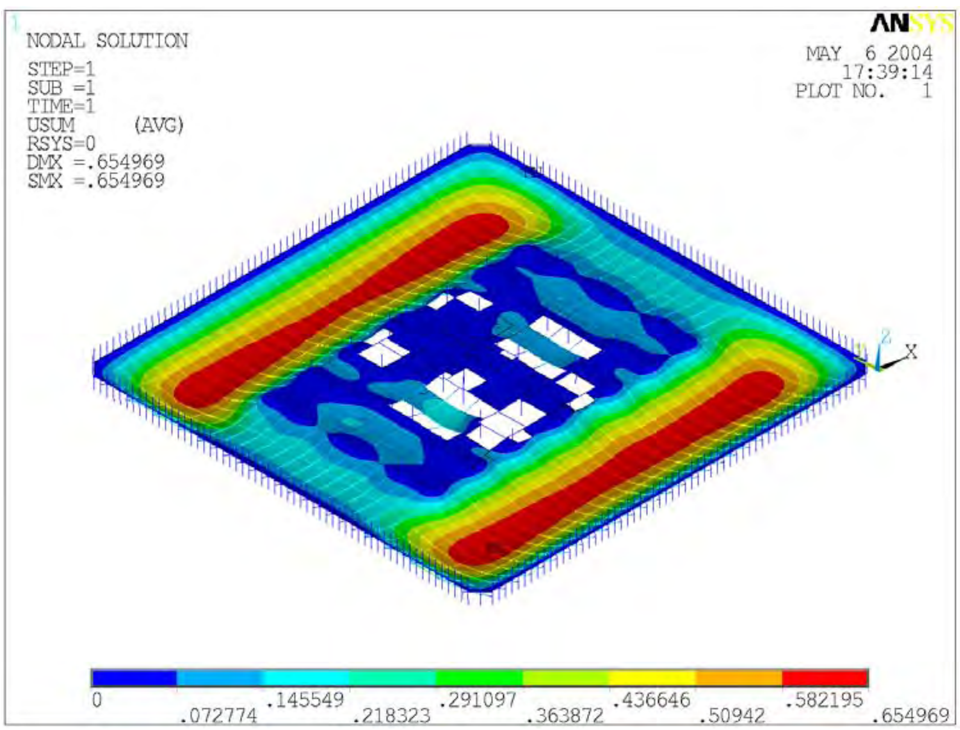


Figure K–13. Deformed shape (x100) of gravity load case for ANSYS floor model with BEAM44 (Euler beam) elements.

Table K–5 summarizes the comparison of the SAP and ANSYS results for the modal analysis. The total masses of the SAP and ANSYS models are within 0.02 percent of each other. The dominant natural frequency of the floor predicted by the ANSYS model is 2.5 percent higher than that obtained from the

SAP model. This discrepancy is consistent with the discrepancy observed for gravity displacement, and is currently under study and is being resolved. The dominant mode shapes of the floor for SAP and ANSYS models are shown in Figs. K–14 and K–15.

Table K–5. Comparison of SAP and ANSYS Modal Analysis Results.

	SAP	ANSYS (BEAM 188)
Total mass, lb·sec^2/in.	5448.7	5447.7 (-0.018 %)
Dominant natural frequency of floor, Hz	4.32	4.43 (+2.5 %)

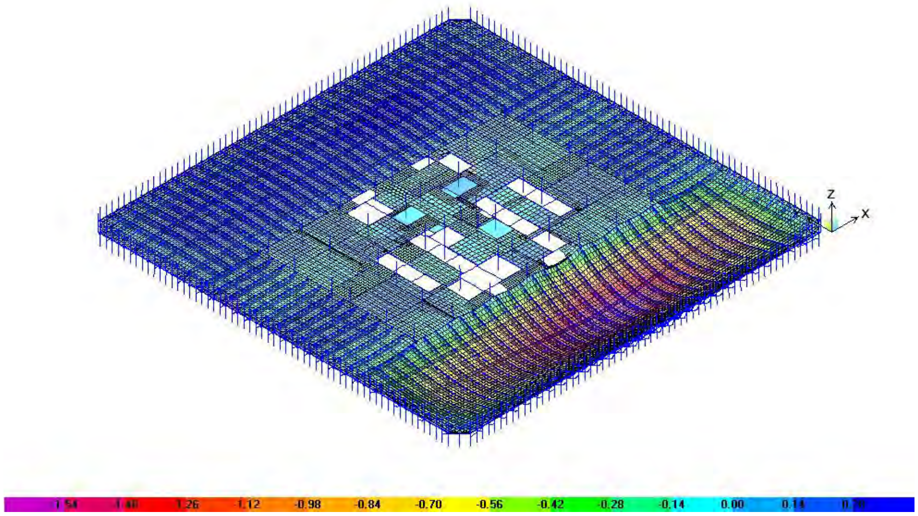


Figure K–14. Dominant mode shape (frequency = 4.32 Hz) of floor structure for SAP floor model.

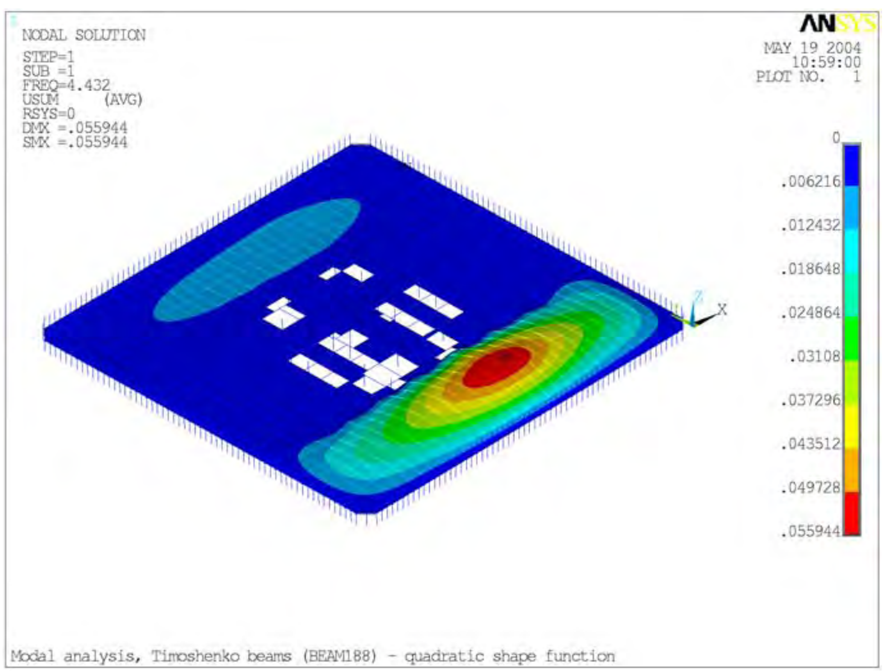


Figure K–15. Dominant mode shape (frequency = 4.43 Hz) of floor structure for ANSYS floor model.

K.7 FULL FLOOR SUBSYSTEMS

The full floor model is analyzed using the ANSYS general purpose finite element program Version 8.0. The objectives of the analysis are:

- To identify the most likely failure modes,
- To evaluate
  - Failure loads,
  - Temperatures at failure,
  - Time-to-failure, and
  - Changes in mechanical properties and geometry at failure.
- To simplify the model and to reduce the computational efforts for incorporation into the global model.

The failure modes and the failure loads of different components of the full floor subsystem are evaluated through analysis of detailed models of those components, using either hand calculations or FEAs. Simplified models that capture the failure loads and failure modes are then developed for each component. These simplified models of components are incorporated in the full floor subsystem model.

In this chapter, after a general description of the full floor model, the analyses of important components are presented and discussed.

K.7.1 Full Floor Model

Model Description

The floor model is developed using the converted SAP2000 model for floor 96, with the following modifications:

1. Combine two adjacent trusses into a single truss. The elements in the truss model have double the areas of elements in each real truss.

2. Change rigid beams at knuckle locations to user-defined elements with the properties of the knuckle determined by the component knuckle model.

3. For compression diagonals, add user-defined elements to account for buckling of diagonals.

4. For truss ends and connections, add user-defined elements to account for truss seat failure.

5. Pin concrete slab for out-of-plane rotation at both its interior and exterior edges.

6. Use user-defined elements along the edge nodes of the concrete slab to model the tensile strength of the concrete slab and the in-plane shear capacity at the connection to the spandrel plate.

7. Remove the spandrels defined as beam sections in SAP2000 model and replace them with SHELL181 elements in ANSYS. (This modification eliminates the need for defining panel zone stiffness.) The new spandrels will wrap continuously around the floor. Each spandrel plate between columns will be represented by 16 elements, 4 in. height and 4 in. width. Material and geometry assignments are carried through to ANSYS.

8. Change the elastic column elements as translated into ANSYS to user-defined sections with BEAM189 elements with plasticity and creep.

9. Make new column sections within the limits of the spandrels with reduced Plate 3 thickness, say 0.005 in. in thickness, to insure correct modeling of torsional stiffness. Spandrel thicknesses should be reduced within the limits of the column by the same thickness. Connect the centerline of column to spandrel with rigid elements.

Material Properties

ANSYS's multilinear isotropic hardening von Mises plasticity with time hardening for temperatures below 350 °C is used for the beam elements representing the truss system, girders, beams, and columns in areas where plasticity is either anticipated or expected to occur by analysis. This material model with creep is used for temperatures above 350 °C. This material model is used for shell elements representing the spandrel plates, when appropriate.

Loading

The full floor model is analyzed for dead and live loads first, and then thermal loads are applied to model the path dependent nonlinear response. The thermal loads are provided by Project 5 and include temperatures and temperature gradient time-histories for all structural members in the full floor model for (1) standard fire, (2) representative building fire scenarios, and (3) different fire protection scenarios.

Boundary Conditions

The beam elements representing the columns are restrained vertically at floor 95. The outward and tangential displacements and all rotations of the column ends at floors 95 and 97 are fixed to restrain thermal expansion. Mass elements defined by the tributary dead and live loads are added to the top of the columns and at connections to floor 96.

Failure Modes

The possible failure modes of the floor subsystem are as follows:

1. Sagging: Floor sagging along the axis of the main trusses may be caused by (1) loss of stiffness and softening of truss at high temperature, (2) catenary action of the truss due to plastic bending or buckling of critical members required for truss action, or (3) loss of composite action of floor-to-knuckle failure. These are discussed in some detail under truss failure modes. Floor sagging may result in component failure due to tension in the truss seats, tension in the floor subsystem, tension on the connections to the exterior walls, lateral

loads on columns, and increased demand on other components of the floor subsystem, for example, bridging trusses and transfer trusses and their connections.

2. Edge Sagging: Edge sagging results from failure of truss seat connections at either the interior or exterior supports and is evidenced in videos. Edge sagging, similar to sagging, increases demand on other components of the floor subsystem, reduces buckling strength of columns, and can lead to failure of a floor.

3. Loss of Support: Abrupt failure of the floor subsystem can result from loss of truss support for a large number of adjacent parallel trusses. Loss of a truss support can occur due to (1) vertical shear load due to debris and/or impact load of the dropping floor above, (2) vertical and horizontal shear loads resulting from slab expansion acting on column truss seats (3) tension acting on column truss seats, and (4) cooling of a truss shortened by plastic deformation and loss of composite action. Failure of truss support will increase the demand on the adjacent trusses and can result in sequence of truss seat failure, edge sagging, and ultimately failure of the floor subsystem.

4. Expansion of Floor System: Expansion of floor results in deformation of columns and forces at corners of the exterior wall subsystem. Such corner forces can initiate a failure sequence of columns near the corners. Such a failure includes development of horizontal shear in the gusset plates and the exterior column truss seats, large forces in the straps, and large lateral x and y forces in columns, especially near the corners.

K.7.2 Knuckle Analysis

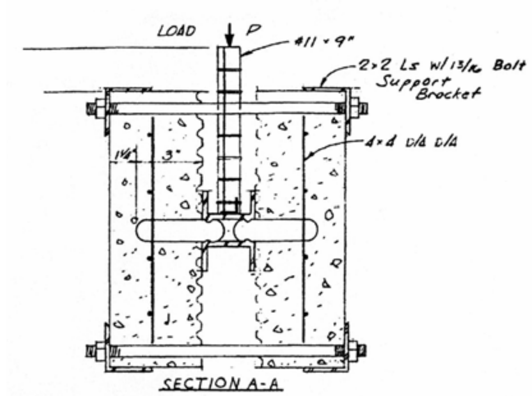
The “knuckle” is formed by the extension of the truss diagonals into the concrete slab and provides for composite action of the steel truss and concrete slab. The composite action is due to the shear transfer between the knuckle and the concrete slab both in the truss transverse and longitudinal directions.

The objective of this analysis is to predict the knuckle capacity when the truss and concrete deck act as a composite member and to develop a simplified model of the knuckle behavior to be included in the full floor subsystem model.

Knuckle Shear Tests

Two sets of experiments were performed in 1967 at Laclede Steel Company in Saint Louis, Missouri, to determine the transverse and longitudinal shear capacities of the knuckle.

The transverse shear test consists of double knuckles placed into two reinforced concrete blocks that were confined on the corners by angles as shown in Fig. K–16. The concrete density of 110 pcf corresponds to the lightweight concrete in the office areas. The concrete compressive strengths reported for 7 day and 27 day cylinder tests were 1,330 psi, and 2,600 psi, respectively. The inner ends of the two knuckles were connected through channels to a #11 rebar and the rebar was loaded until the concrete failed. The tests were conducted at concrete age of 6 and 27 days. The primary failure mode observed was concrete shear failure. The pictures from the tests show formation of the shear crack in one of the concrete blocks and edging of the channel into the concrete. The transverse shear capacity of the knuckle as the average of the

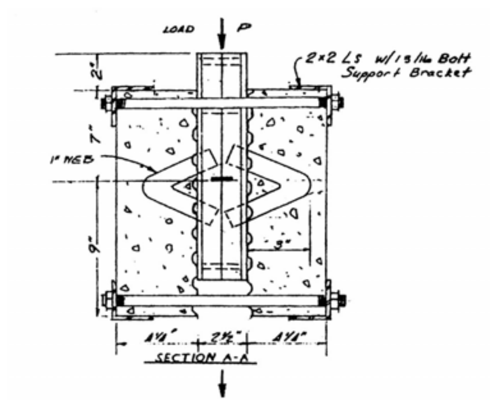


Drawing provided by Laclede Steel.

Figure K–16. Transverse shear test of a knuckle.

two reported tests is 16.9 kip per knuckle. After adjusting it for the strength of in-place, mature, lightweight concrete in the slab of 4,100 psi relative to the average strength of the lightweight concrete used in the test of 1,965 psi, by multiplying by the ratio of 4,100 to 1,965 psi, the transverse shear capacity of the knuckle is approximately 35 kip per knuckle.

The longitudinal shear test consists of double knuckles placed into two concrete blocks as shown in Fig. K–17. The test specification shows corner angles confining concrete blocks and no reinforcement for the concrete. However, the test pictures show reinforcement in both directions for each concrete block, with the corner angles dismantled. The test specification calls for concrete density of 152 pcf, which corresponds to a normal-weight concrete. The slab in office areas is of lightweight concrete. The average strength of two 28 day cylinders tested is 4,290 psi. A third sample, tested after 96 days, showed a strength of 2,850 psi. The test specification does not identify the weld size connecting the inner ends of the two knuckles to two channels. However, the primary failure mode observed for three tests is weld failure. Weld failure is not identified as the failure mode of the knuckle for the other two tests. The results of the shear tests of the knuckle in the longitudinal direction based only on these two tests are approximately 28.3 kip per knuckle. After adjusting for the strength of in-place, mature, lightweight concrete in the slab of 4,100 psi relative to the 28 day corrected average strength of the normal-weight concrete used in the test of 3,707 psi, by multiplying by the ratio of 4,100 psi to 3,707 psi, the longitudinal shear capacity of the knuckle is approximately 31 kip per knuckle.



Drawing provided by Laclede Steel.

Figure K–17. Longitudinal shear test of a knuckle.

The effect of temperature on the knuckle is as follows:

- The steel knuckle conducts the temperature of the diagonal without much loss into the cool concrete. Concrete has a low coefficient of conductivity and does not respond rapidly to the rise of temperature.

- Concrete in the intermediate proximity of the metal knuckle will heat to a temperature close to that of the steel.

- Shear failure of the knuckle is initiated by the failure of concrete in close proximity to the knuckle. Final failure will engage not only the hot concrete in close proximity of knuckle, but the cooler concrete farther away.

- It is reasonable to assume that for gas temperatures in the range of RT to 450 °C, 650 °C, 850 ^{o}C, and 1050 ^{o}C , the knuckle metal temperature is below 375 ^{o}C, 550 ^{o}C, 725 ^{o}C, and 900 ^{o}C, and the average concrete temperature is below 300 ^{o}C, 450 ^{o}C, 600 ^{o}C, and 750 ^{o}C, respectively.

Neglecting the difference in thermal expansion of concrete and steel, for gas temperatures of RT, 450 ^{o}C, 650 ^{o}C, 850 ^{o}C , and 1050 ^{o}C, the expected concrete strength is in the range of 4,100 psi, 3,300 psi, 2,600 psi, and 2,000 psi, and the knuckle capacity in either direction is 30 kip, 24 kip, 19 kip, and 15 kip, respectively.

Knuckle Test Finite Element Model

Finite element models, shown in Fig. K–18, represent one quarter of the knuckle test specimens. The knuckle and channel members in the test set up are modeled by solid steel elements. Concrete Pseudo Tensor model and the LS-DYNA computer program were used for the analysis. An imposed ramped displacement was applied to the angle member.

The concrete strength used in the finite element model for the longitudinal shear of the knuckle was 4,100 psi and for the transverse shear of the knuckle was 2,500 psi. In addition 0.47 percent steel reinforcement representing welded wire fabric reinforcement of the slab was added in a distributed way to the concrete. Also, two different assumptions were made about the interface condition between the concrete and the steel: fully bonded and frictionless.

The results of the analysis are shown in Figs. K–19 through K–22. They show significant dependence on the characteristic of the interface between the steel and concrete. The longitudinal shear test FEA results, shown in Fig. K–21, show that each knuckle has strength in the range of 15 kip to 35 kip, depending on interface. The test results show that the interface is closer to fully bonded case. For the transverse shear, the FEM results, Fig. K–22, show that transverse knuckle strength is about 24 kip for 2,500 psi concrete, corresponding to 39 kip for 4,100 psi concrete. Figure K–20 shows that for transverse shear concrete crushes in a small region next to the knuckle and extending in front of the shear load. Figure K–20 also shows large regions of crushing at the lower boundary of the model. These regions are the result of imposing the boundary condition UY=0. This boundary condition, and the crushing at the boundary, although realistic for the test, would not obtain in a pair of transversely loaded knuckles of the double

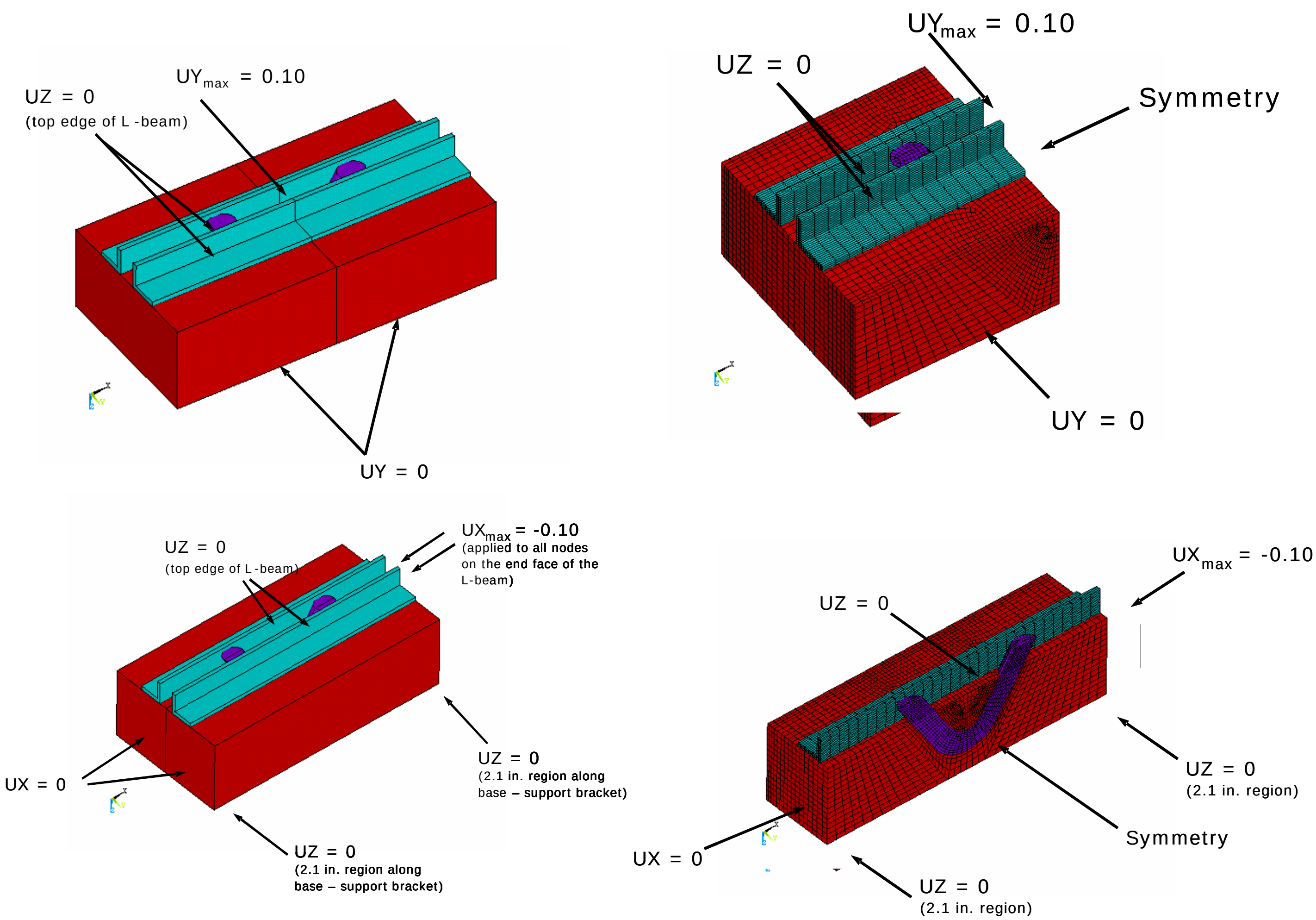


Figure K–18. Finite element models of knuckle shear tests.

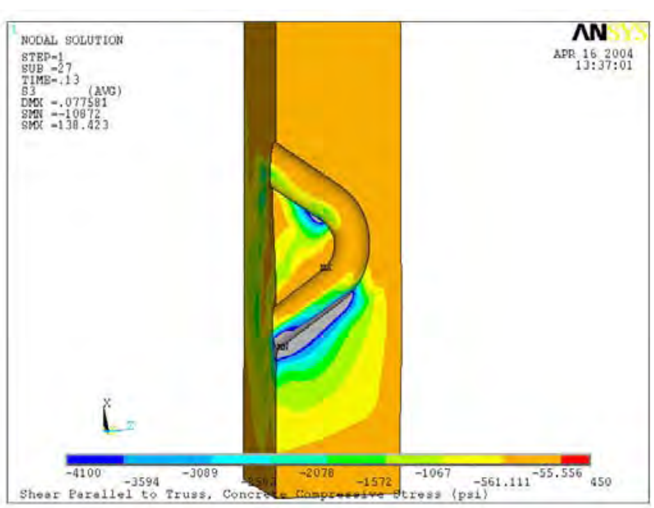


Figure K–19. Compressive stresses in longitudinal shear finite element model.

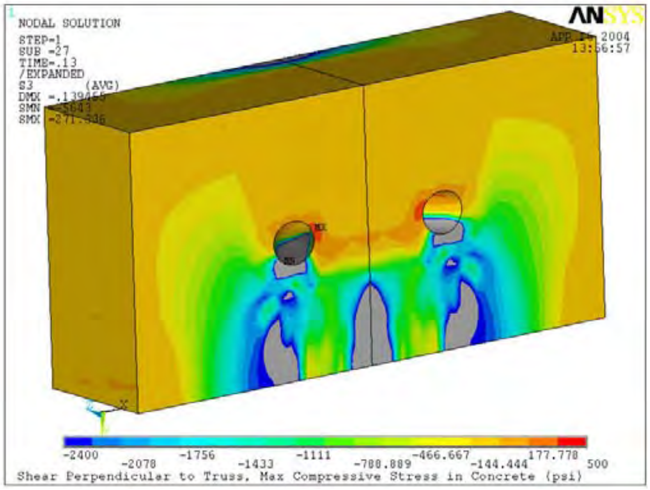


Figure K–20. Compressive stresses in transverse shear finite element model.

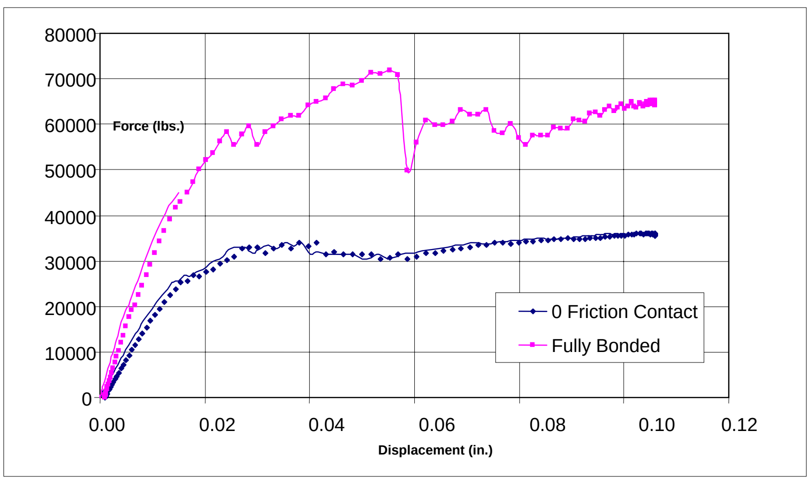


Figure K–21. Shear force versus displacement from finite element model for longitudinal shear of two knuckles.

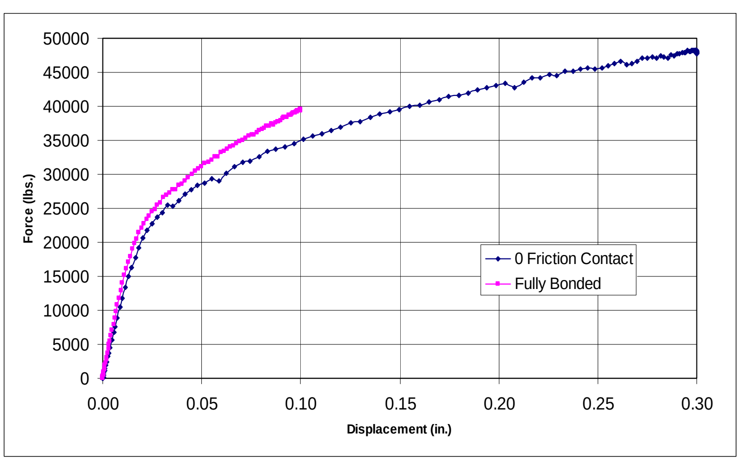


Figure K–22. Shear force versus displacement from finite element model for transverse shear of two knuckles.

truss. The small crushing regions at the knuckle indicate that a pair of knuckles in a double truss can be expected to behave nearly independently of each other, and, therefore, have nearly double the capacity of a single knuckle. Unfortunately, test results are not available that would confirm this conclusion.

Although the analysis shows the sensitivity of the results to the interface assumptions, it justifies the shear capacities computed from the test results.

Knuckle Model

The purpose of the detailed finite element analysis of the knuckle is to provide a basis for deriving a simple model for use in analyses of the full floor. The knuckle model includes segments of concrete floor and truss diagonal that protrudes into the 4 in. thick concrete. The dimensions of the concrete included in the model are one half of the double truss spacing of 40 in. The diameter of the truss diagonal included in the model is 1.09 in., and the center line of the knuckle is 3 9/16 in. from the center line of the double truss. The concrete slab wire mesh reinforcement is modeled by distributed reinforcement properties.

The model is bounded by four planes. Two of these planes are parallel to the chord of the truss, and the other two planes are perpendicular to the chord. Symmetry conditions are applied to these planes consistent with the loads. For the tension loading, in addition to the symmetry conditions, the model is supported vertically at both symmetry planes that are perpendicular to the truss chord. For the shear load parallel to the chord, the model is supported ahead of the shear load in the direction parallel to the chord.

The knuckle has the properties of ASTM A36 (Material ID 20) round bar steels and the concrete has lightweight concrete properties specified for LS-DYNA with concrete-cap model.

K.7.3 Column Truss Seats

In this section, likely failure modes of truss seats are identified, and the corresponding failure loads are determined. The following loading conditions were considered: vertical force, horizontal tensile force, horizontal compressive force, and combined vertical and horizontal force.

Description of Column Truss Seats

The floor truss is supported at the exterior wall and at the core by seats. The truss seat at the exterior wall and at the core will be referred to as *exterior seat* and *interior seat*, respectively.

The interior seat consists of a horizontal plate with two vertical plate stiffeners as shown in Fig. K–23. These plates are fillet welded together and fillet welded to the core channel beam. Two 5/8 A325 bolts (one bolt in each truss) connect the truss to the seat. The bolt connection is a friction type connection with 1 3/4 in. long slotted holes in the seat and 7/8 in. oversize holes in the bearing angles.

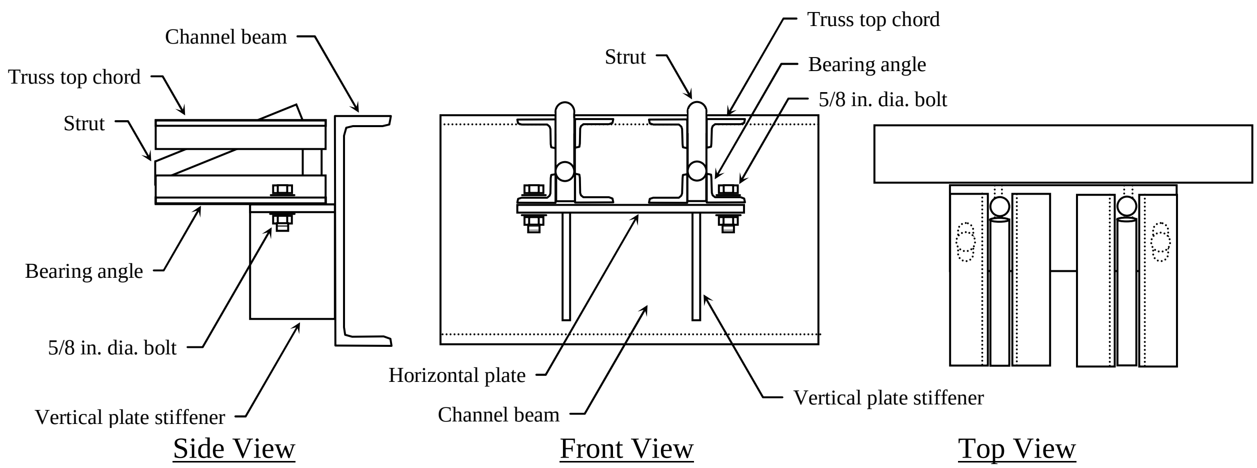


Figure K–23. Interior seat.

The exterior seat consists of a seat angle attached to the spandrel with two vertical plates (stand-off plates), and a gusset plate as shown in Fig. K–24. Fillet welds connect the seat angle to the stand-off, the stand-off to the column/spandrel, and the gusset plate to truss top chord. A complete-joint-penetration groove weld connects the gusset plate to the column/spandrel. Similar to the interior seat, each pair of trusses is attached to the exterior seat by two 5/8 in. A325 bolts. The bolt connection is a friction type connection with 2 in. long slotted holes in the seat angle and 7/8 in. oversize holes in the truss-bearing angle.

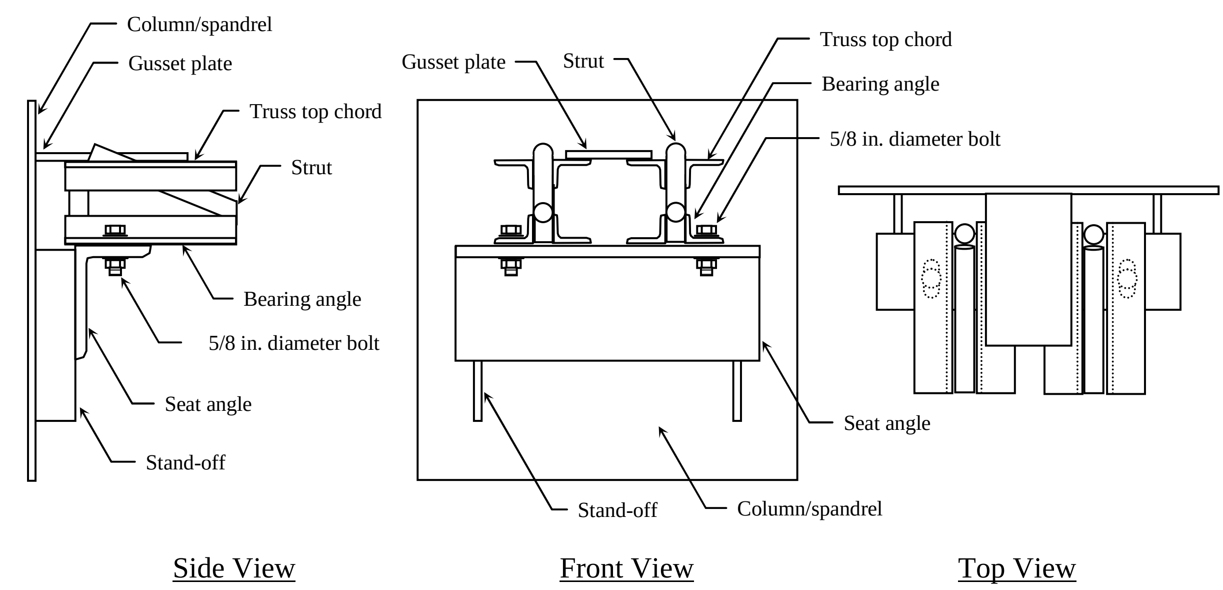


Figure K–24. Exterior seat.

In floor 96 of WTC 1, there are seven types of interior seats and eight types of exterior seats. The different types of interior seats are identified with Detail Numbers 15, 17, 20, 21, 22, 23, and 226A; and the exterior seats with Detail Numbers 1013, 1111, 1212, 1311, 1313, 1411, 1511, and 1611, as shown in Fig. K–25.

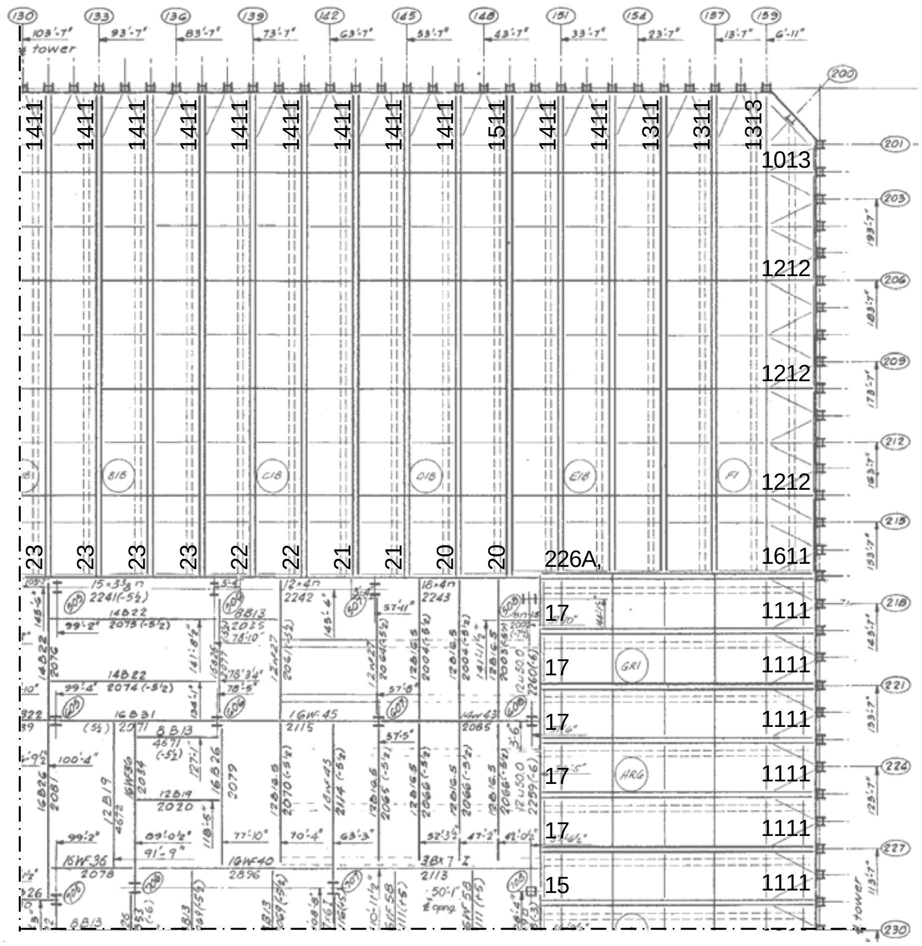


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Figure K–25. Truss seat detail location on northeast quadrant of floor 96 of WTC 1.

All types of interior seats are similar in their design, but are all unique because of the variation in the size of the vertical and horizontal plates, the location of the bolt holes, and the size of the fillet welds. The thickness of the plates ranges from 0.375 in. to 0.75 in.; the distance between bolt holes ranges from 8.5 in. to 10.5 in.; and the size of the fillet welds ranges from 0.25 in. to 0.375 in. All types of exterior seats are also similar in their design, but are all unique because of the variation in the size of the stand-off, and size of the seat angle, the size and shape of the gusset plate, the location of the bolt holes, and the size of fillet welds. The vertical height of the stand-off ranges from 8 in. to 11 in. The smallest seat angle size is L4 × 4 × 1/2, and the largest is L6 × 4 × 3/4. The shapes of the gusset plate are rectangular and trapezoid, and the plate ranges in width from 4.5 in. to 6 in. The distance between bolt holes ranges from 3.25 in. to 10.5 in., where it is 3.25 in. when the seat is supporting a single truss. The size of the fillet welds ranges from 0.2125 in. to 0.375 in.

Truss Seat Material Properties

The material properties used in the calculations were selected from Table K–1 to best match the material properties indicated in the design drawings. Figure K–3 was used to determine the material mechanical properties at high temperature. The material properties used for truss seat calculations are summarized in Table K– 6.

Table K–6. Material properties used for truss seat calculations.

Description		Selected Material ID
Exterior and interior seat	A325 bolts	Material 22
	Fillet welds	Material 7
	Truss bearing angles	Material 21
Exterior seat	Seat angle	Material 1
	Gusset plate	Material 12
	Stand-off	Material 23
	Truss top chord angles	Material 21
	Cover plate for bridging truss top chord	Material 1
Interior seat	Vertical plate stiffener	Material 12
	Horizontal plate	Material 12

Truss Seat Failure Modes and Sequence

The failure modes of different truss seats are identified for vertical force, horizontal tensile force, horizontal compressive force, and combined vertical and horizontal force.

Failure Modes of Interior Seat against Vertical Force: The location of the vertical load on the truss seat is eccentric to the plane of fillet weld connection between the truss seat and the channel beam. Hand calculations have shown that the fillet welds at this connection, which must resist shear and bending, control the truss seat capacity. The failure mode is fracture of the fillet welds at this connection, which results in loss of the truss vertical support.

Failure Modes of Interior Seat against Horizontal Tensile Force: The failure modes considered are (1) bolt shearing, (2) bolt bearing, (3) bolt tear-out, and (4) block shear failure. Hand calculations have shown that the bolt shear strength controls the truss seat capacity. Bolt shear by itself, however, does not cause the truss to lose its vertical support, but it is the prerequisite to the truss walking off the seat. The travel distance required for the truss to walk off of the seat is 4 in.

Failure Modes of Interior Seat against Horizontal Compressive Force: The concrete slab above the truss seat connection provides the compressive force resistance. If the concrete slab fails, the truss seat has resistance against compressive force from bolt friction and surface friction between the seat and bearing angles. Additional resistance is developed when the truss comes into contact with the channel beam. Travel distance for the truss to come into contact with the channel beam is 1/2 in. Under compressive force, the truss will not lose its vertical support.

Failure Modes of Interior Seat against Combined Vertical and Horizontal Forces: Under combined vertical and horizontal forces, the failure modes are a combination of the individual failure modes for vertical and horizontal forces.

Failure Modes of Exterior Seat against Vertical Force: The location of the vertical load on the seat is eccentric to the plane of connection between the seat and the spandrel. Because of this eccentricity, the truss seat must resist both shear and bending. Finite element analysis of the truss seat was used to determine load paths and evaluate the behavior of the seat connection.

Figure K–26 shows the finite element model of the seat connection, where half of the seat was modeled and symmetry boundary conditions were applied. The results of the finite element analysis show that shear force is carried primarily by the stand-off plates shown in Fig. K–24, while the bending moment is resisted by tensile force in the gusset plate and compressive force in the stand-off plate. The seat restrains the moment until horizontal force in the connection causes slippage between the seat angle and bearing angle. Fillet welds at the stand-off to spandrel connection, which must resist shear, bending, and compression, control the seat capacity. The failure mode is fracture of the fillet welds as this connection, which results in loss of truss vertical support.

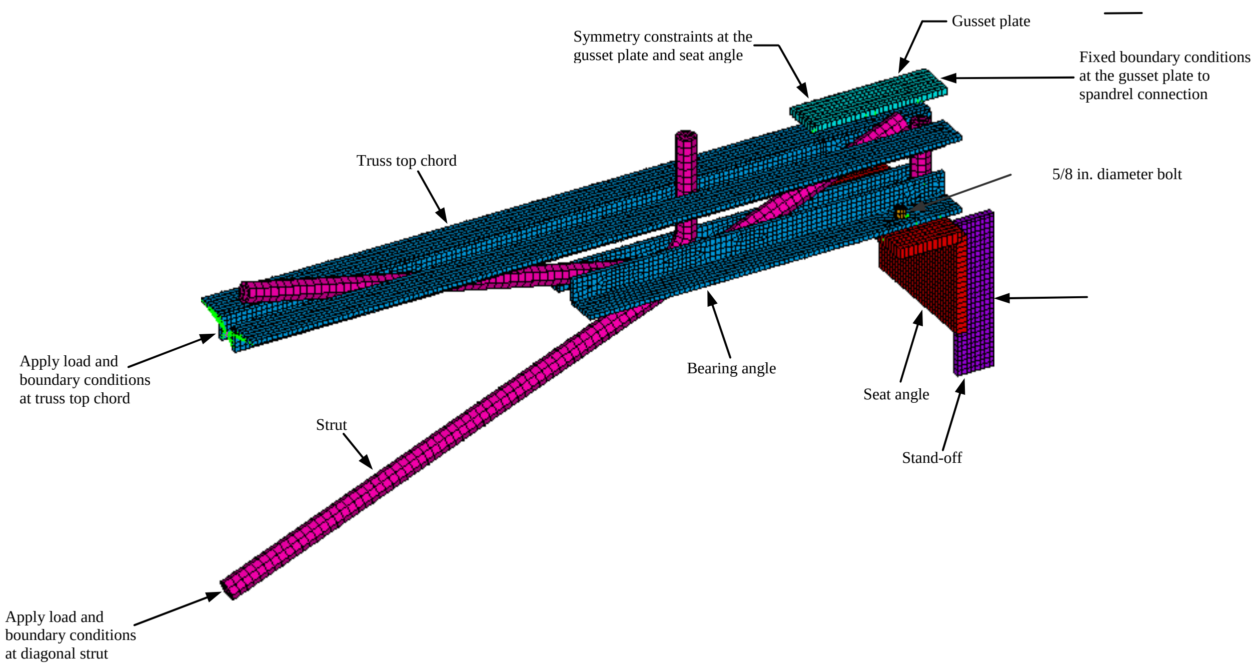
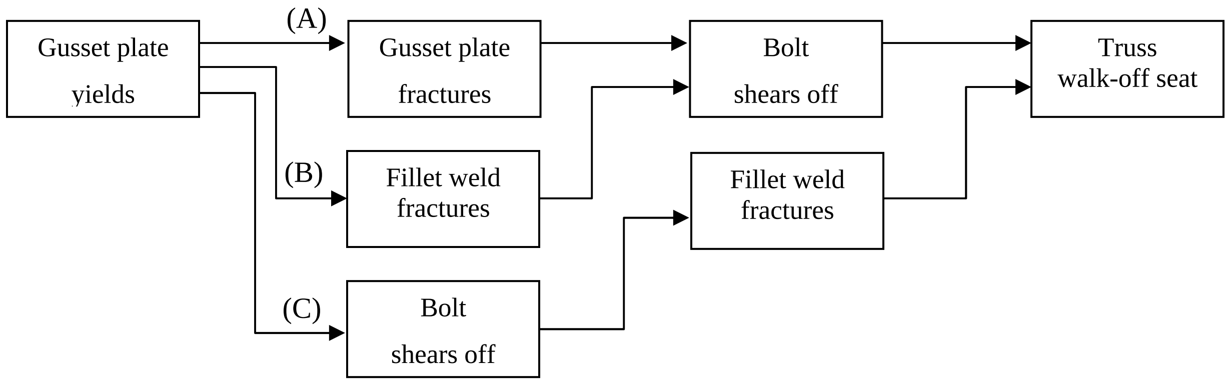


Figure K–26. Finite element model of exterior seat.

Failure Modes of Exterior Seat against Horizontal Tensile Force: The failure modes considered are: (1) failure of the groove weld between gusset plate and spandrel, (2) failure of the fillet weld between the gusset plate and the truss top chord, (3) tensile failure of the gusset plate, (4) bolt shearing off, (5) bolt bearing, (6) bolt tear-out, and (7) block shear failure. For calculation purposes, the bolts are assumed to be centered in the slotted holes. The typical failure sequence of the truss seat is as follows: first the gusset plate yields, then it fractures, followed by truss deformation and bolt bearing against the slotted hole, the bolt shears off, and then finally the truss walks off the seat. The travel distance for the truss to walk off of the seat is 4 5/8 in. This failure sequence is illustrated in Fig. K–27 as path (A) and shown in Fig. K–28, where the relationship between the tensile force resistance from the seat connection and the truss travel distance is plotted. In this plot, frictional resistance between the seat angle and bearing angle was not included.



(A) Seat details 1111, 1311, 1411, 1511, and 1611 at all temperatures.
(B) Seat detail 1013 at temperatures below 100 °C.
(C) Seat details 1212 and 1313 at all temperatures, and detail 1013 at temperatures more than or equal to 100 °C.

Figure K–27. Failure sequence of the exterior seats against tensile force.

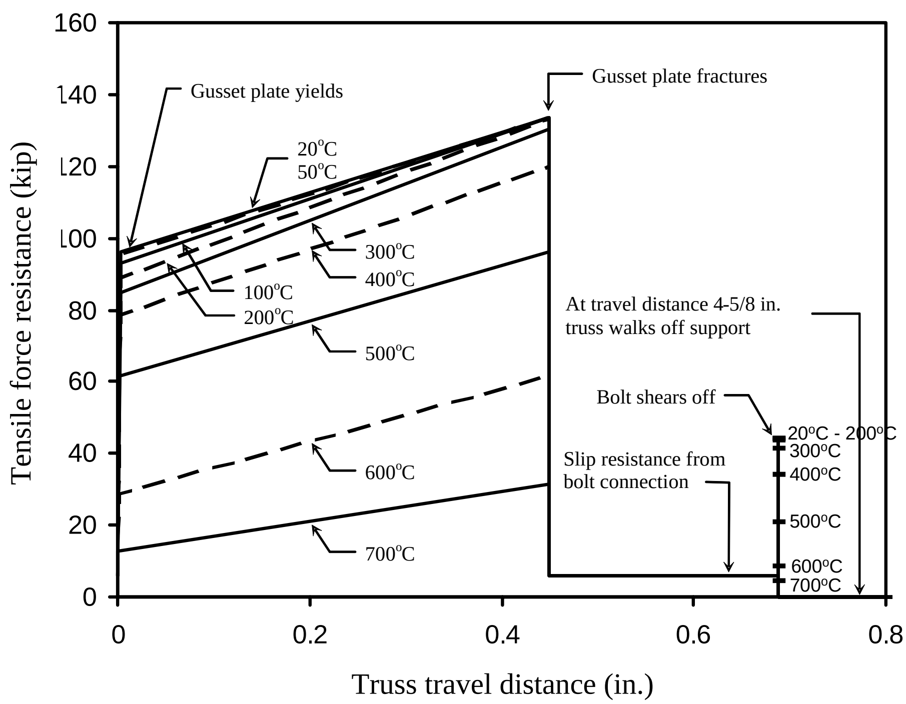


Figure K–28. Typical tensile force resistance from exterior seat (Detail 1411).

Seat details 1212 and 1313 have a wider gusset plate and follow path (C) which differs from the typical sequence where the bolts will bear against the slotted hole then shear off before the gusset plate connection fails. The failure sequence of seat detail 1013 is temperature-dependent. At temperatures below 100 °C, the fillet weld connection between the gusset plate and the truss top chord fractures before bolts shear off. At temperatures greater than or equal to 100 °C the failure sequence is the same as for Details 1212 and 1313.

Failure Modes of Exterior Seat against Horizontal Compressive Force: The concrete slab above the truss seat connection provides the compressive force resistance. If the concrete slab fails, the truss seat has resistance against compressive force provided by the gusset plate until it buckles, and from bolt friction and bolt shear until the bolt bears against the slotted hole and then shears off. Surface friction between the seat angle and bearing angles will also provide some resistance. Additional resistance is developed when the truss comes into contact with the spandrel. Travel distance for the truss to come into contact with the column spandrel is 1 1/2 in. Under compressive force, the truss will not lose its vertical support.

Failure Modes of Exterior Seat against Combined Vertical and Horizontal Force: Under combined vertical and horizontal forces, the failure modes are a combination of the individual failure modes for vertical and horizontal forces.

Truss Seat Capacity Calculations

In this section, truss seat capacities corresponding to the failure modes described in the previous section are given. The capacity is computed for the different types of the truss seat at different temperatures. Calculation of the connection capacity was performed using the methods in the *Manual of Steel Construction: Load and Resistance Factor Design* (AISC 2001) with the resistance factor, ϕ, assumed to be equal to one.

Capacity of Interior Seat against Vertical Force: Failure mode of the truss seat against vertical force is fracture of the fillet welds at the seat-to-channel beam connection. Strengths of the fillet welds at this connection are summarized in Table K–7. The symbol # in this table refers to seat detail number.

Table K–7. Interior seat capacity against vertical force.

Temp. (°C)	Connection Capacity Against Vertical Force (kip)						
	#15	**#17**	**#20**	**#21**	**#22**	**#23**	**#226A**
20	226	226	265	221	187	187	385
50	226	226	265	221	187	187	385
100	226	226	265	221	187	187	384
200	225	225	264	220	187	187	383
300	220	220	258	215	182	182	374
400	201	201	236	197	167	167	343
500	160	160	188	156	132	132	272
600	98	98	116	96	82	82	167
700	45	45	53	44	37	37	76

Capacity of Interior Seat against Horizontal Tensile Force: Failure loads were computed for the failure modes described above. Table K–8 summarizes the results for Seat Detail 22. This table shows that the shear strength of the two bolts controls the horizontal tensile strength of the truss seat connection. As can be seen from this table at temperature 500 °C, bolt shear capacity is reduced by half, and at 600 °C it is reduced to less than a quarter of the original capacity at room temperature. Other seat details also have the same failure mode, and, therefore, the same failure load.

Table K–8. Interior seat capacity against tensile force.

Temp. (°C)	Resistance against Tensile Force (kip)							
	Bolt Slip Critical	Bolt Shearing Off	Bolt Bearing		Bolt Tear-out		Block Shear	
			On Seat	On Truss	From Seat	From Truss	Of Seat	Of Truss
20	6	44	124	69	87	101	60	59
50	6	44	124	69	87	101	60	59
100	6	44	124	69	87	101	60	59
200	6	44	124	69	87	100	59	59
300	6	42	121	68	85	98	58	57
400	6	34	111	62	77	90	53	52
500	6	21	88	49	61	71	42	42
600	6	9	54	30	38	44	24	24
700	6	4	25	14	17	20	10	10

Capacity of Interior Seat against Horizontal Compressive Force: Under compressive force, the truss will come into contact with the channel beam before the bolt bears against the slotted hole. The truss seat connection does not fail under compressive force.

Capacity of Interior Seat against Combined Vertical and Horizontal Force: A typical interaction relationship for combined vertical and horizontal tensile force is shown in Fig. K–29. As can be seen from this figure, the vertical shear strength of the seat reduces because of the additional horizontal tensile force that the fillet weld connection between the truss seat and the channel beam must resist.

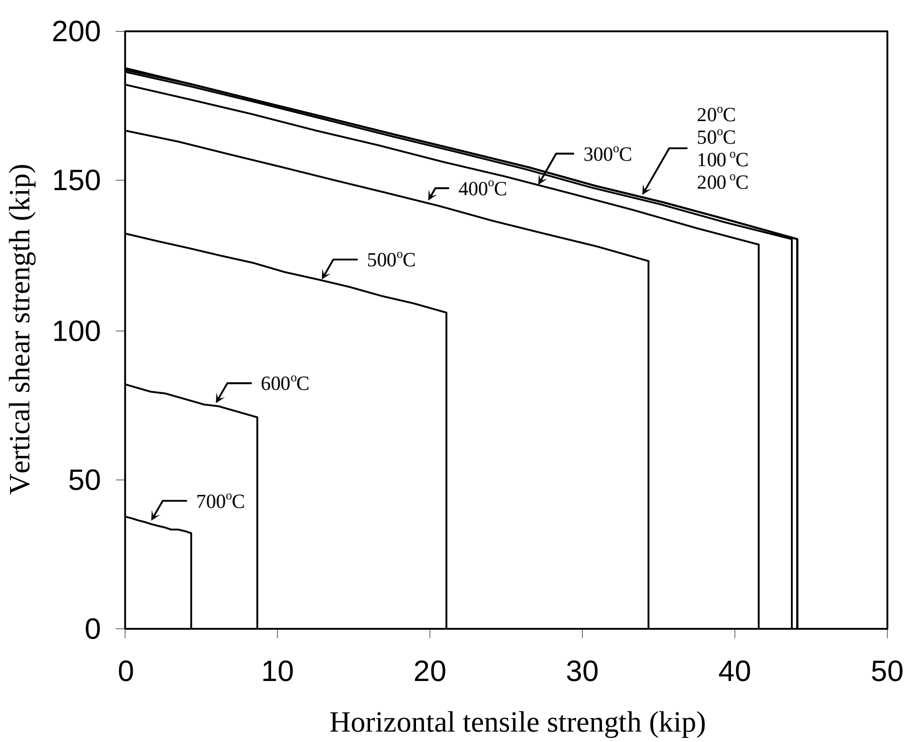


Figure K–29. Strength of combined vertical and horizontal force (Detail 22).

Capacity of Exterior Seat against Vertical Force: The failure mode of the truss seat against vertical force is fracture of the fillet welds at the stand-off-to-spandrel connection. Strength of the fillet welds at this connection is summarized in Table K–9.

Table K–9. Exterior seat capacity against vertical force.

Temp. (°C)	Connection Capacity against Vertical Force (kip)							
	#1013	#1111	#1212	#1311	#1313	#1411	#1511	#1611
20	94	94	111	94	94	140	193	207
50	94	94	111	94	94	140	193	207
100	94	94	111	94	94	140	193	207
200	93	93	110	93	93	139	192	206
300	91	91	108	91	91	136	187	201
400	84	84	100	84	84	126	172	184
500	69	69	81	69	69	102	136	146
600	45	58	53	60	45	78	84	90
700	29	26	34	27	29	35	38	41

Capacity of Exterior Seat against Horizontal Tensile Force: The connection capacity of truss seats that follow failure sequence (A), as shown in Fig. K–27, equals the failure load for mode (3) defined previously. The connection capacity of truss seats that follow failure sequence (B) equals the failure load for mode (2). The connection capacity of truss seats that follow failure sequence (C) equals the failure load for mode (4) plus the developed resistance from the gusset plate. The results of the exterior seat capacity calculations are summarized in Table K–10. Note that the strength of the truss seat #1013 increases by about 38 percent at a temperature of about 100 °C. For temperatures less than 100 °C, the capacity is controlled by the gusset seat fillet weld strength, and for temperatures in excess of 100 °C, the bolt reaches the end of its travel in the elongated bolt hole and increases the capacity of the connection.

Table K–10. Exterior seat capacity against horizontal tensile force.

Temp. (°C)	Connection Capacity against Tensile Force (kip)							
	#1013	#1111	#1212	#1311	#1313	#1411	#1511	#1611
20	100	104	182	134	182	134	134	134
50	100	104	182	134	182	134	134	134
100	138	104	181	134	181	134	134	134
200	135	103	180	133	180	133	133	133
300	130	101	174	130	174	130	130	130
400	115	93	156	120	156	120	120	120
500	84	75	117	96	117	96	96	96
600	42	49	67	62	67	62	62	62
700	20	25	32	31	32	31	31	31

Capacity of Exterior Seat against Horizontal Compressive Force: Under compressive force, the gusset plate will buckle before the bolts shear off. Compression strength of the gusset plate governs the truss seat capacity. The compressive strength of the gusset plate is summarized in Table K–11.

Table K–11. Compression strength of gusset plate.

Temp. (C)	Compression Strength of Gusset Plate (kip)							
	#1013	#1111	#1212	#1311	#1313	#1411	#1511	#1611
20	77	69	99	90	99	90	90	90
50	76	68	98	89	98	89	89	89
100	74	66	96	87	96	87	87	87
200	71	63	91	83	91	83	83	83
300	67	60	87	79	87	79	79	79
400	62	55	80	72	80	72	72	72
500	48	42	61	55	61	55	55	55
600	20	17	25	22	25	22	22	22
700	6	5	8	7	8	7	7	7

Capacity against Combined Vertical and Horizontal Force: Interaction relationships for combined vertical and horizontal forces are under development.

K.7.4 Modeling Connection Failure by Break Elements

In this section, simplified finite element models of the exterior and interior seat, knuckle, stud on strap anchor, and stud on spandrel are described. These connection models were developed for incorporation in the floor truss analysis to capture the connection failure modes and determine the sequence of the failure modes that may lead to the failure of the floor truss.

The developed simplified model of these connections simulates the loss of connection resistance after failure either by exceeding the connection force capacity or by exceeding the allowable deformation (truss walking off the seat). The connection capacity can also be temperature-dependent. The finite element modeling assumptions are as follows:

Break element, a unidirectional linear spring element with the capability of turning on and off during an analysis, is used for modeling connection failure. The element is a part of the structure that connects two “active” nodes in the “on” mode and disconnects them in the “off” mode, depending on the relative displacement of two “control” nodes. The break element is defined as follows:

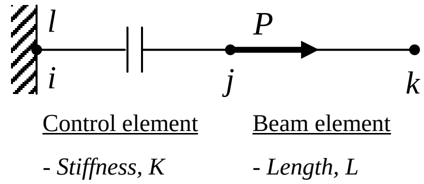
$$B_m[iI,j,dof_{ij});(k,l,dof_{kl});(K,\Delta_0)] \tag{11}$$

where m is the break element number, i and j are the active nodes, dof_{ij} is the degree of freedom for the active nodes, k and l are the control nodes, dof_{kl} is the degree of freedom for the control nodes, K is the elastic stiffness of the break element, and Δ_0 is the differential displacement limit of the control nodes.

A beam element with temperature-dependent thermal expansion material properties is used to make the connection capacity temperature-dependent. This is done by using the deformation of the beam element from thermal expansion to control the status (on/off) of the break element. Figure K–30 illustrates the basic mathematical model of the connection. The connection capacity is made temperature-dependent by defining the thermal expansion of the beam element to be temperature dependent.

Basic mathematical model:

– Let node l and i have same displacement.



– In this example, control node k and l relative displacement has the form:

$$U_k - U_l = \frac{P}{K} + \alpha L \Delta T$$

– Control element status is by definition

$$U_k - U_l \le \Delta_0,\ \text{status: on}$$
$$U_k - U_l > \Delta_0,\ \text{status: off}$$

– Define break element to turn off when capacity, P_u, has been reached

$$\Delta_0 = \frac{P_u}{K} + \alpha L \Delta T$$

– Element capacity at room temperature is given as

$$P_{u.rm} = K\Delta_0$$

– Hence for a given P_u and ΔT relationship, the required thermal expansion to turn element off can be found by

$$\alpha = \frac{P_{u.rm} - P_u}{KL\Delta T}$$

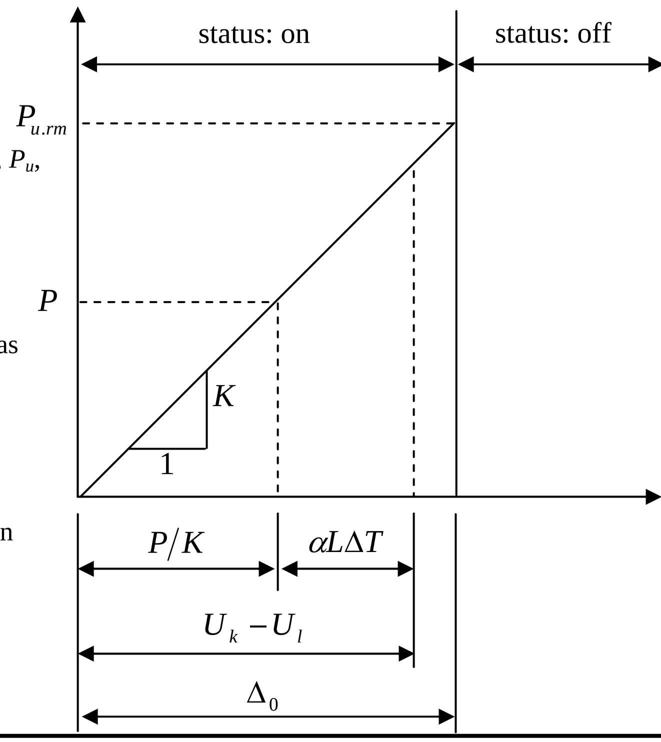


Figure K–30. Basic mathematical model of connection failure.

Multiple connection failure modes require use of different break elements that are connected together in a logical manner. For example, to model independent failure modes, that is, one failure mode that does not cause other failures, break elements are connected in parallel. If one break element turns off, the other break elements remain. For dependent failure modes, break elements are connected in series. If one break element turns off, then all elements turn off.

Simplified Model of the Interior and Exterior Seat

The failure modes of the interior seat include (1) the truss walking off the support, (2) exceeding the vertical temperature-dependent shear capacity of the truss seat, and (3) exceeding bolt temperature-dependent shear capacity when bolt bears against slotted hole. These failure modes are captured by using four break elements and two beam elements as shown in Fig. K–31. Results of the simplified seat model capturing failure from the truss walking off support and failure from exceeding seat vertical shear capacity are shown in Fig. K–32 and Fig. K–33, respectively, which depict the relationship between the horizontal and vertical seat forces and the horizontal truss travel distance.

When truss reaction force on the seat is large in horizontal tension and small in vertical shear, the failure mode is bolt shearing off followed by truss walking off the support as shown in Fig. K–32. Bolt shear is controlling the seat horizontal resistance capacity. Bolt shear by itself however does not cause the truss to lose its vertical support, but it is the prerequisite of truss walking off the seat. The travel distance for a truss to walk off an interior seat is 4 in. When truss reaction force on the seat is large in vertical shear and small in horizontal tension, the failure mode is exceeding the seat vertical shear capacity as shown in Fig. K–33. This failure mode will cause the truss to lose both its vertical and horizontal support from the seat.

The simplified model of the exterior seat is the same as the simplified model of the interior seat, except for an additional beam element and a break element to model failure of the gusset plate shown in Fig. K–34.

Simplified Model of the Knuckle

Knuckle failure modes that must be captured by the simplified model are the horizontal shear and vertical tensile failure, which are both temperature-dependent. Finite element modeling assumptions for the knuckle are: (1) the knuckle has resistance in all translational DOF, (2) the knuckle does not have a vertical compression capacity limit, (3) capacities in the horizontal shear and vertical tension are dependent, and (4) vertical compression resistance is independent of the capacities in the other directions. Knuckle failure is captured by using 15 control elements and 5 beam elements as shown in Fig. K–35.

Simplified Model of the Stud on Strap Anchor and Stud on Spandrel

Simplified models of stud on strap anchor and stud on spandrel were developed using the same technique as described for the knuckle model.

K.7.5 Truss Model

Objectives

The objectives of the truss model study are to:

- Capture the potential failure modes and failure sequence of the truss under combined gravity load and thermal load;
- Develop an understanding of the relative importance of different structural features and failure modes; and
- Develop a simplified model that replicates the expected failure and the limit loads of the truss to be used for analysis of the full floor subsystem model.

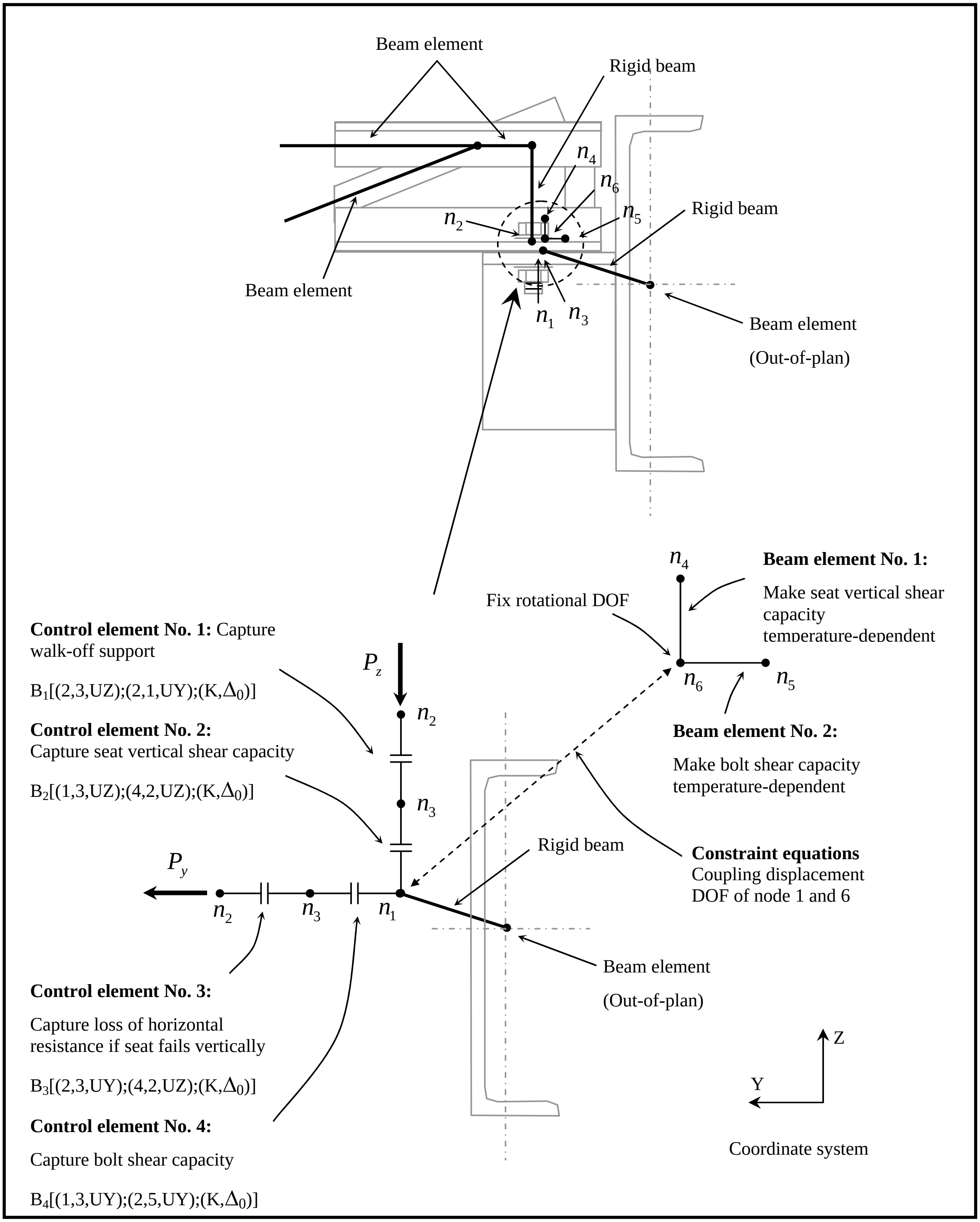


Figure K–31. Simplified model of interior seat.

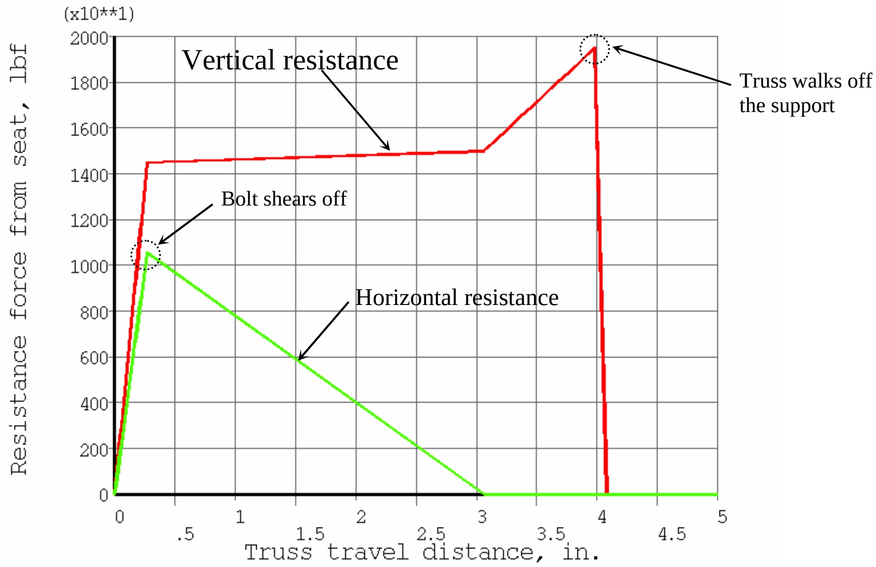


Figure K–32. Results of simplified seat model capturing failure from truss walking off interior seat.

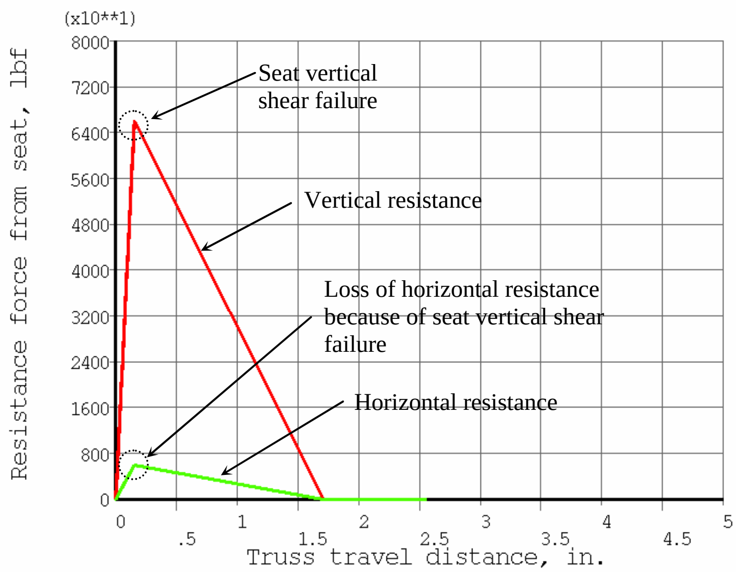


Figure K–33. Results of simplified seat model capturing failure from exceeding the interior seat vertical shear capacity.

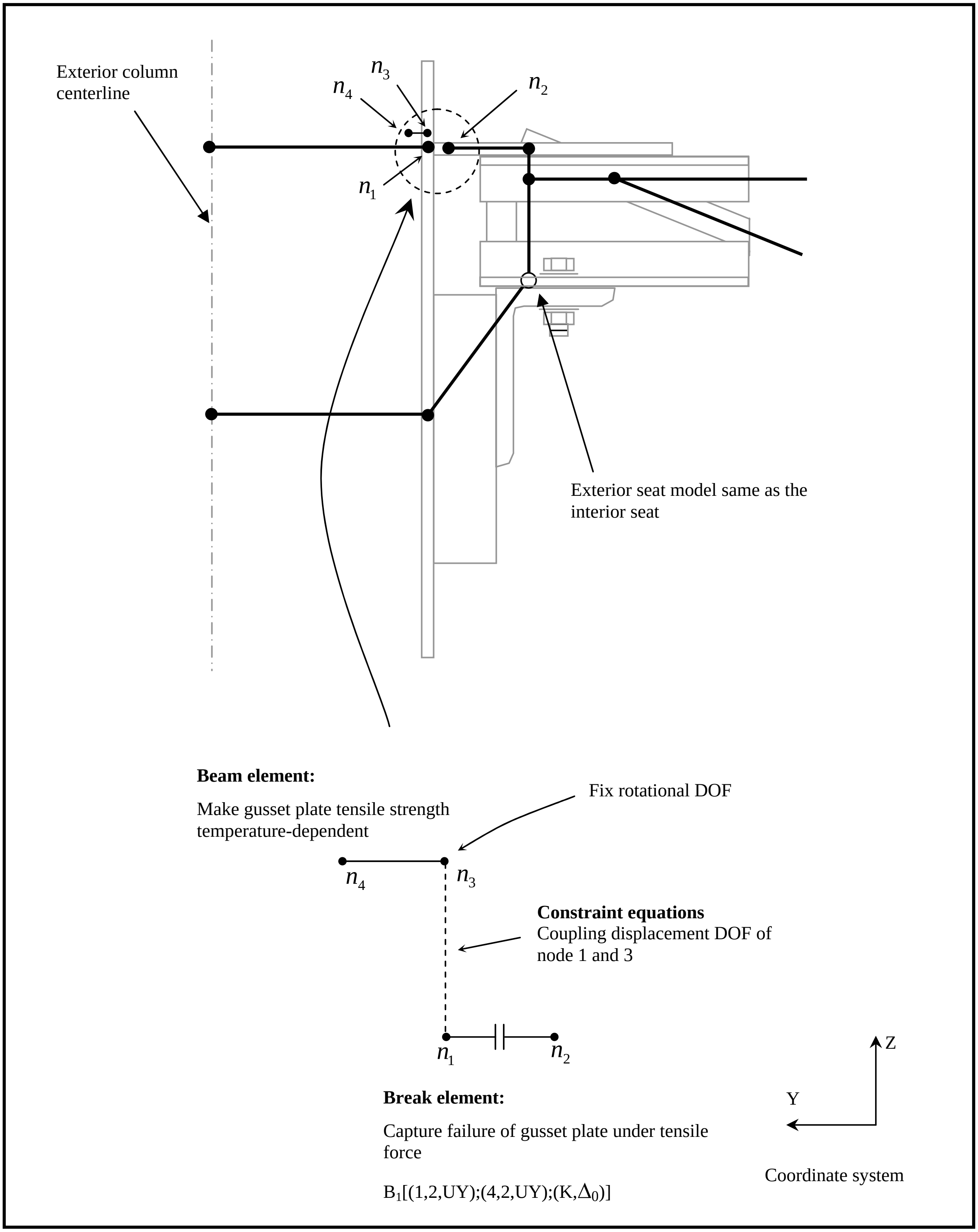


Figure K–34. Simplified model of exterior seat.

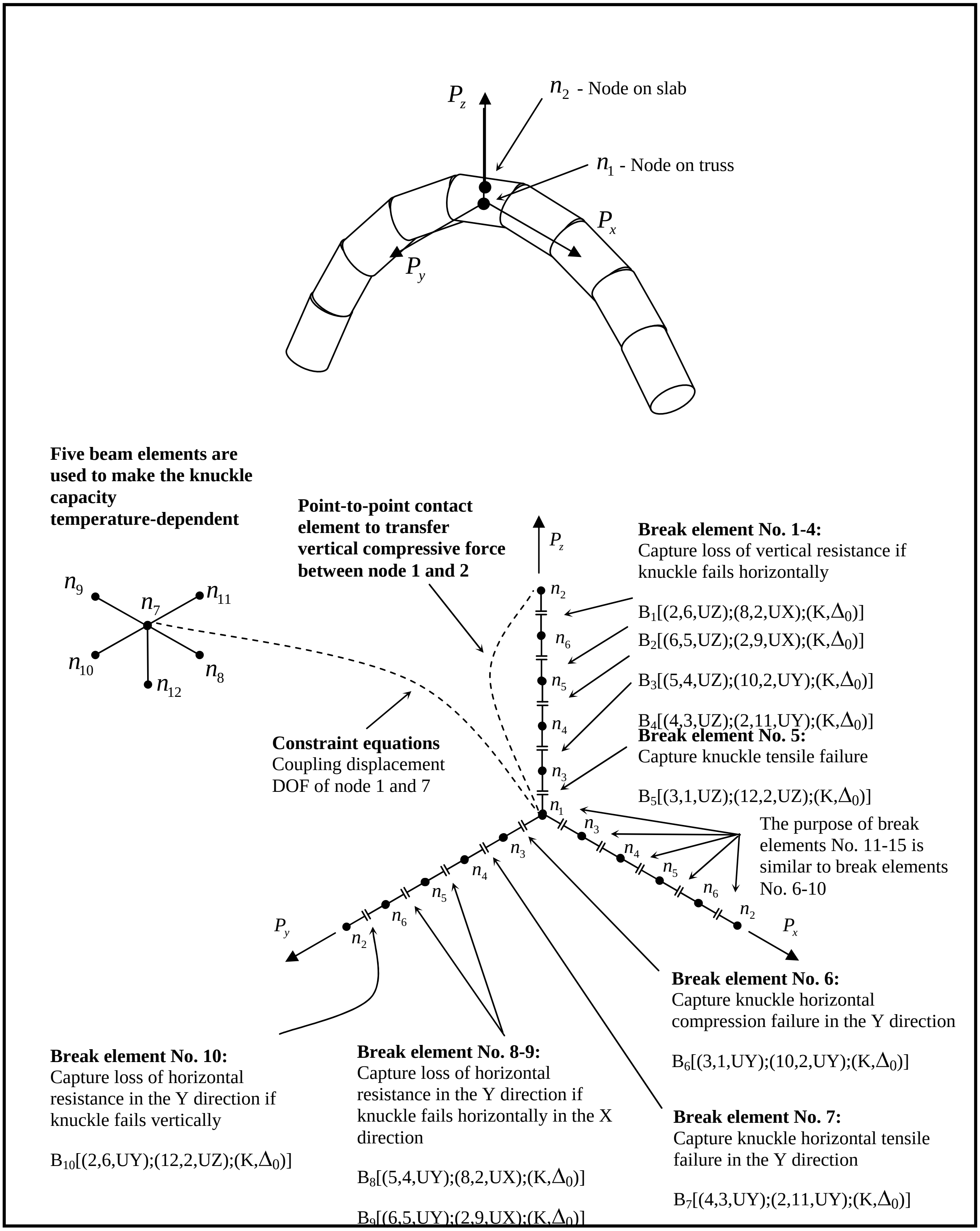


Figure K–35. Simplified model of knuckle.

Failure Modes

The model can capture the following failure modes:

Softening and Sagging of Truss**—**The top and bottom chords and diagonals of the truss are exposed to the hot gas layer below the floor slab. As described in Section K.5, the steel in the truss exhibits stiffness degradation, yield strength reduction, plastic softening, and creep at high temperatures. A truss with softened chords sags. The heat may also reduce the stiffness and strength of the concrete slab, especially its bottom layer where temperature is the highest, and around the knuckle where concrete temperature rises by conduction through the steel.

In addition to direct thermal effects, sagging and weakening of the truss can be caused by the following failure modes:

- Buckling or failure of web diagonal members, which reduces the truss action and causes the truss to act as a catenary;
- Buckling or failure of the top and bottom chord members;
- Knuckle failure and loss of composite action of the concrete slab and the steel truss; or
- Weld failure between the diagonal and the chord.

Loss of Support of Truss**—**The truss can fail by loss of support due to seat failure. Loss of support at either the exterior or interior seat can be caused by the extreme sagging and catenary action of the truss due to plastic deformation and buckling of truss members.

As discussed under Boundary Conditions later in this section, the bottom chord of the truss is restrained in the lateral direction at the bridging truss locations. Although the out-of-plane deformation of the bottom chord due to thermal expansion of bridging trusses will result in a reduction in the vertical load capacity of a primary truss, the truss model studied here cannot capture this phenomenon. The interaction between the bridging trusses and the primary trusses is intended to be captured in the full floor model.

Model Description

Figure K–36 shows the truss model. A typical long-span truss designated C32T1 (SHCR 1973:WTC Drawing Book 7, Sheet AB1–2) is modeled to study its response to failure when subjected to dead and live loads and thermal loads. The model includes the following:

- One truss of the pair of trusses at column line 143 of floor 96 in WTC 1;
- Two exterior columns (columns 143 and 144) with half the area and bending properties, and a length of 24 ft (12 ft above and below the floor level);
- The portion of the spandrel between the two exterior columns;

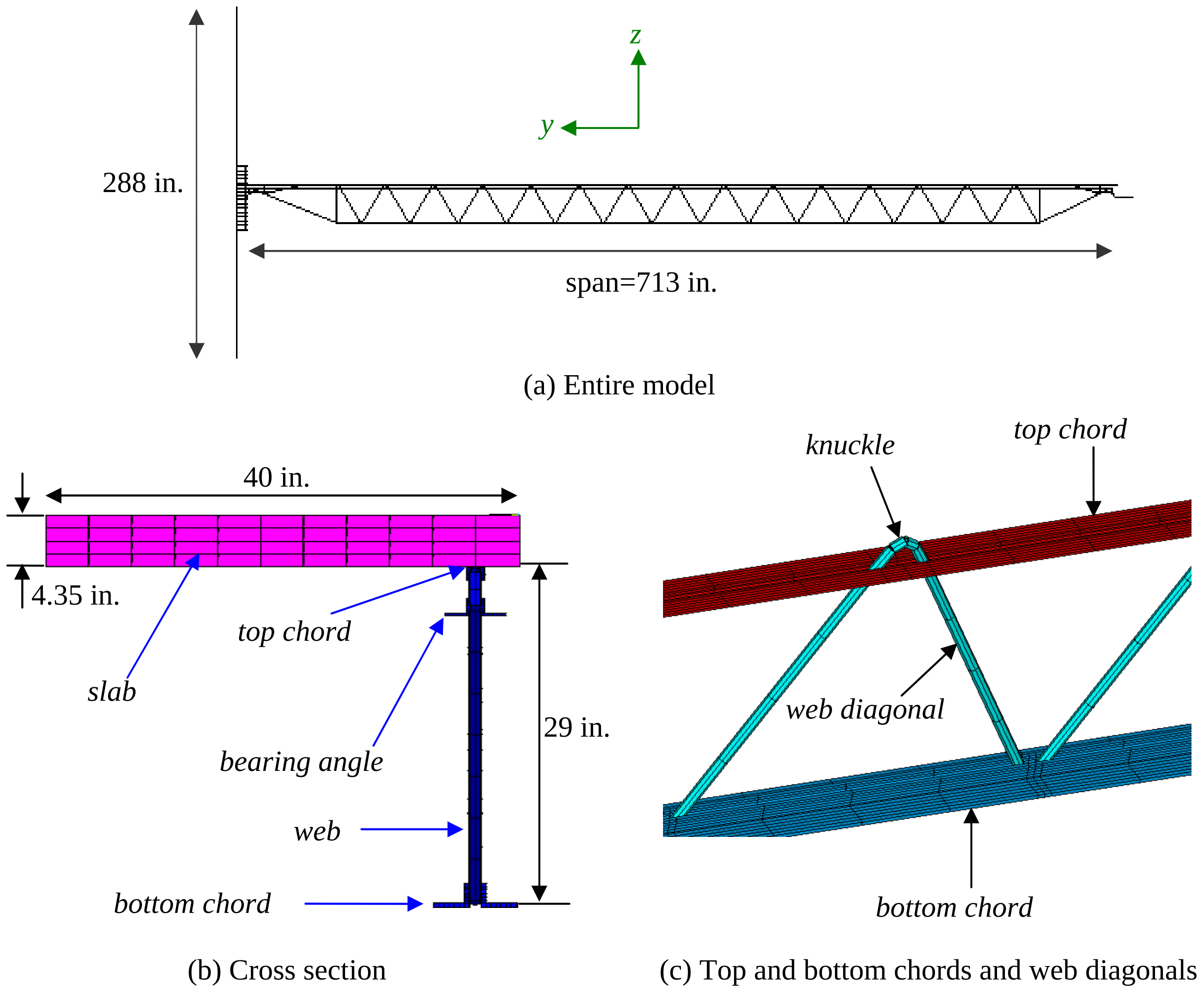


Figure K–36. Truss model.

- The portion of the slab (40 in. wide) between the two exterior columns;
- One strap anchor that is attached to the truss top chord, concrete slab and the adjacent exterior column (column 144); and
- Exterior and interior seats, and the gusset plate at the exterior end.

A typical slab section consists of 4 in. thick lightweight concrete on 22 gauge metal deck with flutes 6.8 in. on centers. Two layers of welded wire fabric were provided in the slab. The reinforcement ratios are 0.21 and 0.735 in the directions along and transverse to the truss, respectively. A flute is 2 in. wide at the top, 1.25 in. wide at the bottom, and 1.47 in. high. An equivalent thickness of 4.35 in. is used as the slab thickness in the truss model. By using the equivalent thickness, the bending stiffness in the direction transverse to the truss is about 15 percent higher than the actual stiffness. However, since the bending in the transverse direction in this truss model is insignificant, the slab is modeled as an isotropic plate. The metal deck and the welded wire fabric are not included in the truss model.

The top and bottom chords and the diagonals of the truss are modeled by 3–D quadratic finite strain beam (BEAM189) elements with temperature-dependent elastic, plastic, and creep material properties. The top chord consists of double angles of 1 1/2 × 2 × 0.25 (long legs horizontal), while the bottom chord consists of double angles of 3 × 2 × 0.37 (long legs horizontal). Web members are round bars of either 1.09 in. or 1.14 in. diameter. A typical diagonal member has a 1.09 in. diameter. Top and bottom chords are divided into four elements between panel points, and a diagonal is also divided into four elements between top and bottom chords. The concrete slab is modeled with 4-node finite strain shell (SHELL181) elements. The nodes of the concrete slab are located at the neutral plane of the concrete slab with an offset relative to the nodes of the top chords. The cast iron model (Hjelm model) can be used with the SHELL181 elements that allow different "yield" in tension and compression. A low "yield stress in tension" is used to simulate cracking.

At knuckle locations, the top chord elements and the elements representing the concrete slab are connected by control elements (COMBIN37) with capacities determined from the detailed knuckle analysis. By including point-to-point contact (CONTA178) elements, compression can always be transferred even after knuckles fail. Studs on the strap between the top chord and column 144 are also modeled by COMBIN37 elements that connect the strap to the slab and have temperature-dependent capacities. The slab and the strap are tied by the COMBIN37 elements horizontally while their vertical displacements are coupled. The exterior and core seats are modeled by COMBIN37 elements that have temperature-dependent capacities determined from the seat analysis. A stud on the spandrel is also modeled by a COMBIN37 element, which ties the spandrel with the slab and has temperature-dependent capacities. Because only one 5/8 in. stud was provided over 80 in. between the slab and the spandrel, a half stud capacity is considered for the stud in the model. In addition, the first stud on the strap is located near this stud on the spandrel. Therefore, COMBIN37 elements between the slab and the spandrel have a capacity of a combination of these studs, including a group effect. Damping unit connecting the truss bottom chord to the spandrel plate is assumed to have little effect on the behavior of the floor truss under sustained loading; therefore, it was not included in the model.

Three–D elastic beam (BEAM44) elements model the exterior columns. SHELL63 elastic shell elements model the spandrel.

Boundary Conditions

Boundary conditions on the truss model are shown in Fig. K–37.

The entire top chord of the truss is supported in the x direction. The bottom chord is supported in the x direction at four bridging truss locations. Two edges of the concrete slab are restrained against rotations about the y and z axes, but can move in the x direction.

The interior truss seat is fixed in all directions. The exterior seat is fixed to the spandrel. The truss is pinned at both exterior and interior truss seats.

The exterior end of the slab is tied to the spandrel by only COMBIN37 elements representing studs. The interior end of the slab is fixed in the z direction and in rotation about the z direction. In the y direction at the interior end of slab, break elements that have temperature-dependent tensile capacities are implemented as show in Fig. K–38. Therefore, the interior slab end is fixed in the y direction until the tensile force exceeds the capacity that is calculated based on the amount of steel reinforcement (#3@10″ top and #4@12″ bottom).

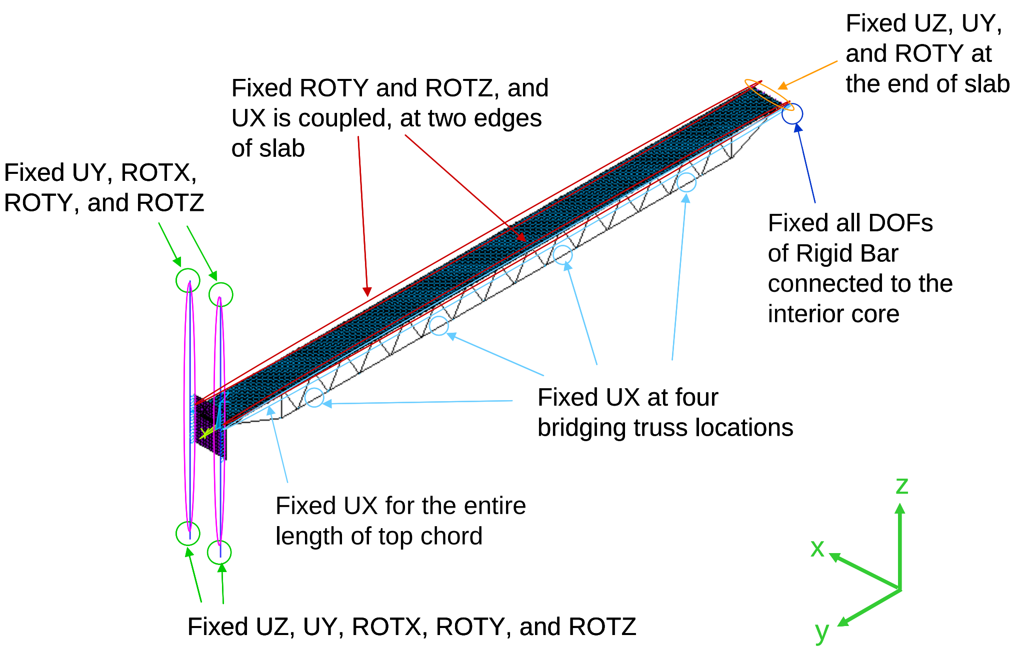


Figure K–37. Boundary conditions.

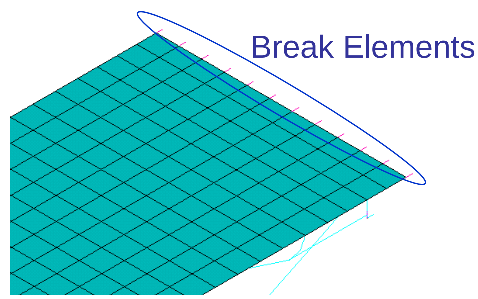


Figure K–38. Break elements at the interior end of slab.

In the analysis with increasing gravity load, a model different from the current model is used, where boundary conditions of the slab are slightly different from the current model described above. In this model, the exterior end of the slab is tied to the spandrel, and the core end of the slab is fixed in the *y* and *z* directions and in rotations about the *x* and *z* directions. Also, COMBIN37 elements for seats are not included in this model.

Loading

The loading on the truss model consists of gravity dead and live loads and temperature time-histories for all steel members, including the truss seats. The gravity loads include weight of the structure, 8 psf superimposed dead load (including nonstructural dead loads due to architectural items and fixed service equipment), and 13.75 psf of live load equal to 25 percent of design live load of 55 psf. The thermal load is a linear temperature gradient through the slab from 300 °C at the top surface of the concrete slab to

700 °C at the bottom surface of the slab. The temperature is ramped from 20 °C to 700 °C in steel members; from 20 °C to 700 °C at the bottom surface of the slab and from 20 °C to 300 °C at the top surface of the slab at 1,800 s; thereafter, the temperatures do not change for another 1,800 s. Temperature is not applied to the columns.

In order to determine the effect of debris load on the truss behavior, a parametric study will be performed.

Material Properties

Table K–12 shows material assignments for different structural components in the truss model.

Table K–12. Material assignments in truss model.

Structural Component	Specified Yield Strength	Material ID
Top chord	50 ksi	21
Bottom chord	50 ksi	21
1.09 in. diameter web	36 ksi	20
1.14 in. diameter web	50 ksi	21
Strap	36 ksi	1
Column 143	65 ksi	15
Column 144	65 ksi	15
Spandrel	42 ksi	11
Lightweight concrete slab	3,000 psi (f’c)	83

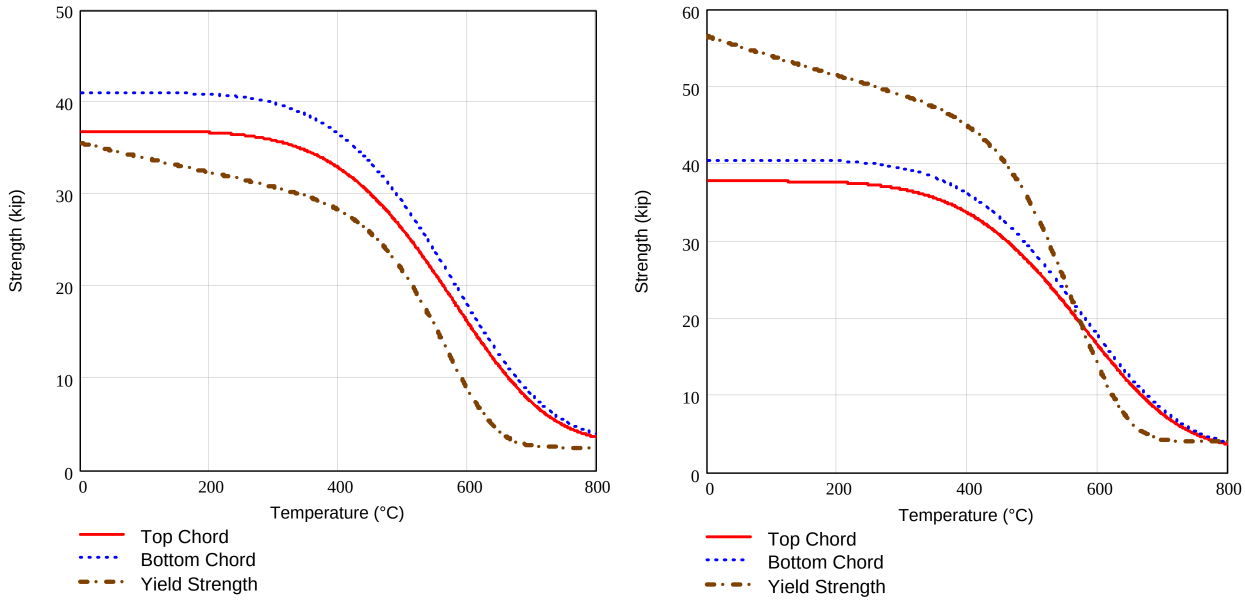
Columns 143 and 144 and the spandrel, use only elastic properties. In the current model, the concrete slab also remains elastic.

Resistance Welds

Table K–13 shows the resistance weld strength between a chord (double angles) and a diagonal based on the test data found at Laclede Steel. Weld strength shown in Table K–13 is the sum of the capacities of two resistance welds. Figure K–39 compares resistance weld strength between top or bottom chord and a diagonal with yield strength of a diagonal at elevated temperatures. As can be seen in Fig. K–39 (a), a typical diagonal (1.09 in. diameter) will yield before the resistance weld fails. For 1.14 in. diameter diagonal, the resistance weld strength cannot yield the bar at temperatures below 550 °C, as can be seen in Fig. K–39 (b). However, shop drawings show additional arc welds between the chord and 1.14 in. diameter bar at most locations.

Table K–13. Resistance weld strength.

Chord	Diagonal Size (in.)	Average Strength (kip)
Top chord	1.09	36.9
Top chord	1.14	37.7
Bottom chord	1.09	41.0
Bottom chord	1.14	40.5



(a) 1.09 in. diameter bar (b) 1.14 in. diameter bar

Figure K–39. Comparison of resistance weld strength and yield strength of web member at elevated temperatures.

Current Status

The truss model can capture the following:

- Temperature-dependent elastic material properties for both steel and concrete;
- Temperature-dependent steel plasticity;
- Buckling of truss members;
- Failure of knuckle—loss of composite action;
- Failure of studs on the strap;
- Failure of stud between the spandrel and the concrete slab; and
- Failure of the exterior and interior truss seats.

The following features are being added to the truss model:

- Crushing and cracking of concrete;
- Creep strain in steel at elevated temperatures; and
- Failure of welds (Calculations show section yielding can occur prior to weld failure in nearly all cases.).

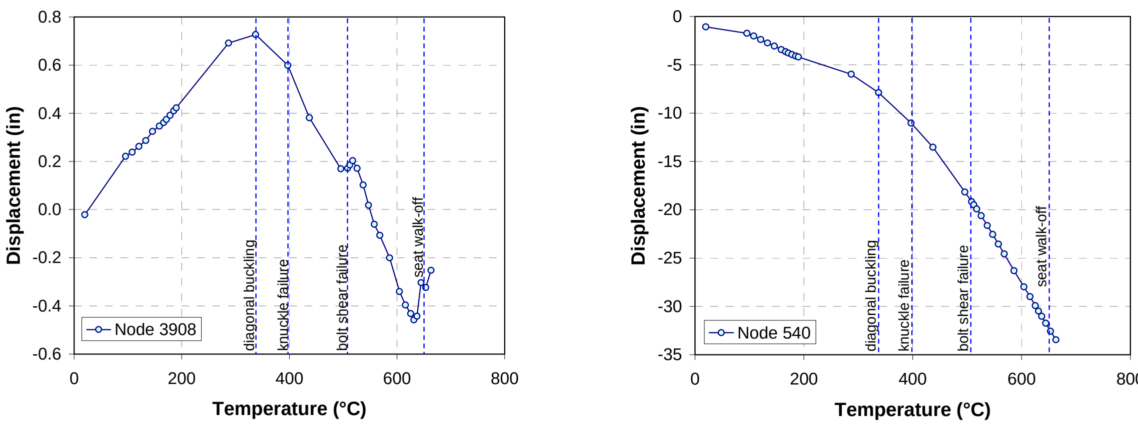
Model Verification

The maximum vertical displacement is checked against the single truss model extracted from the ANSYS full floor model that was converted from the SAP full floor model. The difference in the vertical displacement is only 3.5 percent.

FEA Results

***Gravity Loading*—**The maximum calculated vertical deflection is 1.1 in. downward. The maximum calculated horizontal column deflection is 0.022 in. inward. The maximum forces in top chord, bottom chord, and diagonal are 13,357 lb, 39,514 lb, and 7,647 lb, respectively.

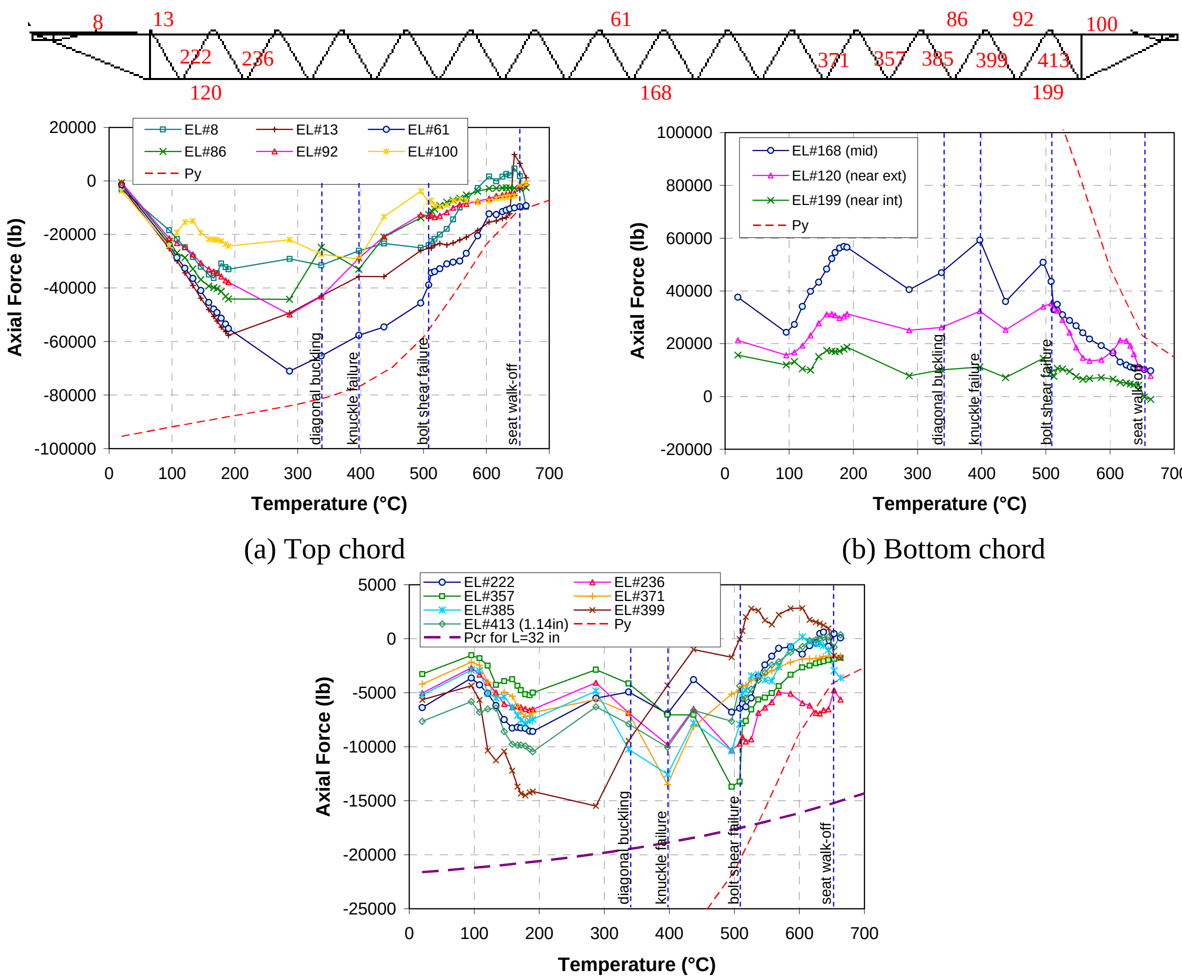
***Gravity Plus Thermal Loading*—**The analysis is carried out dynamically with 5 percent Rayleigh damping. To shorten the run time, the total time period is set to 1.0 s for the temperature ramp. The analysis proceeded to a temperature of T=663 ^{o}C. Figure K–40 shows horizontal and vertical displacement results. A positive horizontal displacement indicates that the exterior columns are pushed out, and a negative vertical displacement indicates that the truss is deflected downward. At 340 ^{o}C, the horizontal displacement at the exterior column starts to decrease. At 560 ^{o}C, the exterior columns are pulled in, and the truss becomes catenary from that point on.



(a) Horizontal displacement at column 143 (b) Vertical displacement at midspan

Figure K–40. Displacement versus temperature.

Figure K–41 shows axial forces in the truss members. In the figure, Py is the axial force at yield and equals the product of the net area of the member and the yield strength which varies with temperature. Pc is the compressive strength per AISC formula (AISC 2003) for the top chord with fixed ends in Fig. K–41 (a) and for 1.09 in. diameter diagonal bar with pinned ends in Fig. K–41 (c).



(a) Top chord

(b) Bottom chord

(c) Compression diagonal

Figure K–41. Axial force in truss members versus temperature.

Figure K–42 (a) shows the top chords yielding beyond 300 °C. This is due to a significant difference of coefficients of thermal expansion (CTE) between concrete and steel. At 500 °C, the CTE of steel is twice that of lightweight concrete. Bottom chords are still in the elastic range at the end of analysis. Some diagonals are bent significantly in the plane of the truss by high axial force and end moments (see Fig. K–42 for the deformed shape at the interior end). This diagonal buckling starts at approximately 340 °C.

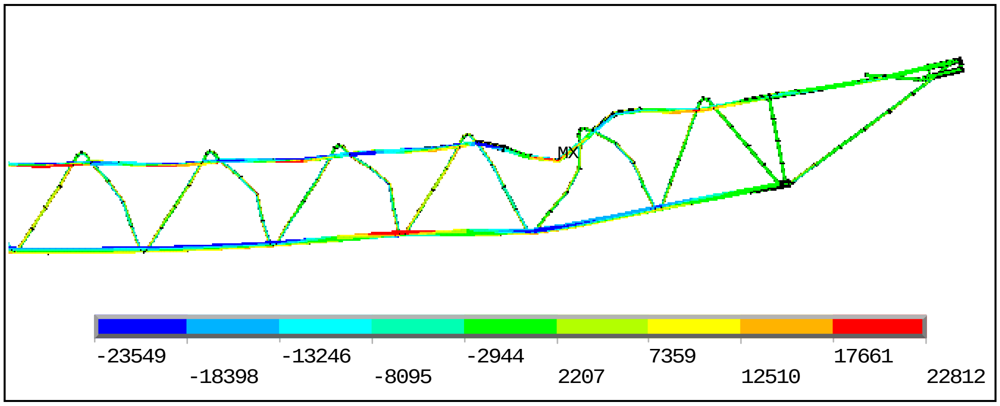
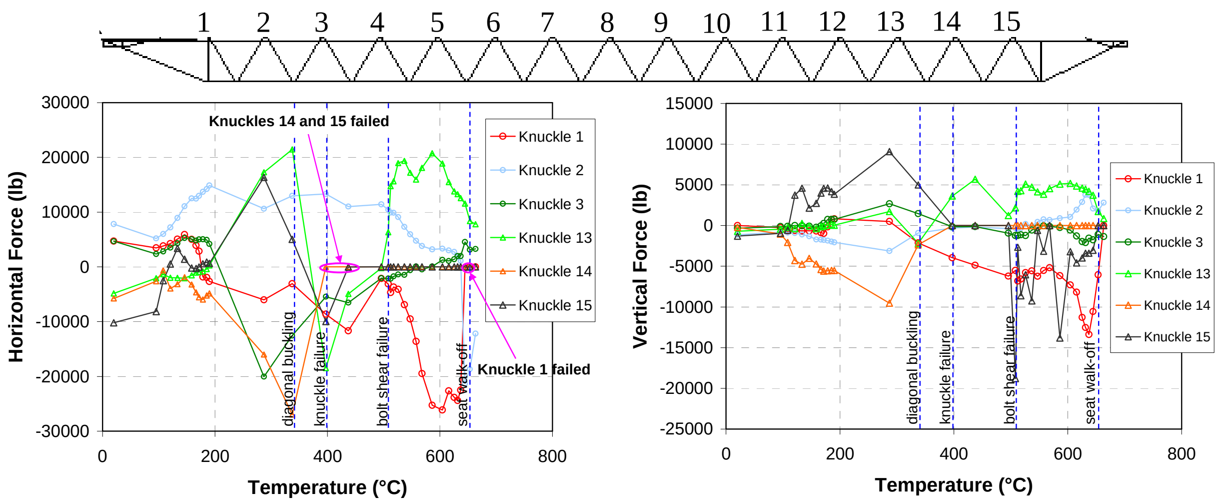


Figure K–42. Axial stress contour in the truss members at 663 °C (displacement magnification factor = 1.0).

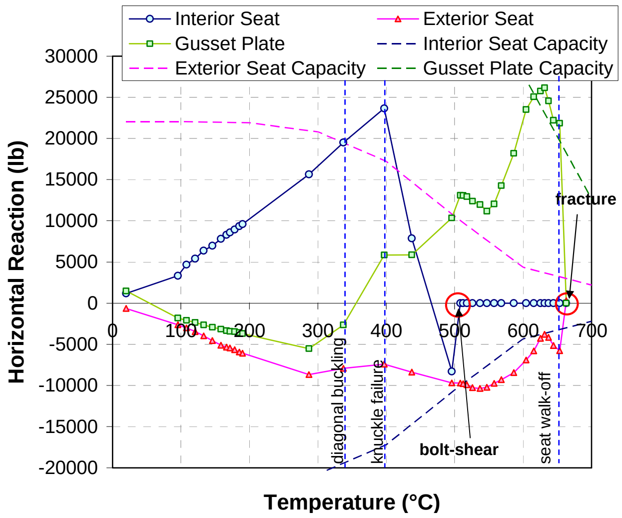
Figure K–43 shows knuckle forces in the y direction (longitudinal truss direction) and the z direction (vertical direction). The capacity of a knuckle in the y direction is assumed to be 30,000 lb, and in the z direction 15,000 lb in tension. Knuckles 14 and 15 fail due to horizontal shear around 400 °C. Knuckle 1 also fails due to the horizontal shear around 650 °C.

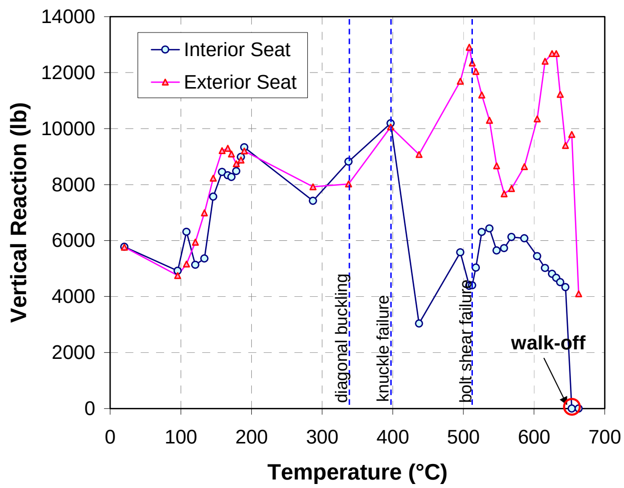


(a) Horizontal Force (b) Vertical Force

Figure K–43. Force in the knuckles versus temperature.

Figure K–44 (a) and (b) show horizontal and vertical reaction forces at seats, respectively. At 510 °C, the interior seat bolt shears off. At 650 °C, the truss walks off the interior seat. At 660 °C, the gusset plate at the exterior end fails in tension.



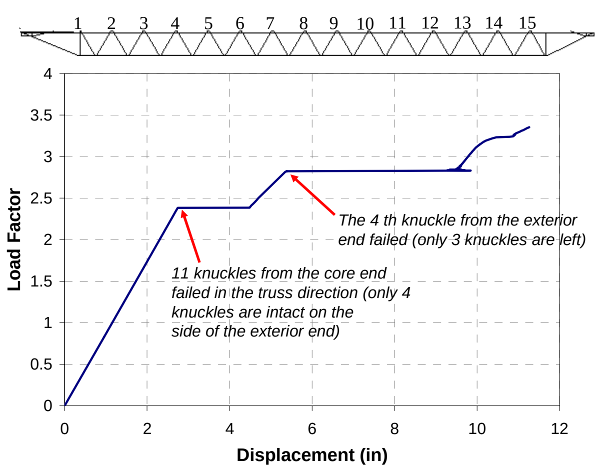


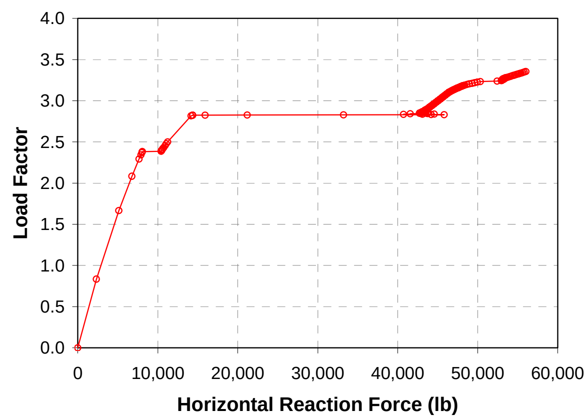
(a) Horizontal reaction force (b) Vertical reaction force

Figure K–44. Reaction forces at seats.

Additional Debris Load

The capacity of the truss model against additional debris load is determined by increasing the gravity loading at room temperature. The analysis is performed with the previous model, where boundary conditions of the slab are as described in the section "Bounding Conditions." Let us define load factor as the ratio of the gravity load plus debris weight to the gravity load, where gravity load includes self weight, superimposed dead load, and 25 percent of the reduced live load. The analysis was terminated at a load factor of 3.4. Figure K–45 (a) shows midspan vertical displacement versus load factor. At 2.4 times the gravity loading, 11 knuckles from the core end fail in the truss direction. At 2.8 times the gravity loading, the fourth knuckle from the exterior end fails. Figure K–45 (b) shows the sum of horizontal reaction forces measured at the exterior columns. Note that seat capacities are not modeled in this analysis.





(a) Load factor versus midspan vertical displacement (b) Load factor versus horizontal reaction force

Figure K–45. Finite element analysis results from increasing gravity.

Summary and Discussion

The truss behavior under the gravity plus thermal loading, where the temperature is ramped up to 663 °C can be summarized as follows:

- Top chords yield above 300 °C due to the difference in CTEs of steel and lightweight concrete.
- Compression diagonals start to buckle in the plane of the truss due to a high axial force and end moments at 340 °C.
- At 400 °C, knuckles start to fail.
- The interior seat bolt shears off at 510 °C.
- The truss walks off the interior seat at 650 °C, followed by fracture of the gusset plate at the exterior end at 663 °C.

The results for the additional debris weight show that the knuckles start failing when the load factor is 2.4. Most knuckles fail before load factor reaches 3.0. After the knuckle failure, the truss loses composite action between the truss and the concrete slab, and the vertical displacement increases significantly. As a result, horizontal reaction force increases.

Models of the truss including knuckles with temperature-dependent capacities, diagonal weld failure, and concrete cracking and crushing are under study.

Simplified Model

To be used in the full floor subsystem model, the truss model will be simplified based on the results from the truss model analysis. Characteristics of the simplified truss model are listed in the following:

- The geometry of the truss will be preserved.
- Pin-ended Link elements will be used for truss members.
- User-defined elements will be used to model failure modes of knuckles, seats, and diagonal members. They will be implemented at the ends of link elements.
- Slab softening or cracking will be incorporated into the model.

K.8 EXTERIOR WALL SUBSYSTEM

The exterior wall subsystem represents the impact zone and includes nine prefabricated wall panels, three panels high by three panels wide.

The exterior wall subsystem model includes nine columns, extending vertically from the column splice located below floor 91 to the column splice above floor 99, and nine spandrels, extending horizontally from the spandrel splice located at mid-span between columns 149 and 150 to the spandrel splice at mid-span between columns 158 and 159, of the WTC 1 exterior wall.

Figure K–46 shows the subsystem pictorially. Tables K–14 through K–16 give the properties of the column component plates, the spandrels, and the column and spandrel splices. Figure K–47 shows pictorially the spandrel plate thickness, nominal yield strengths, and spandrel splice types. Figure K–48 shows the column plates notation used.

The odd numbered columns support floor trusses. Pairs of strap anchors extend diagonally from the top chord of truss pairs to the even numbered columns. The trusses and the straps partially brace the columns both in-plane and out-of-plane of the exterior wall.

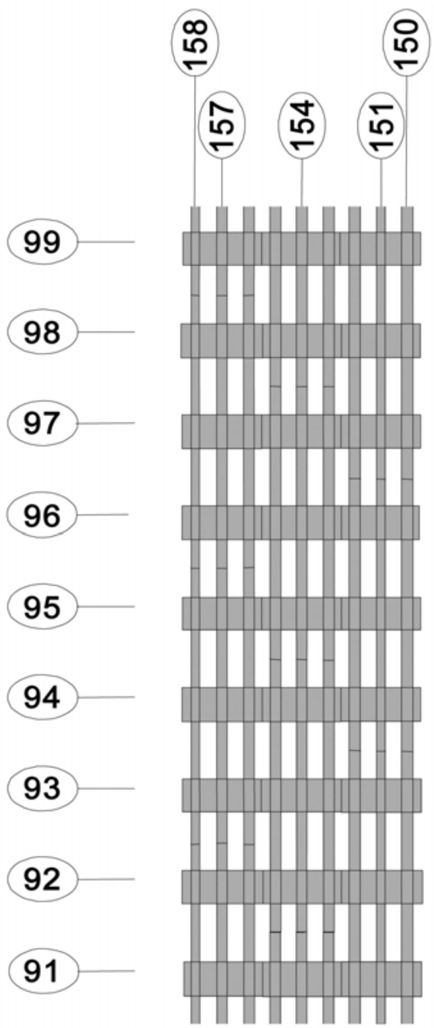
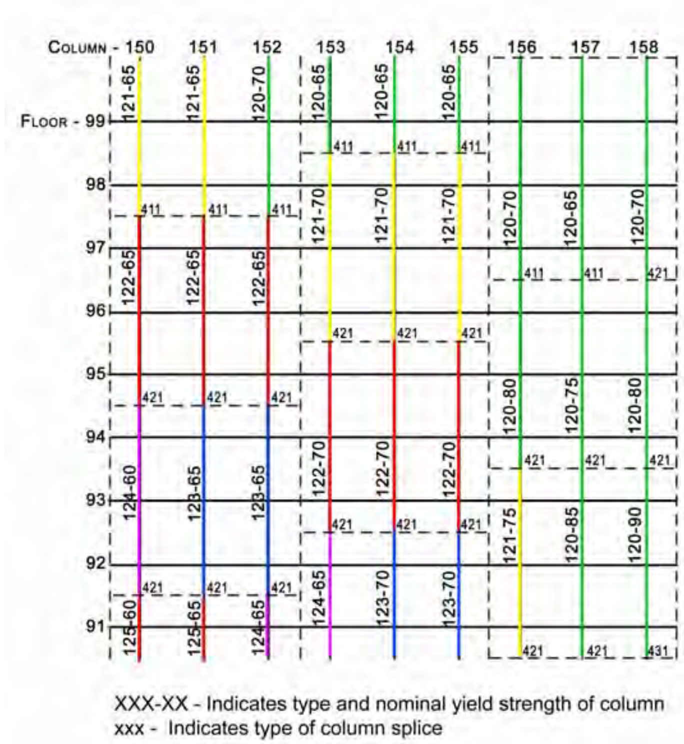
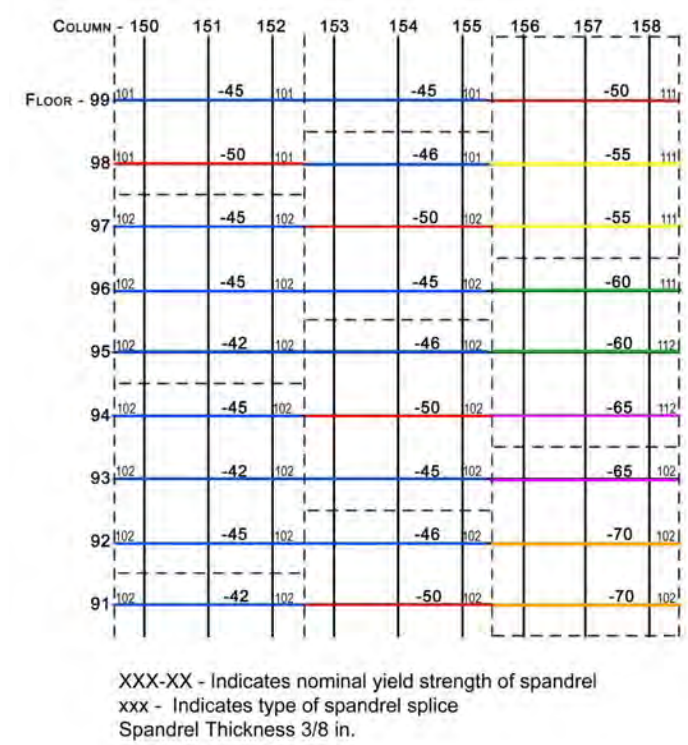


Figure K–46. Exterior wall subsystem structure.



a) Columns and Column Splices



b) Spandrels and Spandrel Splices

Figure K–47. Column and floor number materials and splice types.

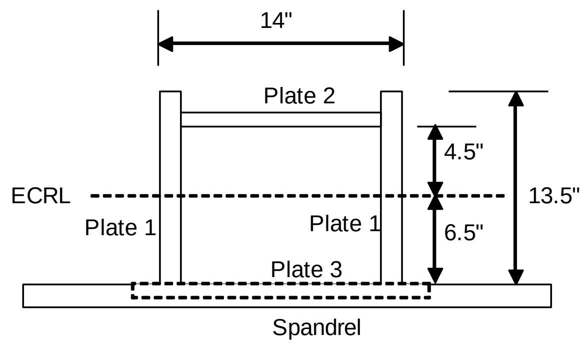


Figure K–48. Spandrels and spandrel splices.

Table K–14. Column sectional properties.

Column Type	Plate 1 l × t	Plate 2 l × t	Plate 3 l × t	Col. Type ID
120	13.5 × 0.25	13.5 × 0.25	15.75 × 0.25	0
121	13.5 × 0.3125	13.375 × 0.25	15.75 × 0.25	1
122	13.5 × 0.375	13.25 × 0.25	15.75 × 0.25	2
123	13.5 × 0.4375	13.125 × 0.25	15.75 × 0.25	3
124	13.5 × 0.5	13 × 0.25	15.75 × 0.25	4
125	13.5 × 0.5625	12.875 × 0.25	15.75 × 0.25	5

Note: All spandrels in wall model are 52 in. deep × 3/8 in. thick.

Table K–15. Spandrel splice details.

Spandrel Splice Type	Number of Bolts/Row	Total Number of Rows	Bolt Spacing	Gage	Overall Splice Plate Dimensions	Bolt to Centerline of Splice	Gap B/W Spandrels	Spandrel Splice ID
101	6	2	5@9		49 × 6.75 ×.25	1.875	0.75	101
102	8	2	3,6,3@ 9,6,3		49 × 6.75 ×.25	1.875	0.75	102
111	6	4	5@9	3	49 × 12.75 ×.25	1.875	0.75	111
112	8	4	3,6,3@ 9,6,3	3	49 × 12.75 ×.25	1.875	0.75	112

a. All spandrel splices use 7/8 in. A325 bolts; spandrel plate yield strength is 36 ksi.
b. Holes in spandrel are 1/4 in. larger than bolts; holes in plates are bolt + 1/16 in. or option to match spandrel holes.

Table K–16. Column splice details.

Column Splice Type	Butt Plate Thickness	Number of Bolts	Bolt Diameter	Gage	Bolt Spacing	Column Splice ID
411	1.375	4	0.875	3.5	6	411
421	1.625	4	0.875	3.5	6	421
431	1.875	4	1	3.5	6	431

a. Butt plates have specified yield strength of 50 ksi.
b. Bolts are A325.

K.8.1 Description of Exterior Wall Subsystem Model

Figure K–49 shows the model in elevation. BEAM189 elements model the columns. SHELL181 plate elements model the spandrels. Figure K–50 shows the number of elements used to model columns and spandrels. MPC184 rigid elements connect the center of gravity of a column to the mid-plane of a spandrel at each shell element. Figure K–51 shows this use of the MPC184. MPC184 rigid elements also model the spandrel connections. A simplified model, consisting of two BEAM189 elements for each of the four bolts, four pairs of CONTA178 contact elements at the faying (contact) surfaces, and MPC184 rigid elements connecting the tops of the bolts to the CONTA178 contact elements, model the column splice. COMBIN37 elements model the fracture of the column splice bolts.

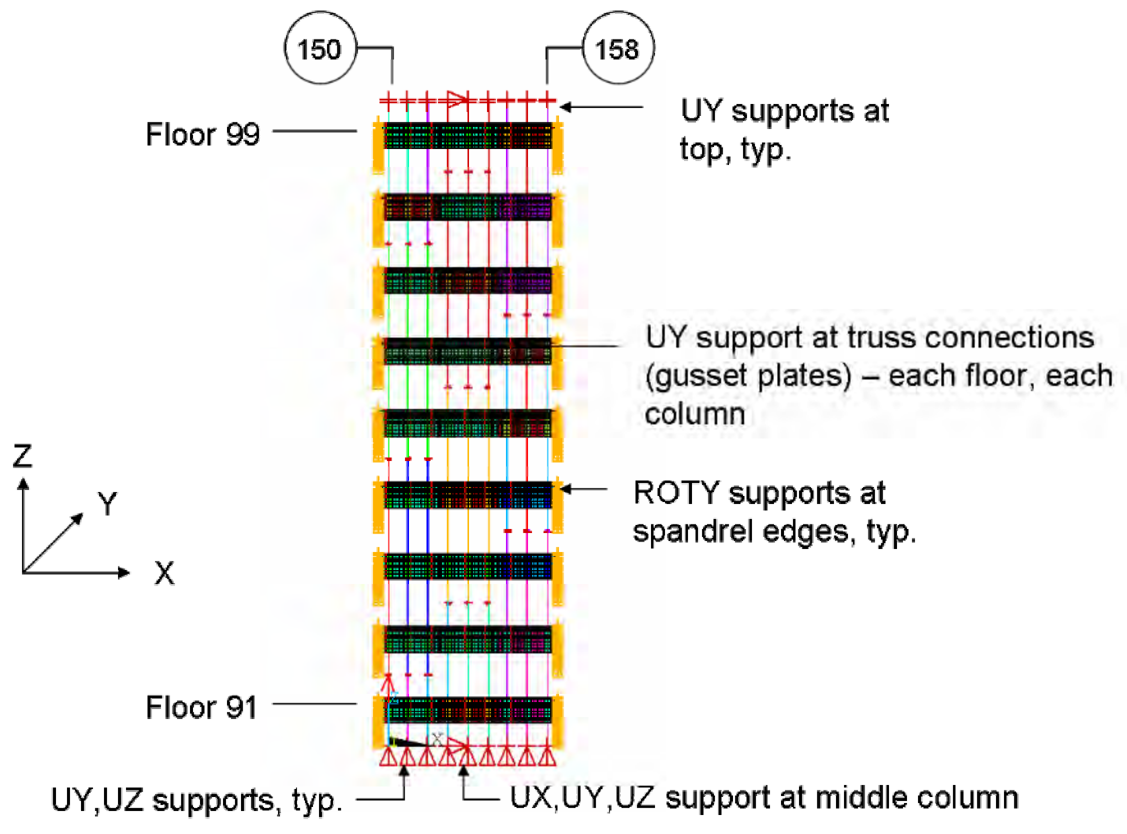


Figure K–49. Exterior wall subsystem model, viewed from inside of WTC 1.

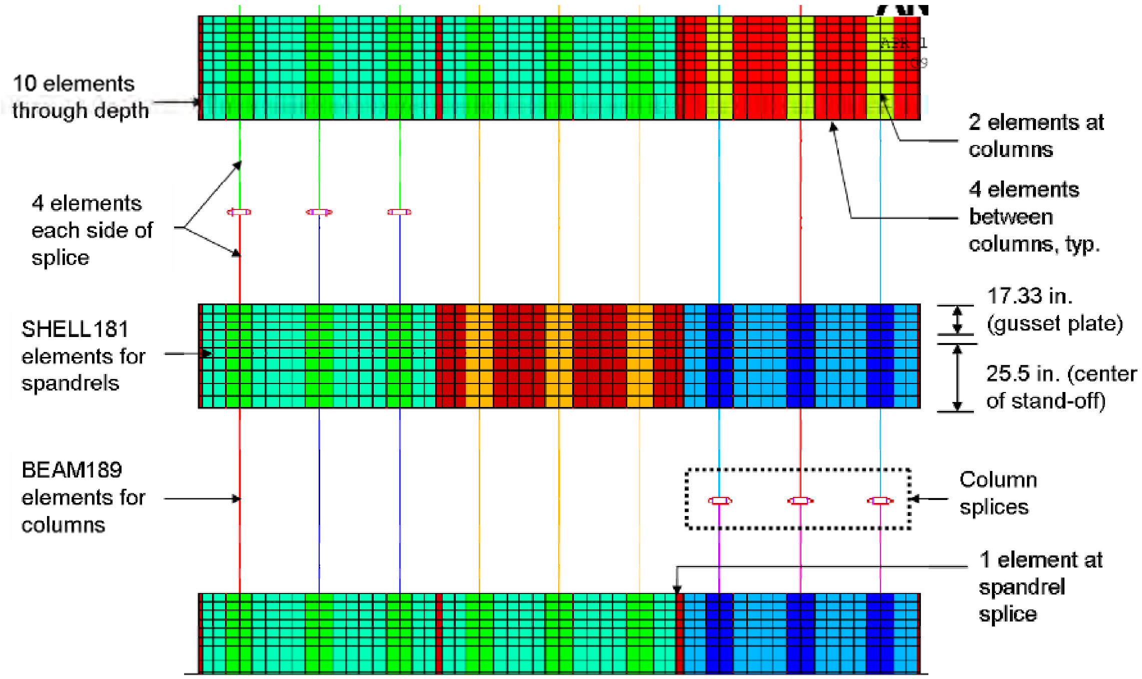


Figure K–50. Portion of exterior wall subsystem model showing number of elements used.

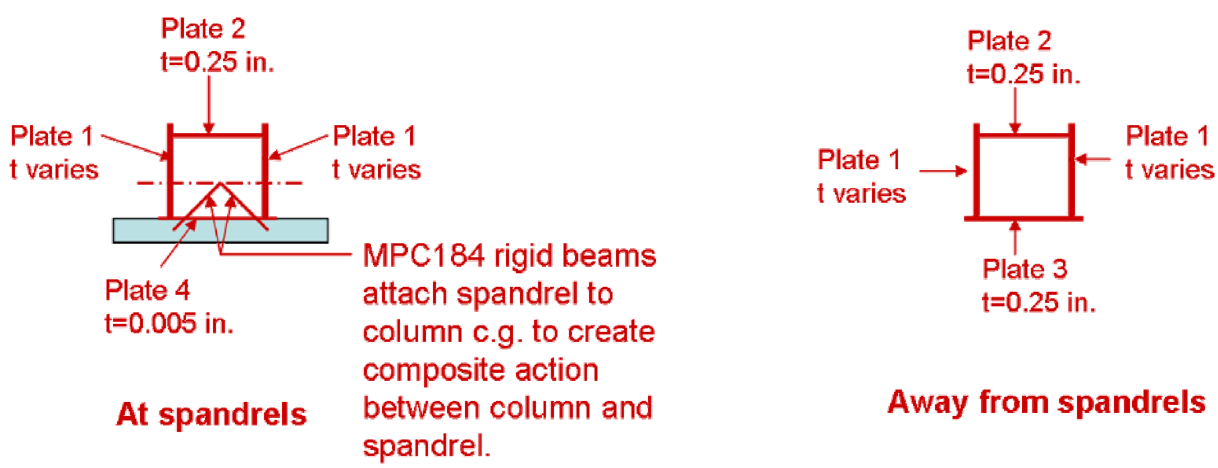


Figure K–51. Schematic representation of columns used in the exterior wall subsystem model.

The capabilities of the BEAM189 and SHELL181 elements include large deflections, plastic deformation, and creep at elevated temperatures. Materials are assigned as described in Section K.5.

The loads on the model include the following:

- Self weight;
- Dead load of floor trusses;

- 25 percent of floor live load;
- Column splice bolt preload; and
- Temperatures of fire scenarios.

A concentrated vertical load and an out-of-the-wall-plane moment due to the dead and live load of the structure above floor 99 load the top of each column. A concentrated vertical load and an out-of-the-wall-plane couple due to the dead and live load of the floor truss load each odd numbered column at the truss seats. Mean temperature at the center of gravity of the column and a linear gradient in each of two directions through the section of the column strain the BEAM189 elements at each node. Temperatures at the nodes strain the SHELL181 elements. Loads and/or deflections at the truss seats model the outward motion or the caternary action of the floor truss due to fire scenarios. The 7/8 in. diameter column splice bolts are preloaded with 36.05 kip (AISC 1964).

Simple supports out of the plane of the wall restrain the tops and the bottoms of all columns in the model. In addition, supports horizontally in the plane of the wall restrain the top and the bottom of central column 154. Simple supports in the vertical direction restrain the bottoms of all columns in the model. Symmetry conditions are imposed on the spandrels at the extremities of the model, except that the spandrels are free to expand in the plane of the wall. In the plane and out of the plane of the wall restraints brace the column at floor truss seats and diagonal straps.

The model captures the following failure modes:

- Column collapse due to large lateral deformations;
- Column buckling due to loss of bracing at floor truss seats and diagonal straps;
- Failure of column splice bolts; and
- Failure of spandrel splice bolts.

The model does not capture the local buckling of the column plates and the formation of plastic hinges due to the interaction of local plate buckling and high stresses in the column from axial load and bending moments. Section K.8.4 below includes the justification for excluding this structural behavior from the wall subsystem model.

K.8.2 Validation of the Exterior Wall Subsystem Model

The behavior of models of the following components of the exterior wall subsystem validate the exterior wall subsystem model:

- Model of a one-story-high exterior column.
- Model of a nine-story-high exterior column.

- Detailed and simplified models of the column splice.
- SAP2000 and ANSYS models of a prefabricated wall panel.

K.8.3 Model of One-Story High Exterior Column

Figure K–52 shows the model of a one-story-high exterior column. The model includes a one-story-high portion of column 151 extending from floor 95 to floor 96 and portions of spandrels at floor 95 and floor 96. The model also represents column 151 from floor 96 to floor 97 since the dimensions, plate thicknesses and material properties are identical to those of column 151 from floor 95 to floor 96. SHELL181 plate elements model the plates of the column and the spandrels. CERIG rigid elements connect the center of gravity of the column to its component plates and the spandrel at both the top and bottom of the model. The column is simply supported in three directions at the bottom and simply supported in the horizontal direction at the top. Increments of axial displacement applied at the top load the model.

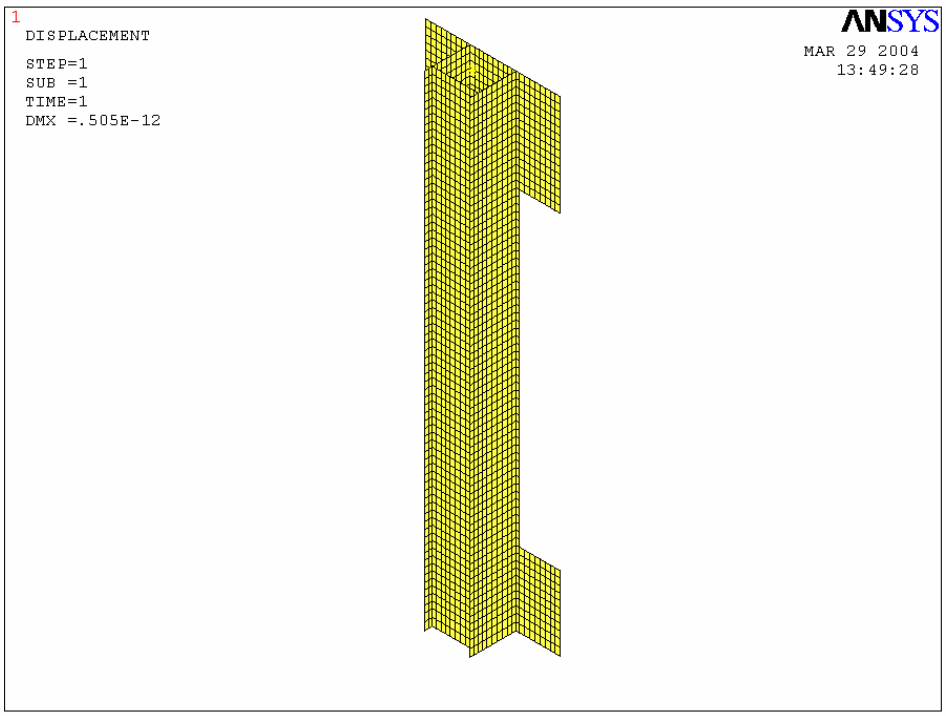


Figure K–52. One-story exterior column model.

Figure K–53 shows the variation of axial load with enforced axial displacement and resulting lateral deflection at room temperature and 700 °C. This figure also shows the hand calculated column load levels at room temperature and 700 °C for:

- Local buckling of Plate 2 and Plate 3;
- Uniform yielding of the column; and
- Axial load due to dead and live load at floor 96 in the exterior wall subsystem model.

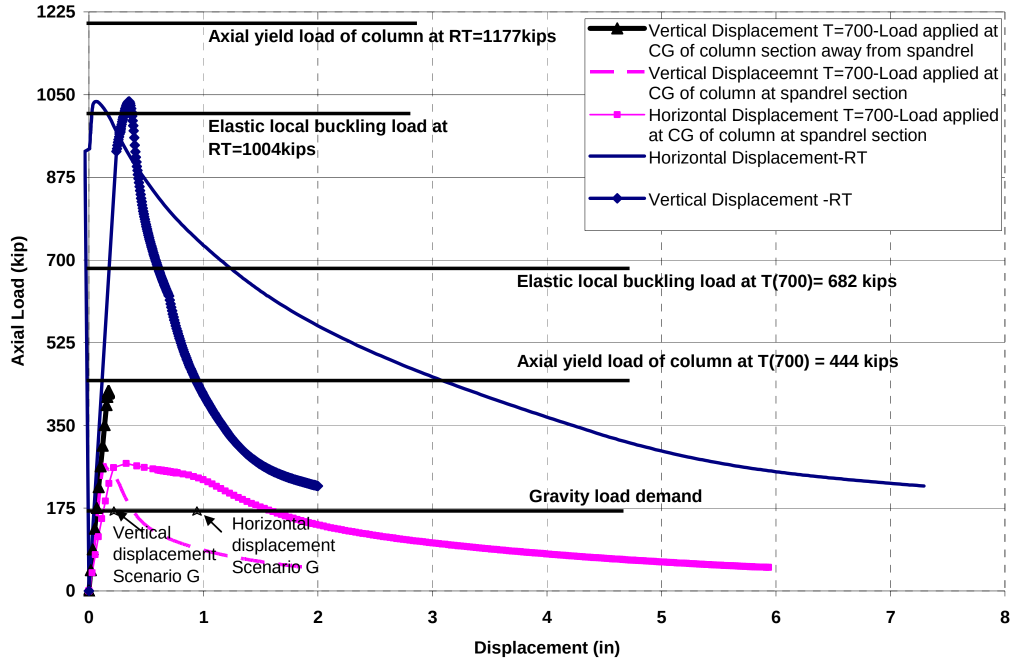


Figure K–53. Load-deflection of column at room temperature and 700 °C.

Figure K–54 shows the local bucking deformation of Plate 2 and Plate 3 at the maximum load level. Figure K–55 shows a plastic hinge at mid-height of the column for an axial displacement of 2 in. Figure K–56 shows the presence of local buckles in Plate 2 and Plate 3 at the maximum load.

Figure K–53 shows that at room temperature Plate 2 and Plate 3 buckle locally at a load that is less than the maximum column load, but that at 700 °C the column yields before it buckles locally. This figure also shows that the expected column demand load of 175 kip is substantially lower than the local buckling load at room temperature and the column yield load at 700 °C. For these results, the axial displacement was applied along the center of gravity of the column cross section away from the spandrel. If axial displacement is applied at center of gravity of the column cross section at the spandrel, there will be additional bending moment in the column section away from the spandrel. The presence of moments reduces the axial load capacity of the column. The resulting load-deflection diagram is also shown in Fig. K–53.

K.8.4 Model of Nine-Story High Exterior Model

Figure K–57 shows the nine-story-high exterior column model. The model includes column 151 extending from near mid-height between floor 91 and floor 92 to mid-height between floor 100 and floor 101, spandrels at floors 92 through 100, and column splices located at the mid-height between floors 94 and 95 and floors 97 and 98. SHELL181 plate elements model the plates of the column, the spandrels, the butt plates at the column splice, and the stiffeners. BEAM189 elements model the column splice bolts. CONTA174 and TARGE170 elements model the faying surfaces of the column splice. MPC184 rigid elements connect the tops of the bolts to the butt plates. At the bottom the column is restrained from displacement and rotation in all three directions. At the top the column is restrained from translating in the horizontal directions and from twisting.

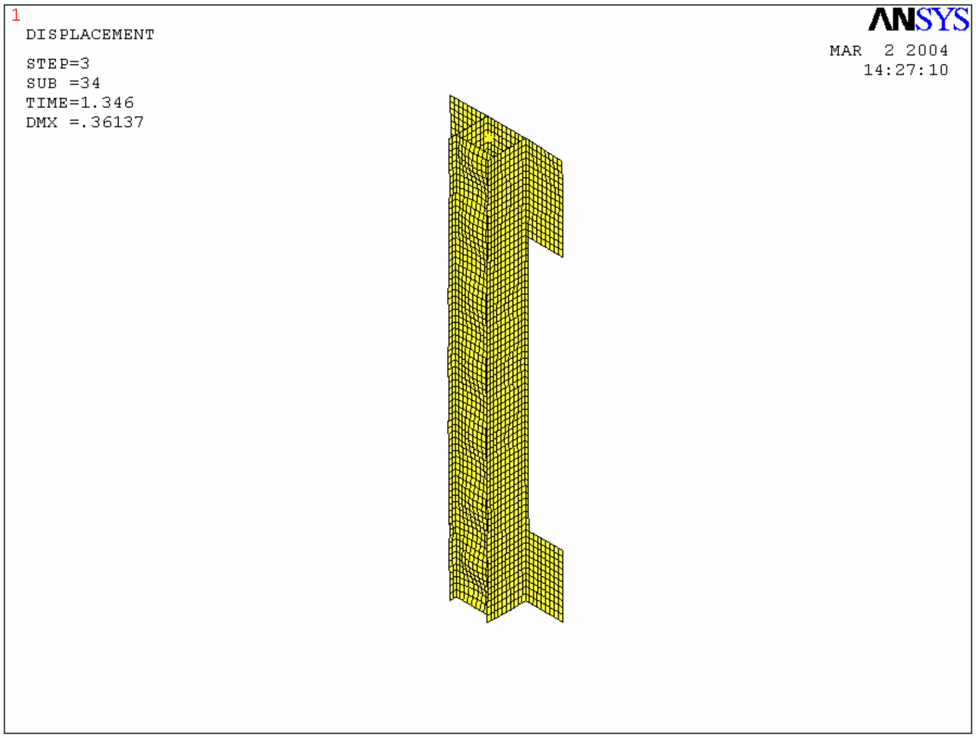


Figure K–54. Local buckling of column at room temperature.

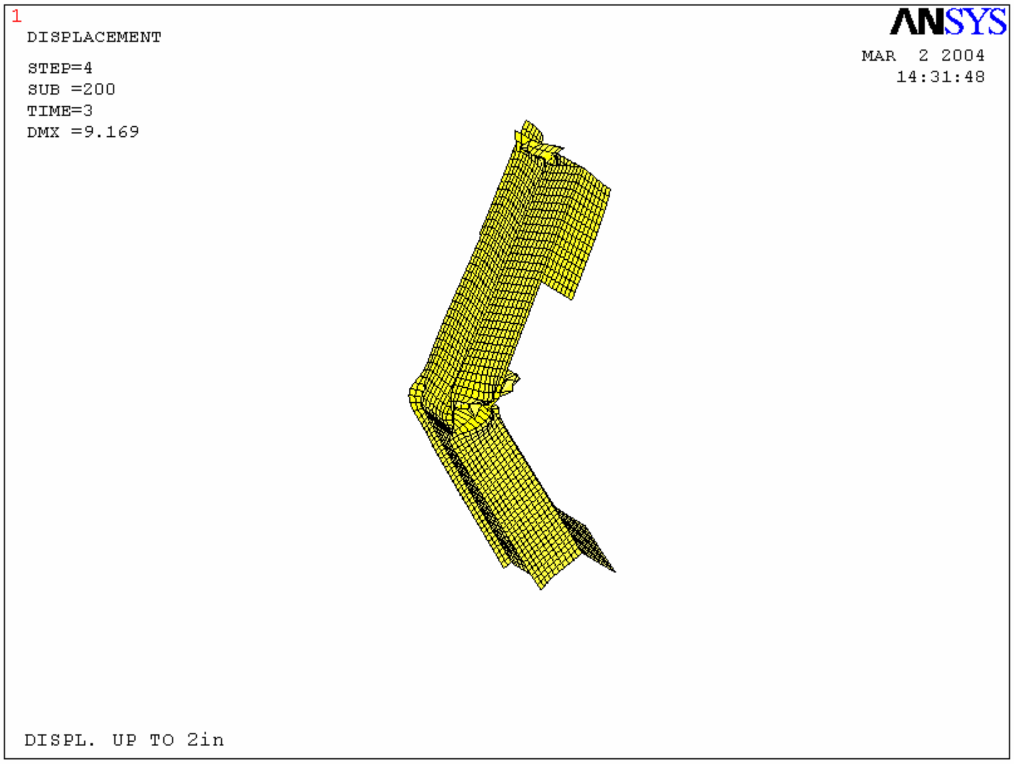


Figure K–55. Plastic hinge in column at room temperature.

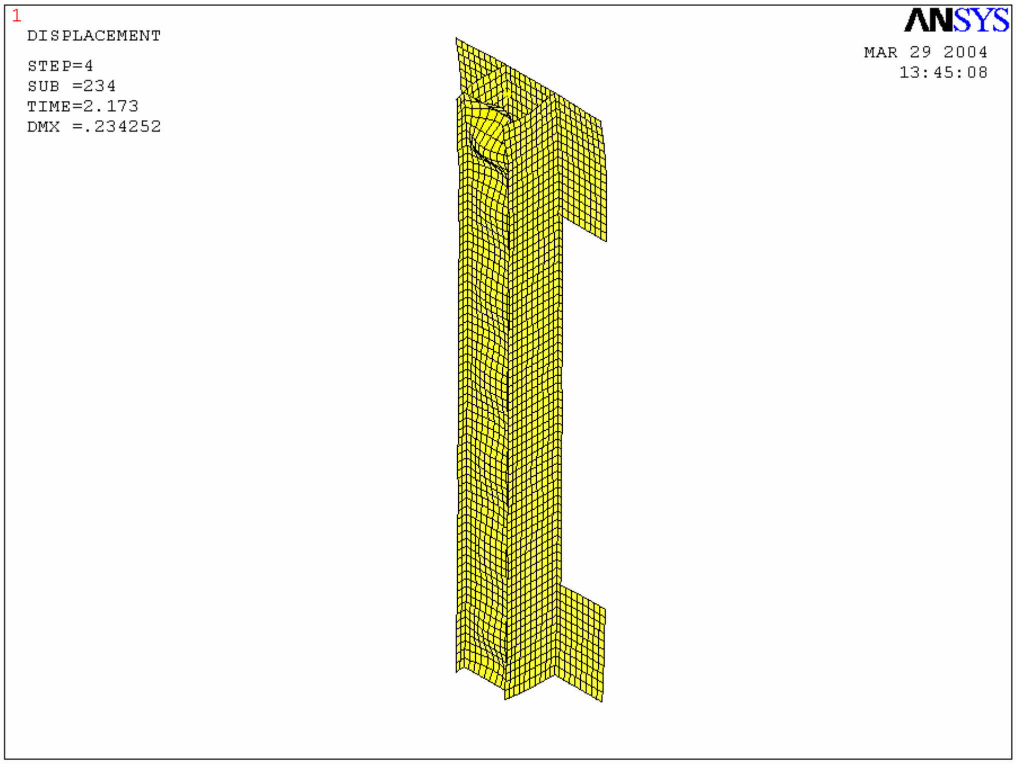


Figure K–56. Deformed shape of column at maximum axial load at 700 °C.

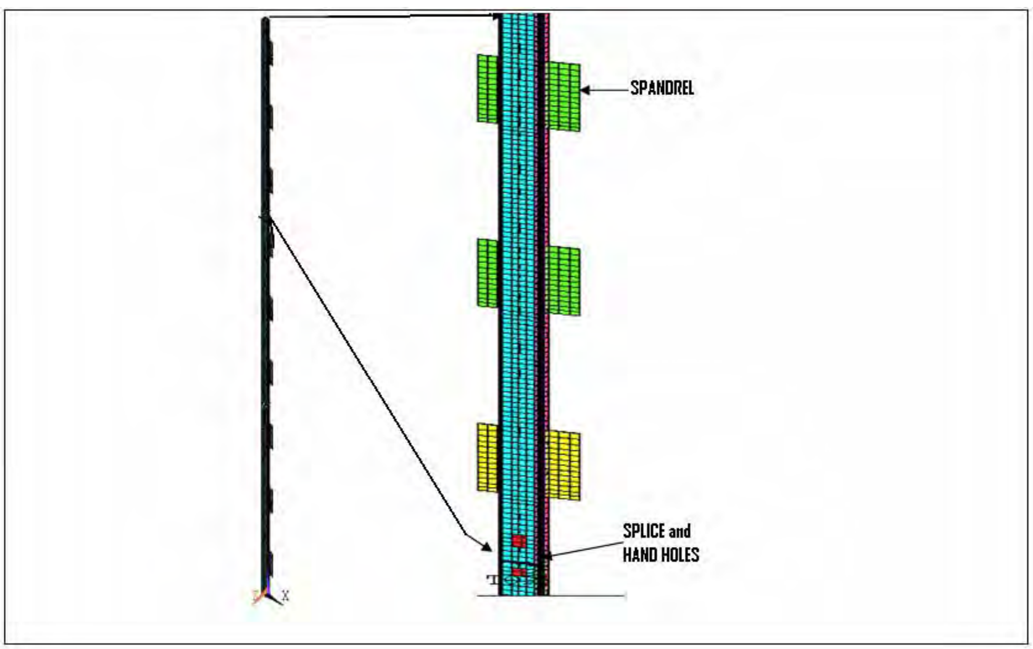


Figure K–57. Nine-story column model.

The capabilities of the BEAM189 and SHELL181 elements include large deflections and plastic deformation. For these elements, gives the material property identification numbers, which in turn are described in Chapter 4 above.

The loads on the model include the following:

- Self weight;
- Dead load of floor trusses;
- 25 percent of floor live load;
- Column splice bolt preload; and
- Temperature of Fire Scenario G.

In Fire Scenario G, the fire starts on floors 95, 96, and 97 and spreads to floors 93 through 98. Gas temperature reaches 1,100 °C. Convection cools the outside face of the column. Radiation heats the other three faces. The inside face of the column is not fireproofed. Temperatures are provided at 200 s intervals up to 5,000 s. Figure K–58 shows the variation of the maximum temperature anywhere in the column with time and the yield stress at the point of maximum temperature. The temperature reaches a maximum of 706 °C at 5,000 s.

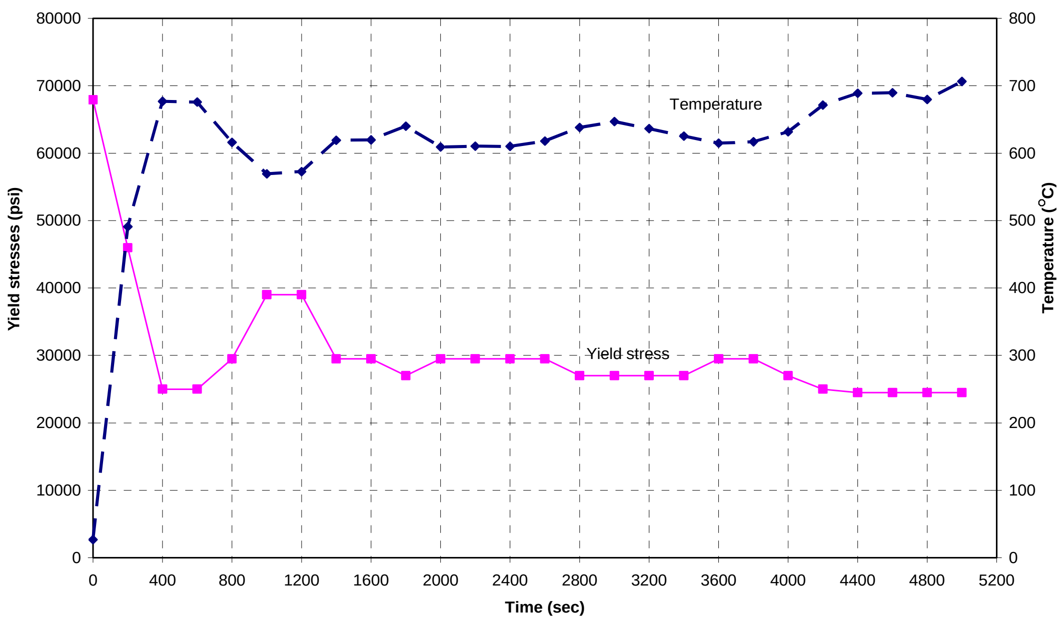


Figure K–58. Variation of maximum temperature and corresponding yield stress with time, fire scenario G.

To account for the dead and live load of the structure above floor 100 and of the floors that connect to the column, concentrated vertical loads and bending moments about a horizontal axis in the plane of the wall are applied to the top of the column and at all truss seats. Furthermore, the 7/8 in. diameter column splice bolts are preloaded to 36.05 kip (AISC 1964).

Figure K–59 shows the variation of maximum tensile and maximum compressive stresses with time and the corresponding yield stress. Figure K–60 shows the deformed shape of the column at 400 s when the compression stress in the column is a maximum. Figure K–60 also shows the deformed shape of the column at 3,200 s when the tensile stress in the column is a maximum. Figure K–60 also shows the deformed shape of the column at 5,000 s when the temperature in the column is a maximum.

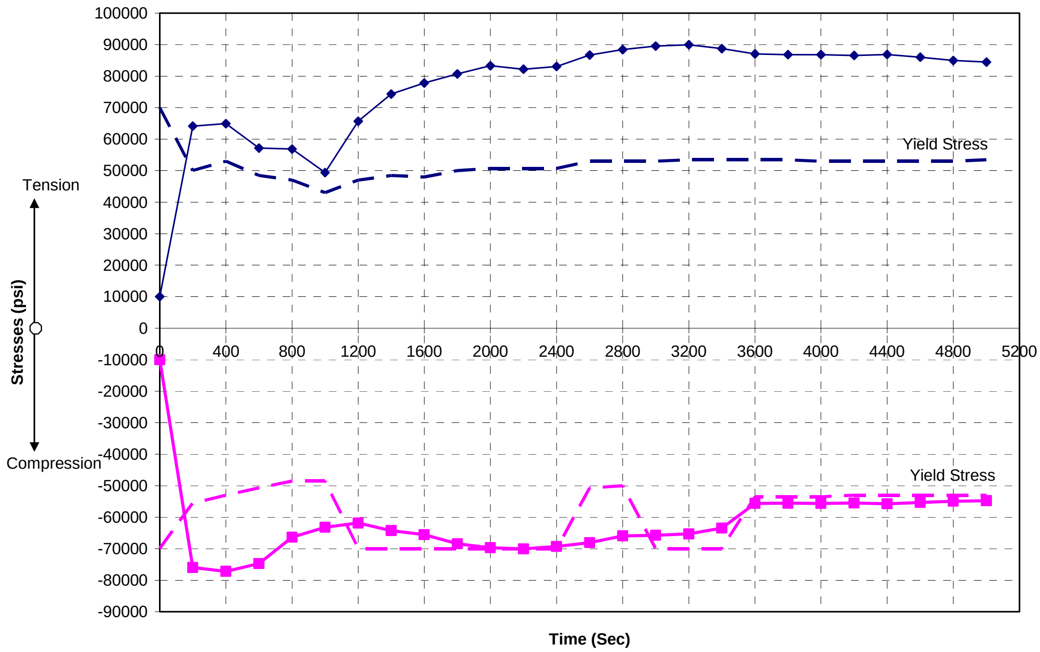


Figure K–59. Maximum compressive and tensile axial stress and corresponding yield stress with time, fire scenario G.

Figure K–59 shows that the tensile and compressive stresses exceed the yield stress for most times during the duration of the fire.

Figure K–60 shows that for Fire Scenario G, an extreme scenario that assumes no fireproofing on the inside face of the column, plastic hinges do not form in the column. This justifies the exclusion of local buckling of the column plates from the wall subsystem model.

K.8.5 Models of the Column Splice

The plate model of the column splice, shown in Fig. K–61 includes a 92 in. tall section of the nine-story column model centered on the column splice located below floor 98.

Figure K–62 shows a simplified model of the column splice. The simplified model consists of two BEAM 189 elements for each of the four bolts, four pairs of CONTA178 contact elements at the faying surfaces, and MPC184 rigid elements connecting the ends of the bolts to the CONTA178 contact elements. A BEAM189 element extends from each side of the splice to match the length of the plate model. Figure K–63 shows the details that model the faying surfaces of the splice.

Figure K–60. Deformed shape of column at 400 s, 3,200 s, and 5,000 s (floors 95–97).

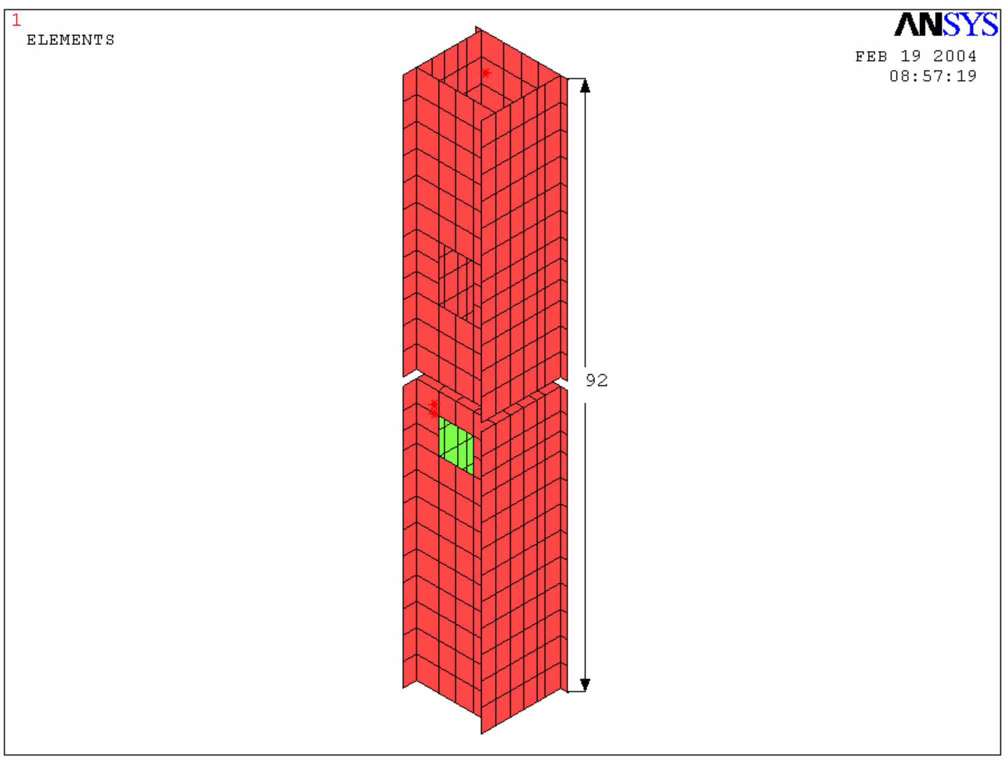


Figure K–61. Plate model of column splice, floors 97-98.

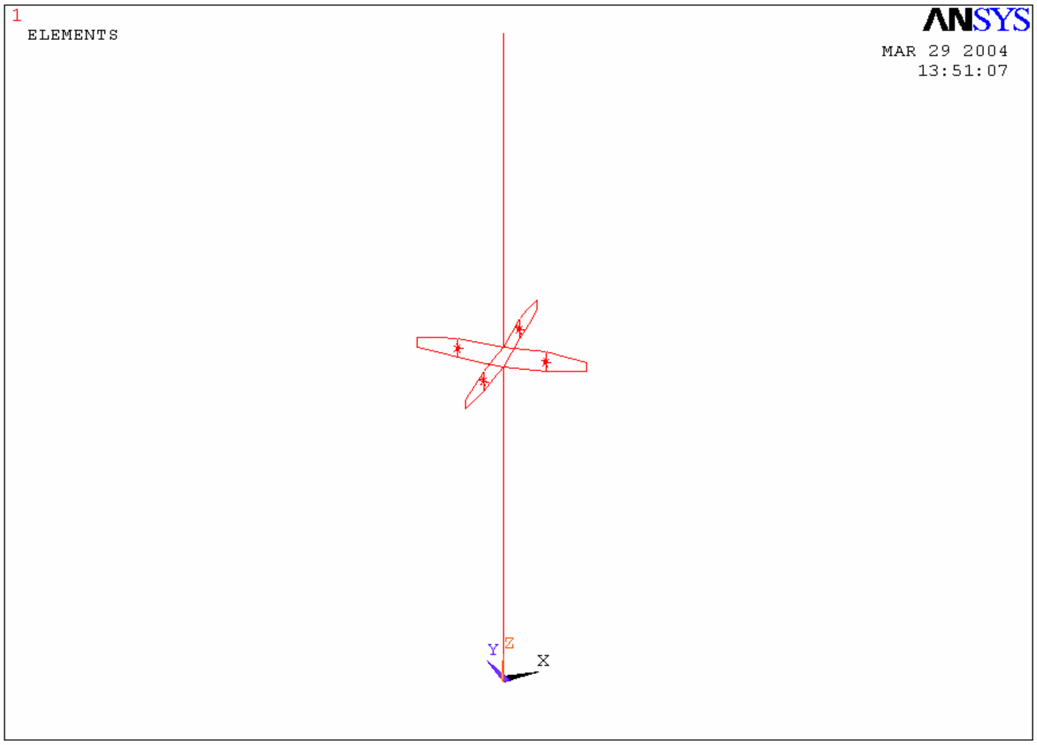


Figure K–62. Simplified model of column splice.

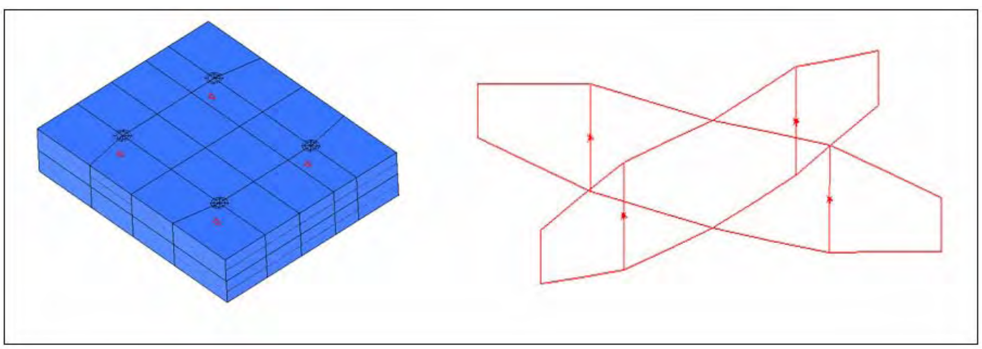


Figure K–63. Column splice details, plate model and simplified model.

Both models are subjected to the following loads:

- Axial tension;
- Shear transverse to the plane of the wall;
- Moment out of plane of the wall;
- Moment in plane of the wall; and
- Torsion.

Figure K–64 shows the variation of axial displacement with axial force load. Figure K–65 shows the variation of transverse displacement with transverse shear fore. These figures show excellent agreement of the simplified model with the plate model.

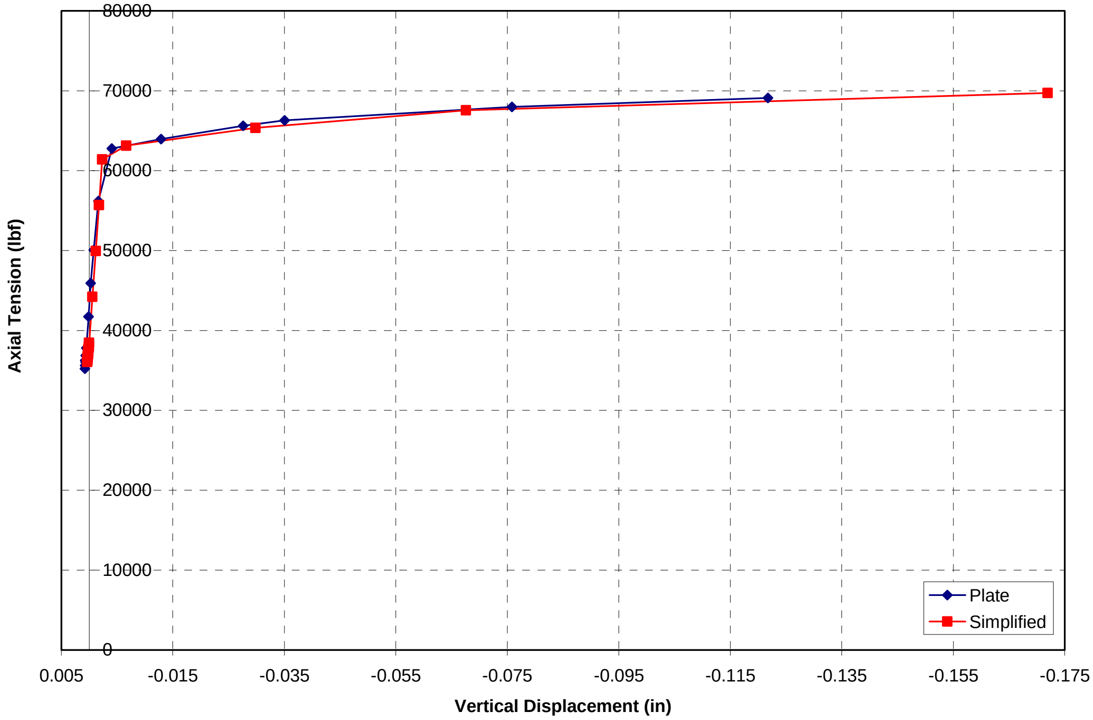


Figure K–64. Variation of axial displacement with axial load.

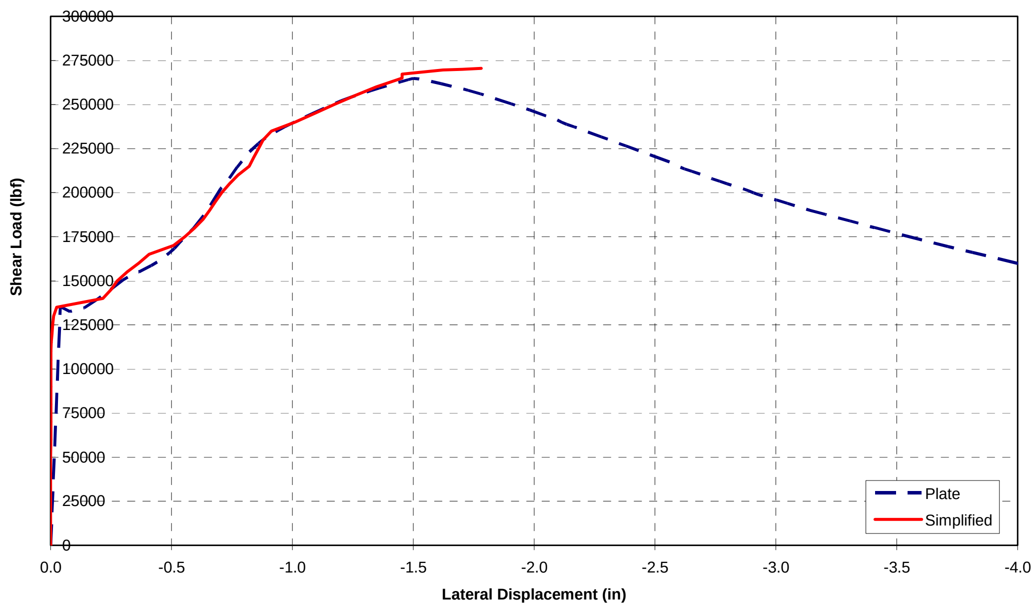


Figure K–65. Variation of lateral displacement with shear load.

Figure K–66 shows the rotation variation with out-of-plane of the wall moment. Figure K–67 shows the rotation variation with in-plane-of-the-wall moment. Figure K–68 shows the twist variation with torque. Figures K–66, K–67, and Fig. K–68 show large differences between the results of the simplified and plate models of the column splice. These differences are due to the fixed locations of pivot points in the simplified model, provided by pairs of CONTA178 point-to-point contact elements, about which the faying surfaces rotate. The CONTA174 and TARGE170 surface contact elements for the faying surfaces in the plate model permit the location of the pivot point to adjust to the demand of the applied moment. Adjusting the location of the point-to-point contact elements can minimize these differences, but they cannot be eliminated. In the exterior wall subsystem model, the locations of the single point contact elements in the column splices will be adjusted and the sensitivity of the response of the model results to these locations computed.

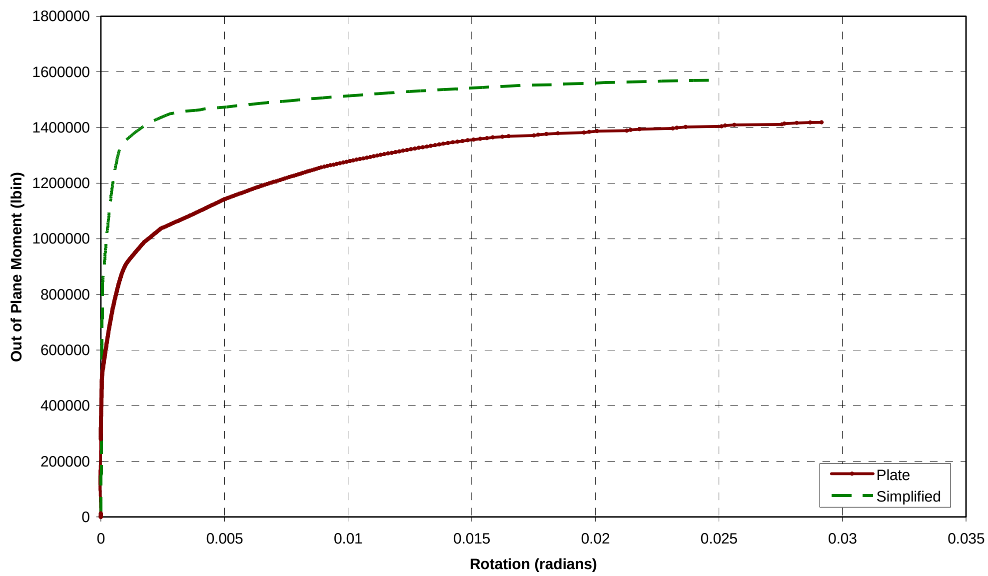


Figure K–66. Variation of rotation with moment, out of wall plane.

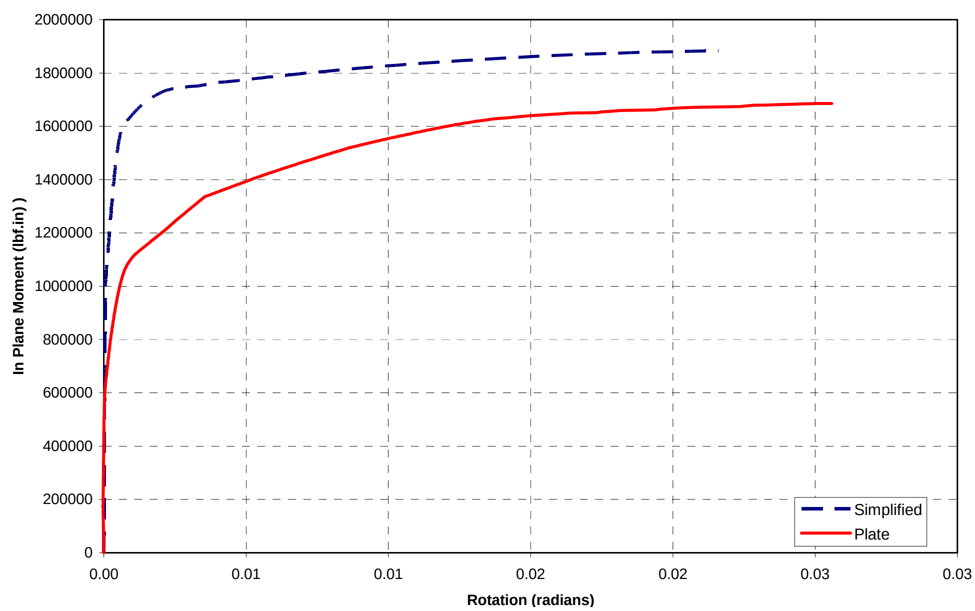


Figure K–67. Variation of rotation with moment, in plane of wall.

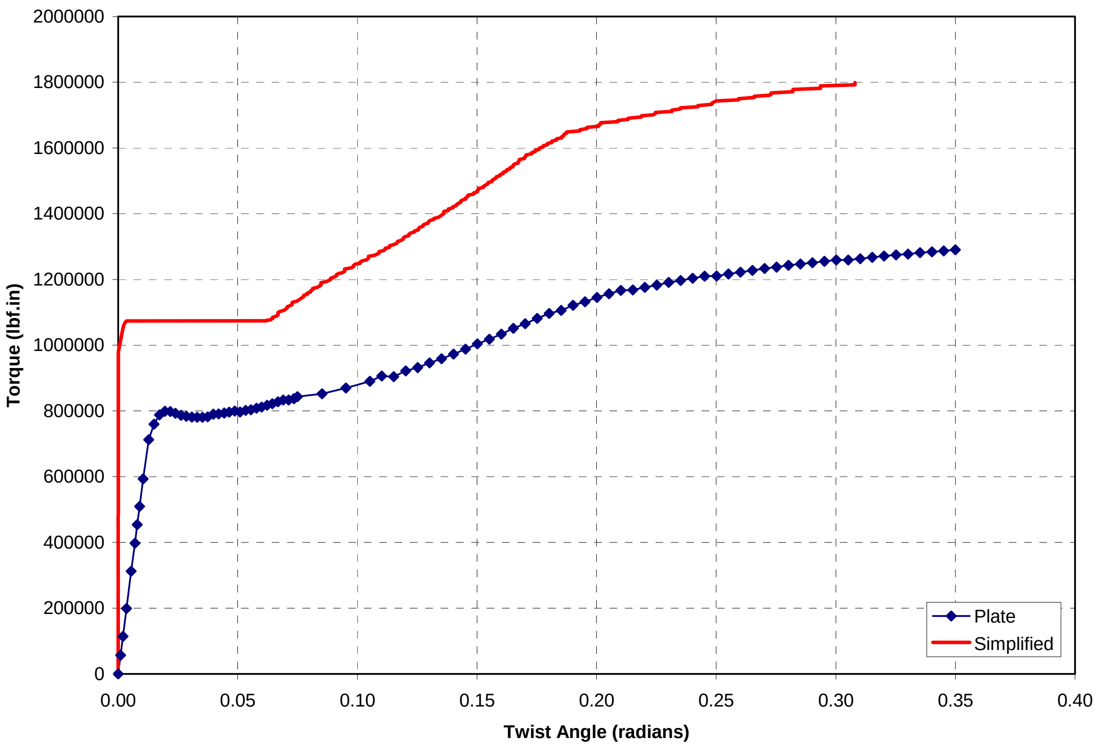


Figure K–68. Variation of twist angle with torque.

K.8.6 Prefabricated Panel Model

Description of Model

Figure K–69 shows the SAP2000 model of a typical prefabricated panel at floors 79 to 82. The model is modified as follows:

- Eliminated self-weight from loading conditions.
- Provided stiff members at the tops of the columns and replaced the four concentrated loads with a single concentrated load.
- Added out-of-plane of the wall supports (UY) at top of columns for out-of-plane loading.

Figure K–70 shows the ANSYS model for matching the behavior of the SAP2000 exterior wall subsystem model. In the ANSYS version of the panel model BEAM189 elements model the columns and MPC184 rigid elements attach the spandrels to the columns.

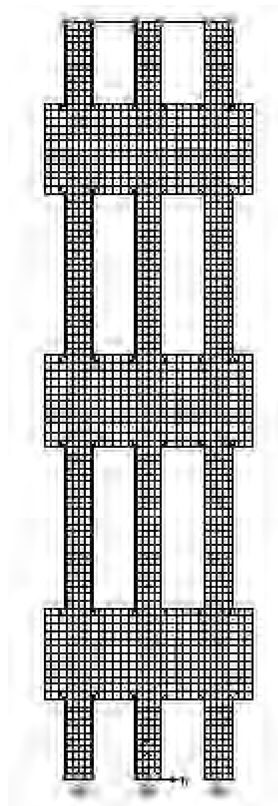


Figure K–69. SAP2000 model of prefabricated panel.

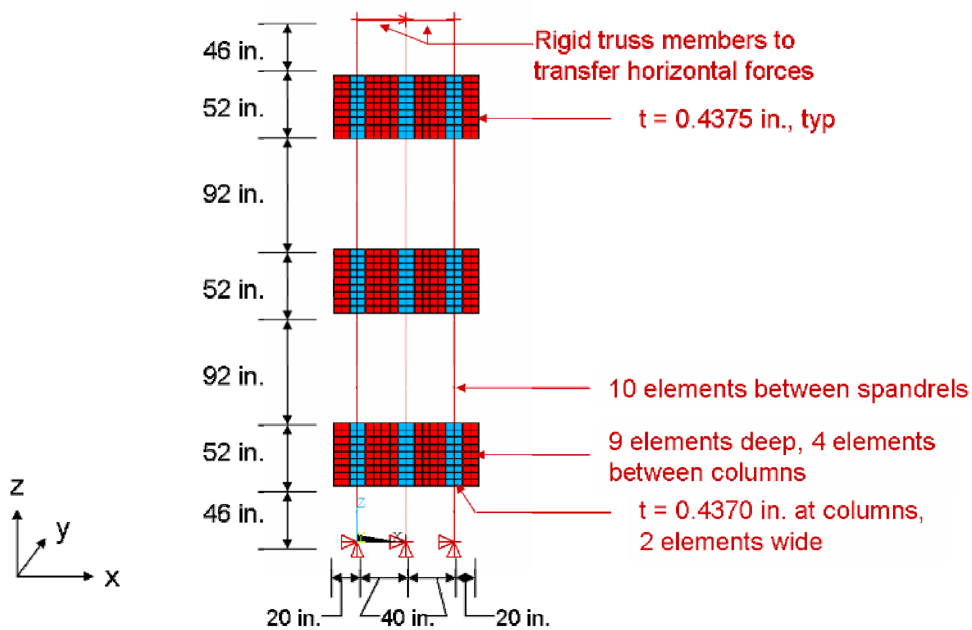


Figure K–70. ANSYS model of prefabricated panel showing geometry and number of elements used.

Both models are subjected to the following loadings at room temperature:

- A concentrated vertical load (FZ) at the top of one of the outside columns.
- A concentrated horizontal load in the plane of the wall (FX) at the top of one of the outside columns. The stiff members described above distribute this shear load evenly to the tops of all three columns.
- A concentrated transverse load (FY) on the middle column at floor 81.

The above loadings do not include self-weight. Figure K–71 shows the various loadings applied to the ANSYS model.

Simple supports in the plane and out of the plane of the wall (UX,UY) restrain the tops of the columns. Simple supports in all three directions restrain the bottoms of the columns. The spandrels at the extremities of the model are free. See Figure K–71.

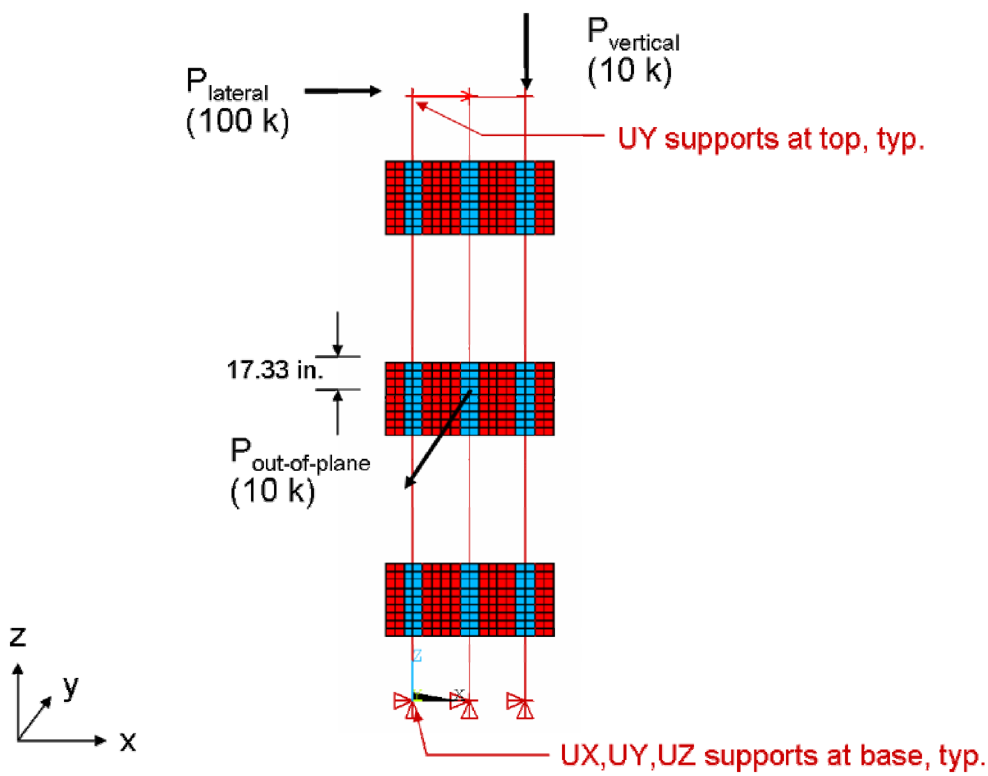


Figure K–71. ANSYS model of prefabricated panel showing loading and boundary conditions.

Validation Results

Figures K–72 through K–74 show deflected shapes and indicate the displacement at the points of applied load for the SAP and ANSYS models. Table K–17 summarizes the differences in reactions and displacements between the SAP and ANSYS models.

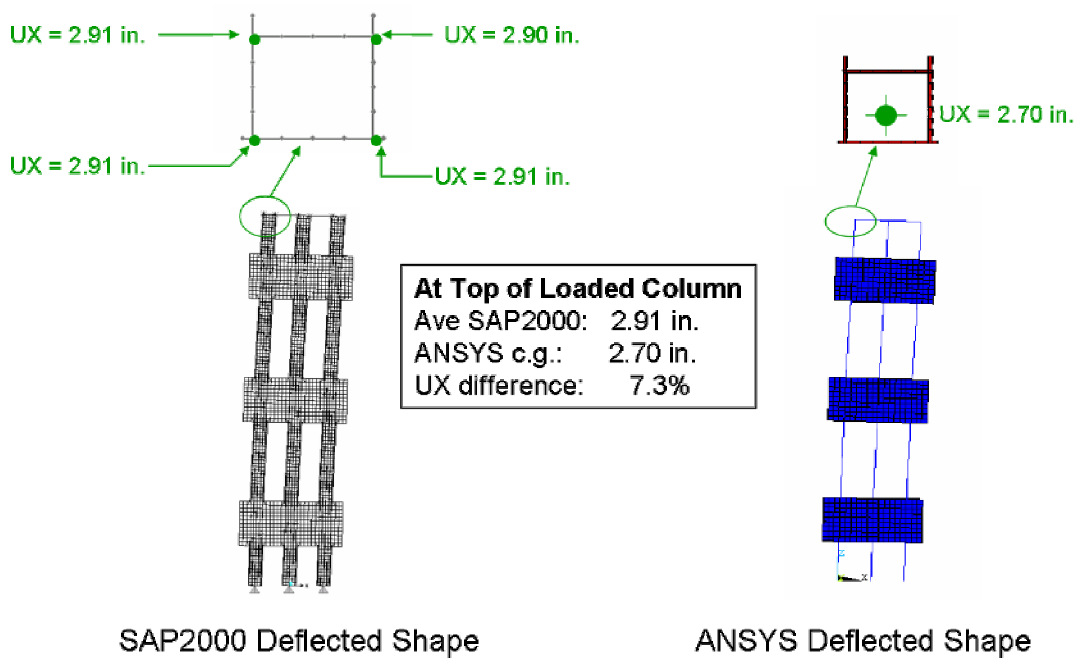


Figure K–72. Deflection of prefabricated panels under 100 kip lateral load.

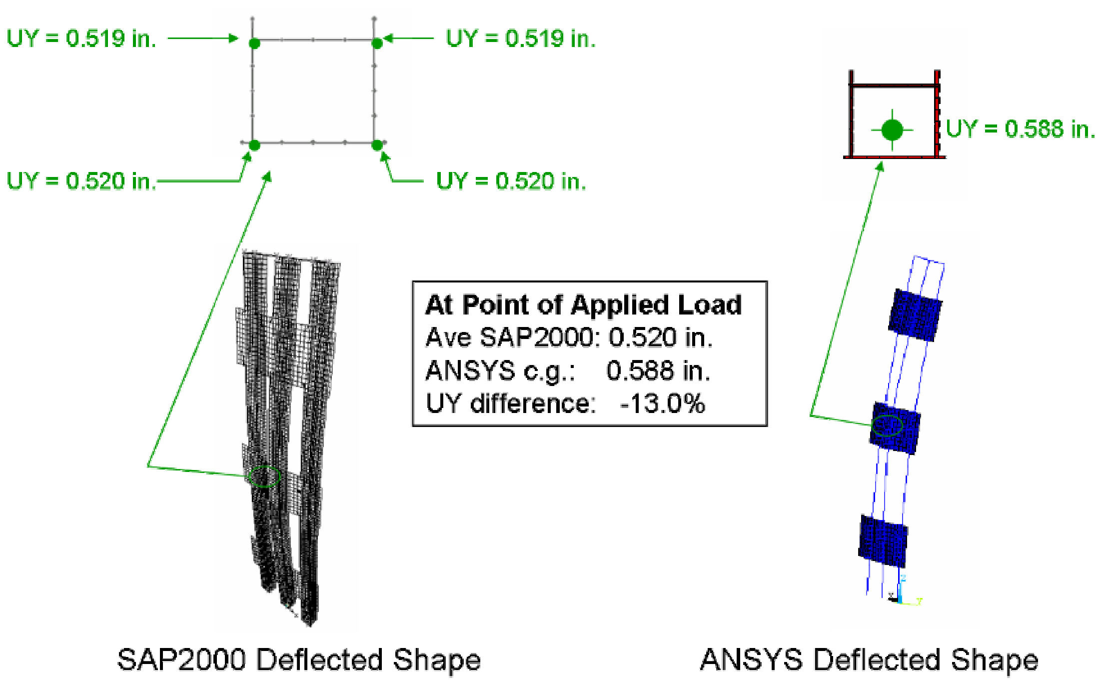


Figure K–73. Deflection of prefabricated panels under 100 kip transverse load.

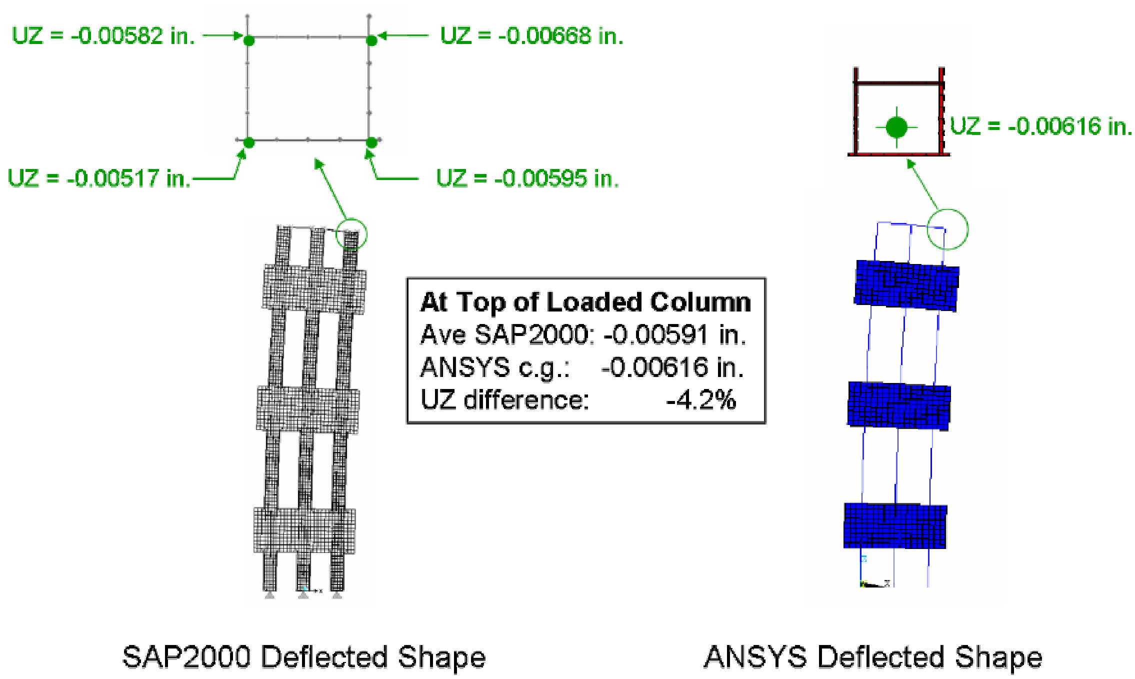


Figure K–74. Deflection of prefabricated panels under 10 kip vertical load.

Table K–17. Validation results.

Loading Condition	SAP2000/ANSYS Displacements[a] Difference Range	
Lateral FX	RX: -2 % to +1 %	UX: 7 %
Transverse FY	RY: -6 % to +7 %	UY: -13 %
Vertical FZ	RZ: -1 % to +2 %	UZ: -4 %

a. Displacements considered at tops of columns for FX and FZ, and at points of load application for FY.

K.8.7 Ongoing Work on the Exterior Wall Subsystem Model

The ongoing work includes the following:

- Stability of a two-story-high exterior column unbraced at the middle floor.
- Stability of a three-story-high exterior column braced at the top and bottom floor levels only.
- Stability of nine-story-high exterior column (floor 92 to 100) unbraced at floors 96 and 97 and subjected to fire scenarios.
- Response of exterior wall model to fire scenarios

K.9 FLOOR TRUSS DYNAMIC RESPONSE DUE TO IMPACT OF DROPPING FLOOR

K.9.1 Impact of Dropping Floor

The failure of dropping floor may occur due to thermal response and/or additional debris weight on the truss, and/or as a result of the aircraft impact. A floor truss or a group of floor trusses could lose support at both the exterior and interior supporting ends and drop onto the floor below. This failure mode, which is shown in Fig. K–75, will be referred to as "full truss drop." Alternatively, a floor truss or a group of floor trusses could lose support on one side and drop down to impact the floor below. This failure mode, which is also shown in Fig. K–75**,** will be referred to as "partial truss drop."

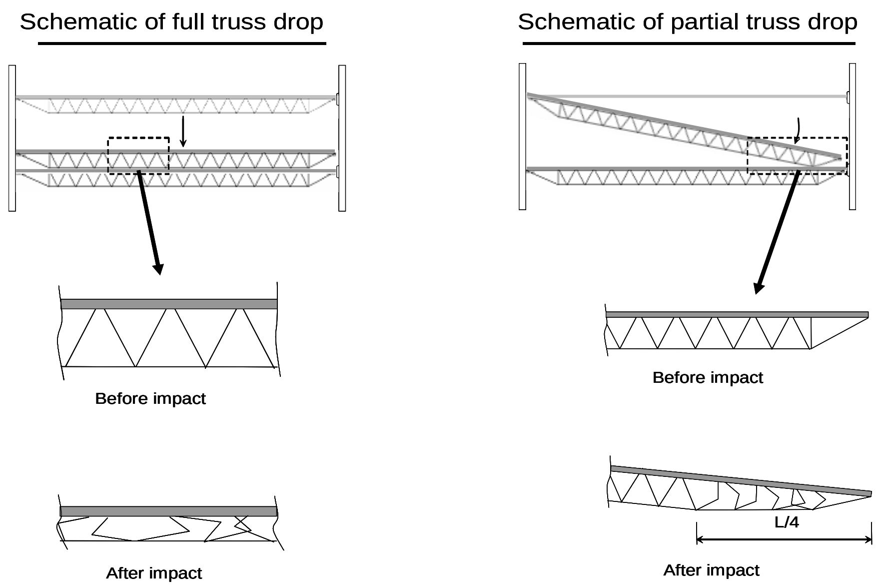


Figure K–75. Schematic of full truss or partial truss drop and diagonal crushing at impact.

K.9.2 Purpose and Scope

The purpose of this study is to determine the dynamic response of the target truss from the impact of full and partial truss drop, to determine whether the target truss seats can resist such an impact load and to determine whether the target truss will lose its composite action, become a catenary, and thus fail to restrain the exterior column to which it is connected against instability.

K.9.3 Method of Analysis

The simulation of a floor drop is idealized with a truss drop. This has the inherent assumption that all seats for the floor fail simultaneously to cause a full or partial drop. The dynamic response of the target truss from the impact of a dropping truss is calculated using conservation of energy principle. The potential energy of the truss just before drop, which is a function of drop height, converts to the kinetic energy of the truss just before impact. As the dropping truss starts to impact the target truss, the diagonal members of the dropping truss are assumed to deform plastically to absorb some of the kinetic energy.

The energy absorption due to crushing of the furniture and partitions are neglected in this study. The energy absorption due to diagonal member crushing reduces the kinetic energy available at impact to deform the target truss. All the diagonal members are assumed to deform plastically for the full truss drop case, while only one quarter of the diagonal member are assumed to deform plastically for partial truss drop, representing one quarter of the length of the truss that may come in contact at impact with floor below. The kinetic energy loss at the time of impact of the dropping truss and the target truss is calculated based on conservation of momentum. The two trusses are assumed to travel together after the impact, at one-half of the velocity of the dropping floor before impact.

The dynamic load due to the impact of the dropping truss onto the target truss will result in the target truss to deform plastically beyond the static load due to the weight of the two trusses. The maximum dynamic deformation of the trusses is calculated by conservation of energy principle assuming that the resistance of the truss is a bilinear function of displacement. This assumption is based on fitting the FEA calculated acceleration-deflection relationship of target truss as shown in Fig. K–76.

K.9.4 Results

The ratios of demand-to-seat capacity for the gravity loads of the dropped and impacted trusses moving together for temperatures of 20 ^{o}C, 400 ^{o}C, 600 ^{o}C, and 700 ^{o}C; and the gravity plus dynamic impact loads for temperatures of 20 ^{o}C and 400 ^{o}C, are calculated. The demand-to-capacity ratio of less than one shows that the truss seat has sufficient capacity to resist the load, and the demand-to-capacity ratio of larger than one, implies that the seat could fail. The range of the demand-to-capacity ratios are due to the different assumptions for the amount of energy loss due to crushing of the diagonal members of the dropped truss.

The demand-to-capacity ratio of the long-span truss for gravity loads is shown in Table K–18 and for gravity plus impact load is shown in Table K–19. The result for gravity load alone shows that both the exterior and interior truss seats have sufficient capacity to support the weight of two floors for all temperatures considered. The result for gravity plus impact load shows that at temperatures below 400 ^{o}C neither the exterior nor interior truss seat is expected to fail. Peak deflection response due to gravity and the dynamic impact of the dropping truss is given in Table K–20. The results show that at room temperature, and more so at 400 ^{o}C, the impacted truss will deflect to an extent that it loses composite action, and become a catenary. At 400 ^{o}C the truss walks off the interior seat. Obviously, a catenary truss is not able to restrain the exterior column against transverse movement and cannot restrain it from instability. Although a truss response to increasing acceleration at 700 ^{o}C has not yet been developed, the strength reduction of the truss seats clearly indicates that the failure of truss seats will occur. The results for long-span truss, for partial truss drop, and for the short-span truss are in progress.

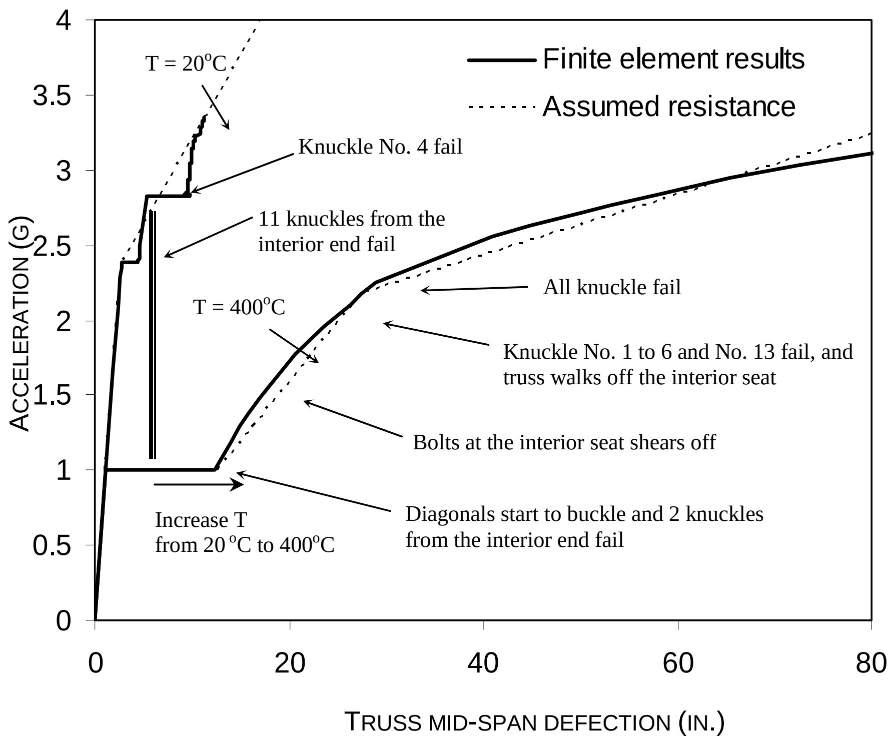


Figure K–76. Target truss resistance against increasing acceleration.

Table K–18. Demand-to-Capacity ratio of long-span truss for static gravity load.

Temp. (°C)	Demand (kip)	Capacity (kip)			Demand/Capacity	
		Int. Seat	Ext. Seat		Int. Seat	Ext. Seat
20	26.4	187.3	140.0		0.14	0.19
400	26.4	166.9	125.7		0.16	0.21
600	26.4	81.6	77.8		0.32	0.34
700	26.4	37.2	35.5	0.71	0.74	

Table K–19. Demand-to-Capacity ratio of long-span truss for dynamic impact load from full truss drop.

Temp. (°C)	Demand (kip)			Capacity (kip)		Demand / Capacity					
				Int. Seat	Ext. Seat	Int. Seat			Ext. Seat		
20	38.6	-	65.3	187.3	140.0	0.21	-	0.35	0.28	-	0.47
400	39.1	-	45.2	166.9	125.7	0.23	-	0.27	0.31	-	0.36

Table K–20. Peak deflection response due to static gravity and dynamic impact.

Temp. (°C)	Static Deflection (in.)	Dynamic Deflection (in.)		
20	2.3	7.6	-	25.4
400	24.2	66.4	-	89.6

K.9.5 Conclusions

At room temperature, the impact of a dropping truss will not cause failure of truss seats, but will cause the impacted truss to deform into a catenary. At 400 ^{o}C, the impacted truss will walk off the interior seat. In either case, the impacted floor will not restrain the exterior column against transverse movement and instability. The impact of a dropping truss at 700 ^{o}C will cause failure of truss seats.

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Appendix L
INTERIM REPORT ON WTC 7

L.1 BUILDING DESCRIPTION

L.1.1 Purpose

Project 6 addresses the first primary objective of the technical investigation led by the National Institute of Standards and Technology (NIST) of the 47-story World Trade Center (WTC) disaster: to determine why and how WTC 7 collapsed. Specifically, the objective of this Project is to determine the response of structural components and systems to the impact damage and fire environment in WTC 7, and to identify probable structural collapse mechanisms.

L.1.2 Scope of Work

The structural response of WTC 7 to damage from debris and fires is being evaluated to identify possible collapse sequences and critical components that are consistent with the videographic and photographic records, interview accounts by individuals that were in or around WTC 7, and other available data. This work is being conducted in two tasks:

- Task 1, Structural response analysis to identify critical components
- Task 2, Structural analysis of possible collapse initiation hypotheses

The analytical work is being conducted with the assistance of Gilsanz Murray Steficek LLP.

The scope of work under Task 1 includes (a) develop a nonlinear global structural model of WTC 7 and evaluate its performance under design gravity loads, (b) identify credible failure sequences for the structural model with service loads and initial structural damage by analyzing the effect of component failures (that may have occurred directly or indirectly from fires) on the structural system stability, (c) identify dominant failure modes for critical components and subsystems determined in (b) for service loads and elevated structural temperatures, (d) conduct parametric studies of critical subsystems to identify influential parameters, and (e) develop approaches to simplify structural analyses for global modeling and analyses.

Selected technical results and finding for progress on Task 1 (a), (b), and (c) are presented in the following sections: a description of the WTC 7 structural design, observations of damage, fires, and the structural collapse, and the working collapse hypothesis developed to date.

L.1.3 Introduction

WTC 7 was a 47 story commercial office building, completed in 1987. Its location relative to the WTC Plaza is shown in Fig. L–1. It contained approximately 2 million ft^2 of floor area. The overall dimensions of WTC 7 were approximately 330 ft long, 140 ft wide, and 610 ft tall. The typical floor was similar in size to a football or soccer field (see Fig. L–2). The gross floor area was about 75 percent of that contained in the Empire State Building. The building was constructed over a pre-existing electrical substation owned by Con Edison. The original plans for the Con Ed Substation included supporting a high-rise building, and the foundation was sized for the planned structure. However, the final design for WTC 7 had a larger footprint than originally planned. Section L.1.4 describes the WTC 7 foundation.

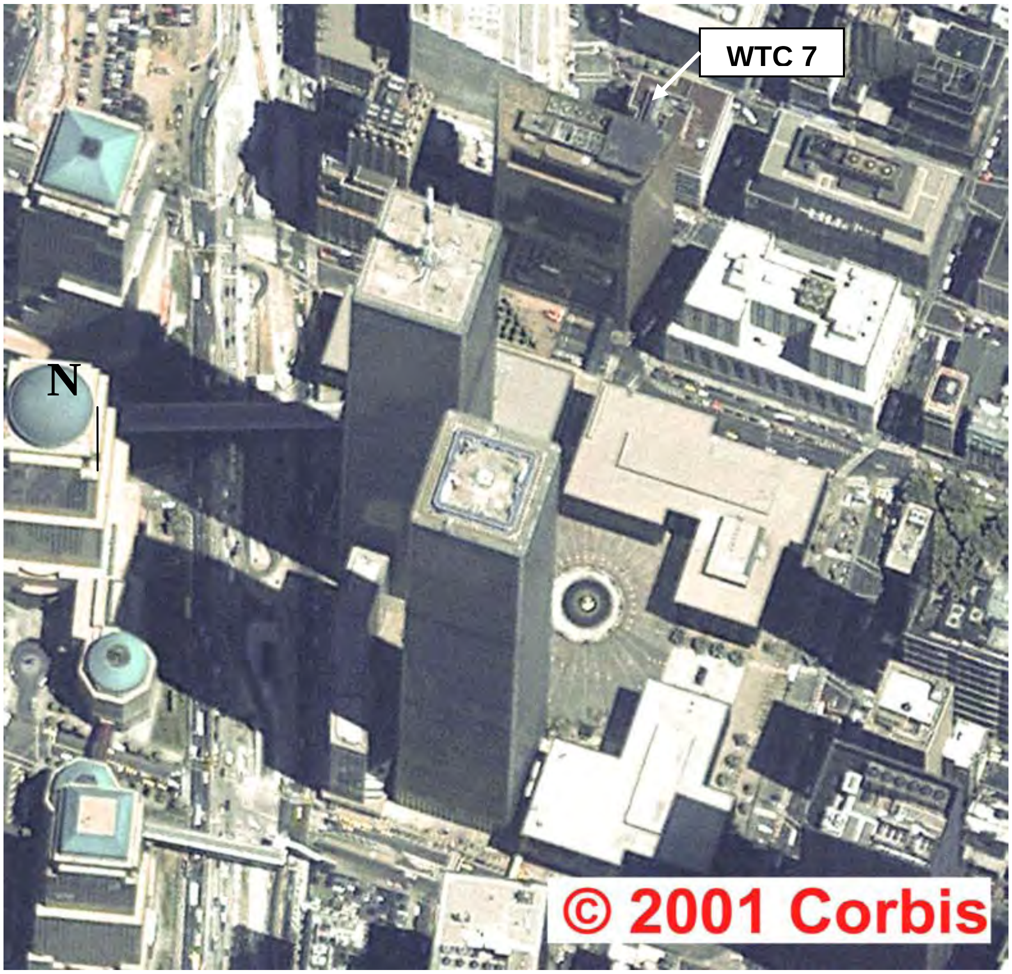
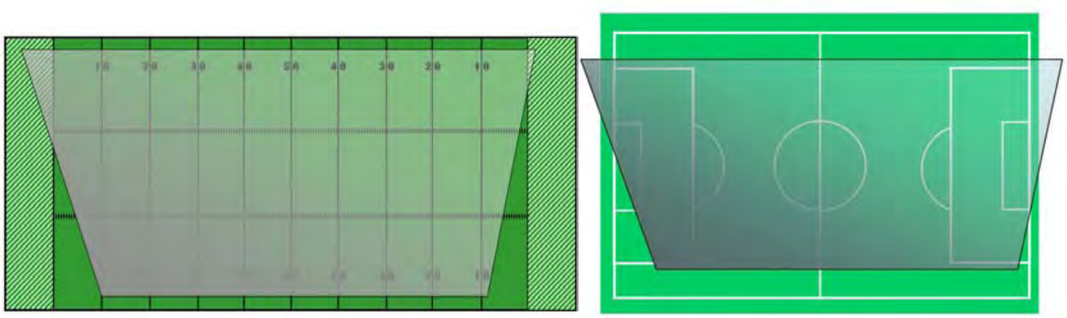


Figure L–1. WTC complex.



A. Comparison to Football Field B. Comparison to Soccer Field

Figure L–2. Size comparison of WTC 7.

WTC 7 was located immediately to the north of the main WTC Complex, approximately 350 ft from the north side of WTC 1. It occupied the block bounded by Vesey Street on the south, Barclay Street on the north, Washington Street on the west, and West Broadway on the east. It was connected to the WTC complex with a 120 ft wide elevated plaza at the Floor 3, and a 22 ft wide pedestrian bridge, also at Floor 3.

Above Floor 7, the building had typical steel framing for high-rise construction. The floor systems had composite construction with steel beams supporting concrete slabs on metal deck, with a floor thickness of 5.5 in. The core and perimeter columns supported the floor system and carried their loads to the foundation. The perimeter moment frame also resisted wind forces. Columns above Floor 7 did not align with the foundation columns, so braced frames, transfer trusses, and transfer girders were used to transfer loads between these column systems, primarily between Floors 5 and 7. Floors 5 and 7 were heavily reinforced concrete slabs on metal decks, with thicknesses of 14 in. and 8 in., respectively. The following sections describe the components and subsystems of WTC 7.

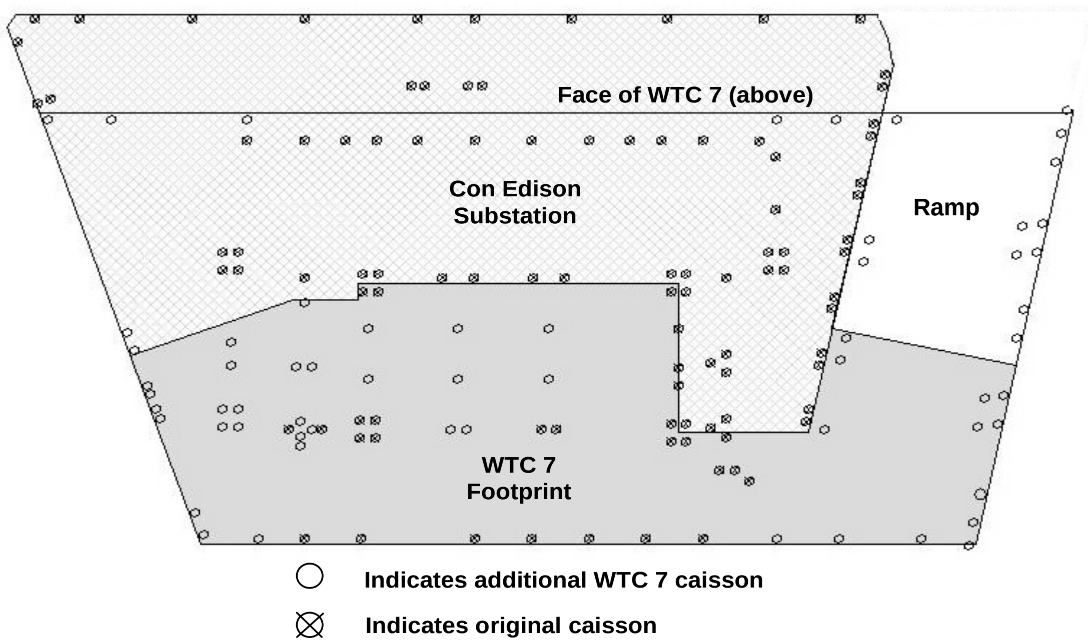
L.1.4 Foundations

WTC 7 and the electrical substation were supported on caisson foundations. When the substation was constructed in 1967, provision was made for a future office tower by including capacity to carry both the substation and the weight of a future building. Caissons were also installed in the property adjacent to the substation, for the proposed future building. When WTC 7 was constructed approximately 20 years later, it was significantly larger than the originally proposed building, and required additional caissons to be installed, as shown in Fig. L–3.

The typical caisson consisted of several components: a 30-in., 36-in., or 42-in. diameter steel casing, a heavy rolled or built-up steel core shape, vertical reinforcing bars, spiral rebar, and concrete fill. At the base of the caisson core, a pattern of shear studs was placed to help transfer the load from the steel caisson core into the encompassing concrete, from which it passed into the rock. The caissons extended through the soil, and were socketed (seated) in the bedrock, approximately 60 ft below the surface. There were vertical caissons as well as battered (or sloped) caissons to carry the lateral load. Above the caissons were heavy grillages composed of built up steel girders. Grillages transferred loads between the building columns and the caissons.

The distance between the caisson grillages and the first floor varied between 8 ft and 30 ft. This region was braced by reinforced concrete walls with thicknesses varying from 1 ft to 2.5 ft. Many of the WTC 7 steel columns were embedded in these walls, and supporting steel braces were made composite by the addition of shear studs along the height of embedment.

Areas between the concrete walls were backfilled with compacted gravel fill and then covered with a concrete slab on grade or framed slab to form closed cells and bring the structure up to the required elevation. In some cases, the area was left unfilled and used to house fuel tanks.



Source: McAllister 2002.

Figure L–3. WTC 7 to foundations.

L.1.5 Con Edison Substation

The Con Ed Substation was constructed in 1967 and consisted of a steel framed structure with cast-in-place concrete floors and walls. It was placed on the northerly portion of the site and extended approximately 40 ft north of the north facade of WTC 7, as shown in Fig. L–4. Its southerly boundary was irregular, but extended approximately one-third to two-thirds of the width of WTC 7. The Con Ed Substation was three stories in height.

The substation's lateral system consisted of a moment frame along the northern row of interior columns. Along the south edge of the substation there was a braced frame. This braced frame was coincident with the north side of the WTC 7 core, at columns 64, 67, 70, and 73. Lateral loads from WTC 7 were passed directly from the core above to the Con Ed braced frame below. There were also two moment frames within the substation oriented in the north-south direction, one on each end of the WTC 7 core.

The WTC 7 columns, which were within the perimeter of the substation, were supported by substation columns. During the construction of WTC 7, heavy plates were welded to the tops of the existing substation columns which then received the new building columns.

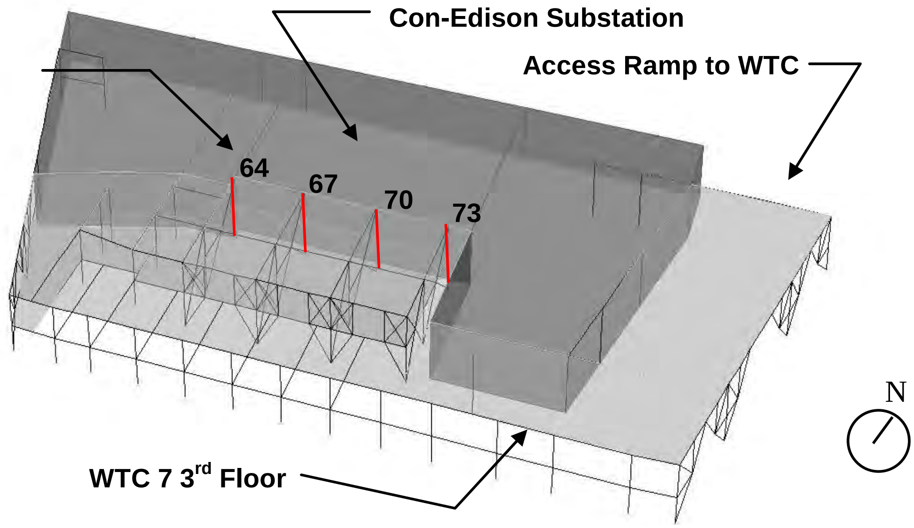


Figure L–4. Con Ed substation location relative to the WTC 7 building.

L.1.6 Floor Systems

Typical Floor Systems Above Floor 7

The typical floor framing system, shown in Fig. L–5, was composed of rolled steel wide-flange beams with composite metal decking and concrete slabs. Floors 8 through 45 had essentially the same framing plan, but the core layout varied over the height of the building.

Floors 8 through 45 had floor slabs that were composed of 3 in., 20 gage metal deck with 2.5 in., 3,500 psi normal weight concrete, for a total thickness of 5.5 in. There was one layer of 6x6 W1.4xW1.4 welded wire mesh within the concrete. The drawings show a second layer of mesh placed over girders at the slab edges. The fastening requirements for the metal deck are not shown on the drawings, but standard practice provides puddle welds 12 in. on-center at the beams and side lap welds, screws, or button-punching at 36 in. on-center between adjacent panels of deck. The drawings contain a note calling for 1.5 in., 20 gage deck with 4 in. concrete topping (5.5 in. total) in the elevator lobbies, where there was a 3 in. floor finish specified by the architect.

Typical floor framing for Floors 8 through 20 and Floors 24 through 45 consisted of 50 ksi wide-flange beams and girders. Between the core columns was a grid of beams and girders. Core girders ranged in size from W16x31 to W36x135, depending on the span and load. (W16x31 describes a steel wide-flange beam, sometimes referred to as ‘I’ beams; the nomenclature indicates the cross-section is nominally 16 in. deep and weighs 31 lb per lineal foot.) Beams spanned directly between the core and the exterior of the building, at approximately 9 ft on-center spacing. On the north and east sides, the typical beam was a W24x55 with 28 shear studs, spanning 53 ft. On the south side, the typical beam was a W16x26 with 24 shear studs spanning 36 ft. Between the exterior columns were moment connected girders that formed part of the lateral system of the building. On Floors 10, 19, and 20, a portion of the floor framing was

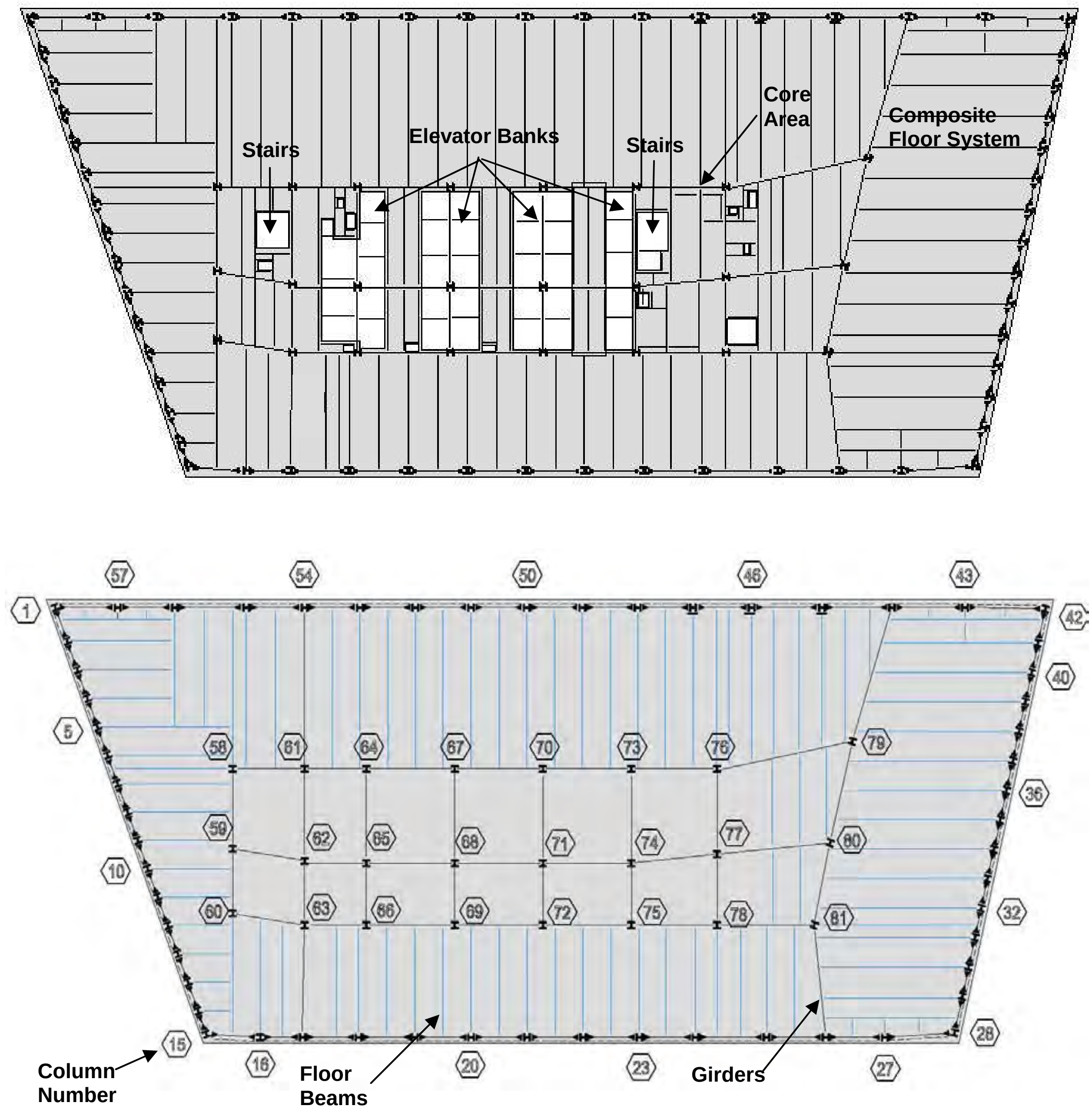


Figure L–5. Floors 8 to 45 plan.

reinforced with plates attached to the bottom flange. Certain connections at these floors were also reinforced.

Floors 21 to 23 had slightly heavier steel framing than the typical floors. Core girders were generally one size class larger than the typical floor; the beams between the core and the south facade were W16x31 instead of W16x26. There were additional studs on the W24x55 beams on the north and west sides.

Most of the beams and girders were made composite with the slabs through the use of shear studs. Typically, the shear studs were 0.75 in. in diameter by 5 in. long, spaced 1 ft to 2 ft on center. Studs were not indicated on the design drawings for many of the core girders. The design drawings specified design forces for connections and suggested a typical detail, but did not show specific connection designs; this is standard practice on the U.S. east coast. The erection drawings indicate that design shear forces for the typical beam and girder connections were to be taken from the American Institute of Steel Construction (AISC) beam design tables for beams without shear studs, using 1.5 times those forces for beams with shear studs.

According to a paper by Salvarinas (1986), who was the project manager for Frankel Steel, which fabricated the steel for WTC 7, the typical floor beam to girder and girder to core column connection was a single shear plate, although end plate and double angle connections were also used. The typical beam to exterior column connection was a seated connection. The typical bolt size for the simple shear connections is cited as 0.875 in. in diameter ASTM A325, where A325 is a standard specification for a structural bolt specified by ASTM International. The bolt size used for heavier brace and moment connections was 1 in. in diameter ASTM A490. Information on the specific connection details used is unavailable at this time.

Other Floors

The remaining floors, Floors 1 to 7 and Floors 46 to 47, were atypical and are described below and in Figs. L–6 through L–15.

Floor 1 was built adjacent to the substation and included the truck ramp for the WTC complex. The first floor is shown in Fig. L–6. The floor was framed with steel beams that were encased in a formed concrete slab. The floor slab was 14 in. thick, with typical #5 reinforcement bars (5/8 in. rebar) at a 10 in. to 12 in. spacing and #6 rebar at 9 in. spacing for the bottom reinforcement; #5 rebar at 12 in. spacing was used for temperature reinforcement. The southeast portion of the floor above the WTC truck ramp had a 6 in. formed concrete slab with #4 rebar at 12 in. spacing for top and bottom reinforcement; #4 rebar at 18 in. spacing was used for temperature reinforcement.

The floor slab for Floors 2, 3, 4, and 6 had a 3 in., 20 gage metal deck with 3 in. 3,500 psi normal weight concrete, for a total thickness of 6 in. Floors 2 and 3 were also partial floors adjacent to the substation. In addition, they had a floor opening on the south side to form the atrium above the ground level lobby (see Figs. L–7 and L–8). Floor 4 was above the substation and had a large opening over most of the south side of the building, to form a double-height space above the 3rd floor lobby (see Fig. L–9). Floor 6 had two openings on the floor to form a double-height mechanical space, one at the east side and the other one at the southeast corner (see Fig. L–12). Truss #2 and column 80 were located in this double-height mechanical space.

The 5th floor slab was 11 in. of 3,500 psi normal weight concrete on top of a 3 in., 18 gage composite metal deck for a total slab thickness of 14 in. The slab was heavily reinforced, with #7 rebar at 12 in. spacing for top reinforcement in both directions and #9 rebar at 12 in. spacing for bottom reinforcement that acted as additional diaphragm chord reinforcement in many areas. This floor also had 36 ksi steel WT sections (W, or wide-flange, sections cut in half to look like a ‘T’ section) embedded in the 11 in. concrete slab above the deck. The WT sections were designed to act as a horizontal truss within the plane of the floor between the perimeter and core columns (see Figs. L–10 and L–11).

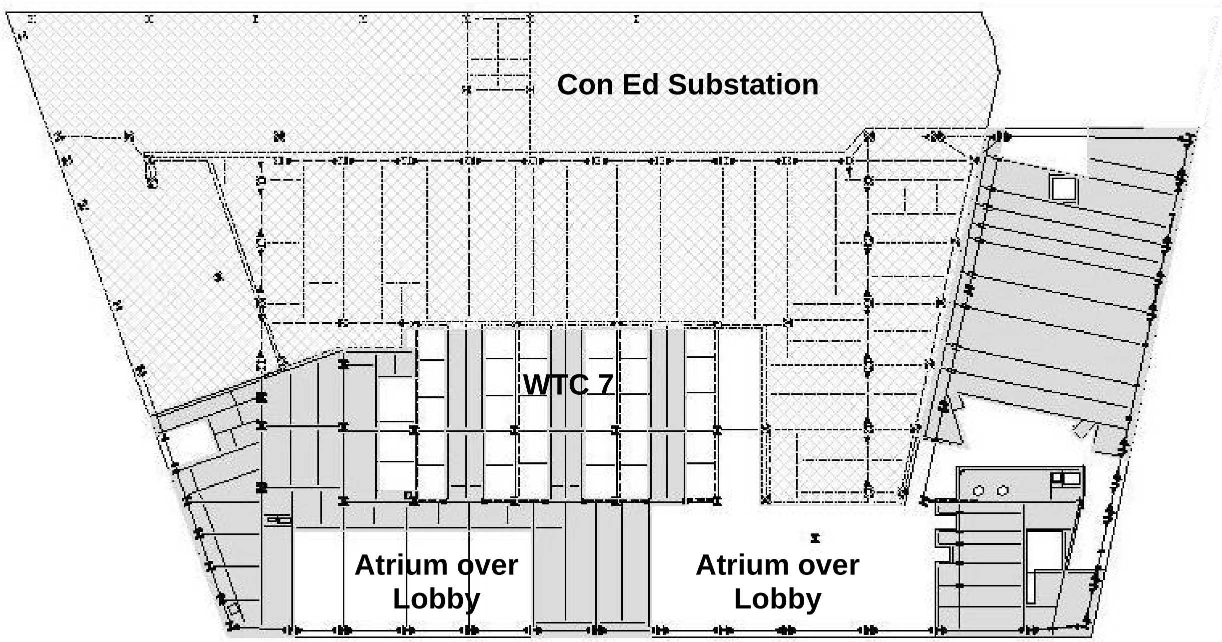


Figure L–6. Floor 1 plan.

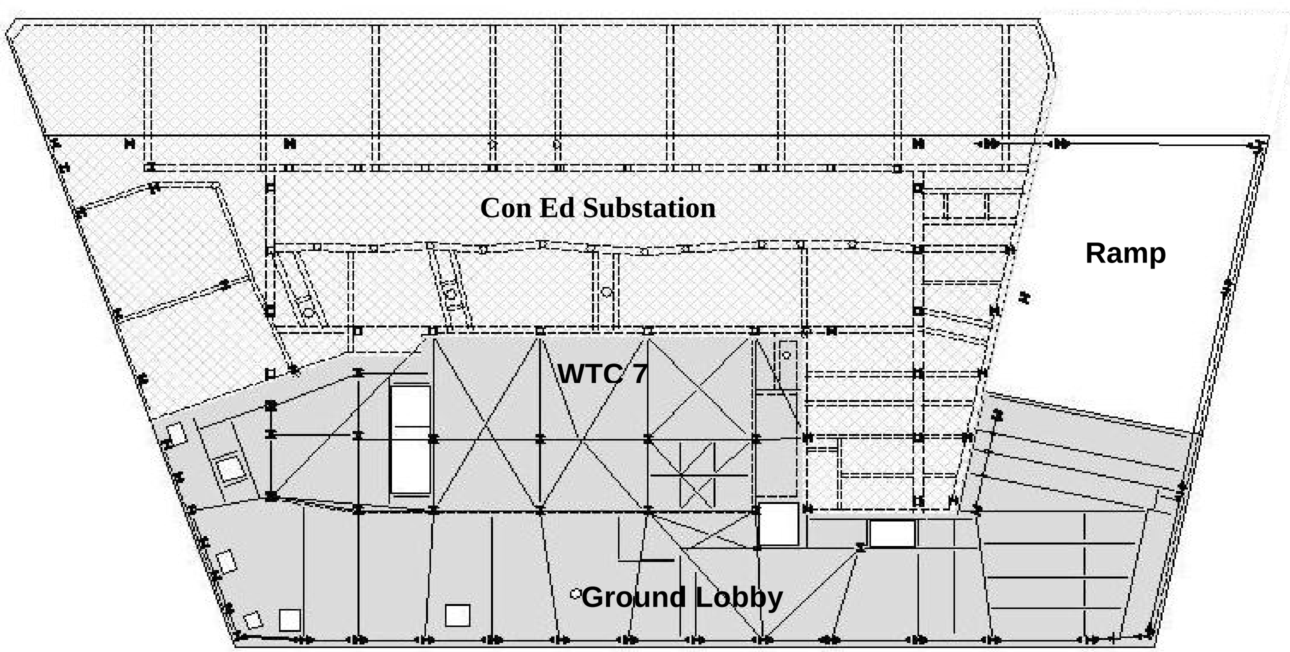


Figure L–7. Floor 2 plan.

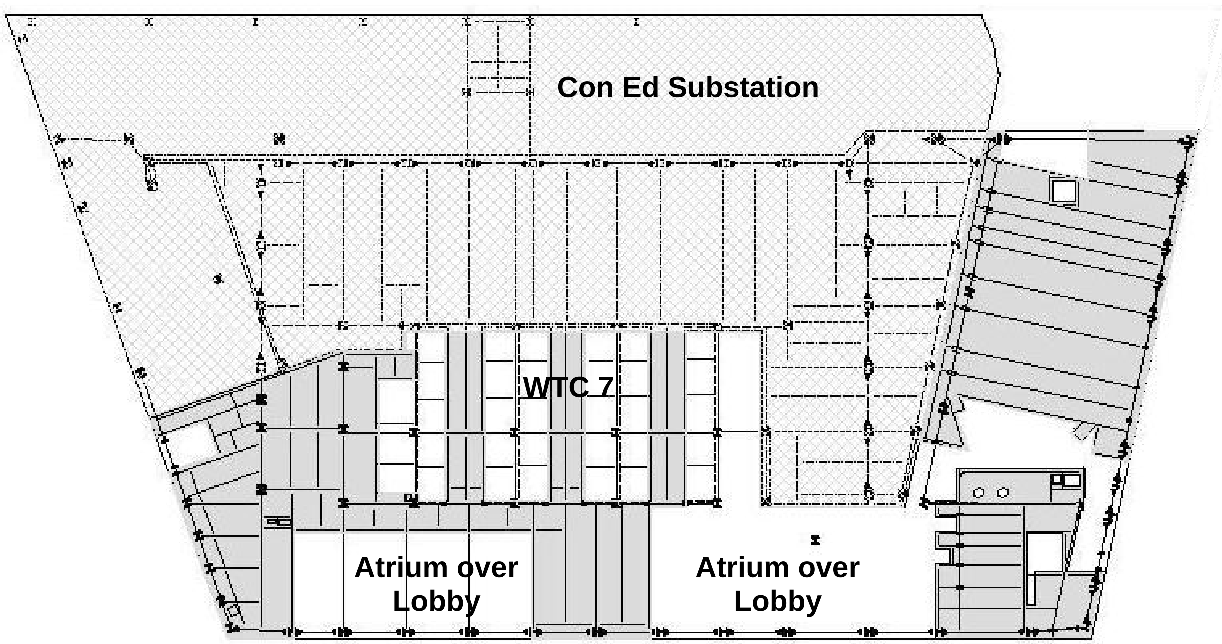


Figure L–8. Floor 3 plan.

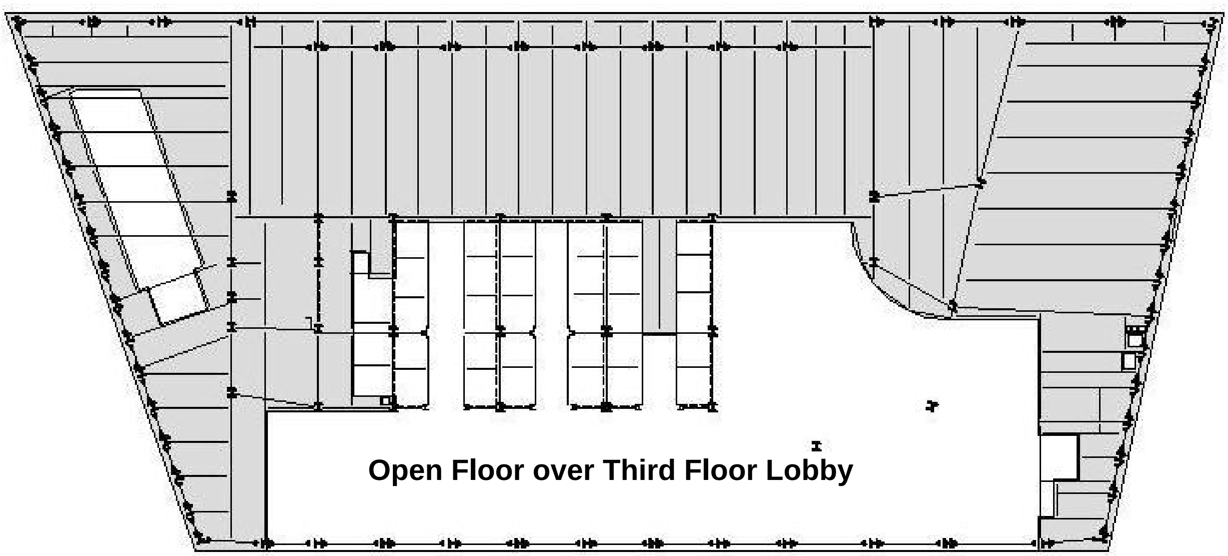


Figure L–9. Floor 4 plan.

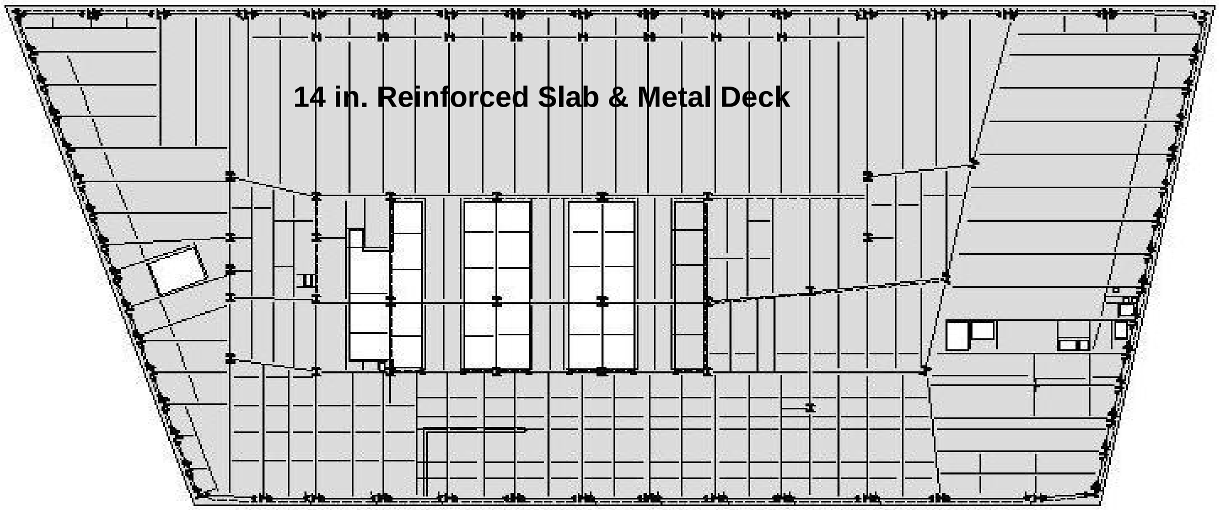


Figure L–10. Floor 5 plan.

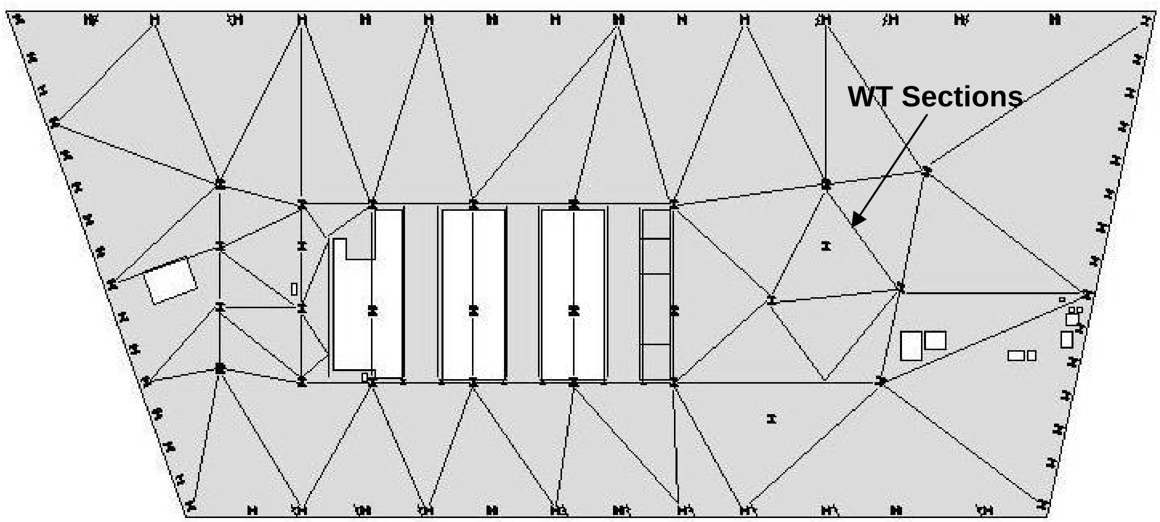


Figure L–11. Floor 5 diaphragm plan.

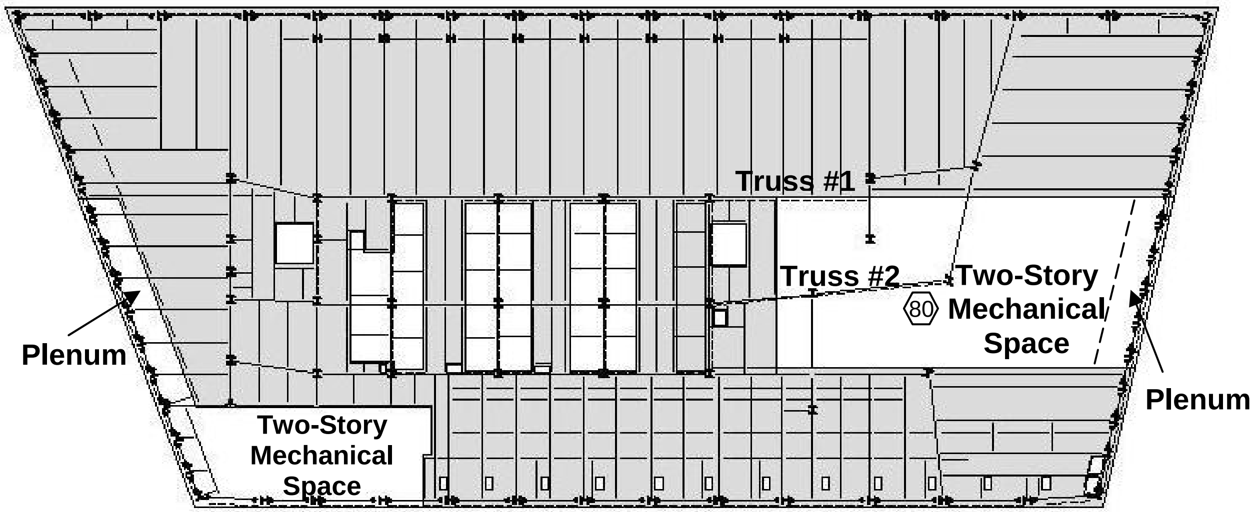


Figure L–12. Floor 6 plan.

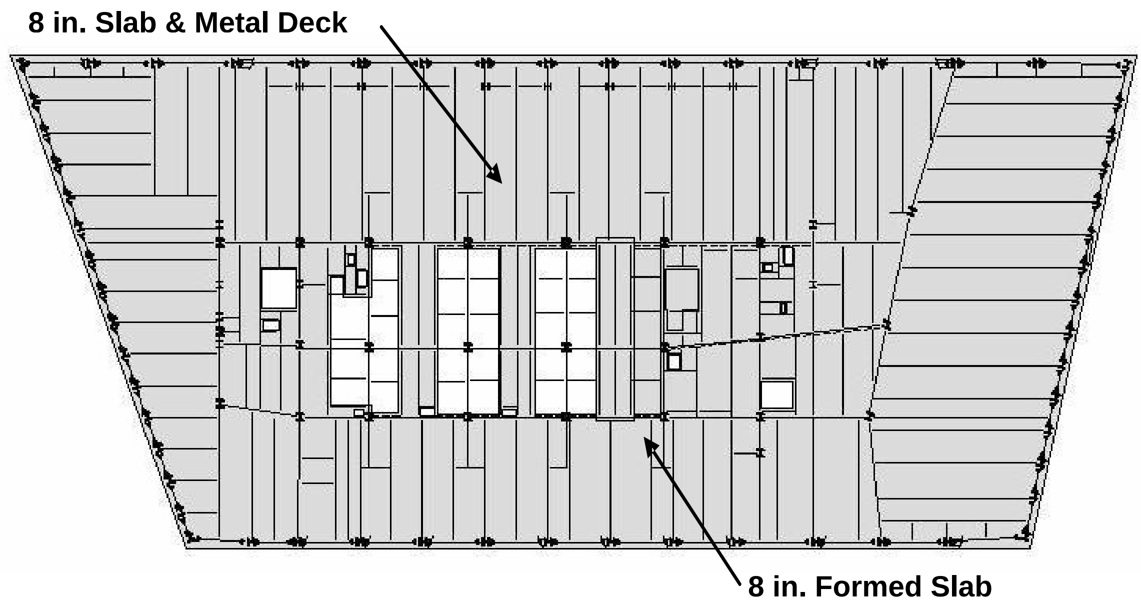


Figure L–13. Floor 7 plan.

The 7th Floor slab consisted of 5 in. of 3,500 psi normal weight concrete on top of 3 in., 18 gage composite metal deck, for a total thickness of 8 in. The slab was reinforced with #5 rebars at 6 in. on-center in both directions. Regions of the slab on the south side of the building had 8 in. of formed concrete without any metal deck. In these regions two layers of steel reinforcing were provided (see Fig. L–13).

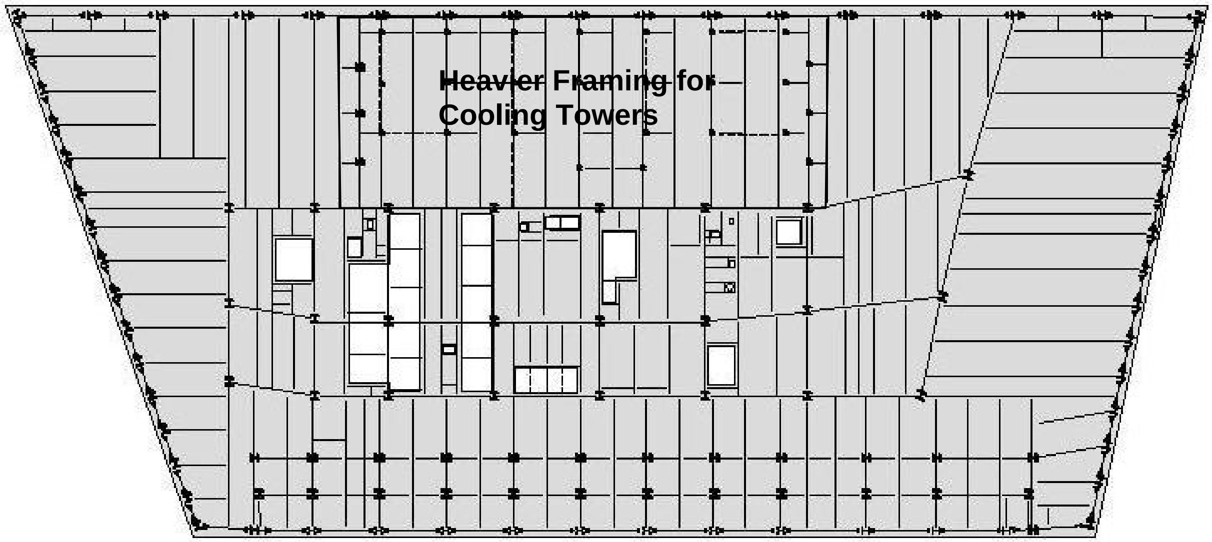


Figure L–14. Floor 46 plan.

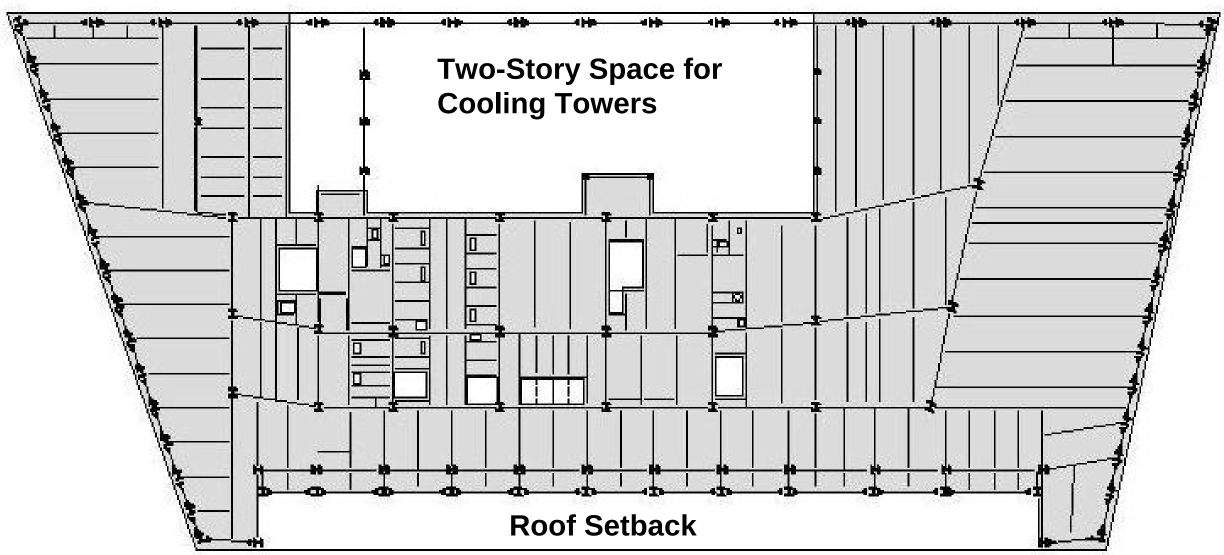


Figure L–15. Floor 47 plan.

Floors 41 and 43 had the east half removed to provide double height spaces. Columns in these areas and areas of Floors 40 and 42 had been reinforced to provide adequate capacity for the additional height and change in use by tenants. By 2001, Floors 41 and 43 had been reconstructed to provide full floor space. Specifics of this reconstruction are not available at this time.

The 46th Floor had heavier framing to support the cooling towers and dunnage on the north side, (alternating W36x150 with W36x260 under the posts) and the setback roof on the south side (alternating W21x44 with W36x150 under the posts). There was a 6 in. reinforced concrete slab in a portion of the core and under the cooling towers (see Fig. L–14).

Floor 47 had a double height space extending from the 46th Floor to the underside of the roof for the cooling towers on the north side. There was also a setback roof on the south side at Floor 46 (see Fig. L–15).

Roof and Penthouses

The roof had a concrete slab on metal deck, the top of which sloped 3 in., from an 8.5 in. thickness to a 5.5 in. thickness, to provide drainage. The wire mesh in this slab was 6x6 W2.4xW2.4, which was 70 percent heavier mesh than at the typical floor. There were slab openings for the cooling towers on the north and the setback roof on the south. The area above the cooling towers was framed in steel, with areas of grating spanning between the beams. A series of diagonal WT 6x9 members under the grating provided diaphragm action in this area. The east side of the floor was reinforced to carry the east penthouse and its contents. Specifics of this reinforcement are not available at this time.

The west penthouse roof was framed in steel with the floor slab increased to a 6 in. thickness. The framing and roof reinforcement for the east penthouse and the mechanical equipment screenwall are not available at this time. Layout of these areas has been determined from photographs, as shown in Fig. L–16.

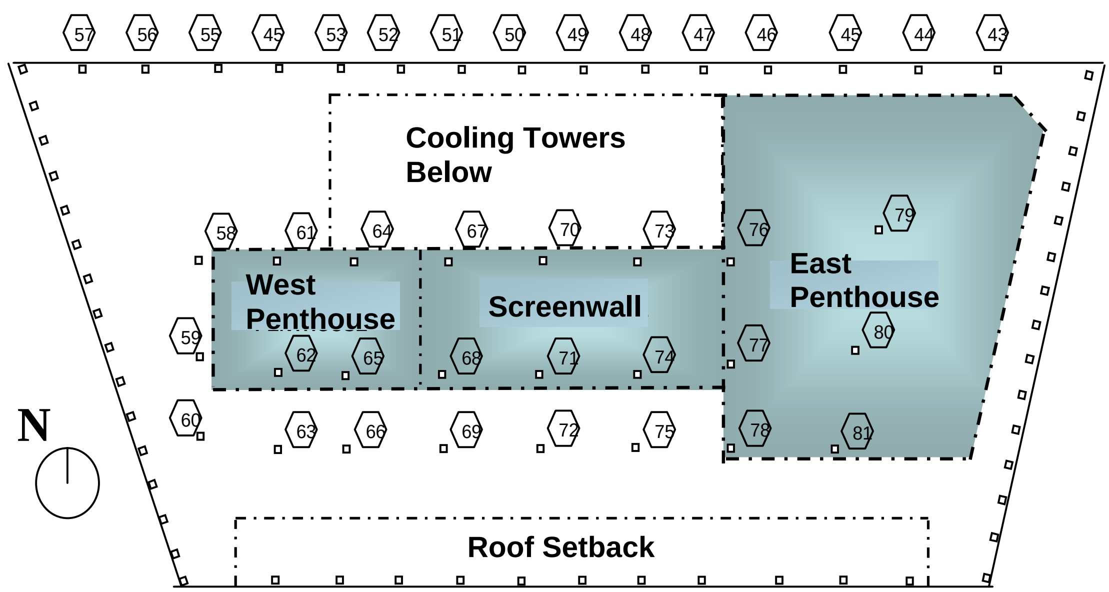


Figure L–16. Roof layout.

L.1.7 Columns

Core columns were primarily rolled wide-flange shapes of grade 36 or 50 steel. As the loads increased towards the base of the building, many of these column sizes were increased through the use of built-up shapes. These built-up columns had a W14x730 core with cover plates welded to the flanges (to form a box) or web plates welded between the flanges as shown in Fig. L–17. The reinforcing plate welds were specified to be continuous 0.5 in. fillet welds at the cover plates and 0.313 in. minimum at the web plates. Plate thickness ranged from 1.5 in to 8 in. Reinforcing plates were specified as follows:

Plate thickness t (in.):

2 < t < 4	ASTM A588 Grade 50
4 < t < 6	ASTM A572 Grade 42
t > 6	ASTM A588 Grade 42

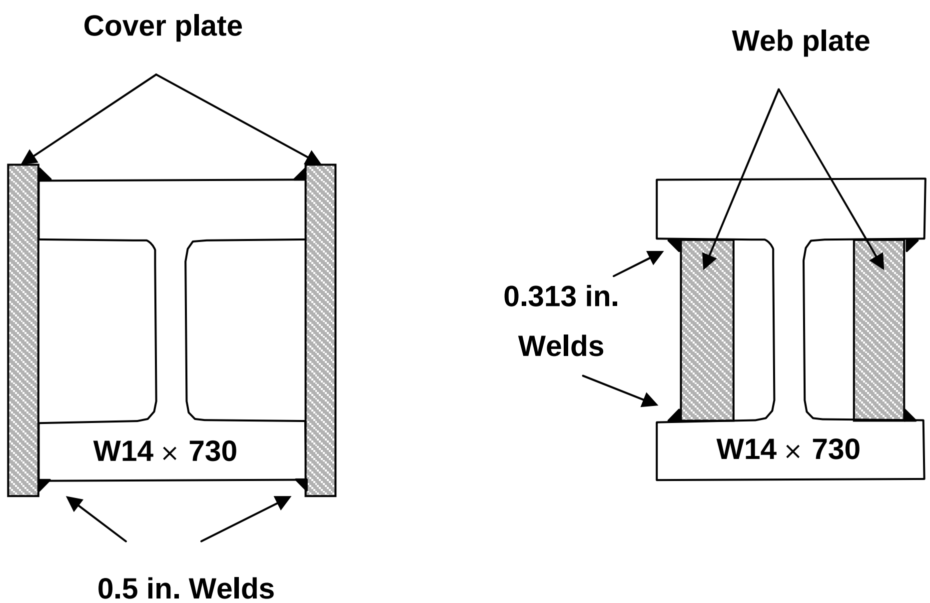


Figure L–17. Typical built-up column details.

Typical core column splices were shown on available erection drawings. The adjoining surfaces of columns were specified to be milled. The splice plates were welded or bolted to the outsides of the column web and flanges. Built-up columns were also milled at their bearing ends but the splice plates were fillet welded to the cover plates.

Perimeter columns were nominally 14 in. wide-flange shapes (W14) of ASTM A 36 steel. Perimeter column splices were similar to the core column splices.

L.1.8 Column Transfer Trusses and Girders

The layout of the substructure and Con Edison columns did not align with the column layout in the upper portion of WTC 7. Therefore a series of column transfers were constructed. These transfers occurred primarily between Floors 5 and 7. See Fig. L–18 for a schematic rendering of the transfers.

Columns 47 through 54, at the north facade, were transferred at Floor 7 by cantilever girders to bring them in line with the substation columns, offset 6 ft to 9 ft to the south. The back-span of these cantilevers was supported by the north side core columns. The eastern most cantilever girder was connected to truss #1, and the western most cantilever girder was connected to truss #3 (see Fig. L–18).

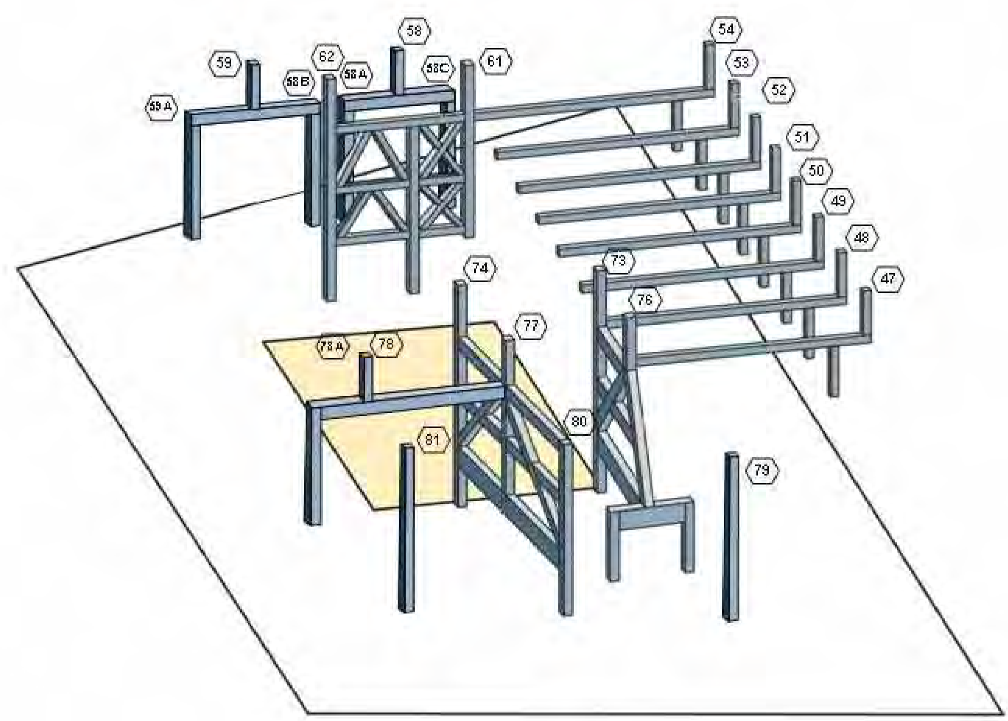


Figure L–18. 3D schematic view of transfer trusses and girders between Floors 5 and 7.

Column 76 was supported at Floor 7 by truss #1. The west side of truss #1 is supported by column 73, while the east side is supported by a transfer girder running north-south which is, in turn, supported by columns E3 and E4 at Floor 5.

Columns 58, 59, and 78 were transferred by simply supported girders at Floor 7. Column 78 was supported at Floor 7 by a transfer girder that was supported at its north end by truss #2. Column 77 was also supported by truss #2. Truss #2 was supported by column 74 at its west end and by column 80 at its east end.

Column 61 was supported by truss #3. Truss #3 runs north-south and was supported by columns 62 and 61A. Truss #3 has a 10 ft cantilever span between column 61 and column 61A and an 18 ft back span to column 62.

L.1.9 Lateral System

Above Floor 7, WTC 7 had a perimeter moment frame. Exterior columns were typically rolled W14 shapes of ASTM A36 grade steel. Column trees were fabricated for the east and west facades with field splices occurring every other story in the columns and at the spandrel beam midspan between columns, where the tree stubs were spliced with a bolted connection. On the north and south facades, the moment frames were constructed with spandrel connections at the face of the columns. Some column splices were shown on the erection drawings to be partial penetration groove welds between the column flanges.

At Floors 5 to 7 and Floors 22 to 24, there was a perimeter belt truss, shown in Fig. L–19. Below Floor 7, a combination of moment and braced frames around the perimeter and a series of braced frames in the core, is shown in Fig. L–20. The strong diaphragms of Floors 5 and 7 transferred load from the perimeter to the core. Above the loading dock at the south facade, two of the columns were hung from the belt truss at Floors 5 through 7. Above the Con Edison vault at the north facade, eight columns were also hanging from the belt truss between Floors 5 and 7.

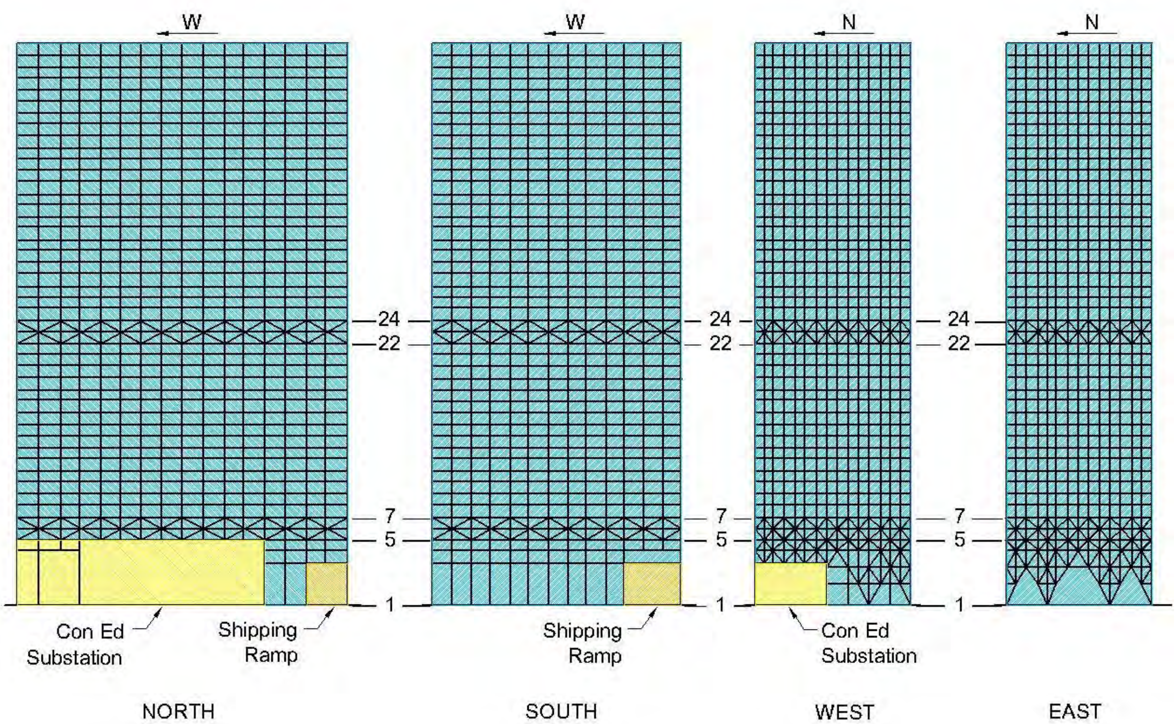


Figure L–19. Perimeter lateral system elevations.

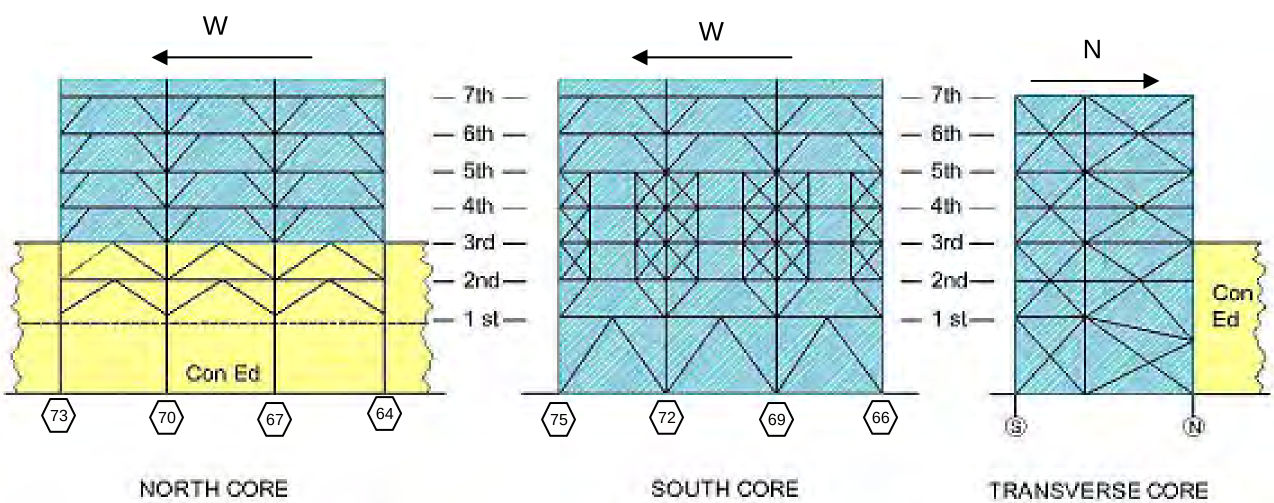


Figure L–20. Core lateral system.

L.2 OBSERVATIONS OF STRUCTURAL COLLAPSE

This section presents observed data and events from available drawings, photographic and videographic records, interviews, and other data sources for WTC 7 to identify damage and fire locations. Damage to WTC 7 from debris impact from WTC 1 and WTC 2 is summarized in Section L.2.2, followed by known fire growth and progression in Section L.2.3. The observed exterior sequence of collapse events from photographic and videographic records are described in Sections L.2.4 and L.2.5, where collapse observations are considered from the plan and elevation views of the structure, respectively. These observations have been used for developing possible collapse initiation locations and progression mechanisms, which are presented in Section L.3.

L.2.1 Damage from WTC 1 and WTC 2 Collapses

To place the events leading to the global collapse of WTC 7 into context, it is helpful to summarize the events of September 11, 2001:

8:46 a.m.	WTC 1 was struck by an aircraft
9:03 a.m.	WTC 2 was struck by an aircraft
9:59 a.m.	WTC 2 collapsed
10:28 a.m.	WTC 1 collapsed
5:21 p.m.	WTC 7 collapsed

After WTC 1 collapsed, the south face of WTC 7 was obscured by smoke, making direct observation of damage from photographs or videos difficult or impossible. The source of the smoke is uncertain, as large fires were burning in WTC 5 and WTC 6, as well as those noted below in WTC 7. The light but prevalent winds from the northwest caused the smoke to rise on the leeward, or south, side of the building. The following information about damage seen in WTC 7 was obtained from interviews of people in or near the building:

After WTC 2 collapsed:

- Some south face glass panes were broken at lower lobby floors
- Dust covered the lobby areas at Floors 1 and 3
- Power was on in the building and phones were working
- No fires were observed

Reported close to time of WTC 1 collapse:

- East stair experienced an air pressure burst, filled with dust/smoke, lost lights
- West stair filled with dust/smoke, lost lights, swayed at Floors 29 through 30, and a crack was felt (in the dark) on the stairwell wall between Floors 27 through 28 and Floors 29 through 30
- Floors 7 and 8 had no power, air was breathable but not clear

- Phone lights on Floor 7 were on but could not call out

After WTC 1 collapsed:

- Heavy debris (exterior panels from WTC 1) was seen on Vesey Street and the WTC 7 promenade structure at the third floor level

- Southwest corner damage extended over Floors 8 to 18

- Damage was observed on the south face that starts at the roof level and severed the spandrels between exterior columns near the southwest corner for at least 5 to 10 floors. However, the extent and details of this damage have not yet been discerned, as smoke is present.

- Damage to the south face was described by a number of individuals. While the accounts are mostly consistent, there are some conflicting descriptions:

  – middle one-fourth to one-third width of the south face was gouged out from Floor 10 to the ground

  – large debris hole near center of the south face around Floor 14

  – debris damage across one-fourth width of the south face, starting several floors above the atrium (extended from the ground to 5th floor), noted that the atrium glass was still intact

  – from inside the building at the 8th or 9th Floor elevator lobby, where two elevator cars were ejected from their shafts and landed in the hallway north of the elevator shaft, the visible portion of the south wall was gone with more light visible from the west side possibly indicating damage extending to the west

At 12:10 to 12:15 p.m.:

- Firefighters found individuals on Floors 7 and 8 and led them out of the building

- No fires, heavy dust or smoke were reported as they left Floor 8

- Cubicle fire was seen along west wall on Floor 7 just before leaving

- No heavy debris was observed in the lobby area as the building was exited, primarily white dust coating and black wires hanging from ceiling areas were observed

Photographs support some of these reports and show additional damage at the upper portions of the building. Figure L–21 is an aerial view of WTC 7 after the collapse of WTC 1. There is no visible debris on the roof; some minor damage is seen on the south side at the parapet wall. Figures L–22a and L–22b show the reported damage between Floors 8 to 18 at the southwest corner. Much of the damage above Floor 18 appears to be nonstructural. The black areas on the facade indicate areas of burned out fires. Note the heavy smoke obstructing any observations along the south face. Study of this photograph indicates that at least two exterior columns were severed. Figures L–23a and L–23b show the debris on Vesey Street in front of WTC 7 after the collapse of WTC 1. The pedestrian bridge (L–23a) and the promenade (L–23b) appear to be standing, although damaged. Exterior panels from WTC 1 can be seen on Vesey Street and on the promenade. The approximate extent of possible damage due to debris from WTC 1 is shown in Fig. L–23c.

Figure L–21. Photograph of roof after WTC 1 collapse.

Figure L–22a. Debris damage around Floor 18 of the southwest corner.

Figure L–22b. Debris damage around Floor 8 of the southwest corner.

Figure L–23a. Pedestrian bridge and debris on Vesey Street after WTC 1 collapsed.

L.2.2 Observed Fire Locations

Photographs and videos were used to determine fire locations and movement within WTC 7. Most of the available information is for the north and east faces of WTC 7. Information about fires in other areas of the building was obtained from interviews, and is summarized as follows:

From 11:30 a.m. to 2:30 p.m.:

- No diesel smells reported from the exterior, stairwells, or lobby areas
- No signs of fire or smoke were reported below the 6th Floor from the exterior, stairwells or lobby areas
- In the east stairwell, smoke was observed around Floors 19 or 20, and a signs of a fully involved fire on the south side of Floor 23 were heard/seen/smelled from Floor 22.
- Interviews place a fire on Floor 7 at the west wall, toward the south side, at approximately 12:15 p.m.

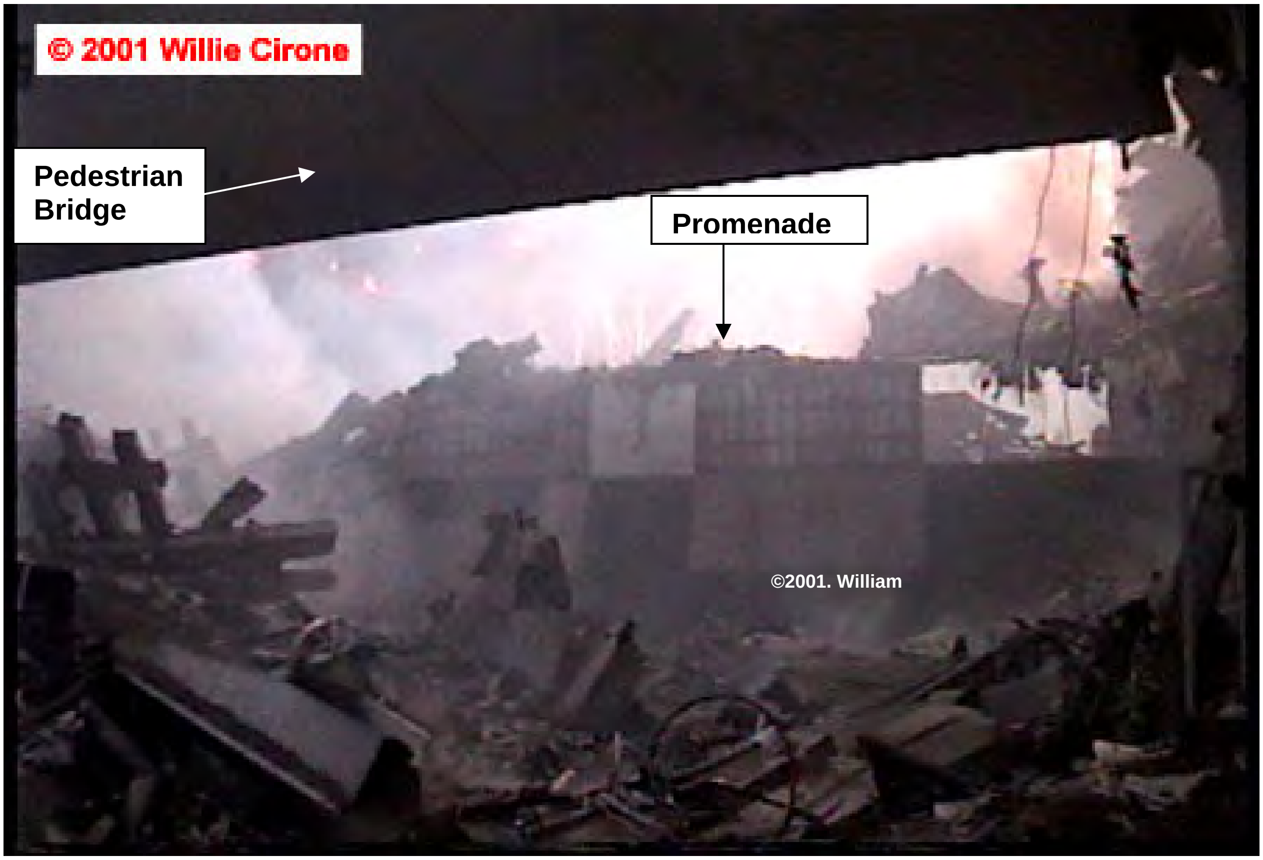


Figure L–23b. WTC 7 Promenade and debris on Vesey Street after WTC 1 collapsed.

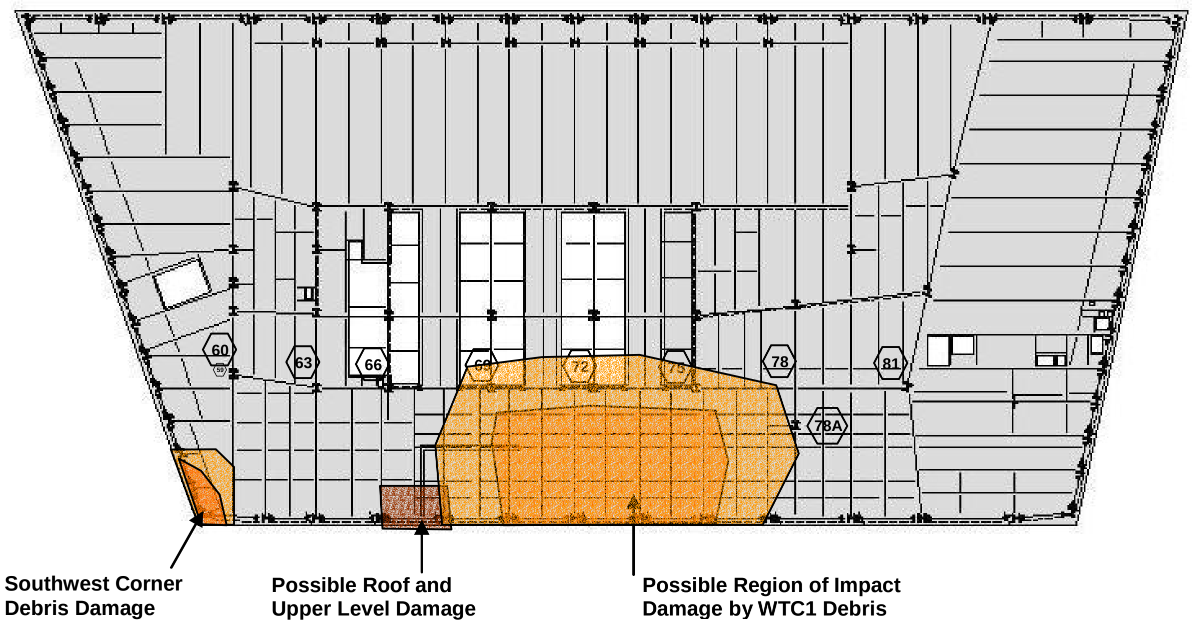


Figure L–23c. Possible extent of debris damage in plan.

- From West and Vesey Streets near the Verizon Building, fires were observed in floors estimated to be numbered in the 20s and 30s.

Looking from the southwest corner at the south face:

- Fire was seen in the southwest corner near Floor 10 or 11
- Fire was seen on Floors 6, 7, 8, 21, and 30
- Heavy black smoke came out of a large, multi-story gash in the south face

Looking from the southeast corner of the south face:

- Fire seen on Floor 14 (reported floor number) on south face; the face above the fire was covered with smoke
- Fire on Floor 14 moved towards the east face

Looking at the east face:

- Fire on Floor 14 (reported floor) moved along east face toward the north side

Photographs and videos were used with these interview accounts to document fire progression in the building. The fires seen in photographs and videos are summarized:

Before 2:00 p.m.

- Figures L–22a shows fires that had burned out by early afternoon on Floors 19, 21, 22, 29, and 30 along the west face near the southwest corner.

2:00 to 2:30 p.m.

- Figure L–24a shows fires on east face Floors 11 and 12 at the southeast corner. Several photos during this time show fires progressing north.

3:00 to 5:00 p.m.

- Around 3 p.m., fires were observed on Floors 7 and 12 along the north face. The fire on Floor 12 appeared to bypass the northeast corner and was first observed at a point approximately one third of the width from the northeast corner, and then spread both east and west across the north face.
- Some time later, fires were observed on Floors 8 and 13, with the fire on Floor 8 moving from west to east and the fire on Floor 13 moving from east to west. Figure L–24b shows fires on Floors 7 and 12.
- At this time, the fire on Floor 7 appeared to have stopped progressing near the middle of the north face.

Figure L–24a. Fires on Floors 11 and 12 on the east face.

Figure L–24b. Fires on Floors 7 and 12 on the north face.

- The fire on Floor 8 continued to move east on the north face, eventually reaching the northeast corner and moving to the east face.

- Around 4:45 p.m., a photograph showed fires Floors 7, 8, 9, and 11 near the middle of the north face; Floor 12 was burned out by this time.

L.2.3 WTC 7 Collapse Observations

The collapse of WTC 7 was recorded on several videos from locations northeast and northwest of the building. Study of these videos led to the development of the timeline in Table L–1, which lists the visible external sequence of events. Figures L–25 to L–28 are images from a CBS News Archives video that show key points observed during the collapse.

The deformed shape of the east penthouse roof shows that the middle fell before the sides (see Fig. L–25), as the whole penthouse drops into the main building (see Fig. L–26). This may imply that support initially remained on the east and west edges of the east penthouse. Therefore, the perimeter columns on the east side of the building which have not already been considered least likely, may be considered less likely locations for collapse initiation.

Table L–1. Timeline of WTC 7 collapse as observed from the northwest.

Time Interval (s)	Total Time (s)	Observation from CBS Video
0.0	0.0	- First movement of east penthouse roofline downwards
0.9	0.9	- East penthouse kink between columns 44 and 45 (Fig. L–25) - First 2 windows at Floor 40 fail between columns 44–45 (windows 9 and 11 from east end)
0.3	1.2	- 4 more windows fail at Floor 40 - East penthouse submerged from view (now inside building)
0.5	1.7	- 3 windows break at Floor 41, Floor 43, Floor 44
0.5	2.2	- East penthouse completely submerged (Fig. L–26**)**
1.8	4.0	- Windows break along column 46 at Floors 37 and 40
3.0	7.0	- West penthouse and screenwall begin to move downward into building - Movement of entire north face of WTC7 (visible above Floor 21)
0.2	7.2	- West end of roof starts to move
0.5	7.7	- East end of roof starts to move - Kink formed in north facade along column 46-47
0.4	8.2	- West penthouse and screenwall submerged - Windows fail between Floors 33–39 around column 55 - Global collapse initiates (Fig. L–27 and L–28**)**

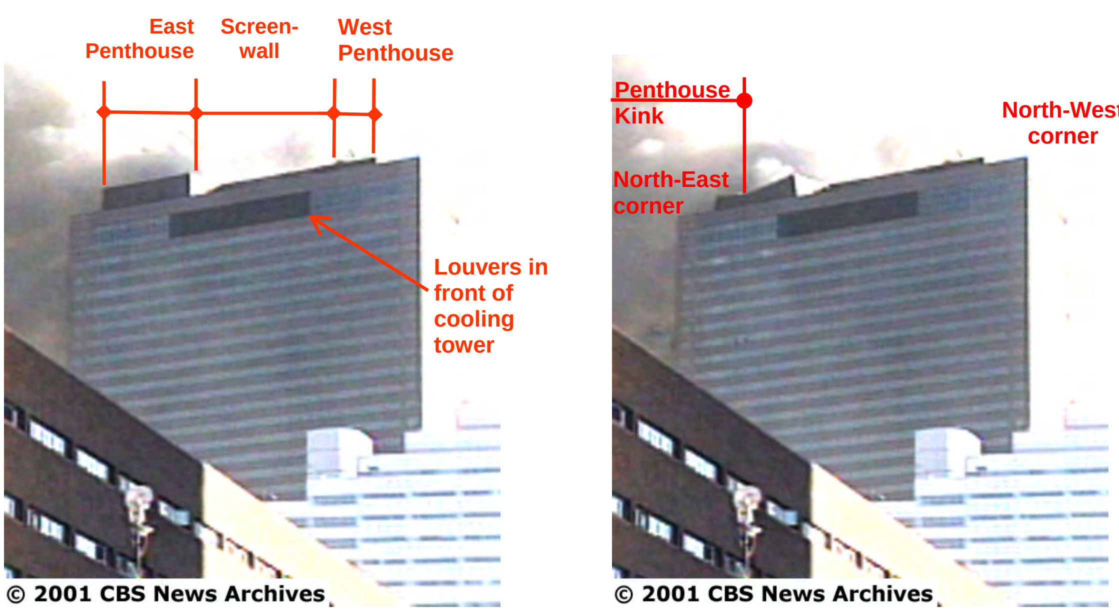


Figure L–25. East penthouse kink.

Figure L–26. East penthouse sinks (2.2 s).

Figure L–27. Center screenwall and west penthouse sink (7.9 s).

Figure L–28. Global collapse.

Possible Locations of Collapse Initiation

Columns 76, 77, 78, 79, 80, and 81 appear to have direct influence on the collapse initiation of the east penthouse. A failure of any of these columns, truss #1 or #2, or the east transfer girder, or some combination of these components, with possible contribution of adjacent framing and floor systems, could be considered possible locations of the initiating events that led to the observed collapse of the east penthouse.

L.2.4 Interpretation of Collapse Initiation Observations in Elevation

In addition to determining some possible locations of the collapse initiation locations within the plan of the structure, it is also helpful to use the available collapse documentation to identify possible locations in the building elevation for the initial failure.

Least Likely Locations of Collapse Initiation–Penthouse Failure Mechanism

Because the first visible failure is in the east penthouse, one possible collapse initiation mechanism involves a local failure of the penthouse framing, which then progressed down the structure with floors sequentially impacting upon those below. There are two reasons that this scenario may be considered unlikely.

First, there was no visible abnormal loading locally applied to cause a local failure at the East Penthouse. The photograph in Fig. L–21 shows the east penthouse sustained no damage due to the collapse of the WTC towers. The videographic records do not show any visible fire in or near the penthouse prior to collapse.

Second, Fig. L–25 shows a snapshot as the east penthouse starts to collapse. When the roof of the penthouse starts to fall, a line of windows (roughly in line with columns 79 to 81) has broken over the entire height of the visible region. In free fall, it would take 3 to 4 seconds for an object to fall from the roof elevation to the height of the bottom visible broken window, around Floor 33. Since the bottom window is broken nearly simultaneously when the kink is seen at the east penthouse, the initial failure may be assumed to have propagated upward from the lowest window breakage rather than propagated downward from the top of the building. Therefore, initial failure within the penthouse may be considered unlikely.

Less Likely Locations of Collapse Initiation–High Elevation Column Failure Mechanism

Another possible collapse initiation mechanism may be the failure of a column in the upper elevations of the building. The collapse could have progressed vertically upward by pulling down the floors above the failed column as debris landed on and sequentially crushed the floors below.

The timing required for this mechanism, in accordance with gravitational acceleration, requires that any column locations significantly above the 13th floor (the lowest visible floor in photographic and videographic records) may be considered unlikely failure initiation locations. The lack of observed fires in the floors above Floor 13 also reduces the likelihood of failure initiating in this region of the building.

Possible Locations of Collapse Initiation Mechanism

Based on review of the photographic and videographic records, a failure of any column within the plan area shown in Fig. L–29, and below Floor 13, likely contributed to the collapse initiation. This includes columns 76, 77, 78, 79, 80, and 81, truss #1, truss #2, column 78A, the east transfer girder and adjacent framing and floor systems within this region (see Fig. L–30).

L.2.5 Interpretation of Collapse Progression Observations

Interior columns 79, 80, and 81, were located directly below the east penthouse on the roof and supported large tributary areas. It appears that some sequence of component failures in the region identified in Figs. L–29 and L–30 led to the failure of one or more of these columns, as discussed above. The failure progressed vertically upward within the failed bay to the roof level, based upon observations of window breakage relative to failure of rooftop structures, and was first visible from the exterior when the east penthouse lost support (see Fig. L–26).

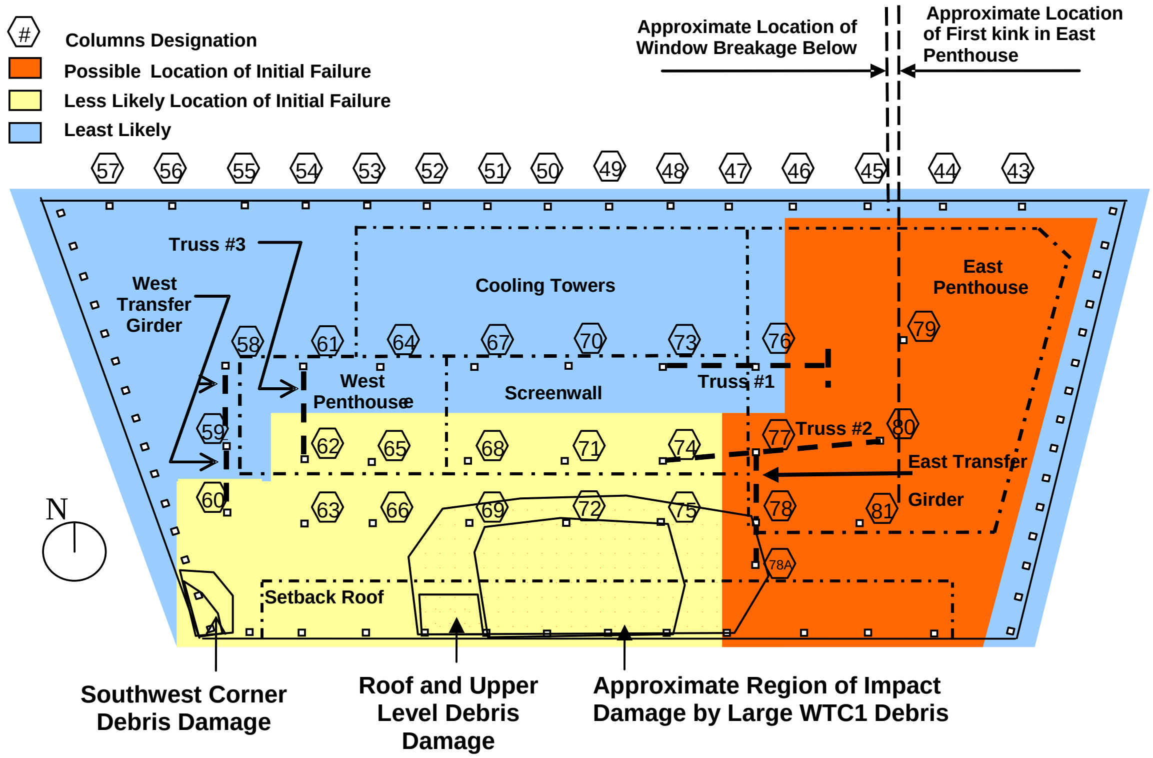


Figure L–29. Plan view of regions for collapse initiation.

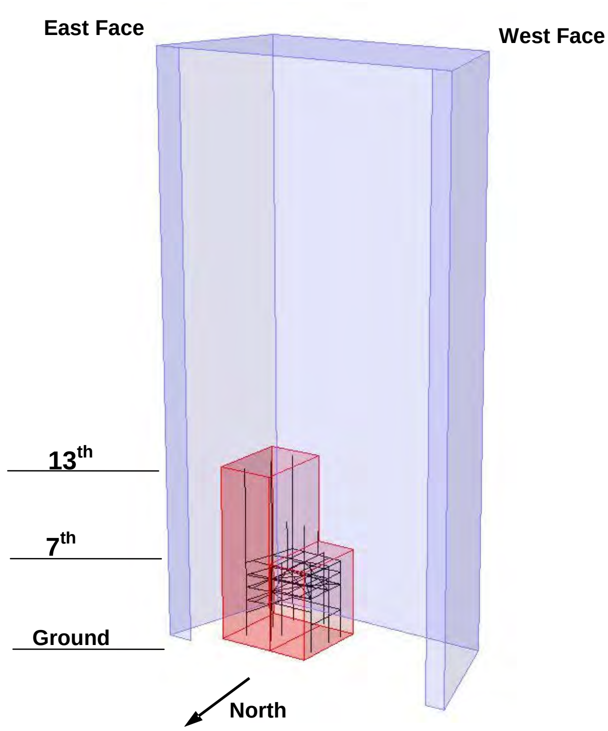


Figure L–30. Likely region of initiating component failures based on videographic and photographic records of fire and collapse.

The 5 s to 6 s delay between the failure of the east penthouse and the failure of the screenwall and west penthouse (shown in Fig. L–27) approximates the time it would take for the debris pile from the vertical failure progression on the east side of the building to reach Floors 5 to 7 and damage the transfer trusses and girders in this area.

A kink developed in the north facade approximately where column 76 projects to the north face. The kink may have formed in the plane of the north facade or it may represent a displacement in the structure along this line towards the south. The area of this kink correlates to the easternmost cantilever transfer at Floor 7. All of the Floor 7 cantilever transfer girders had back spans supported along the line of the north core columns, of which the easternmost one was supported by truss # 1. This north facade kink also coincides with the girders at the eastern edge of the cooling tower area at Floor 46.

When the screenwall and the west penthouse sank into the building, a line of windows broke from Floor 44 down to the bottom of the visible range, which is approximately at Floor 33 on the west side of the structure (see Fig. L–27). This area aligns with column 61, which is supported by the cantilevered end of transfer truss #3 between Floors 5 and 7, as shown in Fig. L–31. This suggests that the observed window breakage may be related to the failure of column 61 or truss #3.

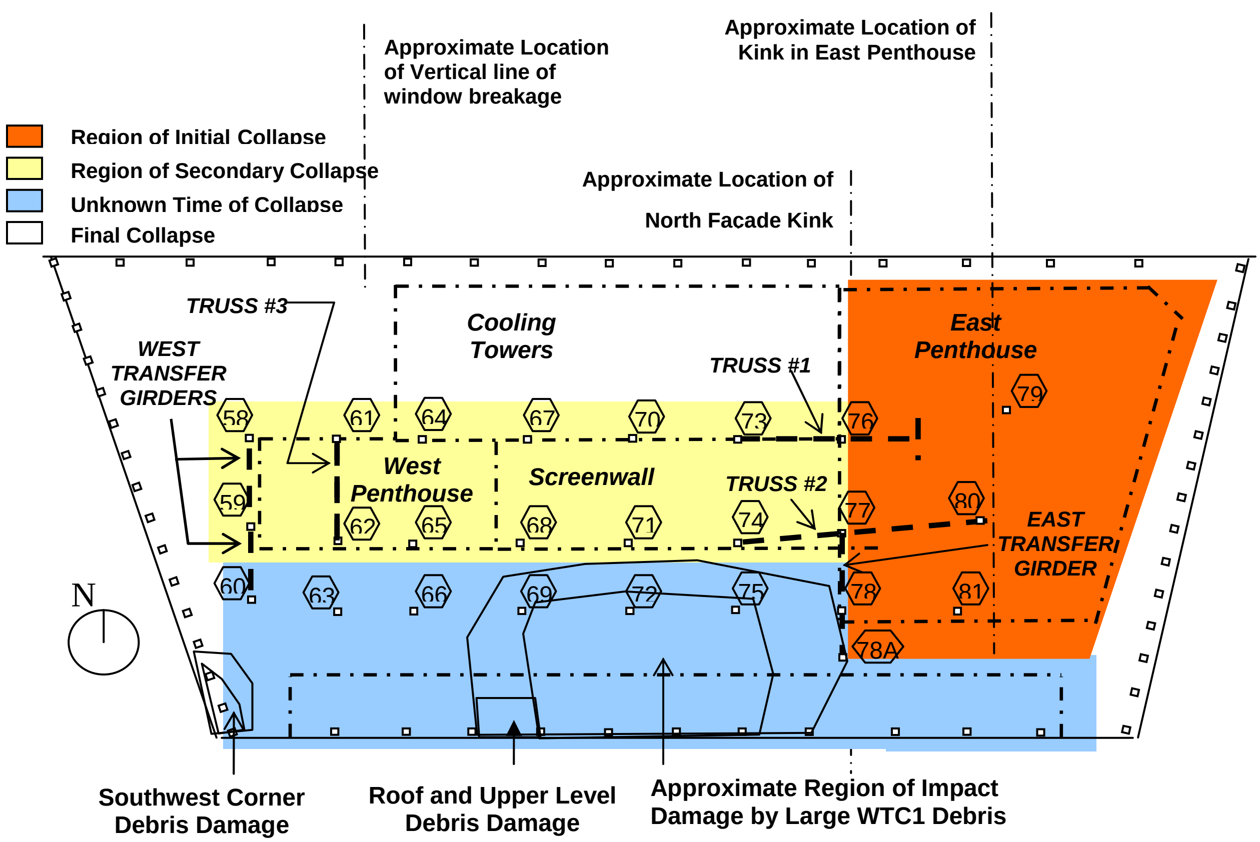


Figure L–31. Plan View of Collapse Progression.

The simultaneous failure of screenwall and west penthouse structures, window breakage on the west side of the north facade, and initiation of global collapse (see Fig. L–28) indicates that the building loads could no longer be supported. Horizontal progression of the collapse appears to have occurred after the vertical collapse on the east side of the building. The greater strength of Floors 5 and 7 relative to the other floors and the transfer trusses between these floors suggests that this region of the building played a key role in destabilizing the remaining core columns, and the global collapse occurred with few external signs prior to the system failure.

All of the photographic and videographic records show the north facade collapsing from below the visible area; the facade appears to sink into the ground without any sign of the other floors in the visible portion of the building collapsing. This may indicate that the collapse of the facade starts below the area visible in the photographic and videographic records.

L.2.6 Debris Field

The debris of WTC 7 was mostly contained within the original footprint of the building. From aerial photos, the debris visible on top of the pile is mostly façade structure. This failure sequence suggests that the interior of the building collapsed before the exterior. See Fig. L–32.

L.2.7 Summary

The possible region of collapse initiation and progression has been refined and can be limited based upon available data as follows:

- Based upon the observed fire locations, it appears that the initiating collapse event may have occurred on Floors 5 through 13.
- Due to the pattern of window breakage, it appears that the initiating collapse event may have occurred below Floor 13 and then progressed vertically upward to the east penthouse.
- Since the middle of the east penthouse roofline appears to fall first, it is possible that the initiating collapse event occurred at columns or transfer components with direct influence on the footprint of the east penthouse.
- The north facade kink and the window breakage on the west side of the north facade as the screenwall and west penthouse began to fall into the building core suggest that a horizontal collapse mechanism occurred between Floors 5 and 7, as there are vertical discontinuities in line with each of these elements between Floors 5 and 7.
- The relatively small debris field, with the exterior moment frame visible on top of the building debris, an internal collapse mechanism is likely.

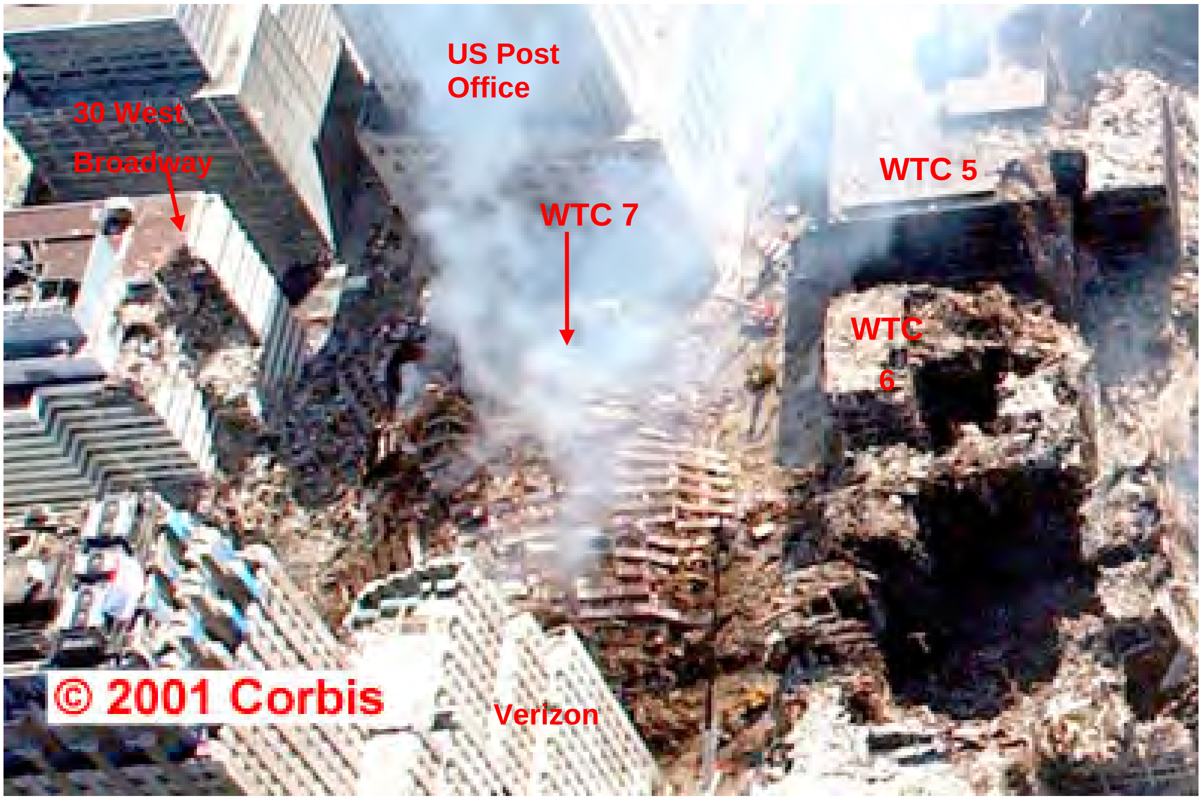


Figure L–32. Aerial view of WTC 7 after collapse.

The working collapse hypothesis can be summarized in Figs. L–33 and L–34, which illustrate the components of the observed collapse event: collapse initiation and vertical progression, horizontal progression, and global collapse.

L.3 COLLAPSE HYPOTHESIS

L.3.1 Introduction

WTC 7 suffered a global collapse. The initiating cause or causes of this collapse, and its sequence of events, are still being investigated though fire appears to have played a key role and there may have been some physical damage on the south side of the building.

To develop a working hypothesis for the collapse sequence, it is useful to subdivide the problem into several phases. Many factors and structural components may have contributed to the start of the collapse, but there must have been an initiating event. After the collapse initiated, it progressed to other parts of the building, leading to their failure as well. From the observations of the collapse (see Section L.2), it appears that first there was a vertical failure progression, from some point in the lower eastern portion of the building up to the east penthouse. After a time lag of approximately five seconds, the screenwall and west penthouse were observed to begin sinking into the core area. This suggests that there was a horizontal progression of the collapse towards the west. Since the screenwall and west penthouse fell almost simultaneously, it is reasonable to assume that the horizontal progression captured all the columns that support these building parts.

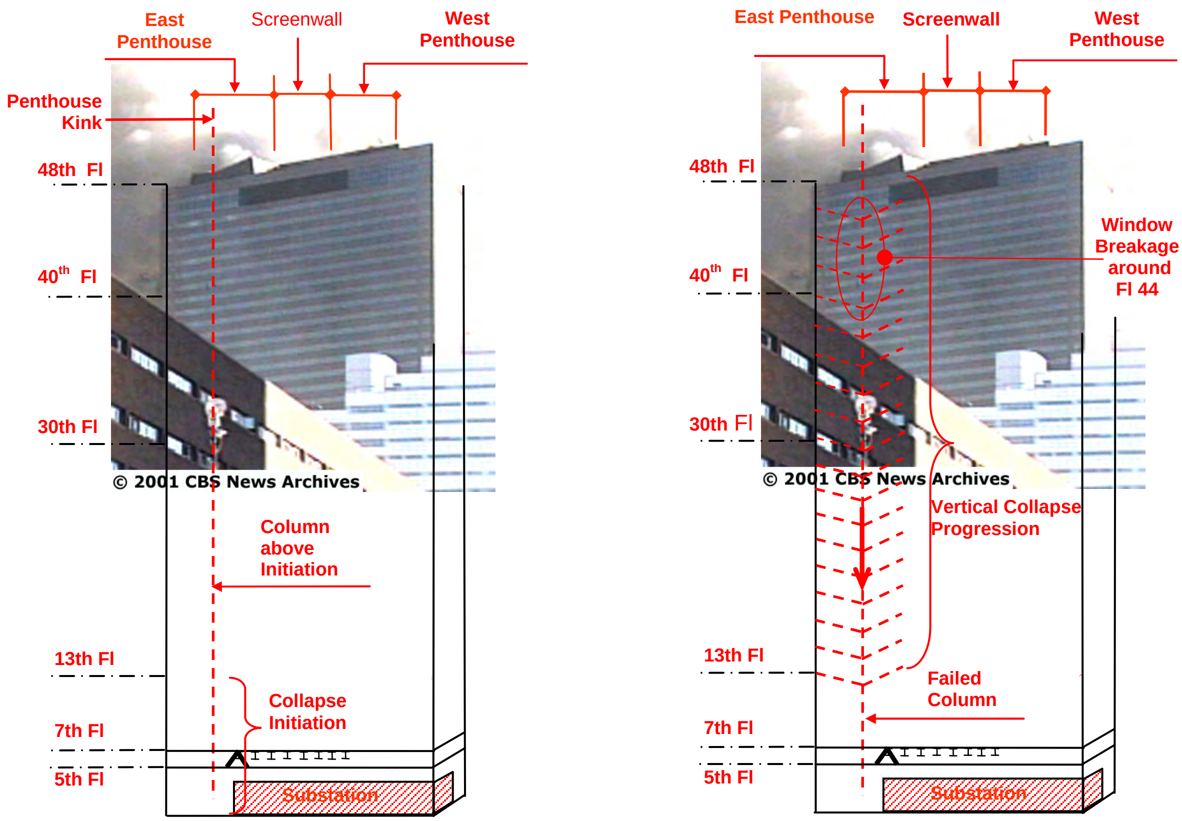


Figure L–33. Collapse initiation and vertical progression on the east side of WTC 7.

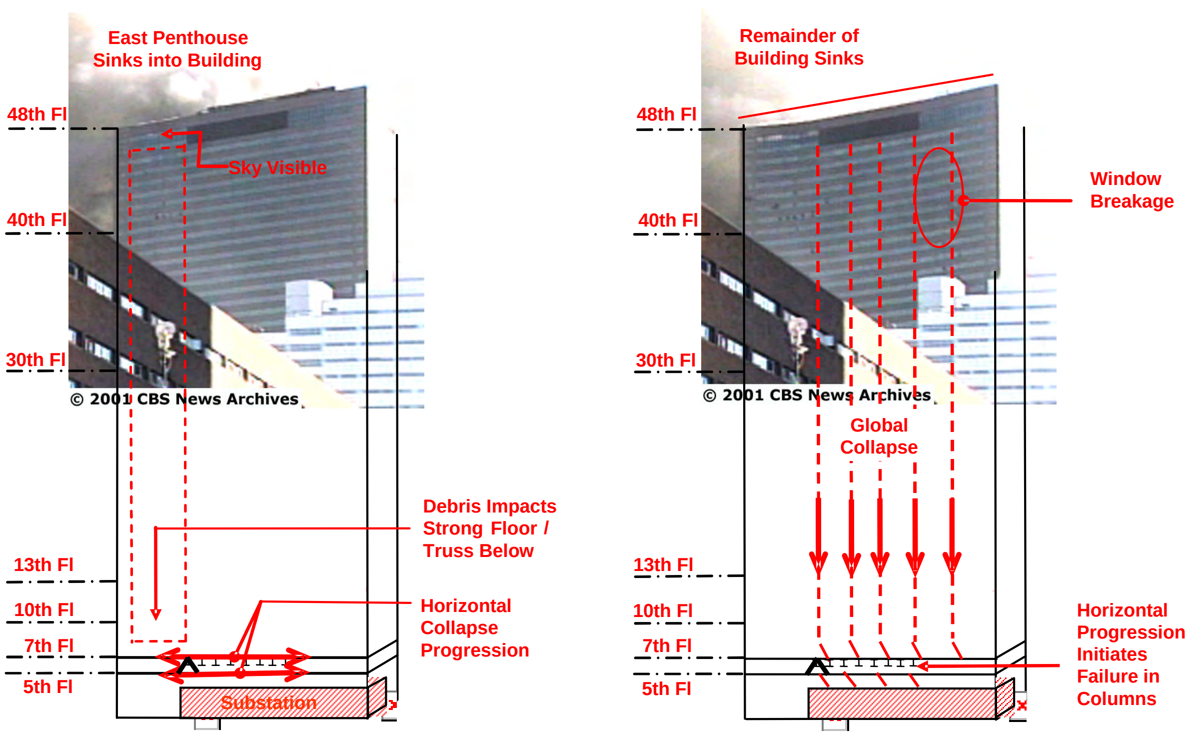


Figure L–34. Horizontal progression to the west side of WTC 7.

Within each phase of the collapse shown in Fig. L–35, the initiating event, vertical progression, horizontal progression, and global collapse, exist many possible scenarios. Scenarios have been developed from available observations of the collapse and are explored with event trees. Preliminary analyses, combined with the observations of collapse, can be used to prune the list of postulated scenarios to a relatively small number of possible collapse hypotheses. These possible hypotheses will then be analyzed in successive levels of detail to try to determine one or more probable sequence of events leading to the building collapse.

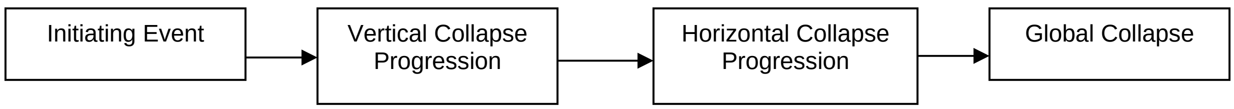


Figure L–35. Phases of WTC 7 building collapse.

L.3.2 Collapse Initiation Scenarios

For the collapse to have started, there must have been a component or group of components that failed first, referred to here as the initiating event, as shown in Fig. L–36. The initiating event may have included structural components severed or damaged by falling debris (I1.1) and/or structural components affected by fires (I1.2).

I1.1 Initiating Components Fail Due to Debris Damage From WTC 1 of WTC 2: The initiating components may have included perimeter or interior columns that were severed or damaged by falling debris from WTC 1 or WTC 2.

- **I2.1 Debris Damage to South Facade Columns:** Perimeter columns on the south face and the southwest corner were reported or observed in photographic and videographic records to have been severed or damaged after WTC 1 collapsed. If the initiating event was due to damage to the perimeter moment frame, then it would have started along the south or southwest facade. Photographic and videographic records show that columns on the north and east facades were undamaged by debris impact.

  – **I3.1 Perimeter Moment Frame Arrests Failure Progression:** Analysis of the global structure indicates that the structure redistributed loads around the severed and damaged areas. A progression of column failure to adjacent columns would have been arrested by the vierendeel action of the perimeter moment frame, which could span across a sizeable opening due to the strength and stiffness of the frame.

  – **I2.2 Debris Damage to Interior Columns:** Interior columns may have been severed or damaged by impacting debris.

  – **I3.2 Interior Columns Fail Immediately:** If interior columns had been severed or severely deformed, they may have failed immediately.

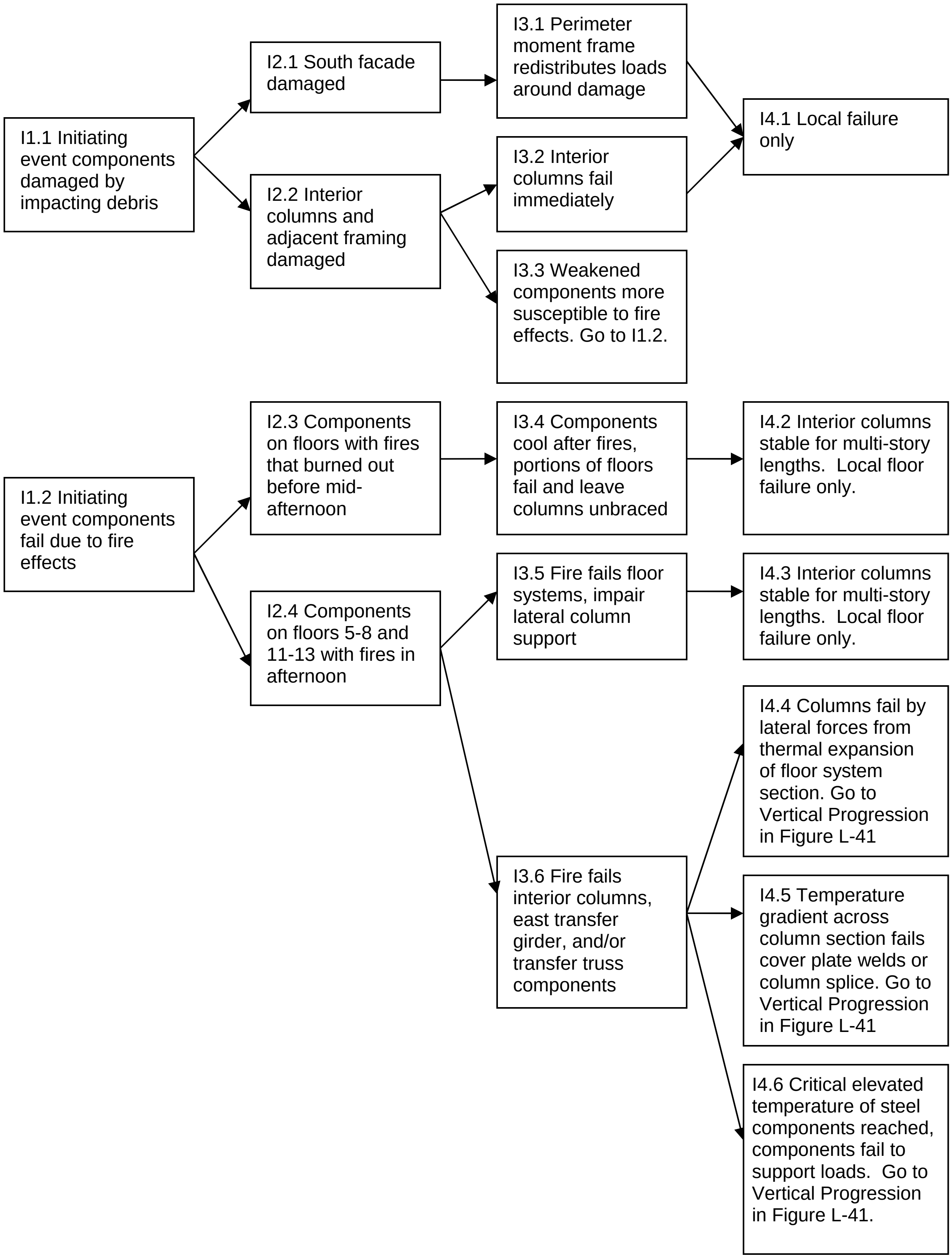


Figure L–36. Collapse initiation scenarios.

- I4.1 **Localized Failure at Interior Columns:** If the interior columns failed just after impact, this likely resulted in a local failure only, since the building continued to stand for almost 7 hours after WTC 1 collapsed. This failure could have progressed vertically upward to the roof level within the bays immediately adjacent to the failed columns, yet from the northern vantage point of the photographic and videographic observations, would not have been visible.

– **I3.3 Interior Columns Remain Standing But Damaged:** If interior columns were weakened by damage from debris, but retained sufficient capacity to carry their loads, then additional loading and/or fire effects would have been required to cause their failure. Debris impact may have damaged the structural steel fireproofing without significantly deforming the structural component.

I1.2 Initiating Components Fail Due to Fire Effects: Fires had been burning in WTC 7 for many hours, as observed in the photographic and videographic records (see section L.2). The initiating event may have been caused by fire effects on structural components.

- **I2.3 Components on Floors With Burned Out Fires:** If the initiating components failed from fire effects, then locations where fires had burned out by mid afternoon could possibly been affected by the cooling which occurs after a fire. No fire was observed or reported in the afternoon on Floors 1–5, 10, or above Floor 13.

– **I3.4 Floor Systems Fail:** The cooling that may have occurred as the fires burned out in an area may have generated thermal contraction forces, which may have induced tensile forces at floor-to-column connections.

- **I4.2 Unbraced Columns:** If floor systems failed, one or more columns may have lost lateral bracing. At a floor where fires were noted, interior columns were comprised of W14x730 cores and reinforcing plates, and could support several stories unbraced without failure. As an example, the column capacity curve of column 79 between Floors 5 to 9 is shown in Fig. L–37. Column load-carrying capacities shown in this figure are based on the AISC column capacity formulas (AISC 2001). The column is not very sensitive to the number of stories of unbraced column length, K. This column, which had a service load stress of approximately 21 ksi, would be approaching its load carrying capacity for an unsupported length of four stories if it was also subject to a uniform temperature of 500 °C.

- **I2.4 Components on Floors With Fire:** If the initiating components failed because of fire effects, then locations with uncontrolled fires would be more likely for the initiating event. From available data of fire locations in WTC 7, likely locations would include Floors 5, 6, 7, 8, 9, 11, 12, and 13. No fires were observed on Floor 5, but the lack of windows and the presence of fuel systems on the south, west, and north floor areas indicate that fire should be considered as a possibility on this floor.

– **I3.5 Floor System Failure:** The fires could have caused the failure of portions of one or more floor system and its framing connections.

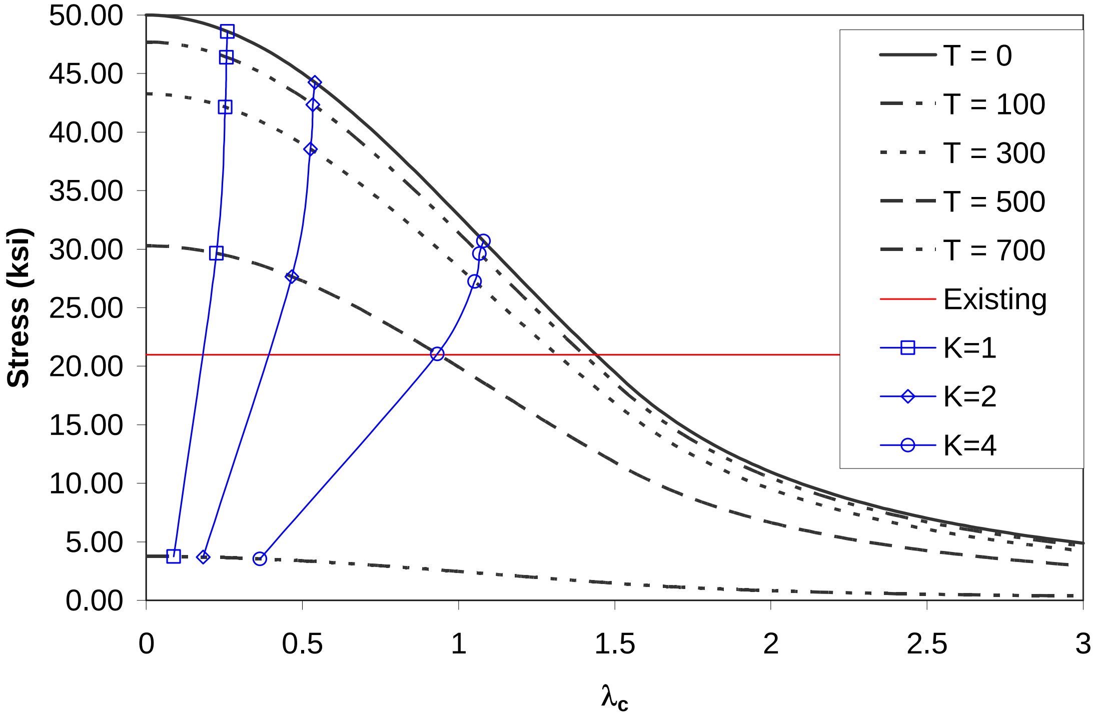


Figure L–37. Column 79 capacity versus temperature and unbraced length K.

- o **I4.3 Unbraced Columns:** If floor systems failed, one or more columns may have lost lateral bracing. See I4.2 for discussion.

– **I3.6 Columns, Transfer Girders or Transfer Trusses Fail:** The fires could have failed interior columns, transfer girders, transfer trusses, or their framing connections.

- o **I4.4 Lateral Displacements:** Fire effects may have caused column instability failure by lateral displacements from asymmetric thermal expansion of the floor system. Such thermally-induced displacements must overcome the restraining effect of the remaining floor system against further lateral deflection of the column.
- o **I4.5 Temperature Gradients:** Fire effects may have caused the failure of columns and other components through the forces induced by temperature gradients through their cross section. Bending and shear forces may be induced that are sufficient to yield either the column splice or reinforcing plate welds. Analysis of a one-story segment of interior column 79 indicates that the cover plate weld would begin to yield at a mean temperature of 490 °C with a 200 °C gradient across the section, as shown in Fig. L–38. Other mean temperature and gradient combinations may also cause this type of failure.

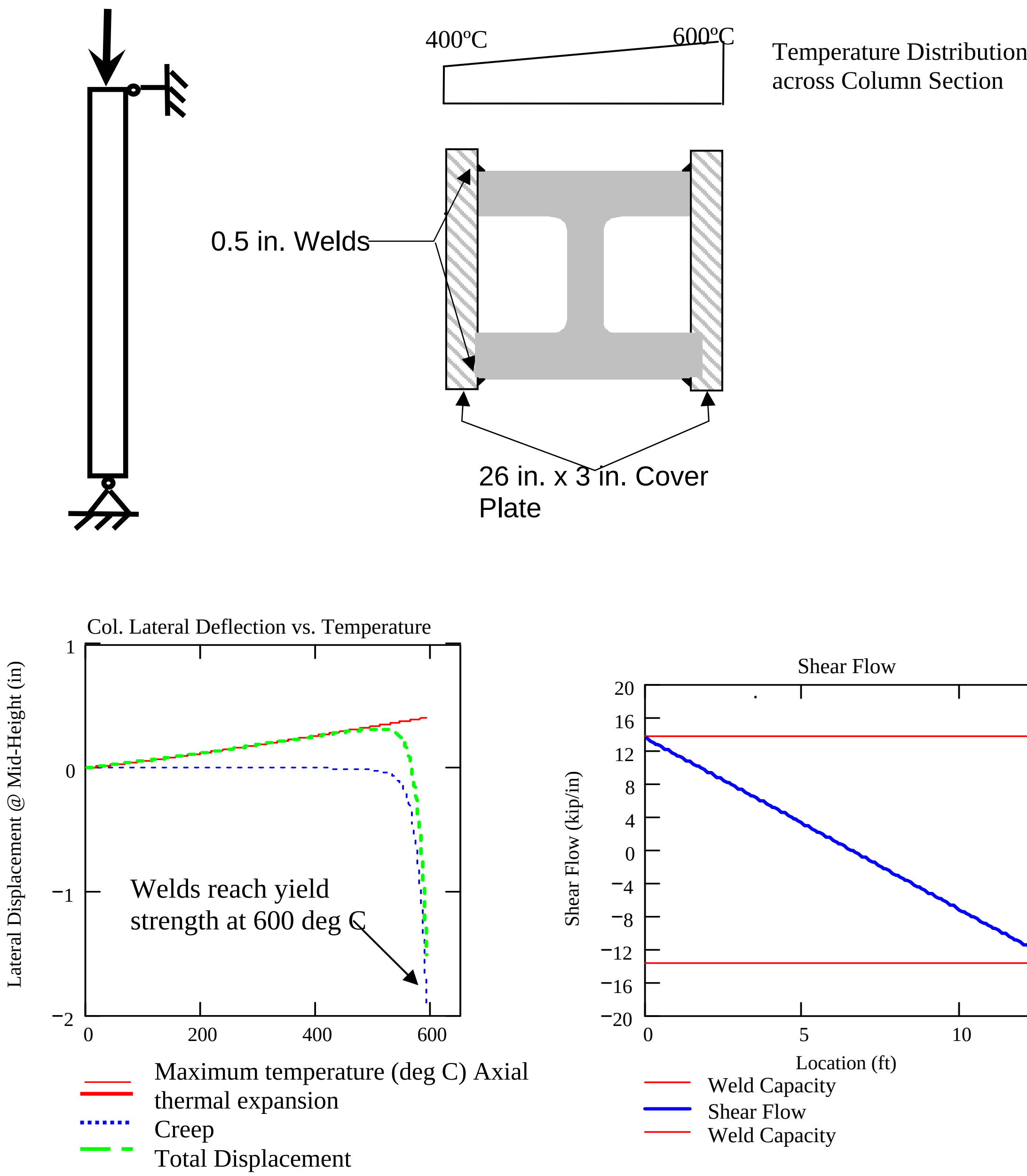


Figure L–38. Effects of temperature gradient on interior column 79.

- **I4.6 Uniform High Temperatures:** If initiating event components were sufficiently exposed to fire effects to be uniformly heated to elevated temperatures, the steel strength would be reduced below that required to support the load. Figure L–39 shows that for interior columns subject to service loads (shown as approximately 20 ksi of compressive stress), uniform steel temperatures of approximately 570 ºC would result in column failure.

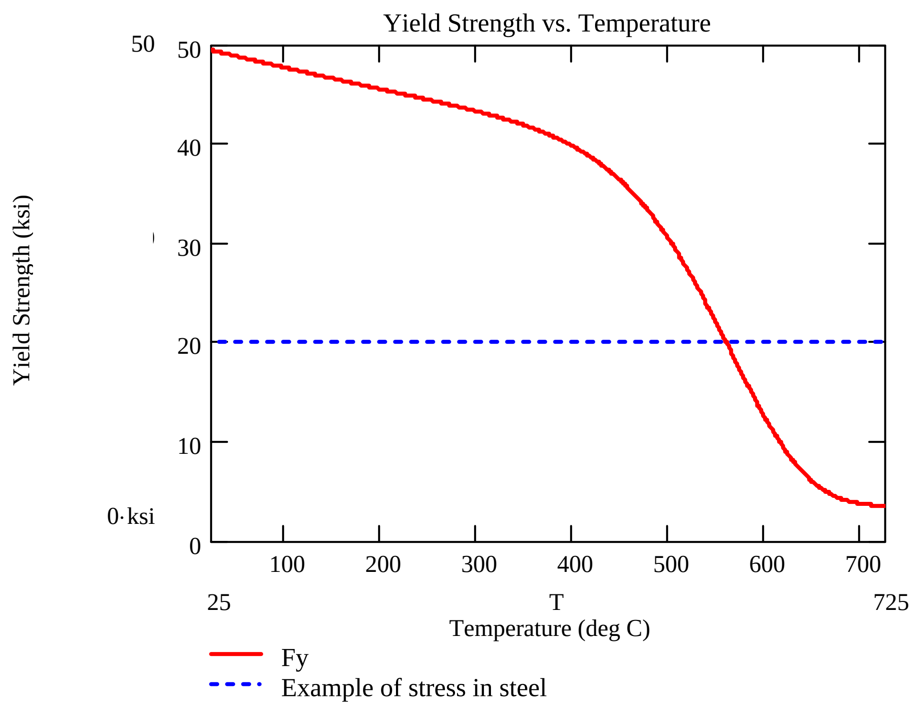


Figure L–39. Steel strength versus temperature.

L.3.3 Vertical Progression Scenarios

After the initiating component or components failed, there must have been a progression of the failure from the initiating event to other locations. To reflect the observed failure of the east penthouse, the failure likely progressed vertically upwards. Figure L–40 shows possible vertical progression scenarios. The initiating component could have failed by any of the failure sequences listed under the collapse initiation scenarios in Fig. L–36. This component could have been one of the columns under the east penthouse. It could also have been one of transfer trusses #1 or #2 under the east penthouse.

A collapse mechanism model was created to capture possible collapse initiation at the roof and the east penthouse. The model seeks to simulate only the kinematics of the collapse mechanism when columns are removed. Several columns were tested for removal. The resulting geometry change was then compared to the observed collapse of WTC 7.

V1.1 Perimeter Columns Fail: Had the initiating component been any perimeter column, most likely it would have been at floor levels with debris impact damage (possible range extends from the ground level up to floors 15 to 20) or the floors possibly experiencing fire (Floors 5, 6, 7, 8, 9, 11, 12, or 13).

- **V2.1 Collapse Does Not Progress:** If a group of perimeter columns failed, the perimeter framing above this area would have redistributed its loads, due to the redundancy of the moment frame.

V1.2 Core Columns Not Directly under East Penthouse Fail: Had the initiating component been a core column that was not under the east penthouse, most likely it would have been at floor levels with debris impact damage (possible range extends from the ground level up to Floors 15 to 20) or the floors

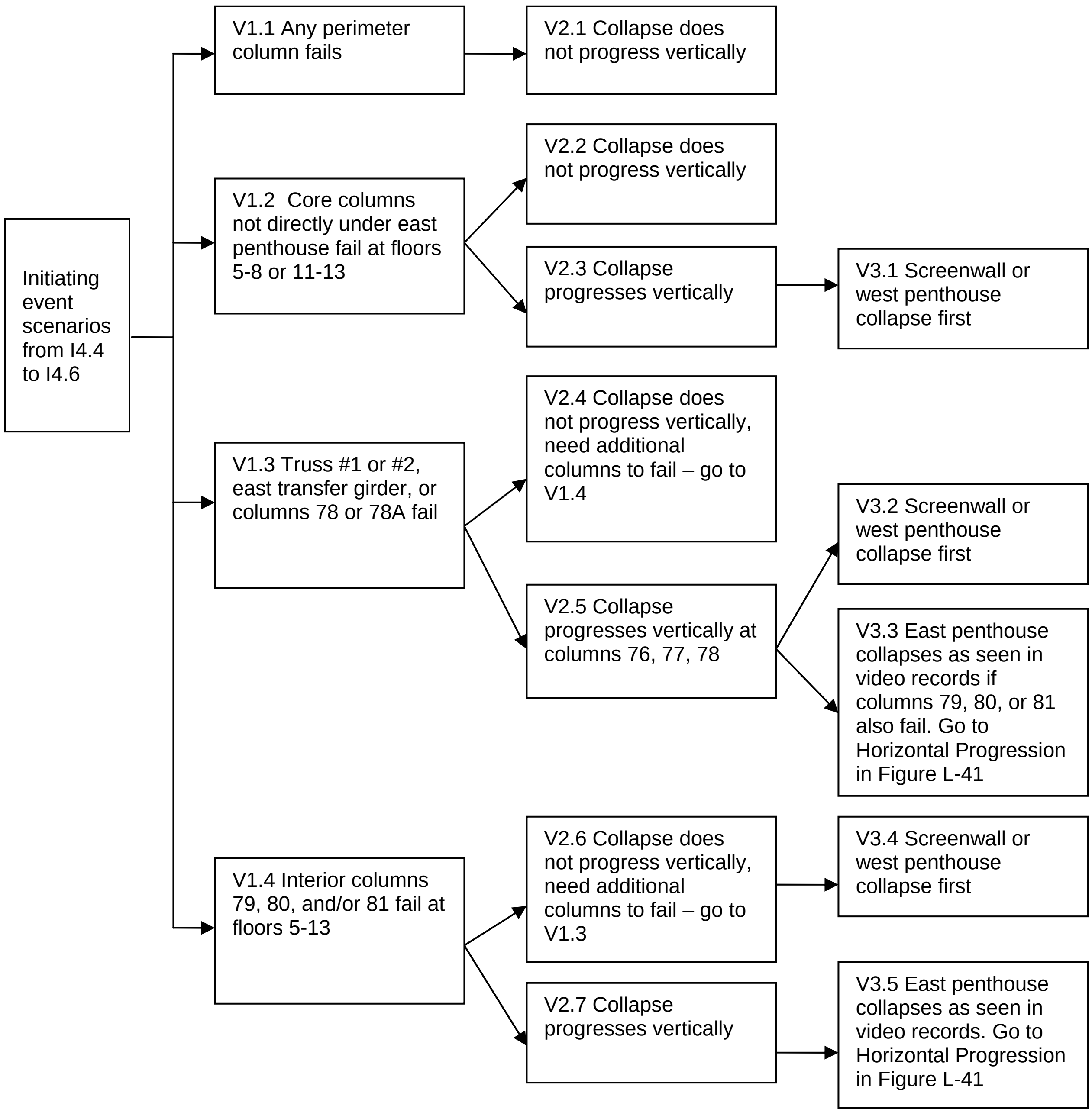


Figure L–40. Vertical collapse progression scenarios.

possibly experiencing fire (Floors 5, 6, 7, 8, 9, 11, 12, or 13). However, a core column may have failed following the failure of adjacent columns or framing members.

- **V2.2 Collapse Does not Progress:** If core columns failed, the loads above the failed columns may have been redistributed to adjacent columns through the core floor system. If the loads could not be redistributed, then additional failures in one or more components would have been necessary to progress the collapse.

- **V2.3 Collapse Progresses:** From this initial failure, the portion of the column above the failure could have fallen, progressing the failure vertically upwards.

- **V3.1 Something Else Besides East Penthouse Observed to Collapse First:** Had the failure of core columns progressed upwards, then the first exterior sign of the internal failure likely would have been seen in the screenwall or west penthouse, which are located above the core columns. A collapse mechanism analysis performed for the removal of columns 61, 64, 67, 70, and 73 produced geometry changes that differed from the observed collapse. For the scenario where each of these columns fails and the failure progresses upwards to the roof line as the adjacent floors cannot redistribute the loads, the screenwall or the west penthouse collapses, and no kink develops in the east penthouse (see Figs. L–41, L–42, and L–43).

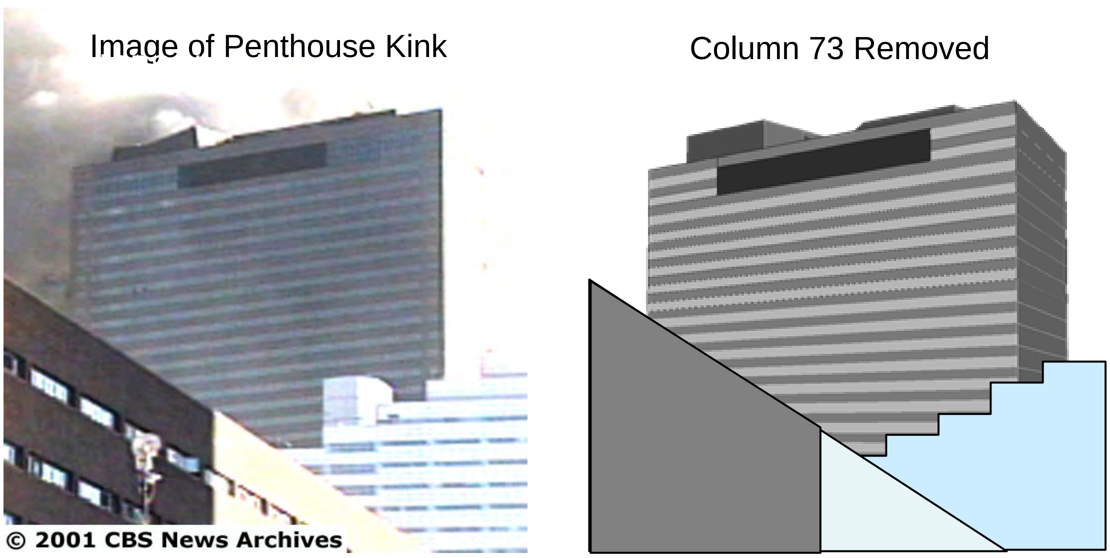


Figure L–41. Geometry changes for removal of column 73.

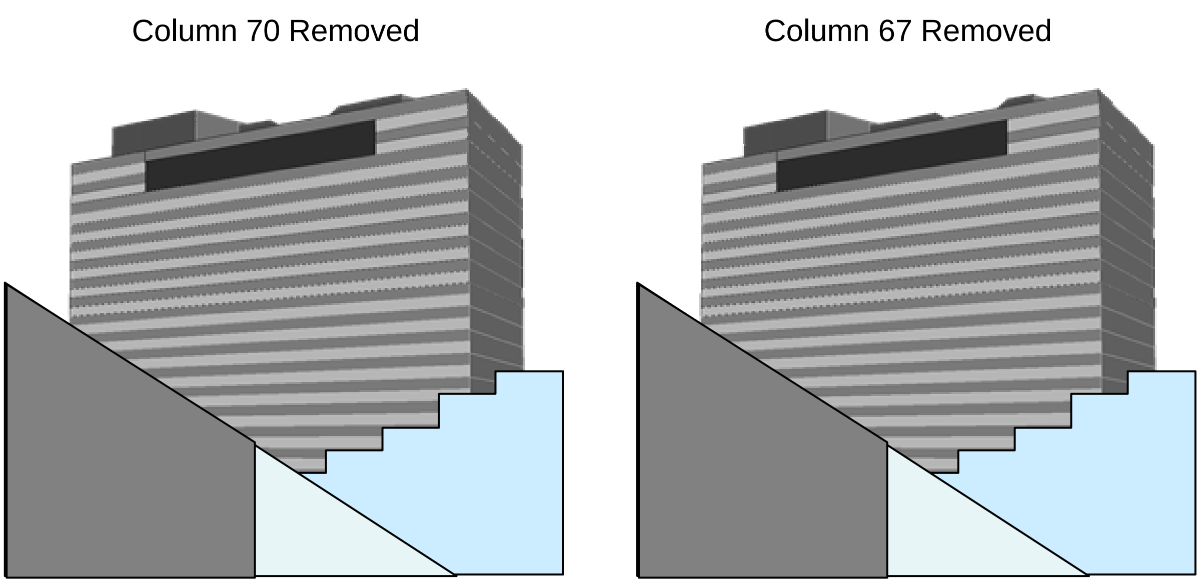


Figure L–42. Geometry changes for removal of columns 70 and 67.

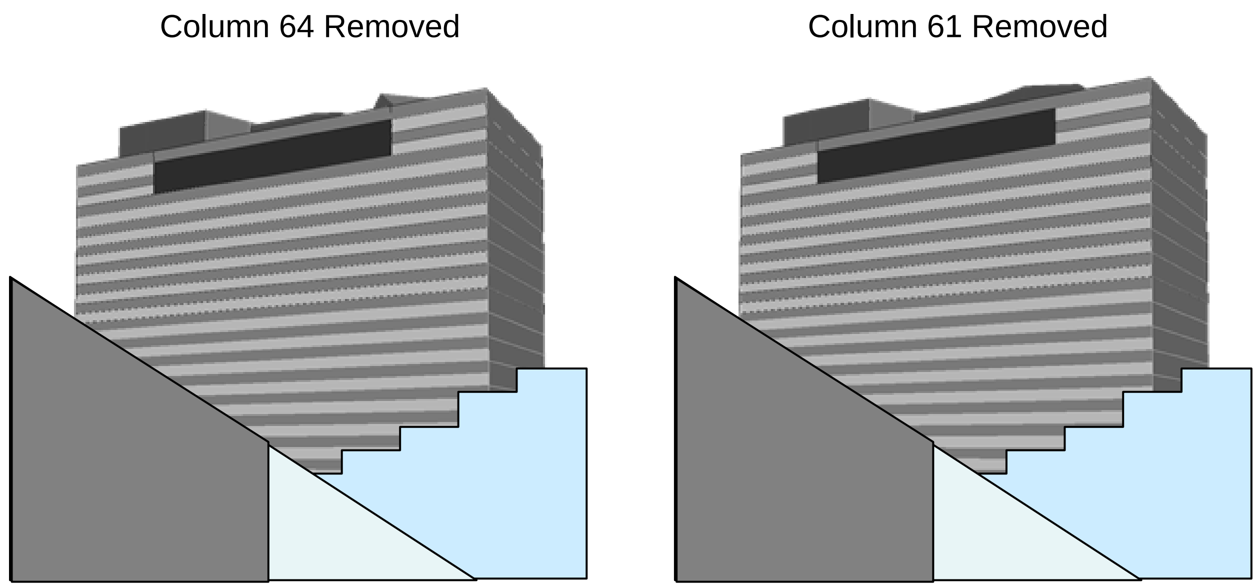


Figure L–43. Geometry changes for removal of columns 64 and 61.

V1.3 Truss #1 or Truss #2 East Transfer Girder, or Columns 78 or 78A Fail: Had the initiating component been truss #1 or truss #2, most likely there would have been debris impact damage or possibly fires at Floors 5 or 6. However, truss #2 failure could have followed the failure of the east transfer girder or columns 78 or 78A.

- **V2.4 Collapse Does Not Progress:** If truss #1 or #2 failed, the floor framing, including the Floor 7 diaphragm, may have redistributed the loads to adjacent columns. Had this occurred, additional failures in one or more components would have been necessary to progress the collapse. For instance, the columns 76, 77, 78, 79, 80, or 81 may also have failed, and the combined effect of both component failures could have been sufficient to overcome the supporting strength of the floor systems.

- **V2.5 Collapse Progresses:** If truss #1 failed, column 76 would lose its support at Floor 7, and the failure could have progressed vertically upwards if the floors could not redistribute column 76 loads. If truss #2 failed, columns 77 and 78 would lose their support at Floor 7, and the failure could have progressed vertically upwards if the floors could not redistribute the loads from columns 77 and 78.

  - **V3.2 East Penthouse Collapses Differently Than Observed:** If truss #1 or truss #2 failed and the failure progressed vertically upward to the roof level, the exterior deformations observed in the roof structures would be different from what was actually observed. Column 76 supported the west side of the east penthouse and the east end of the screenwall. A collapse mechanism analysis performed for the removal of column 76 produced a geometry change that shows the west side of the east penthouse and the east end of the screenwall deflecting downward (see Fig. L–44).

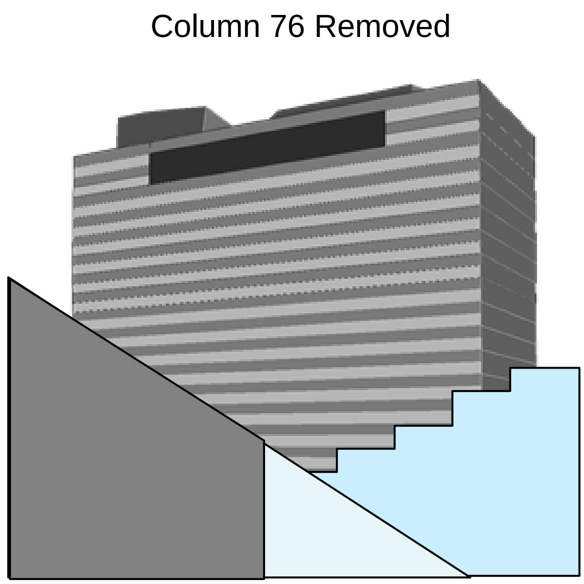


Figure L–44. Geometry changes for removal of column 76.

- **V3.3 East Penthouse Collapses as Observed:** Had the failure of columns 76 or 77 and 78 been followed by the failure of columns 79, 80, or 81, such that the failure of column 79, 80, or 81 progressed upwards, while the vertical progression of failure above columns 76, 77, and 78 was arrested, then the first exterior sign of the internal failures could have been observed at the center of the east penthouse roof.

V1.4 Interior Columns 79, 80 or 81: Had the initiating component been column 79, 80 or 81, most likely the failure would have occurred at the floors possibly experiencing fire (Floors 5, 6, 7, 8, 9, 11, 12, or 13).

- **V2.6 Collapse Does Not Progress:** If only one of columns 79, 80, or 81 failed, the floor systems above the failure area may have redistributed the column loads to adjacent columns. Had this occurred, additional failures in one or more components would have been necessary to progress the collapse vertically upwards. For instance, both columns 79 and 80 may have failed, and the loads of both columns could have been sufficient to overcome the supporting strength of the floor systems.

- **V2.7 Collapse Progresses:** If only one of columns 79, 80, or 81 failed, the floor systems above the failure area may have not been able to redistribute the column loads to adjacent columns. The floor system above Floor 7 had beams and girders, concrete slabs on metal deck, wire mesh in tenant floor areas, and rebar in the core area slabs. These floor systems do not appear to have sufficient bending or catenary action to redistribute loads for failure of column 79, 80, or 81. A calculation of the catenary action that might be developed by the beams and girders framing into column 79, assuming the floors try to redistribute the loads above the area of column 79 failure, found that the girder connections reach their capacity at approximately 10 percent of the estimated service loads and the connections probably fail at approximately 25 percent of the service loads. If the floor-to-column connections had not failed, the beams would have started to yield axially at approximately 40 percent of the service load present.

- **V3.4 East Penthouse Collapses Differently than Observed:** The collapse could have progressed upwards, but the failure caused in the east penthouse could be different than what was actually observed.
- **V3.5 East Penthouse Collapses as Observed:** Had the failure of the column progressed upwards, then it could have been reflected in the observed collapse of the east penthouse, which sits directly above columns 79, 80, and 81. Also, the kink observed in the roof of the east penthouse was in line with these columns. A collapse mechanism analysis performed for the removal of column 79 produced a deformed shape with a kink in the roof of the east penthouse (see Fig. L–45). This is a possible collapse scenario.

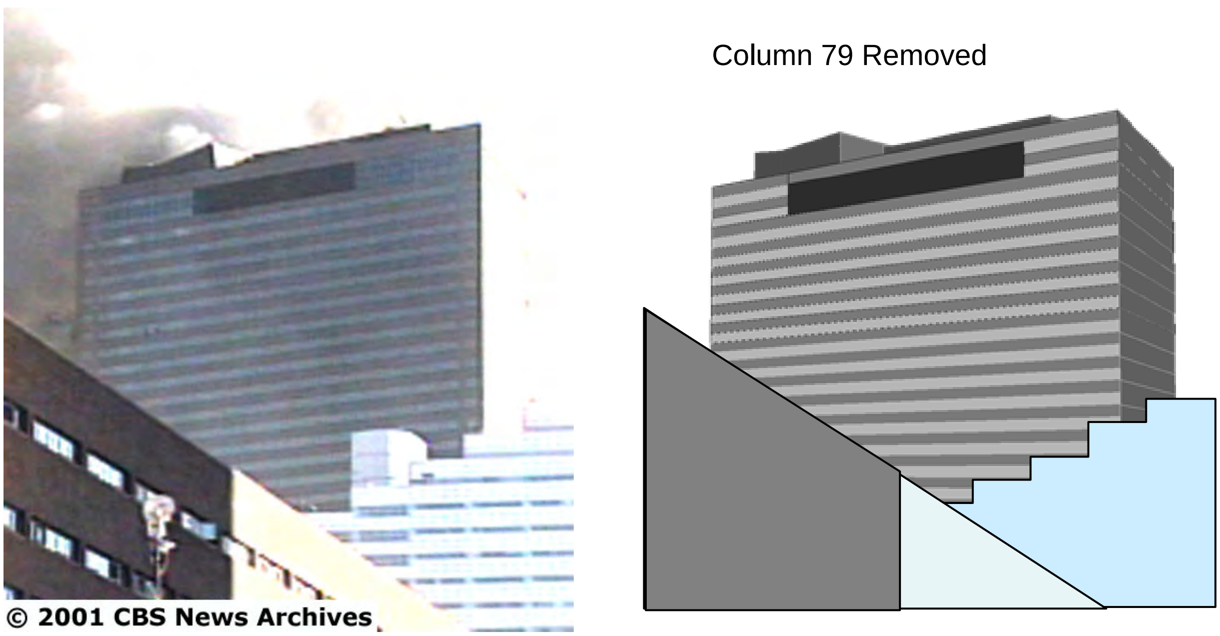


Figure L–45. Geometry change for removal of column 79.

L.3.4 Horizontal Progression Scenarios

After the east penthouse was observed to sink into the building core, approximately five seconds lapsed before the screenwall and west penthouse were observed to also sink into the building core. The screenwall and west penthouse movements occurred almost simultaneously with the global collapse of the structure. From these external observations, it appears that after the vertical progression failure on the east side of the building, the failure progressed horizontally across the core. The horizontal progression of the collapse could have started due to any of the likely vertical progression scenarios, which are shown in Fig. L–46.

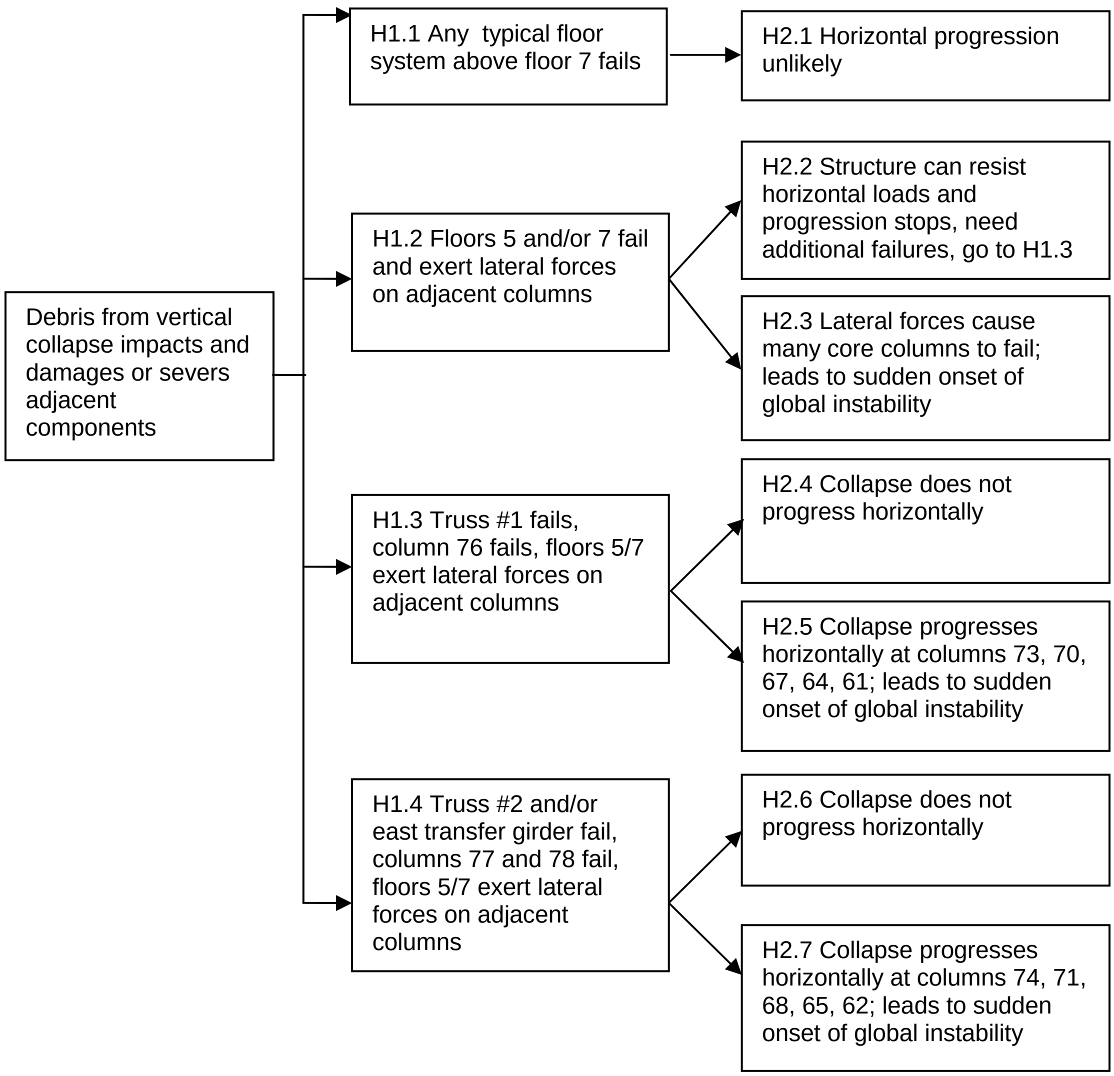


Figure L–46. Horizontal collapse progression scenarios.

The likely region in which the horizontal progression occurred is in the lower portion of the building, around Floors 5 and 7. Floor 5 had a 14 in. reinforced concrete slab on metal deck. The slab was heavily reinforced, and contained steel WT sections embedded in the slab. The WT sections were arranged in a diagonal pattern, like a horizontal truss, within the plane of the floor between the perimeter and core columns. Floor 7 had an 8 in. reinforced concrete slab on metal deck with rebar in each direction. The beams between interior columns at Floors 5 and 7 were much larger than at other floors, and the beam-to-column connections were able to transfer more of the beam axial and bending load capacity. These strong lateral ties between the interior columns may have been able to impose lateral displacements on adjacent columns. Transfer trusses and girders between Floors 5 and 7 transferred loads from the columns above Floor 7 to the foundation columns below Floor 5.

Assuming that a vertical collapse of one or more bays occurred over the height of the building, a large pile of debris would have fallen on Floor 7 and below. Such a large amount of debris is likely to have severed the Floor 7 slab and damaged or severed any transfer truss or girders in the vicinity. For a vertical collapse on the east side of the building, transfer trusses #1 and #2 and the east transfer girder may have been damaged, particularly the east diagonals of the trusses. The scenarios below describe possible responses of Floors 5 and 7 following a vertical collapse of one or more bays.

H1.1 Floor Systems above Floor 7: Typical tenant floors above Floor 7 were constructed with concrete slabs metal deck with wire mesh reinforcement. The steel framing connections were designed for shear loads only, though they could likely resist some degree of tensile catenary forces.

- **H2.1 Collapse Does Not Progress:** For any interior column failure above Floor 7, the tenant and core floor systems are not able to develop sufficient axial tensile loads for imposing lateral deflections on adjacent columns. It is likely that the floor system within a bay will fail before a column failure is propagated horizontally to adjacent columns.

H1.2 Floors 5 and 7: Floors 5 and 7 were thicker and more heavily reinforced than the typical floor systems, and may have been subjected to a large debris load from a vertical collapse within one or more bays.

- **H2.2 Collapse Does Not Progress:** Floors 5 and 7 may fail at connections to adjacent columns before developing any tensile forces large enough to cause other column failures through lateral displacements, halting the horizontal progression.

- **H2.3 Collapse Progresses:** Floors 5 and 7 may impose large tensile forces at the adjacent columns to cause lateral displacements that fail the columns. The failure mechanism could occur at the column splice, located just above Floor 5 and Floor 7, rather than through the column section. The simultaneous occurrence of column instability in many core columns would cause a sudden and large change in the structural system capacity.

H1.3 Truss #1: If one of the diagonals of truss # 1 (see Fig. L–47) was damaged or severed by collapse debris from the vertical progression, there would be a horizontal force developed in the Floor 7 slab as column 76 became unstable. The floor beam between column 76 and column 73 would try to restrain column 76 movement through tensile forces to column 73.

- **H2.4 Collapse Does Not Progress:** The horizontal tensile force would tend to pull the line of columns 73, 70, 67, 64, and 61 towards the east. The continuity of the Floor 7 slab and the presence of braced frames around the north core column line makes the simultaneous lateral displacement of the core columns less likely, as such displacements within a rigid slab may similarly displace other columns, including perimeter columns.

- **H2.5 Collapse Progresses:** The failure of column 76 may create its own vertical collapse, due to the inability of the floor systems above to redistribute the loads and fail at the column splices near Floors 5 and 7 as shown in Fig. L–48. If column 76 cannot be restrained and there is a vertical collapse of the surrounding bay, it would cause a debris pile at the lower floors which may then destabilize adjacent columns.

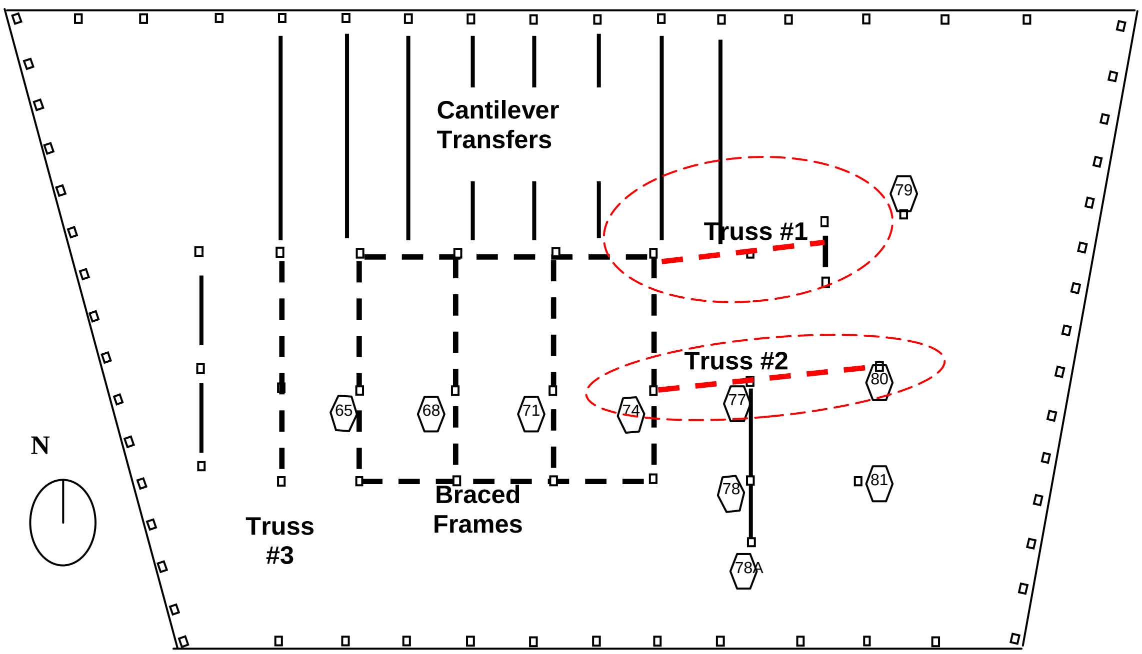


Figure L–47. Transfer components between Floors 5 and 7.

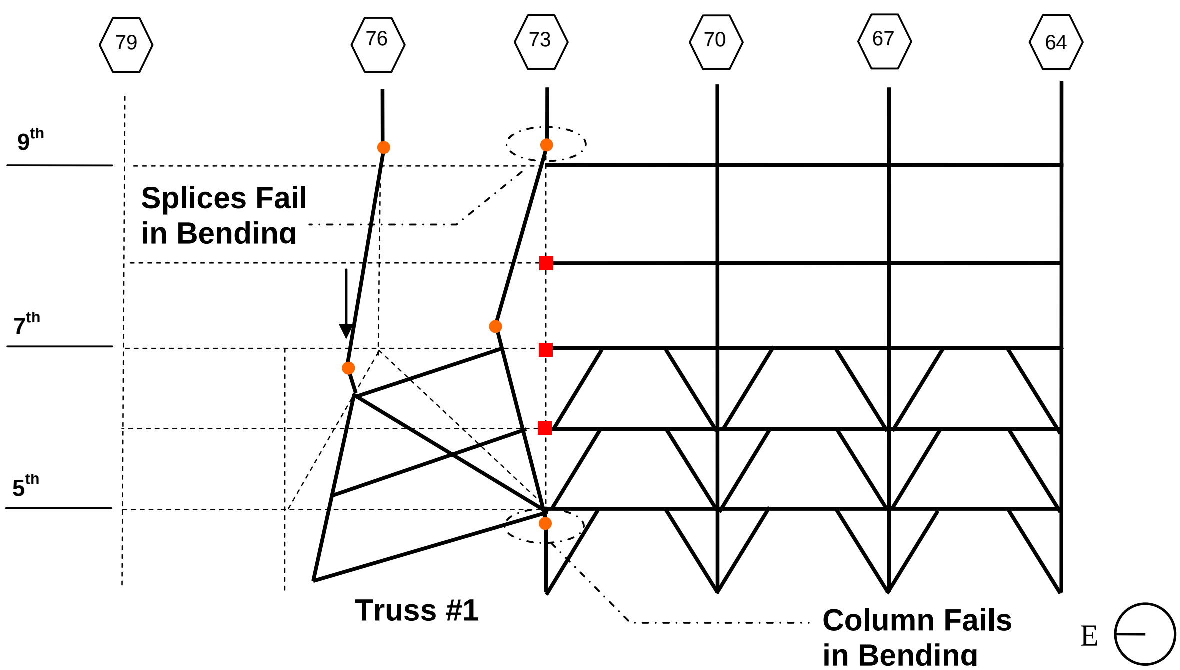


Figure L–48. Horizontal progression mechanism for truss #1 failure.

H1.4 Truss #2 and/or East Transfer Girder: If one of the diagonals of truss # 2 and/or the east transfer girder was damaged or severed by collapse debris from the vertical progression, there would be a horizontal force developed in the Floor 7 slab as columns 77 and 78 became unstable.

- **H2.6 Collapse Does Not Progress:** The Floor 7 slab may fail at adjacent columns prior to imposing lateral displacements sufficient to fail the columns or their splices.

- **H2.7 Collapse Progresses:** The horizontal tensile force would tend to pull the line of columns 74, 71, 68, 65, and 62 towards the east. The general absence of the Floor 7 slab and braced frames around the center core column line, due to the presence of elevators shafts, creates a more likely scenario for the simultaneous lateral displacement of the center core columns without similarly displacing other core columns. The possible result is a failure of all the columns at their splices, as shown in Fig. L–49.

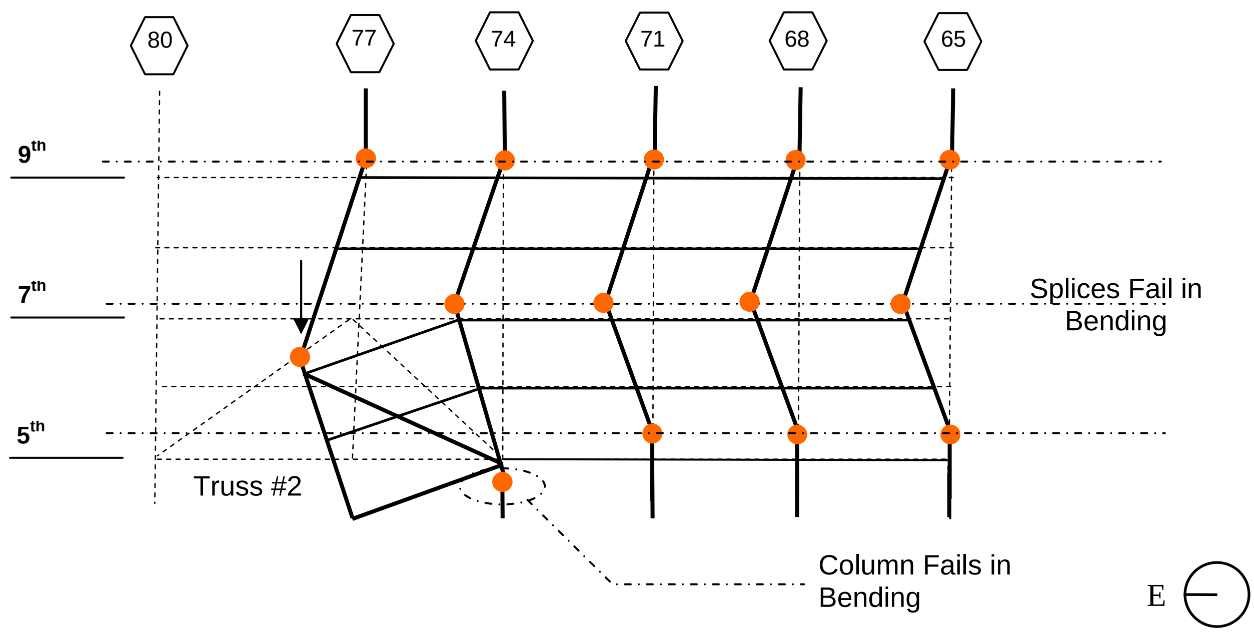


Figure L–49. Horizontal progression mechanism for truss #2 failure.

L.3.5 Summary of Working Collapse Hypothesis

The working collapse hypothesis has been developed around four phases of the collapse that were observed in photographic and videographic records: the initiating event, a vertical progression at the east side of the building, and a horizontal progression from the east to west side of the building, leading to global collapse.

From an analysis of the observed collapse sequence, the following general sequence of events appears possible:

1. Debris damaged the south face of the perimeter moment frame and some interior core framing on the south side. The debris impact severed approximately a quarter to a third of

the south face perimeter columns. The damaged floors are less certain, but reports indicate they occurred between the ground and up to Floors 15 or 20. The extent of damage, both structural and to fireproofing, of core framing is not known, but damage to elevator cars and shafts was reported to have occurred around columns 69 to 78 at Floors 8 or 9.

2. Fires were observed after the collapse of WTC 1. Fires were observed after 2 pm on Floors 7, 8, 9, 11, 12, and 13. Fires were not observed on Floor 5, but this may be due to the lack of windows. The presence of a fuel distribution system and the possibility of damage at the south face from WTC 1 debris impact, indicates that fires may have been present on Floor 5.

3. The initiating event may have included a number of structural components, though the relative role of impact damage and fire need further investigation. Possible components that may have led to the failure of columns 79, 80, and/or 81 include interior columns 69, 72, 75, 78, and 78A, the east transfer girder (which supports column 78A and frames into transfer truss #2), and adjacent framing and floor systems.

4. A vertical collapse appears to have occurred after interior columns 79, 80, and/or 81 failed. This failure mechanism would progress vertically upward within the failed bay to the roof level, as analysis indicates that the floors would not be able to redistribute their loads.

5. The debris from a 40-story vertical collapse on the east side of the building would fall down onto the strong diaphragms at Floors 5 and 7 and possibly onto transfer trusses #1 and #2, and/or the east transfer girder. Damage and loading on these floors and transfer components would generate lateral forces which would cause the failure of the remaining core columns. The horizontal progression requires further analysis and investigation, but observations indicate that the remaining core columns appeared to fail almost simultaneously, approximately 5 second after the east penthouse failed.

6. The core columns failed and redistributed loads until the building loads could no longer be supported. Once the core columns failed, the cantilever girders which supported the north facade also failed. The remaining perimeter columns at the east, south, and west facades were either left unsupported or were pulled down with the interior collapse. The global collapse occurred with few external signs prior to the system failure.

The working hypothesis, for the collapse of the 47-story WTC 7, if it holds up upon further analysis, would suggest that it was a classic progressive collapse that included:

- An initial local failure due to fire and/or debris induced structural damage of a critical column, which supported a large span floor area of about 2,000 ft^2, at the lower floors (below Floor 14) of the building,

- Vertical progression of the initial local failure up to the east penthouse bringing down the interior structure under the east penthouse, and

- Horizontal progression of the failure across the lower floors (in the region of Floors 5 and 7 that were much thicker and more heavily reinforced than the rest of the floors), triggered by damage due to the vertical failure, resulting in disproportionate collapse of the entire structure.

The working hypothesis will be revised and updated as results of ongoing, more comprehensive analyses become available.

L.3.6 Technical Approach for Analysis of the Working Collapse Hypothesis

There are many possible collapse scenarios that have been postulated in the preceding section. Many of the scenarios will not produce the observed sequence of global collapse events and can be classified as unlikely. Likely collapse scenarios will be identified through analyses that test the postulated phases of collapse against observations. It is equally important to test scenarios that are not predicted to match the observed data. The testing of the postulated collapse scenarios will be conducted through hand calculations, simplified nonlinear thermal-structural analysis, and full nonlinear thermal analysis.

L.4 REFERENCES

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Appendix M
INTERIM REPORT ON 2-D ANALYSIS OF THE WTC TOWERS UNDER GRAVITY LOAD AND FIRE

M.1 SUMMARY

A two-dimensional (2-D) finite element model is developed to provide insight and evaluate some aspects of a possible collapse sequence for the World Trade Center (WTC) towers. For a prescribed temperature distribution that corresponds to a two-story, quarter-span fire, and for a three-story fire derived from fire dynamics simulation, diagonals of the heated trusses buckle inelastically, causing considerable sag in the fire floors. This behavior puts a high-tension demand on the truss connections to the perimeter column, which remains at moderate temperatures in this model and does not experience buckling. Because neither the prescribed nor the derived temperature distributions are necessarily representative of the actual fire, and the material properties are approximate, further work is needed to evaluate the collapse sequence and develop findings regarding the actual event.

M.2 INTRODUCTION AND REVIEW

Within days of the collapse of the WTC towers on September 11, 2001, publications postulating the mechanism of the collapse began to circulate. A substantial effort was launched by the Federal Emergency Management Agency (FEMA) and the American Society of Civil Engineers (ASCE), culminating in a preliminary building performance study (McAllister 2002). Quintiere et al. calculated the elastic buckling strength of a single diagonal of a floor truss, assuming pinned end conditions, and suggested that the buckling of such thin members exposed to fire might have initiated the collapse (Quintiere et al. 2002). More recently, Usmani et al. performed a series of 2-D, nonlinear finite element analyses of a 12-story vertical frame that comprises a perimeter column and, at each floor, a truss and floor slab supported by the column and the tower core (Usmani et al. 2003). The temperature distribution in the steel and concrete members was characterized by an assumed time-dependent profile. Usmani et al. concluded that column instability caused by the loss of bracing normally provided by floors led to overall structural collapse (Usmani et al. 2003).

The objective of this report is to present a simplified analysis approach to evaluate some aspects of the collapse sequence of the WTC towers. The analysis is based on a 2-D model that is simple and can be easily used to evaluate a wide variety of conditions. The structural system was modeled independently of connection details. At this stage, connections are the object of a separate analysis that can draw on the results presented here concerning demand upon connections at various stages of fire development.

M.3 STRUCTURAL MODEL

The vertical plane considered in the model includes perimeter column 109 on the North face of WTC 1, and five longitudinal floor trusses and slabs (floors 94 to 98). The center of the airplane impact was at floor 96, and column 109 was the intact column closest to the edge of the initial damage zone (McAllister 2002). The column extends 22 m (72 ft) to a height of six floors, and both its upper and lower ends are pinned, with the upper end free to translate vertically. The upper chords of the floor trusses are simply supported at the internal end, and connected to the perimeter column by hinges. In the actual structure, a double floor truss carries a tributary floor slab 2 m (80 in.) wide and is supported by two perimeter columns, whereas in the present model a single truss supported by a single perimeter column carries a 1 m (40 in.) wide slab. The floor spans 18.3 m (60 ft) from the perimeter column to the core. The model is similar to that of Usmani et al. (2003), except it has fewer floors.

The principal reason for including only five floors in the analysis is to have the simplest model that will still capture salient features of the collapse of the towers. The fire applied to the model only heats two floors, and the remaining floors remain cool and provide lateral restraint to the perimeter column under study. Since the ends of the perimeter column in this simple model are hinged, the model ignores the rotational restraint supplied by the continuous column if additional floors are considered. Thus the short model is less stiff than a taller model (such as the 12-floor model developed by Usmani [2003] and would buckle sooner (or at a lower mechanical or thermal load), if global buckling should occur at all. As far as translational restraint is concerned, only a very small amount of lateral bracing can have a tremendous effect on the buckling strength (Winter 1958), and the cool floors one or two stories away from the fire can be replaced by a support that does not allow horizontal translation. One additional reason for including only five floors is to provide a guide for and allow comparisons with the results of a three-dimensional study, where several full floors are included. The size of the three-dimension model is a concern.

The trusses, slabs and the column that supports them are simulated by three-node beam finite elements, capable of modeling a wide variety of cross sections, with a mesh density and number of integration points specified by the user. One particularly attractive feature of these elements is the capability of supporting linear temperature gradients across their section and along their length.

M.4 MATERIAL PROPERTIES

The various steels range in nominal yield strength from 250 MPa (36 ksi) in the floor trusses to 450 MPa (65 ksi) in the column (McAllister 2002). They are all modeled by bilinear stress-strain curves, with a tangent modulus about 0.5 percent of the elastic modulus. Figures M–1 and M–2 show the steel properties for the temperature range used in the analysis. Usmani et al. (2003) used similar steel properties.

The lightweight concrete slab is also modeled as a bilinear, ductile material (Fig. M–3), with compressive strength of 20 MPa or 3,000 psi (McAllister 2002). The top chord of the floor truss is assumed to act in a perfectly composite way with the slab and allow the tensile strength at the bottom of the slab to be equal in magnitude to the compressive strength at the top. This choice of a simple, bilinear material overestimates the tensile capacity of the slab. As well, the simplification inherent in transforming the steel top chord into an equivalent concrete section disregards the differential thermal expansion between steel and concrete. A more accurate concrete model (currently being developed) may show slab failure or a smaller horizontal tension at the connection between floor and column than the present results.

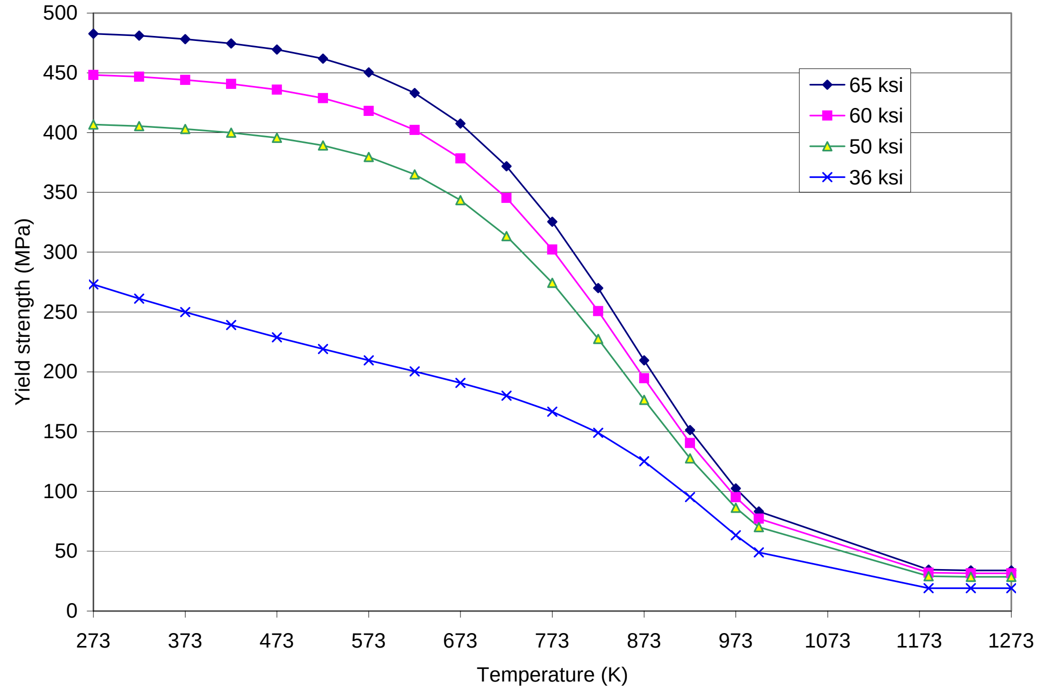

Figure M–1. Yield strength of steels used in model.

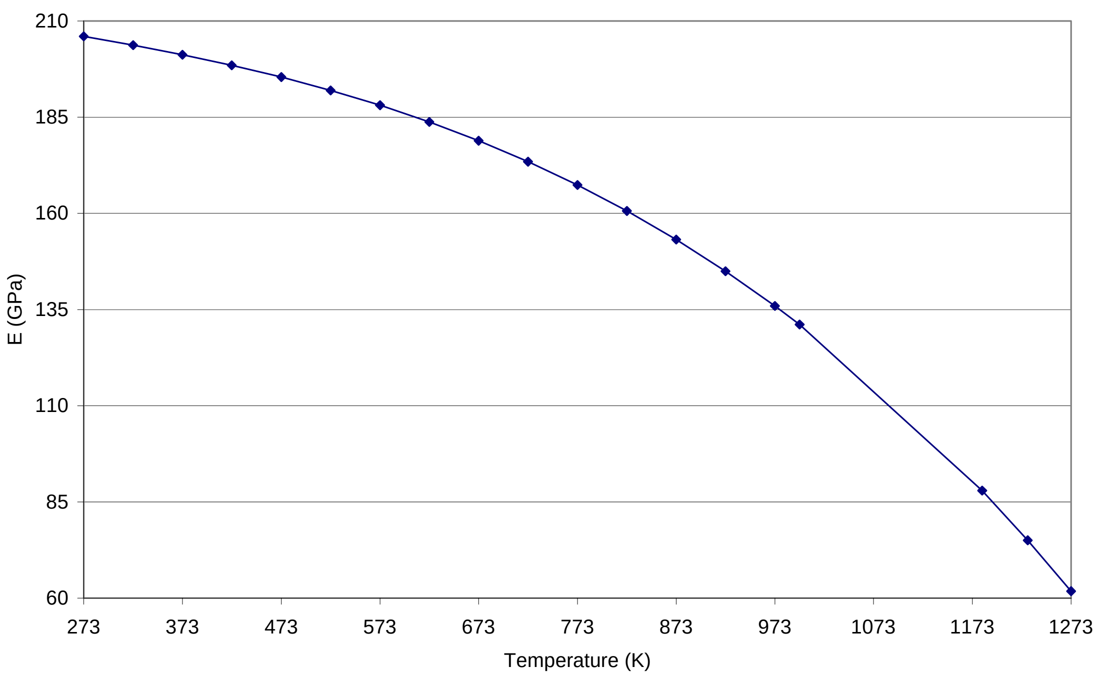

Figure M–2. Modulus of elasticity of steels used in model.

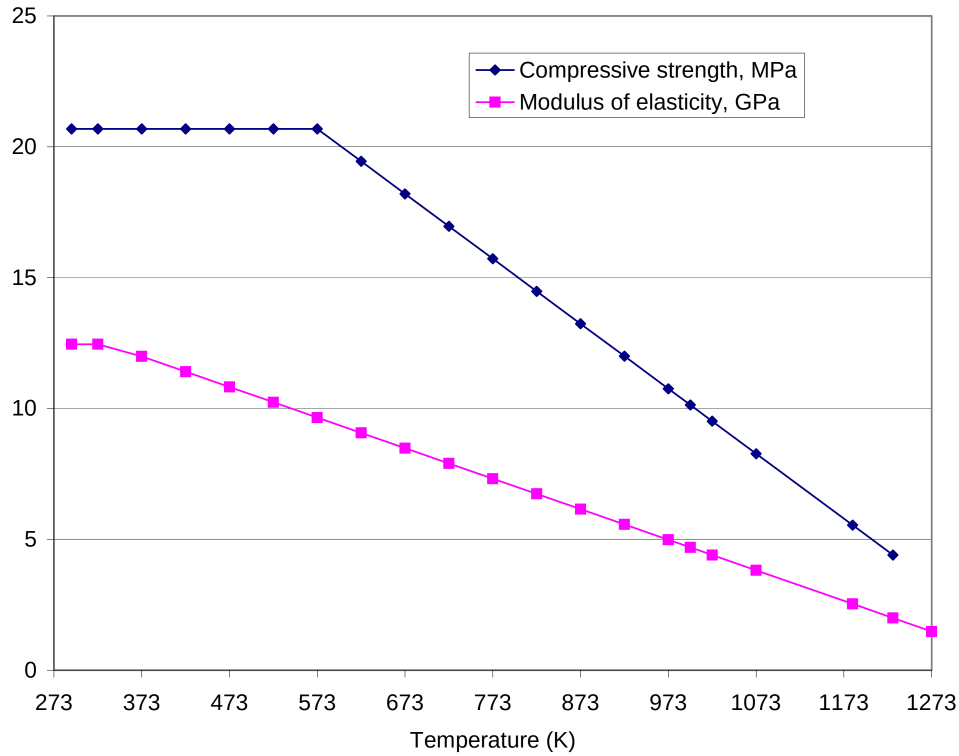

Figure M–3. Mechanical properties of concrete slab as modeled.

M.5 LOADING

The floor slabs are acted upon by a dead load of 3.3 kPa (70 psf) and a live load of 720 Pa (15 psf). The column load, determined by a linear, static finite element analysis of the global, damaged structure, includes the weight of the floors above and a surcharge due to load transfer from the columns damaged or missing after the airplane impact (Appendix D, Section D.2.4 of this report). The top of the column is loaded by a 1,100 kN (250 kip) axial compressive force and a 2,000 N·m (18 kip·in.) clockwise moment (compared to 540 kN or 120 kip, and 1,800 N·m or 16 kip·in. counter clockwise moment before damage). In addition, the column self-weight is applied along its length. For comparison, Usmani et al. (2003) used loading consistent with the FEMA report (McAllister 2002) and applied 40 percent of the gravity loads of the tributary floor strips above the model to the top of the perimeter column.

The behavior of the structure and its eventual collapse are greatly influenced by thermal loads. This report first performs an analysis based on a conventional fire, which provides a useful first approximation to the behavior of the building in fire. For comparison with the work of Usmani et al. (2003), a single temperature distribution T represented by an exponential function of time t, with a reference temperature T_0 = 300 K, is used. The time rate of change of the temperature, represented by coefficient a = 0.005, depends, among other factors, on the location and intensity of the fire, and the quality of the insulation.

$$T(t) = T_0 + (T_{\max} - T_0)(1 - e^{-at}) \tag{1}$$

A two-floor fire, with maximum temperature T_{max} = 1,273 K, heats the structure on floors 95 and 96, over the quarter-span closest to the perimeter column. Over that span, the slab of floor 95 is uniformly heated, whereas the slabs of floors 94 and 96 have linear temperature gradients across their thickness, with the bottom of slab 94 and the top of slab 96 remaining at 300 K at all times. In the three-quarters of the span not directly under fire, the temperature decreases linearly from the maximum at quarter-span to room temperature at the core. Between floors 94 and 96, the column temperature is also described by Eq. (1), with T_{max} = 400 K, whereas the rest of the column remains at 300 K at all times.

The finite element model was used further by applying to it a second temperature distribution (Fig. M–4) that corresponds to a more realistic, physics-based fire, generated by NIST's Fire Dynamics Simulator in a manner consistent with initial conditions appropriate for the WTC towers following the aircraft impacts. The gas temperatures associated with the fire were used to calculate structural member temperatures and temperature gradients, assuming an insulation thickness of 19 mm (3/4 in.) for truss members and 36 mm (1.4 in.) for the perimeter column. The fire considered in this application is more widely spread than the conventional fire, and covers four floors, with the entire floor span heated. Floor 94 (lowest) remains unheated, and the column is only moderately heated. Linear temperature gradients are modeled across the column section and the slab thickness of heated floors. The peak temperature of 1,230 K is obtained at the end of 25 temperature load steps, each 200 s apart.

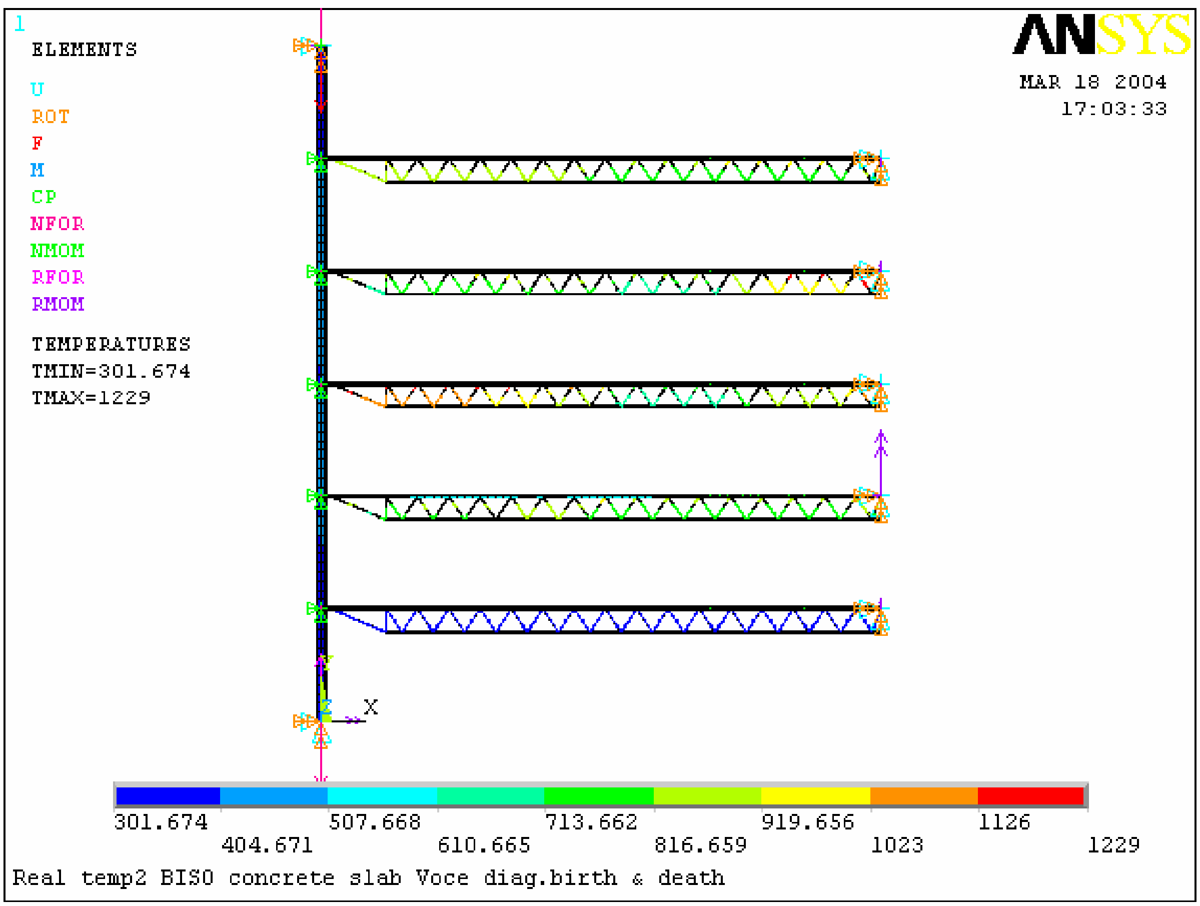

Figure M–4 Temperature distribution (K) of "real fire" scenario 2-04.

M.6 RESULTS

Nonlinear, static, large deformation analysis accounting for the magnification of flexural deflections due to axial load (P-delta effect) was performed. Member stiffness matrices are updated during the analysis to account for the P-delta effect, and when a member stiffness gets close to zero, excessive lateral deflection occurs and the member is considered to have buckled. The first analysis proceeded in eight load steps, the first corresponding to gravity loads at the start of the fire (normal room temperature). Subsequent steps occurred at 200 s intervals, with the maximum temperature attained, to within 1 K, at 1,400 s. As required in the computation, the load steps are further divided into substeps (up to several hundreds). Results for the conventional fire are shown in Figs. M–5, M–6, and M–7.

At room temperature, even under the severe load redistribution due to the damage caused by the airplane impact, the structure still behaves linearly. The maximum floor sag is 35 mm (1.4 in.), causing the horizontal span to decrease and the column to pull in slightly. Approaching 200 s and a temperature of 915 K (the temperatures referred to in these results are the hottest temperatures in the structure at any given time), the heated truss begins to show distress, especially in the compressed diagonal and vertical web members, which buckle inelastically. This means these heated steel members do not buckle elastically, but rather reach yielding in compression. Buckling is then governed by the tangent modulus of steel, which is about 0.5 percent of the elastic modulus, and the members immediately buckle after yielding, in the inelastic range. At 200 s, the maximum floor sag increases to 335 mm (13.2 in.), and the column is pushed out (peak of 38 mm or 1.5 in.) by the thermal expansion of floor 95. At that time, the connection of floor 95 to the perimeter column experiences its maximum compression of 125 kN (28 kip). Because slab 96 has a thermal gradient with its top surface at room temperature, its lateral expansion is much smaller than for slab 95, and its sag is larger. The connection between slab 96 and the column is always in tension (Fig. M–6). As expected, slab 94, heated at the top and cool at the bottom, bows upward. As the temperature continues to rise, more of truss 95 web members buckle inelastically, and the increasing sag begins to pull the column in. The horizontal deflection of the column becomes positive (inward), and the connection force between column and floor 95 turns to tension. This inward movement of the column relieves the tension in the connection between the column and floor 96. Further temperature rise causes further weakening in truss 96, which eventually becomes active in pulling the column in. At the peak temperature of 1,273 K, the maximum lateral deflection in the column (183 mm or 3.3 in.) occurs at floor 96, inward, and the connection between the column and floor 96 experiences a tension of 185 kN (42 kip).

Under the second fire scenario, the structure exhibits similar behavior. Figures M–8, M–9, and M–10 show the resulting deflections. Inelastic buckling of the diagonals causes considerable vertical deflection of the heated floors beyond a maximum temperature of 900 K. At 1,220 K, the sag of floors 96 and 97 overcomes the outward push on the perimeter column due to thermal expansion of floors, and pulls the column inward. This transition causes the column to temporarily straighten up, causing the overall floor deflection to be less. Figures M–11 and M–12 show severe horizontal tension greater than 120 kN (27 kip) at the connection of floors with the internal column (floor 95) and external column (floor 96).

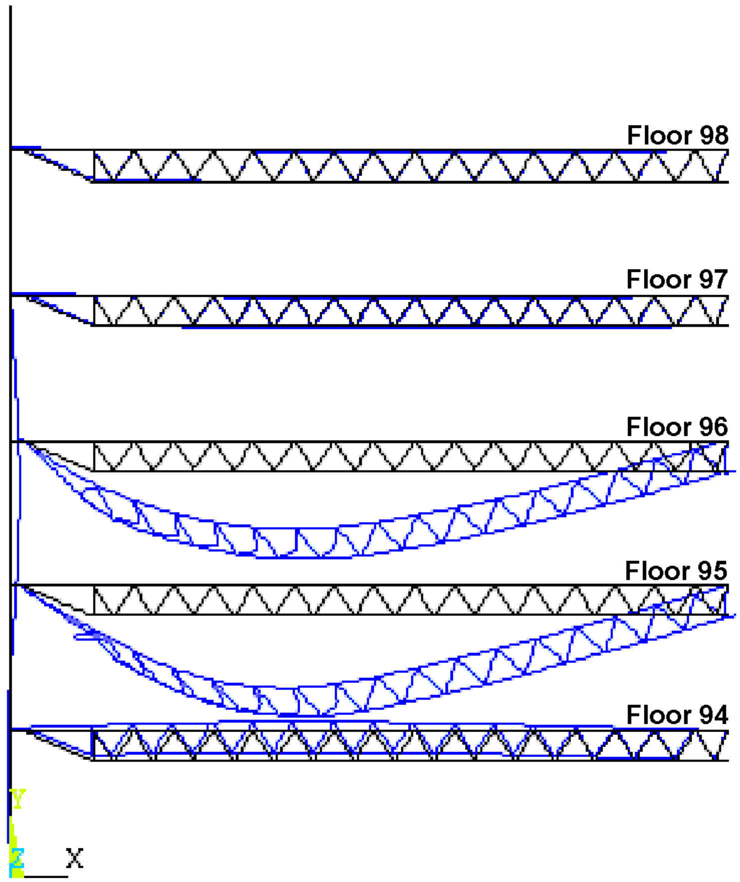

Figure M–5. Deformed shape at 1,273 K (not to scale).

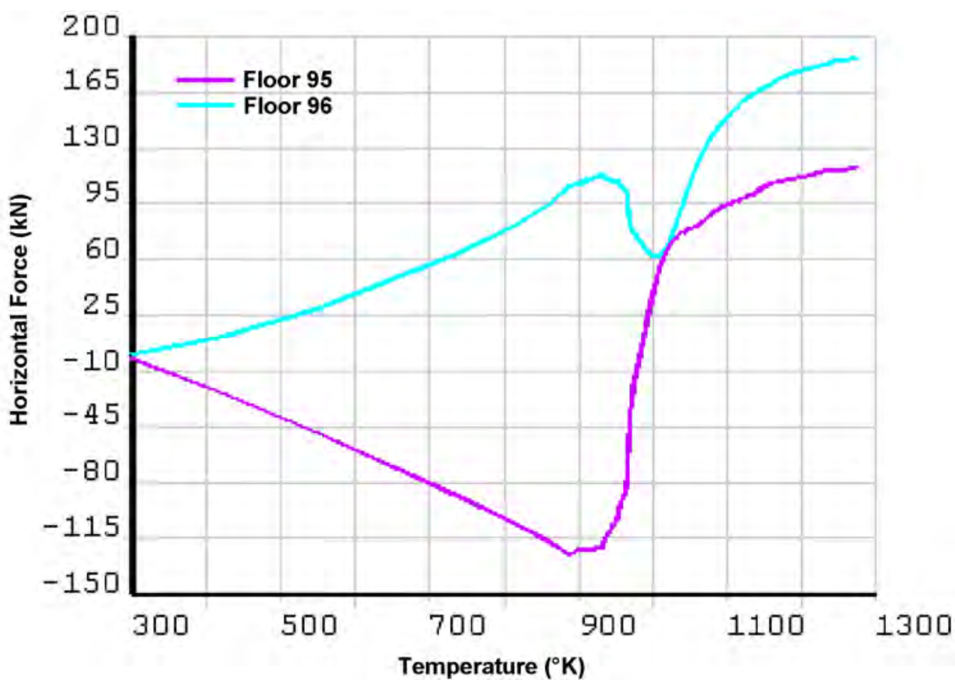

Figure M–6. Horizontal force (kN) between column and floors versus temperature (K).

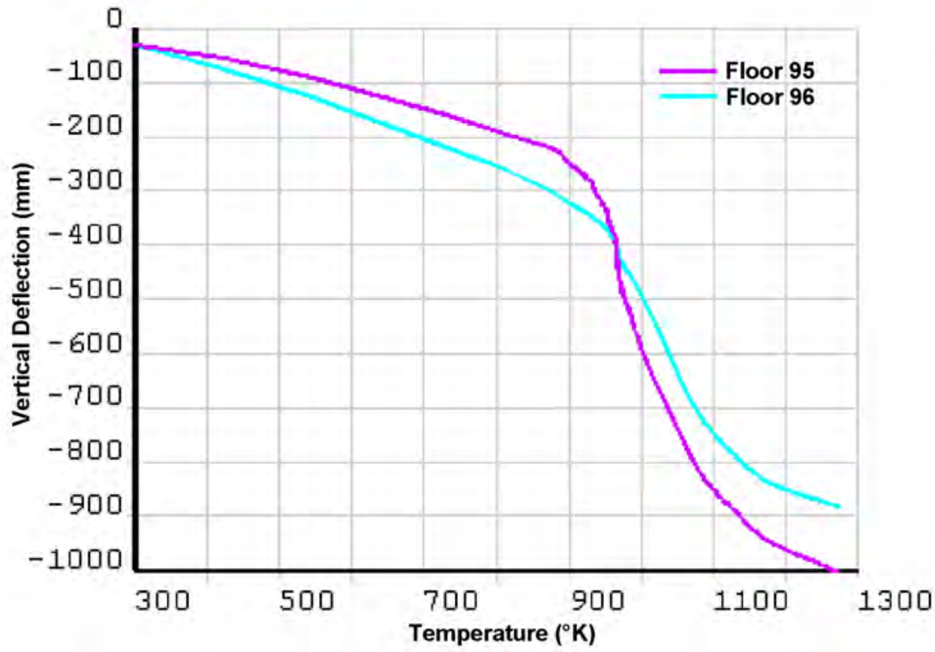

Figure M–7. Vertical deflections (mm) of floors versus temperature (K).

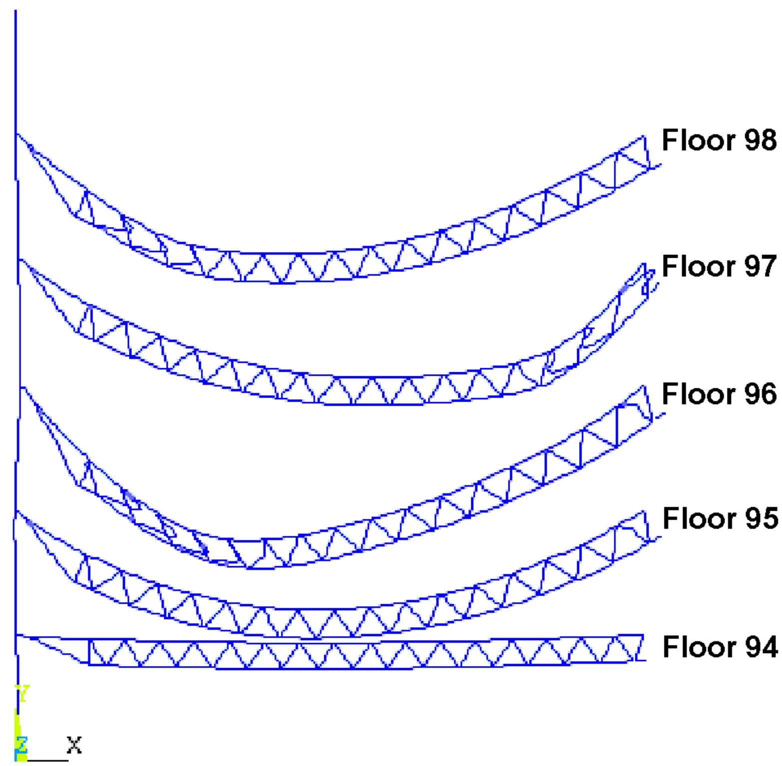

Figure M–8. Overall deflected shape for fire scenario 2-04 (not to scale).

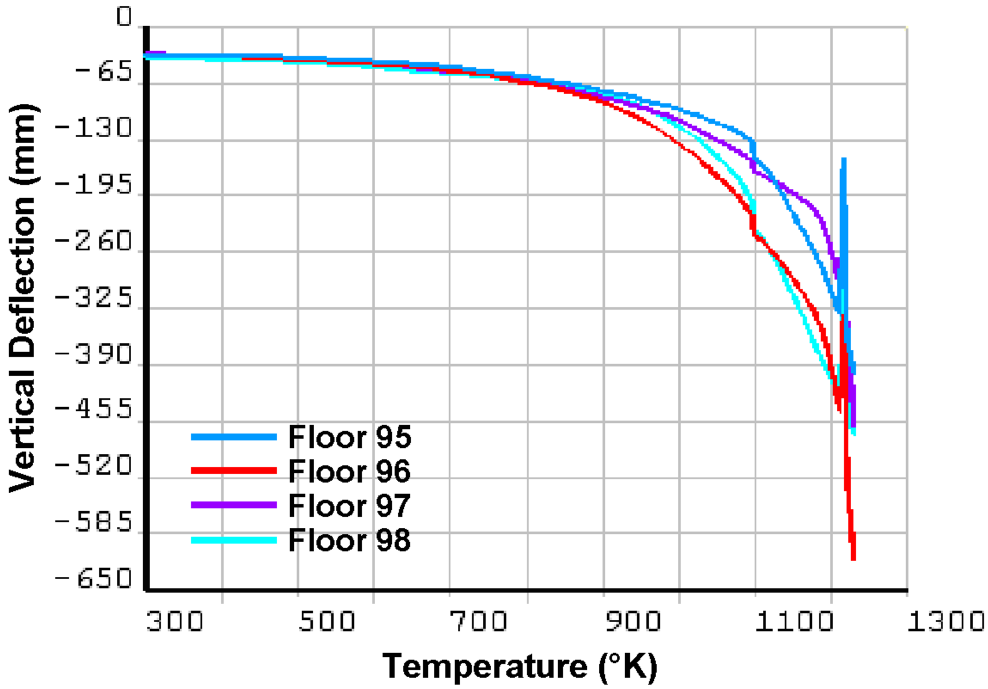

Figure M–9a. Maximum deflection of floors 95–98 for fire scenario 2-04.

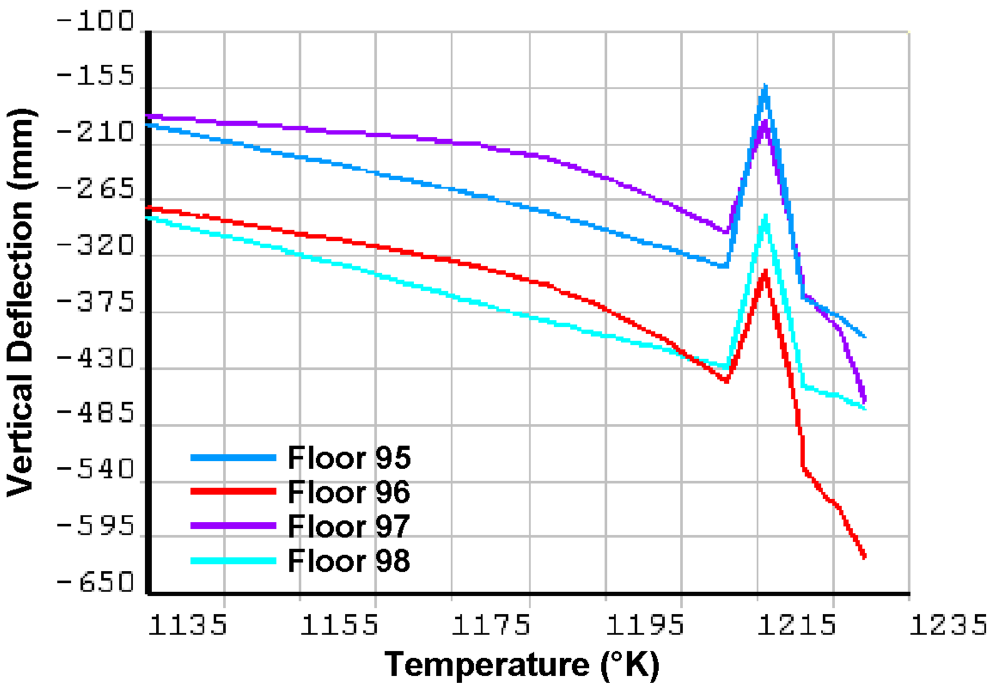

Figure M–9b. Maximum deflection of floors 95–98 for fire scenario 2-04: details of Fig. M–9a at high temperatures.

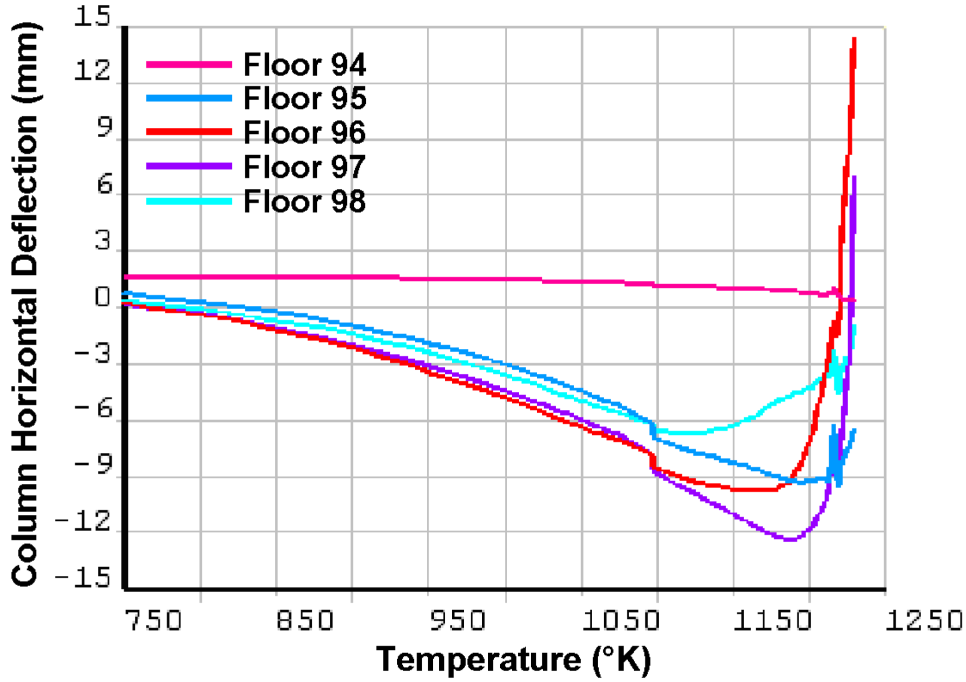

Figure M–10. Perimeter column lateral deflection for fire scenario 2-04.

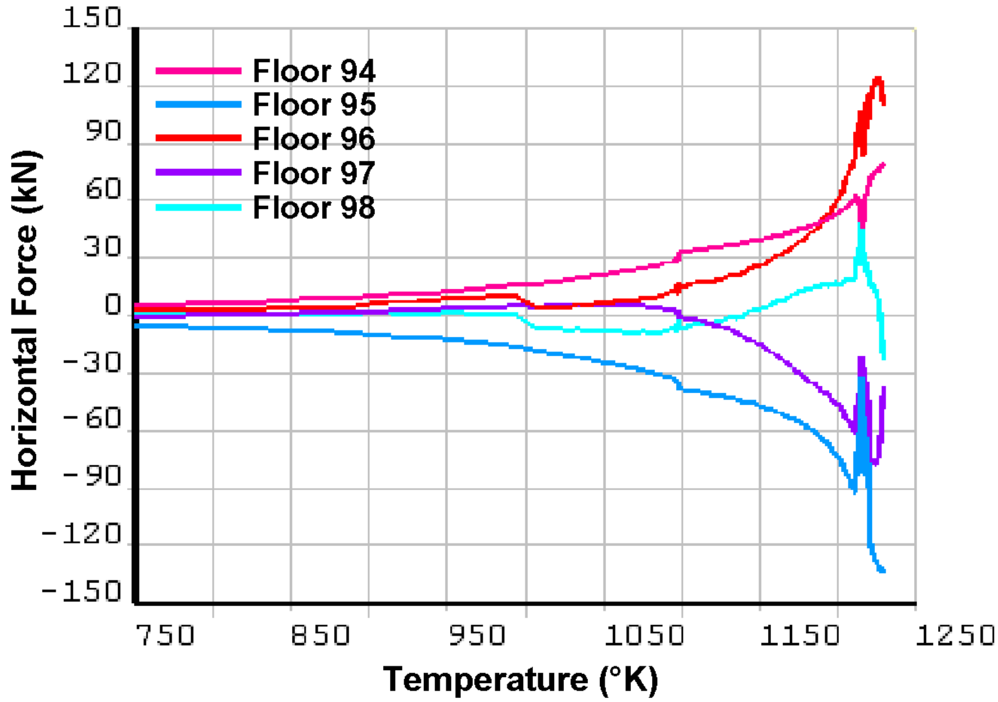

Figure M–11. Horizontal force at connection between floor and internal column for fire scenario 2-04.

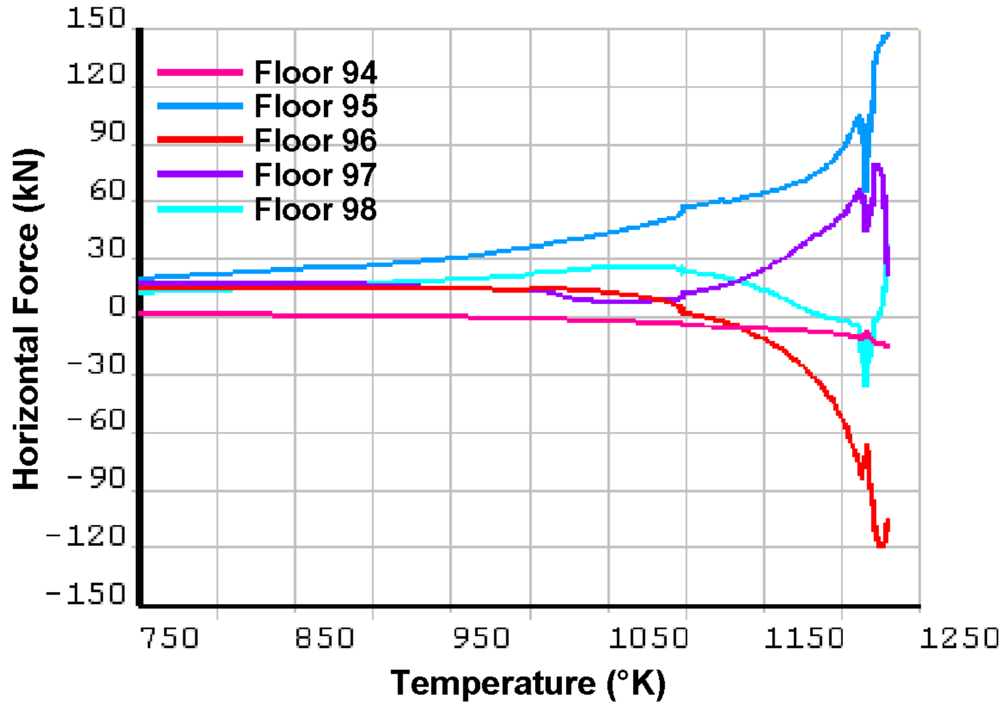

Figure M–12. Horizontal force at connection between floor and external column for fire scenario 2-04.

NIST analysis of connections is ongoing and will indicate whether this or other connections can supply the calculated demand, and if not, at what temperatures connection failures will occur. In this regard, the loss of composite behavior of the concrete slab with the steel truss may occur at a temperature and strain level yet to be determined.

For comparison with Quintiere et al. (2002), the present results show that the truss diagonals buckle *inelastically*, and there is considerable reserve strength after the first diagonal buckles. At the highest temperatures analyzed, seven diagonal and the vertical web members had buckled in each of the floors heated by the conventional fire. This conclusion assumes that the various structural connections maintain their integrity throughout the fire.

M.7 CONCLUSIONS

A model has been developed to provide insight and evaluate some aspects of a possible collapse sequence of the WTC towers. Its results are subject to the following qualifications: (1) the approximate nature of the material properties used, especially the concrete slab; (2) connection failures are not considered, although information is provided on demand experienced by the connections; and (3) the model is two-dimensional. In one of the two cases covered by this report, the temperature distribution of the members is selected from among those assumed by Usmani et al. (2003). In the second case, the temperature distribution is physically based, and was obtained by using the NIST Fire Dynamics Simulator with reasonable initial conditions associated with a damaged tower. In both cases, the diagonals buckle inelastically, causing considerable sag in the fire floors. This behavior puts a high-tension demand on the column, which remains at moderate temperatures in this model (same temperature as in Usmani et al. [2003]), and does not experience buckling. This is the major difference between these results and Usmani et al., even though the heated trusses in the present model are exposed to a much higher temperature and the column to a more severe load that reflects load redistribution in the damaged structure. One possible explanation for the difference is that failure modes may be sensitive to material properties.

M.8 NOTE

For confirmation, a 12-floor model (from floors 91 to 102) was developed and loaded with the same floor load and conventional temperature distribution described by Eq. 1 (Usmani et al. 2003) and mentioned above. Compared to case 1 reported earlier, the hinged column end conditions, the heated floors (95 and 96) and the bending moment applied on top of the perimeter column are the same, but the axial compression is reduced (950 kN or 210 kip) because of the fewer floors above the model. Results of the 12-floor model, shown in Figs. M–13, M–14, and M–15, are very similar to those of the 5-floor model, thus confirming the discussion in Section M.3.

M.9 REFERENCES

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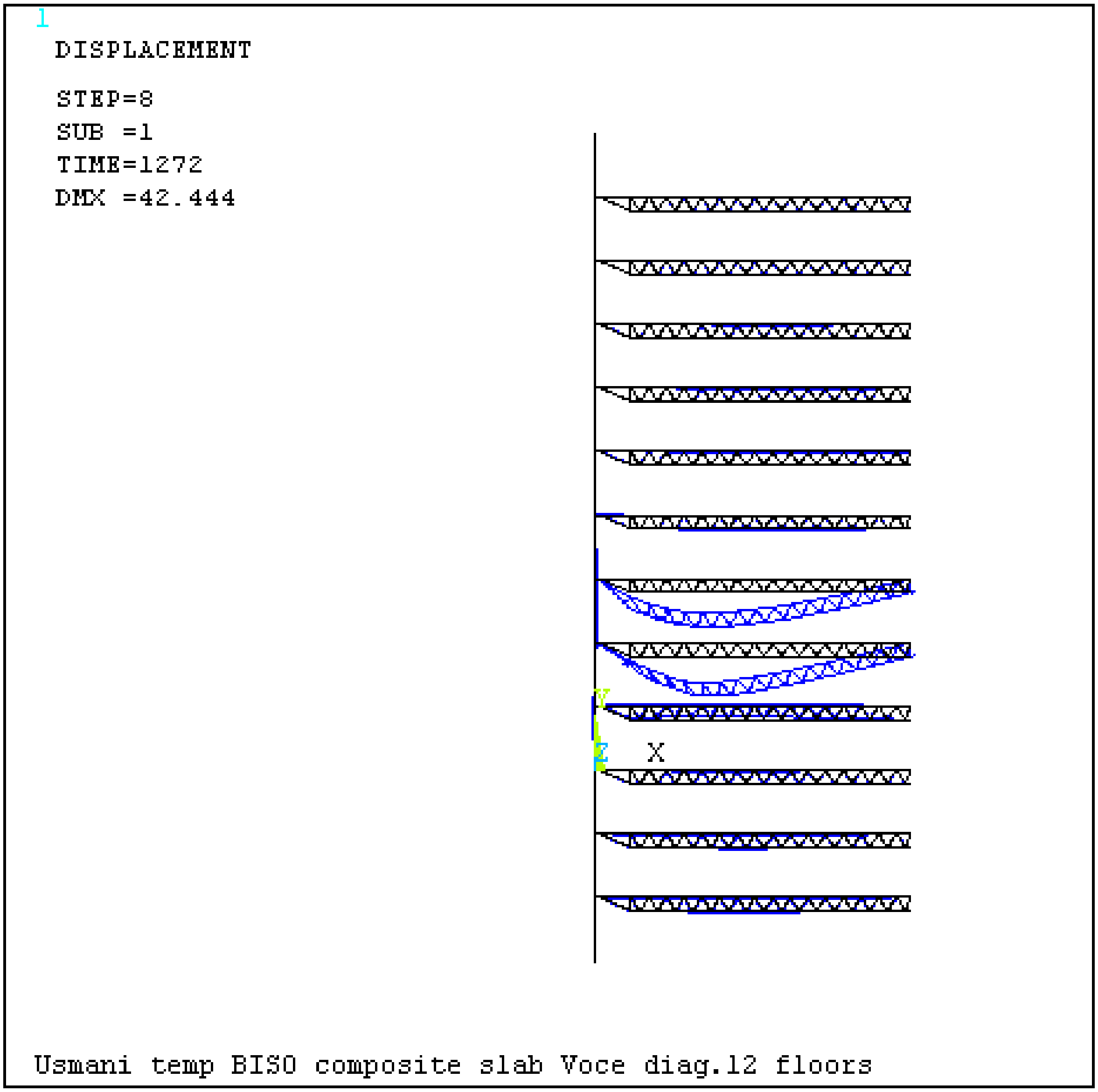

Figure M–13. Overall deflection (not to scale) of 12-floor model (floors 91–102) subjected to gravity loads and with floors 95 and 96 under conventional fire: at the maximum temperature of 1,272 K, the maximum deflection is 1.08 m.

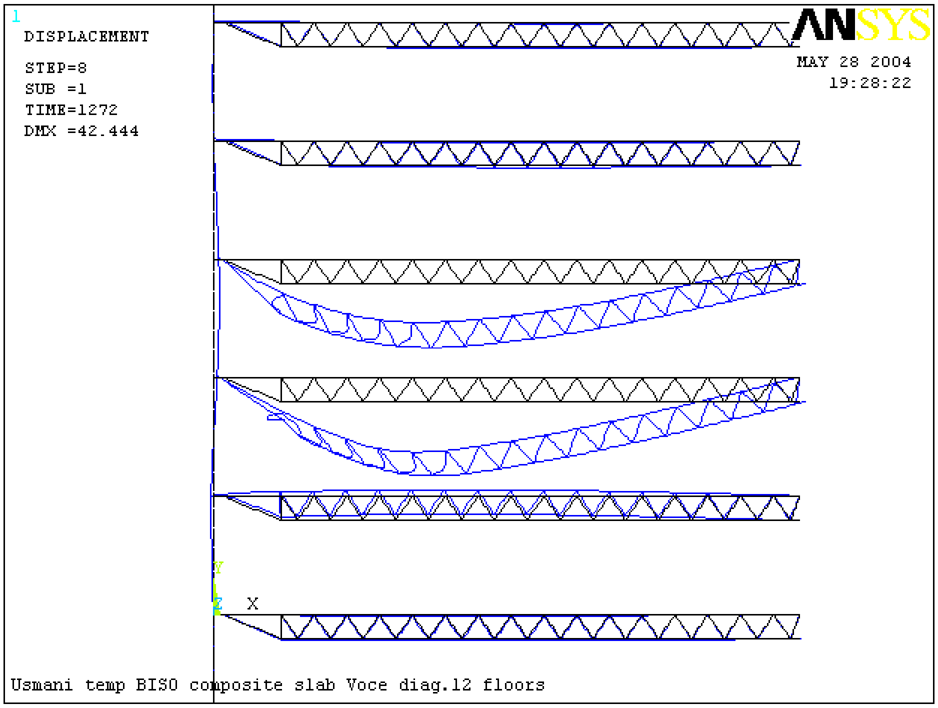

Figure M–14. Details of floors 93–98 for the 12-floor model shown in Fig. M–13 (compare with Fig. M–5).

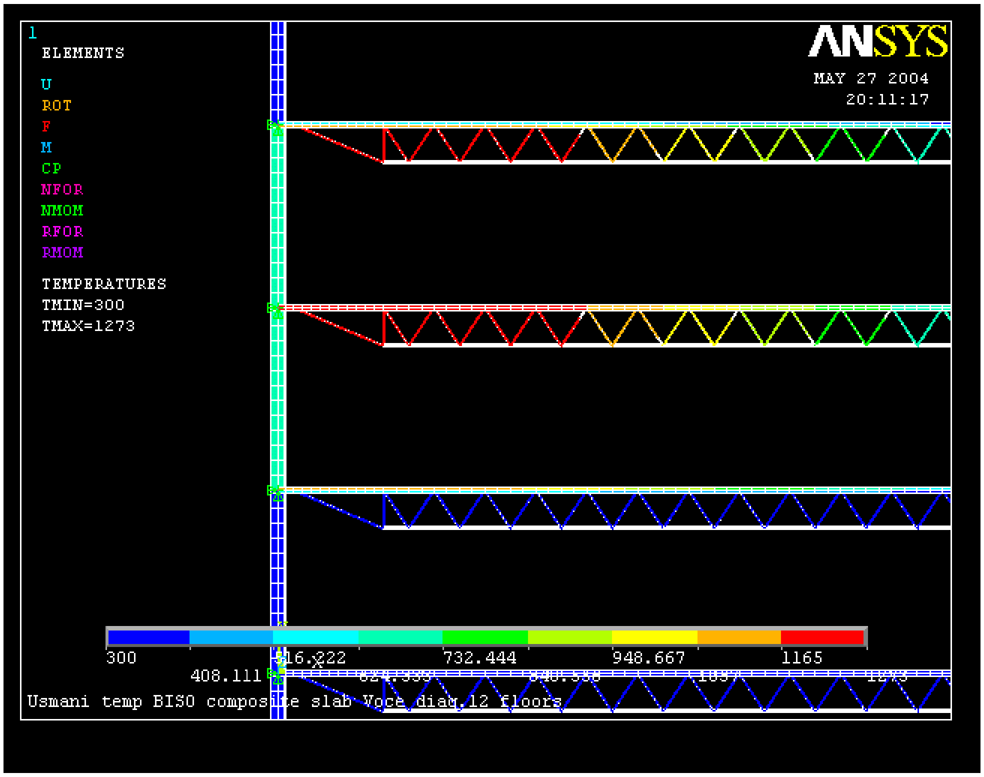

Figure M–15. Details of temperature distribution of floors 92–95 for 12-floor model: note temperature gradients across slab thickness of floors 93 and 95, and cool column.

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Appendix N
INTERIM REPORT ON ANALYSIS OF FIRST-PERSON ACCOUNTS FROM SURVIVORS OF THE WTC EVACUATION ON SEPTEMBER 11, 2001

N.1 INTRODUCTION

In the days following the September 11, 2001, attacks on the World Trade Center (WTC), the National Fire Protection Association (NFPA) in collaboration with the National Research Council of Canada decided to collect survivors' stories to document the event and to use this background material to develop future studies on occupant behavior during the evacuation of the World Trade Center. First-person accounts were collected from newspapers, radio and television programs, e-mail exchanges, and a variety of websites. Additional accounts were received at a later date from the National Institute of Standards and Technology (NIST). This large quantity of material was coded and analyzed to obtain a better understanding of the personal evacuation experiences of different survivors located on the different floors of the two towers. This report documents that analysis. The analysis was solely based on written accounts and does not include first-person interviews conducted as part of the NIST investigation. Rather, it provided background for the development of the telephone and face-to-face interviews conducted for the NIST investigation.

At 8:46 a.m. on Tuesday, September 11, 2001, American Airlines Flight 11, a hijacked Boeing 767, hit WTC 1 of the WTC. This impact caused extensive damage on five floors, from 93 to 99 of the 110-story high tower, trapping those above. Sixteen and a half minutes later, at 9:03 a.m., a second hijacked Boeing 767, United Airlines Flight 175, struck WTC 2 of the WTC, damaging nine floors, from 77 to 85.

Despite the massive localized damage caused by the impact, each structure remained standing. However, as each aircraft impacted the building, jet fuel on board ignited. Part of this fuel immediately burned off in large fireballs that erupted at the impact floors. Remaining fuel flowed across the floors and down elevator and utility shafts, igniting intense fires throughout upper portions of the buildings.

At 9:59 a.m., WTC 2, the second building to be hit, collapsed after burning intensely for 56 minutes. WTC 1 withstood its injury longer than the South tower, collapsing to the ground at 10:28 a.m. after burning for 102 minutes (FEMA BPAT 2002). It is estimated that approximately 2,800 people were killed and 800 others injured by the attacks and eventual collapse of the towers on September 11, 2001, including building occupants and first responders (Cauchon 2001).

Although the events of September 11, 2001, involved the WTC, the Pentagon and the hijacked airliners, the evacuation of the two towers is the focus of this research. The attacks precipitated the evacuation of the entire WTC complex. The evacuation of WTC 1 and WTC 2 was largely initiated by the occupants themselves.

The WTC was a complex of seven buildings, including the two 110-story office towers. Approximately 50,000 people worked in each tower (100,000 total), with an estimated 70,000 visitors to the complex during the course of a normal business day (Yamasaki 2002). However, the occupancy of the towers on the morning of September 11, 2001, was not at its maximum capacity. According to USA TODAY, 5,000 to 7,000 people were in each tower at 8:46 a.m. that morning, the time of first impact (Cauchon 2001). It has been suggested that the towers were not at their maximum capacity for several reasons. That morning marked New York City's mayoral primary, and it is assumed that many people stopped to cast their ballots before heading in to work. The New York Stock Exchange does not open until 9:30 a.m., therefore many people from trading firms had not come into work yet. Tuesday, September 11, 2001, was the first day of school in several primary school districts, and many parents accompany their children to school on this day. Visitor hours had not started yet, as the viewing platform in the South Tower did not open to the public until 9:30 a.m. Perhaps the biggest factor of all was the early hour—many simply had not arrived at work by 8:46 a.m. In addition, dozens of investment firms in the WTC had closed their offices or cut employment sharply. Some offices were leased but empty or under renovation (Cauchon 2001).

By certain measures, the evacuation of the WTC on September 11, 2001, could be termed a success (Cauchon 2001). Under the impacted floors, nearly every occupant who could physically get out did get out. According to USA TODAY, in each tower, 99 percent of the civilian occupants below the crash sites survived. Their analysis shows that two-thirds of WTC 2 occupants started their evacuation of the upper floors during the 16.5 minutes between the attacks, and survived. Among the occupants under the impacted floors in WTC 1, 72 people died, whereas under the impacted floors in WTC 2, 4 people died. It should be noted that some fraction of the deaths below the impacted floors in WTC 1 occurred in the elevators, which were carrying people at the time of impact.

N.2 BACKGROUND LITERATURE

Understanding the basic concepts of human behavior in fire is necessary to envision occupants' likely response during an emergency. Human behavior in fire is a scientific field that identifies facts, concepts and relationships established through systematic observation and experimentation. What is known about human behavior in fire is that the three dimensions of the emergency, namely the occupant, building and fire characteristics, interact to explain or predict occupant response to fires (Proulx 2001).

During a fire, the nature of the information obtained, the limited time to react and the assessment of danger will create a feeling of stress**.** This stress is not an abnormal reaction; on the contrary, stress is regarded as a necessary state to motivate reaction and action. During the course of the event, the intensity of stress experienced will vary as a function of the information newly-perceived and the assessment of the decision taken (Proulx 1993). Key factors which increase stress include: the perception of threat to oneself or others, the suddenness of the threat and the available time to respond or prepare, the amount of sensory input needed to be processed, and the degree of physical effort (aerobic and anaerobic output) that is engaged during the incident. If the individual is physically wounded or injured, the effect will be even greater (Grossman 2002). Taking all of these factors into account, it can be said that most evacuees of the WTC were experiencing extremely high levels of stress.

Most people assume that individuals cease to act in a predictable, orderly fashion in the face of disaster, and that norms which govern our behavior collapse into Durkheim's anomie, a state of normlessness (Fisher 1998). This mindset, known as *disaster mythology*, has been greatly nourished by the mass media and movie industry to capitalize on strong emotional images (Proulx 2002). Today, it is largely known that in the face of the extreme stress of a disaster, there is an absence of widespread, irrational, antisocial and dysfunctional behavior that has often been described as 'panic' (Quarantelli 1998). Thus, the false but common belief that people will panic in disaster situations is a myth (Sime 1980, and Keating, 1982). In human behavior fire research, it is found that panic behavior is extremely rare (Proulx 2002). Decision making during an emergency is, however, different from day to day decision making for three main reasons (Janis and Mann 1977). First, there is much more at stake in emergency decisions—often the survival of the person and of the people he or she values the most is at risk. Second, the amount of time available to make a decision before crucial options are lost is limited. Third, the information on which to base a decision is ambiguous, incomplete and unusual. Further it is usually impossible to look for more appropriate information due to the lack of both time and the means to get information (Proulx 1993).

Turning to the literature of the evacuation of the WTC following the 1993 terrorist bombing, it was concluded that there was a lack of panic flight during the evacuation, even though the occupants had to descend the crowded and smoky stairwells in total darkness. No official evacuation orders were issued by recognized emergency and building officials after the explosion, and no official information was provided about safe or proposed exit routes (Isner and Klem 1993). However, it was found that there was an overall mood of orderly evacuation during the 1993 evacuation (Wenger et al. 1994; Fahy and Proulx 1995). This lack of panic during the 1993 evacuation may be explained by the fact that although the explosion created immediate danger, it was not perceived by participants as particularly severe. Secondly, people were not alone; they were with coworkers, friends and associates. This web of social integration among participants works against the adoption of norms that would support individual, competitive flight behavior to favor the emergence of cooperative, altruistic, helping and orderly behavior (Wenger et al. 1994).

In contrast to the panic scenario of a competitive scramble towards an exit, Dr. Jonathan Sime argues that the most common behavior during a fire is movement towards familiar persons and places (Sime 1985). This is known as the *affiliation* model which suggests that detached groups will attempt to reunite before evacuating, and evacuation movement is most likely to be through a familiar way in and out of the building (Sime 1985). The grouping of people during an emergency is sometimes known as the *milling process*: the communication process that takes place among participants in a crisis setting as they attempt to define the situation, propose and adopt new appropriate norms for behavior and seek coordinated, collective action (Wenger et al. 1994). High levels of social interaction were reported during the 1993 evacuation as the tenants engaged in milling behavior regarding the definition of the situation, the attempt to give meaning to the crisis (i.e. to determine "What is happening?"), the appropriate response to it or proposed cues for action, and the attempt to give direction to the behavior of the participants by offering new, emergent norms (i.e., "What should we do? What is appropriate?") (Wenger et al. 1994).

Identification of the information available to occupants in defining the situation is essential in attempting to understand the decision-making process during an emergency. The social context of the occupants and the opportunity to observe and interact with others are also fundamental considerations when attempting to understand occupant response and the overall outcome of evacuations.

N.3 STUDY OBJECTIVES

This exploratory research project was conducted in order to gain an overall understanding of the circumstances surrounding the evacuation of the WTC towers on September 11, 2001. More specifically, this research project endeavors to gain insight into the variability of human behavior and response time displayed during the evacuation, with the findings to be used as a guide for future research. This study can also provide insight for the NIST investigation into the WTC event. Human behavior data gathered from this project will help to create a better understanding of the individual experiences of occupants in specific locations by documenting, to the extent possible, the information available to occupants, such as conditions on their floor and along their evacuation route, perceived behavior of others, and escape conditions and timing.

N.4 METHODOLOGY

In the moments following the attack of the WTC towers on September 11, 2001, journalists started interviewing survivors to obtain the story of their evacuation. These first-person accounts were presented on television or radio and published in newspapers, magazines, or websites and later reported in books and special media programs. During the three months following the events, over 280 first-person accounts were collected. These accounts included media reports (newspapers, magazines, television and radio), as well as material from books, training videos, personal websites and emails. The information provided in some of these accounts was so detailed that it provided sufficient material for a study. Additional accounts were gathered over the next year for a total of 745 first-person accounts from 465 individuals, as some survivors provided multiple accounts through different sources. The 435 accounts retained for analysis are from evacuees of WTC 1 and WTC 2 only. Although numerous accounts were found from occupants of the surrounding WTC complex, only those civilians who had evacuated the actual towers were considered. For those survivors for whom numerous accounts were found, the information across the accounts was collapsed into one highly detailed account, containing the combined information from all of the given accounts. For instance, 16 survivors provided five to 12 different accounts to the media. These survivors had dramatic accounts and, therefore, were of particular media interest. The study involved no first-person interviews.

It is recognized that the use of first-person accounts published in the media as main sources of information for a study has many limitations. The questions asked by reporters are unknown and can be different for each journalist and with each interview. It is also noted that the date an account was published does not necessarily represent the date that the specific interview took place; the account could have been held at some point and then published at a later date. It is suspected that the most dramatic experiences are reported and that some information may be emphasized or left unreported for the purpose of the article. As stated by Dr. H.W. Fischer, the mass media has greatly reduced the level of flamboyant exaggeration in what they report as typical behavioral and organizational response to disasters over the last 50 years; however, since a larger portion of the news is now devoted to reporting disasters, a less than accurate image is still commonly portrayed both in the print and broadcast media (Fischer 1998). It also must be stressed that the findings in this study are representative only of the <u>individual experiences captured in first-person accounts and cannot be generalized to the population of the two towers</u>.

Recall of details of an event depends on many factors, including the intensity of the event, time since the event, and influence from other information sources. Recollection of extreme events such as the WTC attacks may be far better than ordinary daily events. Conversely, traumatic situations may result in memory impairment or “critical incident amnesia.” Finally, with the intense media attention that the events of September 11, 2001, received, it is highly likely that this coverage influenced survivors’ recollection of events. This phenomenon, referred to as “contamination,” occurs when information outside of the actual experience is integrated into the reconstruction of memory

(Grossman 2002). Since different occupants of the WTC had a range of experiences on and after September 11, it is unclear to what extent memory issues impacted the reports included in this study.

Despite the drawbacks of using media sources for the basis of research, however, some of the accounts contained such a high level of detail, particularly the ones written by survivors themselves, they provided justification for the analysis of this information. It should also be stressed that these media accounts are the only documented descriptions of the WTC evacuation and immediate reactions of the survivors, as no research has been conducted or published 2 years after the events, regarding human behavior surrounding the events of September 11, 2001. Since documenting human behavior is time sensitive and considerable time has passed since the event, it may be said that these initial media accounts may hold significant detailed and accurate information that may only be available in these accounts.

N.4.1 Content Analysis

The most appropriate social research method for analyzing media communications is content analysis. To extract the important content from the accounts, a “questionnaire” was developed to “interview” each account. This procedure was used by Johnson (1987) to analyze police file statements related to the “Who Concert Stampede;”[1] it is also explained in some detail in Gamso’s book “The Strategy of Social Protest” (1975). The approach relies on a series of identical questions used to “interview” each document. Once the information is gathered in a qualitative or descriptive database, codes are developed to reduce the variety of answers to each question to a manageable number. To ensure reliability of the coding, at least two researchers independently review each account and compare their coding. Any disagreement is discussed and resolved.

Questions to “interview” each account were designed to obtain manifest and latent information from the 745 first-person accounts. A majority of the questions, 30 of them, rely on manifest information or elements specifically reported in the account, such as the person's location at certain key moments. They are listed in Table N–1. The remaining three questions called for latent information, such as words describing emotions. They are listed in Table N–2. Data was retrieved from the accounts and entered into a qualitative database. Nominal and ordinal categories were conceptualized, which can be found in the coding scheme presented in Attachment 1. It is important to note that not all questions were answered for each account gathered, as a number of the accounts were incomplete. The fact that an individual’s account is silent on some point does not mean that this factor was not important or relevant in that individual’s evacuation. It simply means that it was not included in the published account or that it was never mentioned by the individual. For those questions not addressed in the account, the category was awarded the code “9” or “99,” accounting for the lack of information regarding that specific question. This lack of information for some items is the equivalent in a questionnaire survey to a respondent who did not answer some of the questions. The information gathered in the qualitative database was coded and transformed into a quantitative matrix from which descriptive statistics were calculated.

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[1] On December 3, 1979, 11 people were crushed to death as fans rushed the entrance of a stadium in Cincinnati, Ohio, to see a sold-out concert.

Table N–1. Questions on manifest information.

What is the date of published account?	Heard fire alarm?
Gender?	Location at WTC 2 impact?
Age?	Location at WTC 2 collapse?
In which building was the person at the time of first cue?	Location at WTC 1 collapse?
On what floor was the person at the time of first cue?	Location when met firefighters?
What was the first cue of event?	At what time person exited the building?
How long did the person take to start evacuation?	Who helped person during evacuation?
Did the person delay start time?	Was the person disabled?
What mode of egress was used?	Was the person injured?
What was the condition on floor?	Location when person placed phone call?
What was the condition on the stairs?	Who was the phone call recipient?
Were obstructions encountered during evacuation?	Was there social influence on decision making?
Heard announcement?	Use other (non-phone) communication technology?
Location when WTC 2 announcement heard?	Was person at the WTC during 1993 bombing?
Action after hearing WTC 2 announcement?	Did the person rest during evacuation?

Table N–2. Questions on latent information.

What was the person's knowledge of the situation in the initial moment?
How serious did the person judge the situation to be?
What was the person's perception of others?

N.4.2 Variables Considered

Conceptualization and operationalization involve precisely defining how the variables were measured and ensuring the attributes within those variables are mutually exclusive and exhaustive. There were 33 questions providing data ranging from demograhics and physical location, to response time and social interaction during the evacuation. Coded data included the evacuees' gender, age and which building and floor they were located in, as well as the date the account was published. The floors of the towers were categorized as lower (basement to floor 42), mid (floor 43 to floor 76), and upper (floor 77 to floor 110) based on the common areas referred to as “skylobbies” on the 44th and 78th floors, which separated the towers into three strata. The skylobbies on floors 44 and 78 served the occupants of floors 43 and 77, respectively. Mode of egress was recorded as stairs, elevators or a combination of both.

The first cue of the event was categorized according to whether the cues were “audio,” such as hearing an explosion, crash or rumbling; “visual,” such as seeing the plane approach the towers, or seeing fire, smoke or debris. Another category of first cue was “building movement” and was represented by feeling the building shake, sway or tremble, whereas moving office furniture, falling ceilings, jolting in the elevator and flickering lights were attributes of the variable category “contents movement.” The remaining three categories were “warned by others” (either verbally or through their behavior), “physically impacted” (e.g., burned, fell from chair, thrown against a wall), and “smelled smoke or fumes or felt heat." These cues were coded as check-off items so that multiple initial cues could be captured. Whether or not evacuees heard a building alarm during their evacuation was recorded in a separate field, if mentioned.

Time to start evacuation was recorded as immediately, shortly after impact (where the occupant took less than 5 minutes to retreive belongings before evacuating), delayed (representing those who took more than 5 minutes to start evacuating, taking time to search floors or gather company documents, etc.), stayed or stuck.

Conditions of floors immediately after the building was hit were recorded in two ways. One field summarized the conditions as follows: "devastated," meaning combinations of conditions such as scattered debris, fire, darkness and fallen ceilings and walls were reported; "abnormal," in that there was some smoke, heat or the smell of jet fuel; and "normal," represented by accounts describing usual conditions on their floors. A series of check-off columns then recorded whether a person's account reported the presence of specific conditions: door jammed, debris (e.g., from wall, floor or ceiling collapses), smoke, dust, no power or darkness, smell of fumes, water, fire, crowds or injured people, entrapment, or normal conditions. If the individual was not on an office floor when the building was struck, that was recorded in an additional check-off column. This allowed the recording of multiple conditions for each individual.

Similar check-off columns were used to record observations of conditions in stairwells during evacuation: normal, door locked or jammed, crowded and/or hot, no power, water, cracked wall, debris, smoky or smell of fumes.

If and where the evacuees heard the announcement stating that WTC 2 was secure were also noted, as were their actions after hearing the announcment (i.e., continued evacuating, continued but saw others return to offices, or returned to or remained in office). The survivors' location at the time of impact, collapse of the towers and meeting of firefighters were also coded, as well as who helped them during the evacuation. Those who provided help were categorized as firefighters, Port Authority employees, external officials such as NYPD, FBI, and coworkers. Individuals' disabilities were coded as "visually impaired," "hearing impaired," "physically challenged" (e.g., obese, pregnant, or with asthma or heart conditions), "wheelchair users," or 'injured." People who mentioned that they had aided a disabled or injured person during the evacuation were also noted in this variable category, as were those who reported seeing injured or disabled people during their evacuation.

Whether or not a person was present at the WTC during the 1993 bombing was recorded, as was whether or not each person delayed his or her evacuation on September 11, 2001. Where the evacuee placed a phone call and the recipient of it were coded, along with whether or not they rested and where they rested. A series of check-off columns recorded if a person experienced obstructions, such as door jams, debris, smoke, no power, smell of fuel, water, fire, crowds, injured and disabled people or became trapped during the evacuation. Multiple entries were possible for each individual.

Other variables included the survivors' knowledge of the situation, recorded as "high" for those who knew a plane had struck the towers or that there had been a terrorist attack; "moderate" for those who thought there was a fire, bombing or judged the situation as a serious emergency; and "low" for those who were not aware of the reasons behind the evacuation. The evacuees' indication of the level of seriousness was recorded as "very serious," "somewhat serious" and "not serious" based on the perceived tone of the account. The variable “perception of others” included the categories of “calm,” in that evacuees felt others to be orderly and composed; “upset,” which represented those who judged others as nervous, anxious or visibly bothered. For survivors who described others as hysterical or pushing and shoving, this field recorded their perception of others as “momentarily panicked.” When accounts reported that those around them lent assistance to others, this field was coded as “helpful.”

Social influence on decision making was categorized according to who influenced the evacuee: authority figures, such as bosses or managers; groups of coworkers; or both authority figures and groups of coworkers. If a person indicated that he or she took on a leadership role, that was also captured. Use of new communication technology including utilizing text messaging over pagers or wireless e-mail devices, TV or radio to gain information was noted. (See Attachment 1 for further variable category definitions.)

The time that people reached the outside was recorded. It must be stressed that most accounts did not report specific times at which people took different actions. However, several occupants mentioned their location at key moments such as where they were when WTC 2 was hit or when WTC 1 or WTC 2 collapsed. For example, one survivor of WTC 1 reports, “When we got to the twentieth (floor) I remember hearing a rumble. One of the fellows looked at me and we knew it didn’t sound good. It must have been WTC 2 coming down” (Fink and Mathias 2002). Thus, it was deduced that this survivor was on the 20th floor of WTC 1 at 9:59 a.m., when WTC 2 collapsed. Similarly, for many people, the time they reached the outside could be estimated from their description of events (e.g., WTC 2 being struck, WTC 2 collapsing) as they reached the outside.

N.4.3 Procedure

Various media avenues were utilized in gathering first-person accounts including television, radio, newspapers, magazines, websites, books and special media programs. Personal websites and e-mails written by survivors themselves were also used and are of particular interest, as they have not been altered by media editors in any way, but appear in their full, original format. During the three months following the events, over 280 first-person accounts were collected. Eventually, a total of 745 accounts were gathered from 465 individuals, as numerous survivors gave multiple stories to different journalists.

The accounts, which were gathered over a period of 18 months, were published up to 14 months after September 11, 2001. The distribution of published accounts over time is shown in Fig. N–1. Among the dated accounts studied, 51 percent were published in the first two weeks after September 11, 2001, with another influx of accounts surfacing around the one-year anniversary, 10 months to 12 months after the disaster.

Content analysis was performed on the 745 accounts using 33 questions for which the data was entered into a qualitative spreadsheet. Duplicate accounts were merged, resulting in a final study size of 435 individuals who were present either in WTC 1 or WTC 2. The data was then coded and transferred into a matrix for analysis.

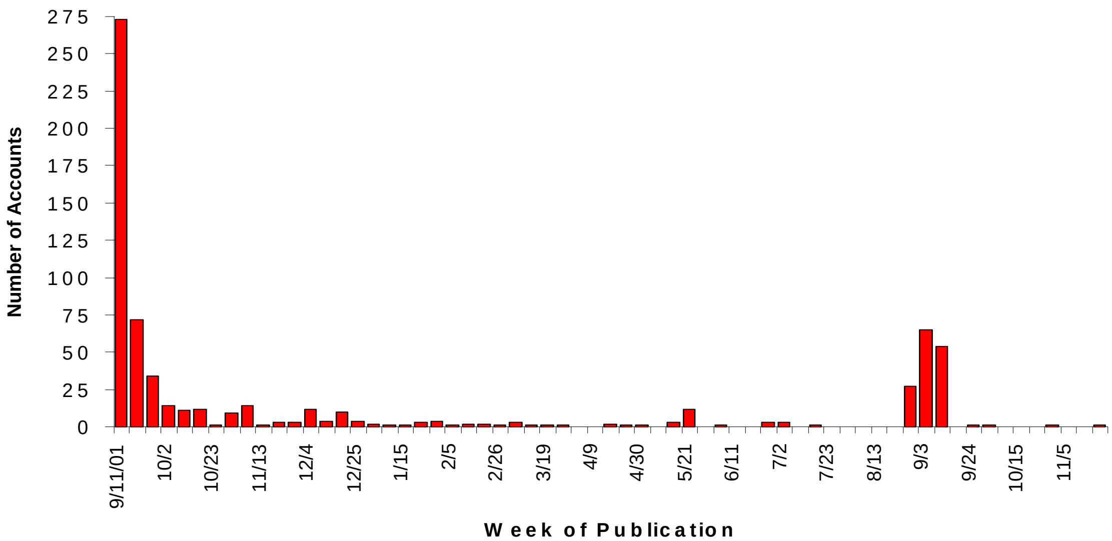

Figure N–1. Distribution of publication dates of accounts.

N.5 STUDY RESULTS

The raw data for each account was entered into an Excel spreadsheet and then coded. The coded data was transferred into SPSS 11.0 for statistical analysis. The statistical analysis conducted was essentially descriptive statistics to organize and summarize the information. Inferential statistical tests were not conducted since the data obtained is not a representative sample of the population. Results presented in this report should not be generalized to all occupants of the two towers on September 11, 2001. Although they are reported using terms such as "the occupants" and "the survivors," the results refer only to the accounts analyzed.

N.5.1 Profile: Gender and Age

The study contained accounts from 435 survivors, ranging in age from 20 to 89 years old (mean = 39.5, standard deviation = 11.8). Included were accounts from 118 women (27 percent) and 314 men (72 percent); three accounts did not mention their gender (1 percent). It is speculated that the substantially higher number of men involved in these accounts occurred because there were more men working in the two towers than women or that men may be more likely to talk to the media than women. The breakdown by gender and age is shown in Fig. N–2.

N.5.2 Location at the Beginning of the Event

There were 251 individuals who were located in WTC 1, comprising 58 percent, with the remaining 42 percent or 184 people from WTC 2. In WTC 1, 90 people (36 percent) were from upper floors (77 to 110), 79 people (31 percent) were from mid levels (43 to 76) and 58 people (23 percent) were from the lower floors of WTC 1. Another 22 people (9 percent) were in elevators and two people did not specify a location. In WTC 2, 94 people (51 percent) were from upper floors, 57 people (31 percent) were from mid floors, 28 people (15 percent) were from the lower levels of WTC 2 and five people did not specify a location. Although the distribution of accounts in the two buildings was not identical, reports were obtained from the three strata in both buildings. It is likely that the higher fraction of individuals in WTC 1 and in higher floors reflects the more dramatic stories of those closest to the airplane impact locations in WTC 1 and WTC 2.

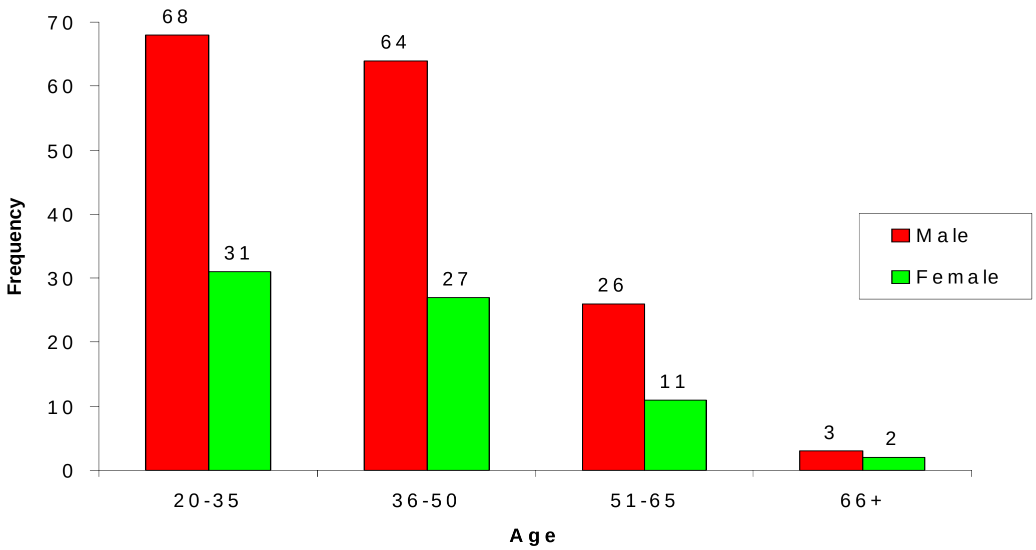

Figure N–2. Gender and age distribution.

N.5.3 Means of Egress Used

On September 11, 2001, almost all individuals from WTC 1 (198 people or 98 percent) reported using the stairs to evacuate while three used both stairs and elevator and one used the elevator only. The person who used the elevator for evacuation reported that he was in an elevator when the building was struck, and the elevator stopped on one of the floors. He was able to use the elevator to move people from that floor to the lobby. Two of the three who used both stairs and elevators were initially trapped in an elevator behind a 50th floor restroom. After freeing themselves, they were directed by firefighters to an elevator to the 44th floor, from which point they walked down. The third person who used both stairs and elevators rode with a person he was assisting from the 52nd floor to the 44th floor. Unable to find a working elevator on the 44th floor, he walked down the rest of the way. In WTC 2, 114 (72 percent of the total for that building) used the stairs while 18 people (11 percent) used elevators and 26 (16 percent) used a combination of elevators and stairs. These results are shown in Table N–3. Of the 44 people who used the elevator to evacuate WTC 2, 37 were from floors served by the 78th sky lobby and 7 were from floors between the 44th and 78th sky lobbies. From these accounts, it seems that the higher up people were in WTC 2, the more likely they were to use the elevator as a means of egress.

Table N–3. Means of egress used within the towers.

	WTC 1, N=202	WTC 2, N=158
Stairs	198 people (98.0 %)	114 people (72 %)
Elevator	1 person (0.5 %)	18 people (11 %)
Stairs and elevator	3 people (1.5 %)	26 people (16 %)

N.5.4 First Cue Reported

The first cues of the event that were mentioned in the accounts were found to differ depending on which tower the person was located. For WTC 1, the first building hit, the most common first cue of the event reported by 146 people (69 percent of people in that tower) was “building movement,” such as feeling the building sway and tremble—many thought the building was going to tip over. WTC 2 occupants most commonly reported first becoming aware of the event from visual cues (96 people) such as fire, debris and smoke, most likely coming from WTC 1. Several people reported more than one first cue, so they may appear more than once in Table N–4 and percentages total more than 100 percent.

Table N–4. First cues of event within the towers.

First Cues	WTC 1, N=212	WTC 2, N=145
Audio cues: heard explosion, crash, rumble	107 (50 %)	69 (48 %)
Visual cues: saw fire, incoming plane, debris, smoke	87 (41 %)	96 (66 %)
Building movement: felt building sway, tremble, jolt	146 (69 %)	30 (21 %)
Contents movement: furniture movement, ceiling falling	66 (31 %)	11 (8 %)
Warning from others	14 (7 %)	34 (23 %)
Impact	29 (14 %)	1 (1 %)
Smelled fumes or felt heat	12 (6 %)	16 (11 %)

Interestingly, only 25 people made any mention of building alarms in their evacuation accounts. Of those, eight in WTC 1 and one in WTC 2 reported hearing alarms but did not specify where. Two in WTC 1 and one in WTC 2 heard alarms while on their floors and one person in each tower heard alarms while in the stairs. Eight people in WTC 1 stated that they did not hear alarms. Three people in WTC 2 said they never heard alarms, but two of them were outside the building when it was hit.

N.5.5 Time to Start Evacuation

After perceiving these first cues, 101 people from WTC 1 (47 percent) immediately started evacuating, while 84 people (52 percent) immediately started their evacuation of WTC 2. As can be seen in Fig. N–3, similar numbers of people from both towers started evacuating shortly after the first cue of the event (28 in WTC 1 versus 27 in WTC 2). Another 46 people in WTC 1 and 40 people in WTC 2 delayed their evacuation. Some 23 people in WTC 1 (11 percent) reported they initially stayed, while 10 people from WTC 2 (6 percent) also said they initially remained on their floors. Of the 16 people who reported being stuck and therefore temporarily unable to start their evacuation, all but one were from WTC 1.

Among occupants who initially decided to stay, it is noteworthy to mention a group in WTC 1. Two survivors reported that a group of about 16 employees gathered in a conference room on Floor 64 of WTC 1. The group stayed in the room discussing the situation for approximately 1 hour before deciding to evacuate the building.

Most of those who were not stuck but who took more than 5 minutes to begin evacuation delayed because they took the time to complete activities such as searching the floor, securing documents, making calls, or giving instructions, or because they felt it was the right thing to do. Twenty-one of 63 people in WTC 1 (33 percent) and 13 of 45 people in WTC 2 (29 percent) delayed starting their evacuation because

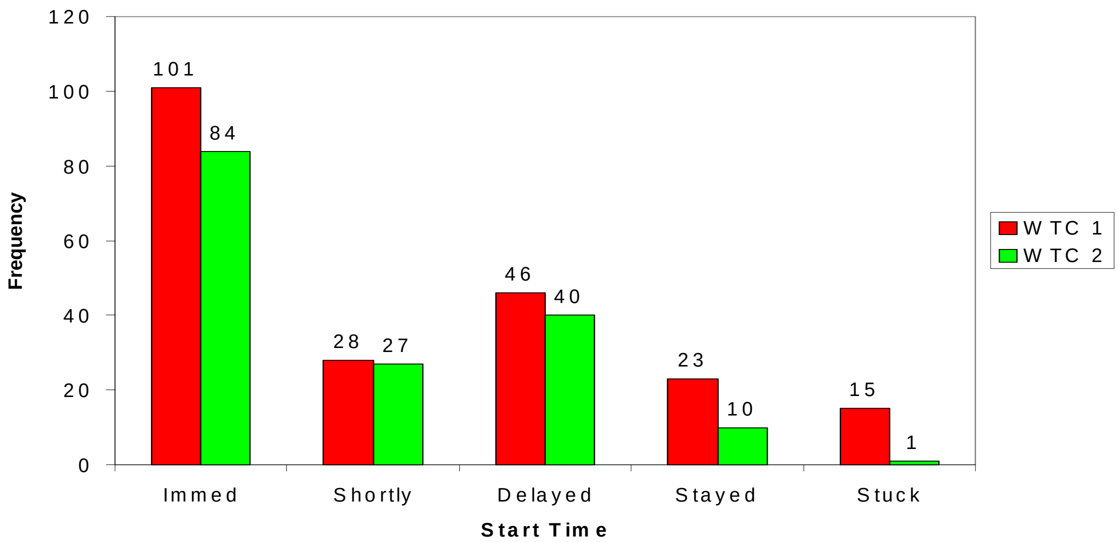

Figure N–3. Distribution of time to start evacuation.

they were completing activities such as those described above. Of those in WTC 1 who did not begin their evacuation within 5 minutes, 12 people simply decided to stay (19 percent), compared to 20 people in WTC 2 (44 percent). In WTC 1, 17 of those who did not begin their evacuation within 5 minutes (27 percent) were helping others or required assistance themselves, compared to only four people (9 percent) in WTC 2.

N.5.6 Conditions on Floors and in Stairwells

It was possible to code multiple reported conditions on floors and in stairwells for each individual. Six people in WTC 1 and seven people in WTC 2 indicated that conditions on their floor were normal after their building was struck. For the 191 evacuees who commented on adverse conditions on their floors after the plane hit their tower, similar results emerged between the towers, in terms of the large proportions reporting smoke or debris and collapse damage on their floor. Specifically, the most frequently reported adverse conditions in WTC 1 were smoke (55 percent or 74 people), debris or collapse of wall, ceiling or floor (72 people or 54 percent), fire (41 people or 31 percent), darkness or loss of power (20 people or 15 percent) and smell of fuel (13 people or 10 percent). In WTC 2, the most frequently reported adverse conditions were debris or collapse of wall, ceiling or floor (38 people or 67 percent), smoke (25 people or 44 percent), darkness or loss of power (18 people or 32 percent), dust (10 people or 18 percent), smell of fuel (7 people or 12 percent) and injured people (7 people or 12 percent). Seven people in WTC 1 who mentioned jammed doors were in the upper strata of the building. Two people in WTC 2 who reported jammed doors had moved to middle floors of their building after the first impact. The complete details on conditions are presented in Table N–5.

A large number of evacuees (106 people) mentioned that the stairwells were crowded and hot during their evacuation (71 people in WTC 1 and 35 in WTC 2). A total of 27 indicated that conditions in the stairwells were otherwise normal. For the 155 evacuees who commented on adverse conditions in the stairwells during their evacuation (other than crowdedness), the majority in both towers reported smoke and the smell of fuel in the stairs (79 people or 72 percent in WTC 1 and 29 people or 63 percent in

WTC 2). For other types of conditions in stairwells, responses between the two towers were quite different, as shown in Table N–6.

Table N–5. Adverse conditions on floor at impact.

	WTC 1, N=134	WTC 2, N=57
Debris (collapse)	72 (54 %)	38 (67 %)
Smoke	74 (55 %)	25 (44 %)
Fire	41 (31 %)	20 (35 %)
No power, dark	20 (15 %)	18 (32 %)
Smell of fumes	13 (10 %)	7 (12 %)
Dust	9 (7 %)	10 (18 %)
Water	7 (5 %)	3 (5 %)
Door jammed	7 (5 %)	2 (4 %)
Crowds, people injured	2 (1 %)	7 (12 %)
Trapped	5 (4 %)	2 (4 %)

Table N–6. Adverse conditions reported in the stairs during evacuation.

	WTC 1, N=109	WTC 2, N=46
Smoke, smell of fuel	79 (72 %)	29 (63 %)
Water	49 (45 %)	4 (9 %)
Dark, no power	14 (13 %)	9 (20 %)
Debris (damage or belongings)	9 (8 %)	14 (30 %)
Cracked walls	5 (5 %)	14 (30 %)
Doors locked, jammed	12 (11 %)	2 (4 %)

N.5.7 Obstructions during Evacuation

Tables N–5 and N–6 display details on the adverse conditions that resulted at the time of impact. These were things that were observed but that might not have presented an obstacle. (For example, a person might have reported seeing smoke or debris, without being impeded by that debris.) Obstructions are things that limited or otherwise affected a person's ability to evacuate. Many of the same items were cited as both adverse conditions and obstructions. More than one obstruction during evacuation could be recorded for each person. A total of 153 people in WTC 1 and 59 people in WTC 2 indicated encountering obstructions during their evacuation. Almost half of the evacuees in each tower reported encountering crowds and injured people in the stairways, and indicated that they interfered to some degree in their evacuation (46 percent in each tower). The next most frequently reported obstructions were smoke and debris. The details are shown in Fig. N–4.

Of the 22 evacuees who reported encountering jammed or locked doors, 20 were in WTC 1, and all but three were located on upper floors. One of the WTC 2 evacuees reported that an elevator door was jammed by debris, and the other reported a locked door on reaching the bottom of the stairs. Of the 25 evacuees who reported being trapped, nine were in elevators, eight were trapped by debris or smoke when their building was hit, five were trapped in the collapse of WTC 2, and three were trapped when WTC 1 collapsed.

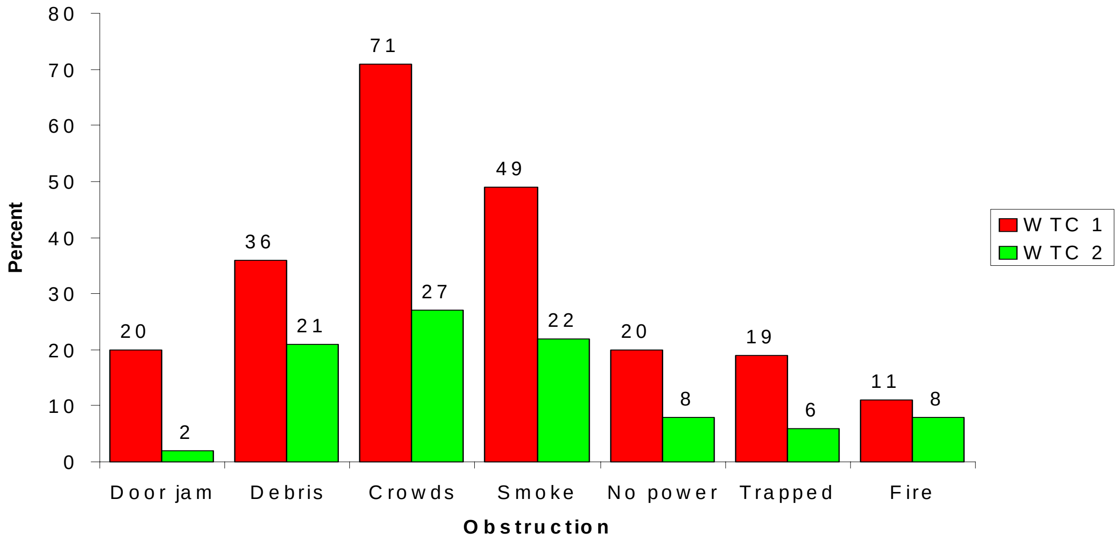

Figure N–4. Obstructions encountered during evacuation in both towers.

N.5.8 Announcement

It is estimated that the WTC 2 announcement came over the public address system at approximately 9 a.m. The majority of survivors said they heard it just minutes before WTC 2 was struck, which occurred at 9:03 a.m. As one survivor from the 103rd floor of WTC 2 describes it, "When we reached the 70th floor we heard the announcement. The building was secure; no one needed to evacuate. We had descended down 3 more floors to the 67th when the second plane hit our tower" (csmonitor.com 2001). Of the 184 WTC 2 occupants, 96 people (52 percent) mentioned hearing this announcement in their accounts. The majority of them, 69 survivors, decided to disregard the instructions of the message and continue their evacuation; however, 16 people (17 percent) said they remained in their offices or decided to return back up to their offices after hearing the message. Those returning did not have time to travel very far before the second plane hit; at that point they all resumed their evacuation.

N.5.9 Location When WTC 2 Was Hit

Of the 273 survivors who mentioned their location at the time WTC 2 was hit, 36 people reported being somewhere inside the stairwells of WTC 1, while 14 people reported being on various floors of WTC 1. Fifty-six did not give a specific location, and 15 had already reached the outside. Of the survivors from WTC 2, 65 people reported they were in the stairs and 52 occupants reported they were on various floors within WTC 2. Four did not give a specific location, and 31 had already left the building. Of the people who were on the floors within WTC 2, 19 were on the upper floors (77th and above) at impact and survived. One of these occupants, who survived the plane impact on the 78th floor of WTC 2, describes the stairwell: "a tornado of hot air and smoke and ceiling tiles and bits of drywall came flying up the stairwell. In front of me, the drywall split from the bottom up" (csmonitor.com 2001).

N.5.10 Location When WTC 2 Collapsed

WTC 2 was the first of the towers to collapse at 9:59 a.m. Of the 296 survivors who mentioned their location at the time of WTC 2's collapse, 230 people (78 percent) were outside of the buildings, on the streets and surrounding areas. Some 47 people (16 percent) were still inside WTC 1 on lower levels from the basement to the 42nd floor, and three people (1 percent) were on mid levels (43 to 76) in WTC 1 when WTC 2 fell. Thirteen did not give exact locations, and one was in an elevator. Three individuals were on the lower levels of WTC 2 (concourse) when it collapsed, and they survived.

N.5.11 Location When WTC 1 Collapsed

WTC 1, the second tower to collapse, fell at 10:28 a.m. As approximately 1 hour and 42 minutes had passed since the initial WTC 1 impact, almost everyone who reported their location at the time WTC 1 collapsed was outside (263 people or 98 percent). Four people were on the lower levels of WTC 1, and two were in the concourse when it collapsed, and they survived.

N.5.12 Location When They Saw Firefighters

For the evacuees who mentioned seeing firefighters during their evacuation, the location where they met them was recorded to gain an understanding of the dispersion of emergency workers throughout the towers. For the 169 people who reported meeting firefighters, 143 people saw them in WTC 1, with only 26 people in WTC 2 mentioning their presence. In terms of floor location within WTC 1, it was found that a majority of the people (76 people) saw firefighters in WTC 1 on the lower levels (basement to 43rd); 74 of them saw firefighters in the stairwells, and two on a floor. Another 21 people saw firefighters on the mid floors (43rd to 76th)—17 of them were in the stairs while the other four people were on floors. Another three people saw firefighters on the upper floors (77th to 110th) in office areas. All three were trapped on the 83rd floor. One survivor stated: "We saw two flashlights belonging to two New York City firemen. They told us to leave all of our possessions and to quickly follow them." (Manning 2001). At the mezzanine, lobby or concourse level, 11 people reported seeing firefighters. The remaining 31 occupants who saw firefighters inside WTC 1 did not give a location.

Among the 26 people who mentioned seeing firefighters in WTC 2, eight saw them on the lower floors (basement to 42nd), and two saw firefighters in the mid floors of the building (43rd to 76th). Seven people saw firefighters at the mezzanine, lobby or concourse levels, while six people in WTC 2 mentioned seeing firefighters but did not indicate their locations. Three people indicated that they met firefighters outside WTC 2.

N.5.13 Time of Exit

For evacuees from both towers who indicated at what time they exited, it was found that as more time passed, a progressively greater number of people exited the building, as shown in Table N–7. Of the 183 WTC 2 occupants who indicated what time it was when they left the building, 77 exited between 9:31 and 9:58 a.m. WTC 2 collapsed at 9:59 a.m. Of the 211 WTC 1 occupants who indicated the time they left their building, 70 exited between 9:59 and 10:27 a.m. WTC 1 fell at 10:28 a.m. The six people who exited the towers after 10:28 a.m. were rescued from the rubble by firefighters up to several hours after the collapse.

Table N–7. Time out of towers.

	WTC 1 (impact - 8:46 a.m.) (collapse - 10:28 a.m.) N=211	WTC 2 (impact - 9:03 a.m.) (collapse - 9:59 a.m.) N=183
8:48 – 9:02 a.m. (before WTC 2 impact)	19	37
9:03 – 9:30 a.m.	45	68
9:31 – 9:58 a.m. (before WTC 2 collapse)	72	77
9:59 – 10:27 a.m. (after WTC 2 collapse)	70	0
10:28 a.m. (after WTC 1 collapse)	5	1

N.5.14 Help Received and Help Given

Among the 435 accounts, 203 survivors described receiving help from others during their evacuation, with some mentioning more than one source of help. Some 84 people (41 percent) were helped by Port Authority personnel. Firefighters provided direct help to 65 people (32 percent). Another 65 people (32 percent) were helped by other first responders such as NYPD or other rescuers. Help from coworkers was received by 34 people (17 percent).

Overall, 166 people mentioned being comforted and reassured by passing firefighters. Several occupants of the two towers helped others during the evacuation. Among the first-person accounts, 20 people said they helped people with disabilities and 14 said they helped people who were injured during the event.

N.5.15 Occupants with Disabilities or Injuries

Among the 27 persons reporting a disability in their account, two were visually impaired, three were hearing impaired, three used wheelchairs and 19 others were physically challenged such as suffering from a heart condition, asthma, obesity, etc. Twenty-two people mentioned seeing people with disabilities.

Another 47 people who provided first-person accounts were injured that morning. Some accounts from people who suffered injuries reported exiting the buildings later in the evacuation process. However, in numerous accounts occupants mention moving aside in the stairwells to let badly injured and burned people pass, thus it is assumed that those with extreme injuries who were mobile exited the building faster than the majority of others. For instance, one survivor from floor 88 of WTC 1 who suffered burns to over 77 percent of her body reported that crowds parted in the stairwell to let her through (Kugler 2002). These victims were all accompanied by coworkers or emergency workers. Twenty-five people mentioned seeing injured people coming down in the stairwells.

Twenty-three individuals with disabilities and 43 with injuries mentioned a time to start. Of these 66 people, 50 percent (13 people with disabilities and 20 injured) started evacuating immediately, 5 percent (two disabled and one injured) left shortly after, 29 percent (7 disabled and 12 injured) delayed evacuating, 14 percent (one wheelchair user and eight injured) initially decided to stay, and 3 percent (two injured people) were initially stuck.

N.5.16 Phone Calls

An overwhelming 87 percent of those who placed phone calls (151 people) were trying to contact their families and friends to let them know their whereabouts and gather information from them. Only 12 people (7 percent) tried contacting authorities, such as building security or calling 911, and 20 people (12 percent) placed calls to their boss or colleagues. Eleven people (6 percent) did not say who they called.

The majority of people who placed phone calls that morning did so once they were outside (93 people or 54 percent); however, many did not get through. Forty-four people (25 percent) mentioned that they placed calls from their offices before evacuating, 13 people (8 percent) called from other floors and 10 people (6 percent) attempted to make phone calls while in the stairwells.

N.5.17 Knowledge of Situation

In judging the evacuees' knowledge of the situation, categories were created. A "high level" of knowledge indicated knowing that planes had hit the towers or that there had been an explosion within the towers. Those who speculated about a bombing saw fire and debris or had reason to believe an emergency was occurring were said to have a "moderate level" of knowledge. Survivors who were not aware of the reasons behind the evacuation were classified as having a "low level" of knowledge. Level of knowledge was coded for 330 people. As shown in Fig. N–5, survivors with "high levels" of knowledge totaled 69; 214 people were judged to have a "moderate level" of knowledge and 47 survivors had a "low level" of knowledge regarding the events of that morning.

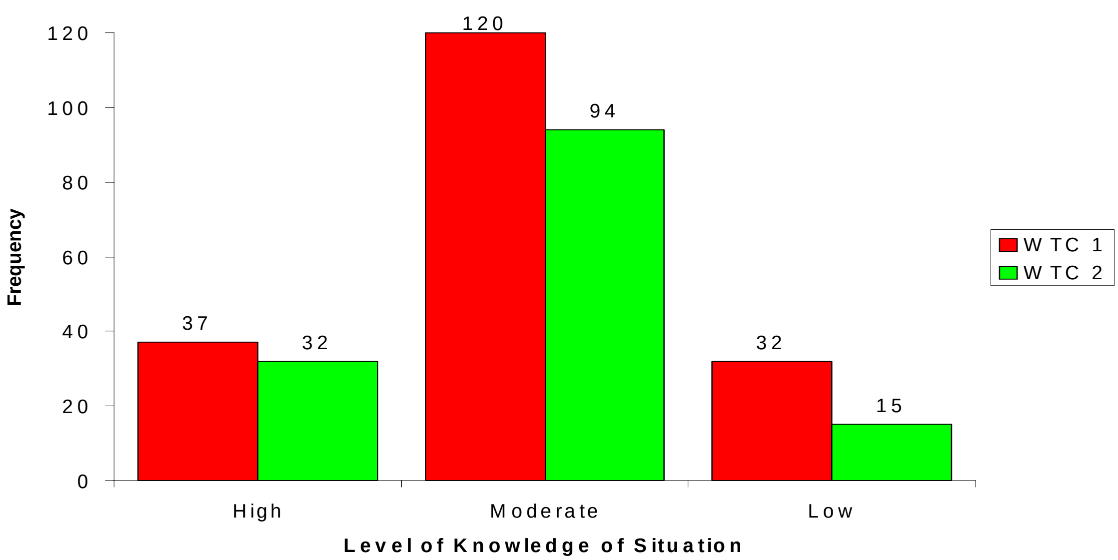

Figure N–5. Knowledge of situation in the towers.

N.5.18 Influence of Others

One hundred and ninety-one survivors reported that their decisions during the evacuation were influenced by others. It appeared that 28 people were influenced by authority figures, such as their boss or manager, and complied with their instructions. Another 97 survivors seemed to be influenced by groups of people and coworkers. One person appeared to have been influenced by both authority figure(s) and the group. Many individuals indicated that they took on leadership roles that morning. Sixty-six people reported they directed people to the stairs, searched for others, gave orders or somehow took part in organizing the evacuation.

Males were more likely to perceive themselves as taking on leadership roles that morning than females (see Table N–8). Thirty-eight women (59 percent of the females for whom influence could be inferred) were influenced by groups of coworkers, whereas only 58 men (46 percent) were apparently influenced by the group. Concerning leadership roles, 52 men (41 percent) reported adopting this behavior, compared to the 14 women who mentioned taking a leadership role (22 percent of the women).

Table N–8. Gender and influence of others.

	Males, N=127	Females, N=64
Authority figures (boss, manager)	17 (13 %)	11 (17 %)
Groups/coworkers	58 (46 %)	38 (59 %)
Both authority and groups	0 (0 %)	1 (2 %)
Took a leadership role	52 (41 %)	14 (22 %)

N.5.19 Perception of Others

How survivors perceived others during the evacuation was recorded for 268 people—others could have been perceived as “calm,” “momentarily panicked,” “upset,” or “helpful.” Multiple responses could be coded for each person. The results show that the majority (154 people or 57 percent) described people around them as calm and orderly. Some 84 people (31 percent) judged others as “upset,” which included crying, shouting, nervous or anxious, but rational. There were 78 people (29 percent) who described others as “momentarily panicked,” in that they were pushing, shoving or generally displaying behavior associated with chaos, while 59 people (22 percent) found others to be “helpful.” More details are presented in Fig. N–6.

It was found that of 155 people in WTC 1, 93 survivors judged others to be “calm,” compared to 61 of 113 people in WTC 2. Only 33 people in WTC 1 described others as “momentarily panicked,” compared to 45 people in WTC 2. For the people in WTC 2, the perception of “panic” occurred before WTC 2 was hit for at least three occupants, while another 29 survivors described others around them as “panicky” after WTC 2 was hit. For two others, the “panicky” behavior was reported at the point in time when each tower collapsed. It was not clear from the other 11 accounts from WTC 2 when the people around them were “panicky.”

This variance in perception of others between the towers is illustrated by contrasting the following two accounts. One survivor from the 65th floor of WTC 1 said that those in the stairwells “maintained their calm really well” and went on to say that, a couple of people started crying a little, but we said, ‘We’re going to get out of here, we just have to take it one step at a time.’ It wasn’t quiet, people were talking–in fact someone was laughing, it was pretty normal (Anderson 2001). It is proposed that the occupants of

WTC 2 observed others "momentarily panicking" mainly once their tower had been hit. One survivor from the 70th floor of WTC 2 said she and her coworkers walked down to the 59th floor and took an elevator to the 44th floor, when at that point, another plane hit their tower and then there was a mad scramble down the stairs with people pushing, shoving and yelling (Black 2001).

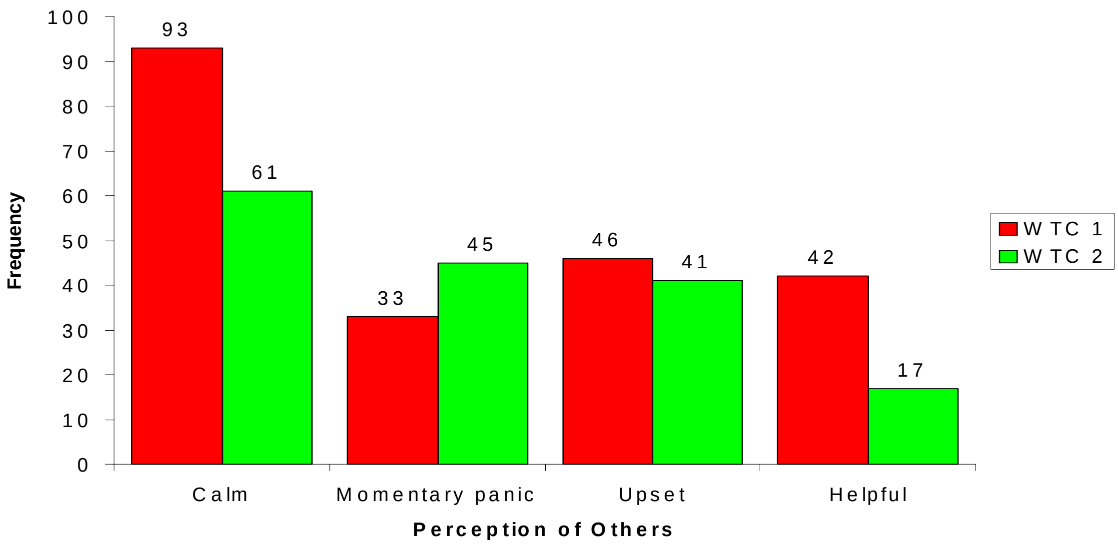

Figure N–6. Distribution of perception of others between the towers.

Perception of others and gender are compared in Fig. N–7.

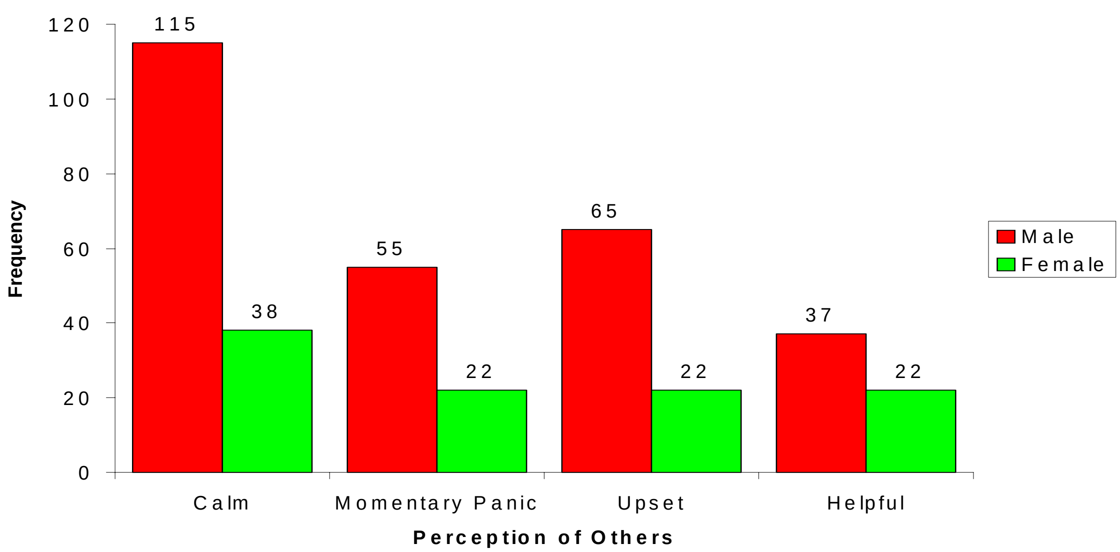

Figure N–7. Distribution of gender and perception of others.

The distribution of perception of others by age group is shown in Table N–9. It is interesting to note that some of the most dramatic language ("chaos," "total chaos," "mayhem") was used by the youngest males.

Table N–9. Distribution of age and perception of others.

	21-35 yrs old (N=74)	36-50 yrs old (N=58)	51-65 yrs old (N=21)
Calm	39 (53 %)	31 (53 %)	9 (43 %)
Panicked	25 (34 %)	14 (24 %)	6 (29 %)
Upset	31 (42 %)	22 (38 %)	4 (19 %)
Helpful	16 (22 %)	17 (29 %)	8 (38 %)

N.5.20 Technology to Gain Information

In addition to the people mentioned earlier who made cell phone calls from the stairwells, 10 people used technology such as wireless e-mail devices and text pagers as a means of gathering information about the situation. Thirteen listened to the radio or watched television, among them three evacuees who stopped to watch TV on the mid floors (43 to 76) of WTC 1 and saw live media coverage of the events.

N.5.21 Impact of the 1993 Evacuation

Only nine percent, 41 people, reported being present during the 1993 bombing and evacuation of the WTC. Of them, three people explained that their experience in 1993 helped them decide to start their evacuation immediately on September 11, 2001. Five people who were present in 1993 mentioned being better prepared this time with evacuation kits. These emergency escape kits were described as being equipped with flashlights, masks, glow sticks, whistles and water (Murphy and Levy 2001). Another 18 people specifically mentioned that 1993 was on their mind during their evacuation, although they were not present during the events of 1993.

Four survivors reported seeing photoluminescent stripes on the stairs, railings and stairwell doors—an improvement the Port Authority made following the 1993 bombing. As one survivor stated, “All you had to do was follow those yellow-green stripes. They were wonderful.” The stripes were especially valuable when the emergency stairs stopped and people had to travel horizontally through mechanical equipment spaces that had many doors (Masetti 2001).

A paraplegic survivor from WTC 1 who was also present for the 1993 evacuation of the WTC commented on the successful use of an evacuation chair on September 11, 2001. The evacuation chairs were part of the improvements made to the WTC evacuation process after the 1993 bombing, and this survivor credits the chair with saving his life. In 1993, he was bounced down the stairs in his electric wheelchair from floor 69 to floor 43, where he was then transferred to a stretcher and carried down the rest of the way. It took him 6 hours to evacuate from floor 69 in 1993. On September 11, 2001, using the evacuation chair enabled him to escape floor 69 of WTC 1 and get to street level in 1 hour and 30 minutes. He went on to say, “If it weren’t for the evacuation chair and the 10 people that brought me down, I would not have made it, that's for sure. That evacuation chair made the difference” (Fink and Mathias 2002).

N.6 SUMMARY RESULTS

Although it is recognized that content analysis of first-person accounts has limitations, and the results cannot be generalized to all occupants of the towers, this methodology was found to be particularly useful in this case. With the large number of accounts that were gathered from a variety of sources (print media, television, radio, internet, emails, etc.), the similar themes and experiences within these texts became more than merely anecdotal stories. Considering that a great majority of the accounts became public within three weeks following the events and that recollection of human behavior is delicately time sensitive, it was important to analyze this information. This methodology could prove useful in future projects dealing with first-person accounts, although events of the magnitude of September 11, 2001, which produced such a large number of first-person accounts, are extremely rare.

For the accounts gathered from media sources, it is recognized that they may represent the most dramatic stories of the evacuation. At the same time, those survivors who have dramatic stories of escape may be more inclined to share them compared to other survivors who may judge their evacuation as less eventful. However, the accounts analyzed were from survivors located in several areas in each tower, providing a distribution of floors from the upper, middle and lower strata of the two towers. In total, 745 accounts were analyzed, representing 435 survivors from WTC 1 and WTC 2.

An interesting and important observation involves the emergence of new first-person accounts from survivors who had not previously shared their stories, around the first anniversary of the event. In trying to explain this phenomenon, it is speculated that survivors who had not previously shared their stories were now prepared to do so after having time to cope and deal with their experience. Many of the evacuees mentioned that telling their stories proved to be a therapeutic exercise. Media sources may have also held accounts gathered from an earlier date or searched for new, untold stories and published them as part of the anniversary coverage.

An important observation stemming from the accounts analysis encompasses the issue of evacuation strategies. It was found that 44 people, about 24 percent of WTC 2 occupants in this study, used the elevators at some point during their evacuation. It has long been accepted among fire safety experts that people know they should not use elevators as a means of egress during an emergency, but those in WTC 2 who chose to use the elevators may have thought it was the quickest or safest route of escape and may have believed that because they were not in immediate danger, they were justified in their decision to use the elevators to evacuate. The same theme is echoed when examining the reactions of the 96 WTC 2 occupants who heard the public address announcement, which told them their building was secure and to return to their offices. Only 16 people took heed of this message and stopped their evacuation, making their way back to, or remaining in, their offices. Through all accounts studied (with the possible exception of one) there was no doubt that people understood the message, as there were no audibility or intelligibility issues; the content of the message was clear. However, the majority of 69 occupants made their decision based on the information that they had at that point in time and decided to disregard the order and continue evacuating. As one survivor stated, "I was thinking that there is a real difference of opinion here about what my eyes are seeing and what the announcement was saying" (Murphy and Levy 2001). The decision to carry on with the evacuation may also reflect the concept of commitment: as these occupants had already made the decision to leave, they pursued this task.

It is also interesting to note that the official procedure for emergencies in the WTC was to meet in the lobby area on each floor and wait for instruction. Nevertheless, the majority of occupants of both towers decided to evacuate on their own after WTC 1 was hit, without waiting for an official building announcement. Thus, this is further evidence that people will make decisions based on what they judge the proper action to take despite official procedures.

Those who had experienced the 1993 terrorist bombing of the WTC left promptly. Although their past experience could have suggested that the evacuation was going to be long and difficult and that people who stayed behind would be evacuated by rescuers later on, very few used this rationale. Instead, most occupants with experience from 1993 felt an urgency to leave immediately.

The results show that 18 people who were identified as having “high levels” of knowledge delayed evacuating. Those who delayed their evacuation reported that they rushed to gather their belongings or went to backup important company files, for they suspected they would not be returning to the building for an extended period of time. These are rational actions; therefore, it is concluded that those with “high levels” of knowledge who delayed evacuating had to have been in areas where the perceived threat to personal safety was not high.

The overall impression of the emotional atmosphere during the evacuation, after reading all 745 accounts, was that of calm and order. Although some reported crying and being anxious or nervous, the majority viewed themselves and others as composed. A stark contrast in perceived behavior was found to exist between the two towers, with the majority of WTC 1 occupants reporting others as “calm” (93 of 155 people), where as a large proportion of WTC 2 occupants perceived others to be “panicked” (45 of 113 people). The perception of “panic” occurred before WTC 2 was hit for at least three occupants, while another 29 survivors perceived others as “panicked” after WTC 2 was hit. After their building had been struck, WTC 2 occupants may have realized they were under attack, which could possibly explain the heightened level of anxiety in the tower. (It is important to note, however, that the colloquial use of the word panic more often describes a state of mind—high anxiety, for example—rather than the irrational actions that more correctly define “panic.”)

Emergency crews disrupted evacuation in the stairwells while going against traffic, but many evacuees who mentioned seeing firefighters felt reassured and safe due to their presence. It is assumed that this counter flow did not prevent occupants from evacuating, as the last people to exit reported being alone in the stairs while they were descending rapidly seconds before the collapse. Evacuees used technology such as cell phones, wireless e-mail devices, and text messaging over pagers during their descent as a means of gathering information about the situation unfolding around them.

N.7 FUTURE WORK

Future research is needed to fully understand the evacuation behavior of the occupants who were in the two towers of the WTC on September 11, 2001. A variety of approaches should be used to gather this information such as interviews and questionnaires. Unfortunately, the extended amount of time that has elapsed since the events is an important factor to mitigate, since occupants’ recollection may be incomplete and contaminated by what has been seen, read, or heard since September 11, 2001.

Technology clearly played a role in providing occupants with information about the event during their evacuation. This phenomenon raises important issues regarding the information age and how new technologies can be taken advantage of to aid in emergency situations. If technology can help to disseminate timely information to the public in times of crises, strategies should be developed to enable authorities to fully utilize such technology.

This major event, which was repeatedly broadcast on television around the world, may also influence fire safety in high-rise buildings in general. It is essential to study how the perception of risk in high-rise buildings has changed since September 11, 2001. Do people who live, work or visit high-rise structures feel more at risk of a potential fire or fear that the building might collapse if there is a fire? If the occupants feel more at risk, what is their likely behavior and response in future emergencies? Studies should be conducted to explore the impact of high-rise risk perception on intended behavior in future emergencies. Are occupants prepared to follow procedures and instructions? Would they comply with a protect-in-place approach or to move to a refuge floor? If all occupants want to evacuate to the ground floor or exit during an emergency, requirements for stair design and building height might need to be revisited. Drills should be conducted to observe unannounced emergency evacuations in high-rise buildings, varying evacuation strategies and information provided to occupants to assess actual response. Longitudinal studies should also be conducted to assess the impact of September 11 over time on high-rise building occupants.

N.8 ACKNOWLEDGMENTS

The authors would like to express their appreciation to Amber Walker, formerly of the National Research Council of Canada, for the work she did developing the original matrix format and undertaking the early analyses that form the basis of this study. They would also like to thank Joseph Molis and Norma Candeloro of NFPA for their contributions to this work.

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Attachment 1
WTC FIRST-PERSON ACCOUNTS CODE BOOK

1. BLDG - 'Building Location at the Time of Awareness / Perception of First Cue'

1 = Tower 1, North Tower

2 = Tower 2, South Tower

3 = Plaza/Outside

4 = Concourse

5 = Mall

6 = PATH Train

7 = Bldg 7 or Bldg 3

99 = n/a

2. FLR - 'Floor Location at Perception of First Cue'

SPLIT COLUMN EXACT FLOOR AND CATEGORY

1 = T1 Lower (basement-42) in stairs

2 = T1 Lower (basement-42) on a floor

3 = T1 Mid (43-76) in stairs

4 = T1 Mid (43-76) on a floor

5 = T1 Upper (77-110) in stairs

6 = T1 Upper (77-110) on a floor

7 = T1 in stairs, level not specified

8 = T1 location not specified

9 = T1 mezzanine, lobby, concourse

10 = T2 Lower (basement-42) in stairs

11 = T2 Lower (basement-42) on a floor

12 = T2 Mid (43-76) in stairs

13 = T2 Mid (43-76) on a floor

14 = T2 Upper (77-110) in stairs

15 = T2 Upper (77-110) on a floor

16 = T2 in stairs, level not specified

17 = T2 location not specified

18 = T2 mezzanine, lobby, concourse

19 = Outside

22 - T1 elevator - lower floors

23 - T1 elevator - mid floors

24 - T1 elevator - upper floors

25 - T1 elevator, level not specified

99 = n/a

3. SEX - ‘Sex of Evacuee’

1 = male

2 = female

99 = n/a

4. AGE_CODE - ‘Age of Evacuee’

SPLIT COLUMN EXACT AGE AND CATEGORY

1 = 21-35

2 = 36-50

3 = 51-65

4 = 66 +

99 = n/a

5. DATE - ‘Date of Record’

SPLIT COLUMN EXACT DATE MENTIONED

1 = Week of (09/11/2001-09/15/2001)

2 = 2 weeks after (09/16/2001-09/30/2001)

3 = 1-3 months after (10/01/2001-12/31/2001)

4 = 4-6 months after (1/01/2002-3/31/2002)

5 = 7-9 months after (4/01/2002-6/30/2002)

6 = 10-12 months after (7/01/2002-9/30/2002)

99 = n/a

6. EGRESS - ‘Evacuation Method’

1 = Stairs

2 = Changed stairwells

3 = Elevator

4 = Combo of stairs and elevator

99 = n/a

7. FSTCUE - ‘First Cue of Event’

COLUMN CHECKED OFF FOR EACH INITIAL CUE MENTIONED

1 = Audio (boom, crash, explosion, thunder, blast, roar, rumbling)

2 = Visual (smoke, fire, bodies, plane approaching, panicked people, debris falling)

3 = Building Movement (impact, sway, shake, earthquake, rocking, jolt)

4 = Content Movement (chairs moving, ceiling falling, bounce in elevator, debris in halls/offices, lights flickering, change in air pressure, burned by fire)

5 = Warn by others (directly told or behavior of others)

6 = Physically impacted (burned, fell or thrown out of chair)

7 = Smelled fumes or Felt heat

99 = n/a

8. ALRM – Heard Alarm

1 = Yes, heard alarm

2 = Heard alarm on floor

3 = Heard alarm in stairs

4 = ‘I did not hear an alarm’

99 = n/a

9. STTIME - ‘Time to Start Evacuation’

1 = Immediately (ran, right away, rapidly): 1 minute

2 = Shortly after (short delay, picked up belongings, warn others): up to 5 minutes after

3 = Delayed (gathered belongings, look out window, make phone calls, watch TV, kept working, checked security, planned with coworkers, shut equip off, Post T2 Impact)

4 = Stayed (to help: headcount, direct people, assisted coworkers, waited to be rescued/given instructions; went up)

5 = Stuck (behind debris, walls, in elevator)

99 = n/a

10. CNDFL - ‘Condition on Floor When Building was Hit’

1 = Devastated (combo of debris, fire, walls collapsed, ceiling/lights down, darkness, water/sprinklers, smoke, jet fuel, glass, bodies)

2 = Abnormal (some smoke, heat, smell fuel, power out, dusty, debris past windows, some reason for alarm/evacuation)

3 = Normal (usual working conditions)

99 = n/a (incl. not on floor when building was hit)

11. CNDFL – 'Condition on Floor'

COLUNM CHECKED OFF FOR EACH CONDITION MENTIONED.

1 = Normal

2 = Door Jammed

3 = Debris – Wall, ceiling collapsed

4 = Smoke

5 = Dust

6 = No power – dark

7 = Smell

8 = Water

9 = Fire

10 = Crowd, injuries

11 = Trapped

12 = Not on a floor

99 = n/a

12. STRS - 'Condition in Stairwell During Evacuation'

COLUMN CHECKED OFF FOR EACH CONDITION MENTIONED.

1 = Normal

2 = Door locked, jammed

3 = Crowd, hot

4 = No power

5 = Water

6 = Cracked wall

7 = Debris

8 = Smoky, smelly

99 = n/a

13. ANCHRD - ‘Heard Announcement’

    1 = T1 Yes

    2 = T1 No (mentioned specifically not hearing message)

    3 = T2 Yes

    4 = T2 No (mentioned specifically not hearing message)

    99 = n/a

14. ANCACT - ‘Action After Hearing T2 Announcement’

    1 = Continued evacuating

    2 = Continued evacuating saw some returned

    3 = Returned to office/Stay on location

    99 = n/a

15. ANCFLR - ‘Location when T2 Announcement Heard’

    10 = T2 Lower (basement-42) in stairs

    11 = T2 Lower (basement-42) on a floor

    12 = T2 Mid (43-76) in stairs

    13 = T2 Mid (43-76) on a floor

    14 = T2 Upper (77-110) in stairs

    15 = T2 Upper (77-110) on a floor

    16 = T2 in Stairs not specified

    17 = T2 Location not specified

    18 = T2 mezzanine, lobby, concourse

    19 = Outside

    20 = T2 in Elevator

    99 = n/a

16. LT2IMP - ‘Location at T2 Impact’

    1 = T1 Lower (basement-42) in stairs

    2 = T1 Lower (basement-42) on a floor

    3 = T1 Mid (43-76) in stairs

    4 = T1 Mid (43-76) on a floor

    5 = T1 Upper (77-110) in stairs

    6 = T1 Upper (77-110) on a floor

    7 = T1 in stairs, level not specified

8 = T1 location not specified (incl. Inside elevator)

9 = T1 mezzanine, lobby, concourse

10 = T2 Lower (basement-42) in stairs

11 = T2 Lower (basement-42) on a floor

12 = T2 Mid (43-76) in stairs

13 = T2 Mid (43-76) on a floor

14 = T2 Upper (77-110) in stairs

15 = T2 Upper (77-110) on a floor

16 = T2 in stairs, level not specified

17 = T2 location not specified (incl. Inside elevator)

18 = T2 mezzanine, lobby, concourse

19 = Outside

99 = n/a

17. LT2COL - ‘Location at T2 Collapse’

1 = T1 Lower (basement-42) in stairs

2 = T1 Lower (basement-42) on a floor

3 = T1 Mid (43-76) in stairs

4 = T1 Mid (43-76) on a floor

5 = T1 Upper (77-110) in stairs

6 = T1 Upper (77-110) on a floor

7 = T1 in Stairs not specified

8 = T1 in Elevator

9 = T1 mezzanine, lobby, concourse

10 = T2 mezzanine, lobby, concourse

11 = T2 Lower (basement-42) in stairs

12 = Outside

13 = Other WTC building

14 = Subway

99 = n/a

18. LT1COL - ‘Location at T1 Collapse’

1 = Lower T1 (basement-43) stairs

2 = T1 mezzanine, lobby, concourse

3 = Outside

99 = n/a

19. LFFS - 'Location When Met Firefighters'

1 = T1 Lower (basement-42) in stairs

2 = T1 Lower (basement-42) on a floor

3 = T1 Mid (43-76) in stairs

4 = T1 Mid (43-76) on a floor

5 = T1 Upper (77-110) in stairs

6 = T1 Upper (77-110) on a floor

7 = T1 in stairs, level not specified

8 = T1 location not specified

9 = T1 mezzanine, lobby, concourse

10 = T2 Lower (basement-42) in stairs

11 = T2 Lower (basement-42) on a floor

12 = T2 Mid (43-76) in stairs

13 = T2 Mid (43-76) on a floor

14 = T2 Upper (77-110) in stairs

15 = T2 Upper (77-110) on a floor

16 = T2 in stairs, level not specified

17 = T2 location not specified

18 = T2 mezzanine, lobby, concourse

19 = Outside

99 = n/a

20. HELP - 'Who Helped Evacuee during Evacuation'

COLUMN CHECKED OFF FOR EACH HELPER MENTIONED

1 = Firefighter

2 = Port Authority (building staff/security)

3 = External Official (police, FBI, EMT, rescue workers)

4 = Coworkers

5 = Passed Firefighters in Stairs

99 = n/a

21. DSBLD - ‘Evacuee Disability and Injury’

1 = Visual impairment

2 = Hearing impairment

3 = Physically challenged (obese, asthma, heart condition)

4 = Wheelchair user

5 = Injured during event (burned, sprained ankle, broken bones, emotional trauma)

6 = Helped disabled (during the evacuation)

7 = Saw disabled (during the evacuation)

8 = Helped injured

9 = Saw injured

99 = n/a

22. B1993 - ‘1993 WTC Bombing Presence’

1 = Yes

2 = Yes, prepared since (evacuation packs)

3 = Yes, reason evacuated early

4 = Yes, reason stayed

5 = No

6 = 1993 bombing in the back of their mind but were probably not there at the time

99 = n/a

23. DELAY - ‘Reason for Delay in Evacuation’

1 = Decide to stay

2 = Activity to complete before leaving (search floor, secure document, made calls, instruct others)

3 = Went Up/Return

4 = Stuck or trapped

5 = Help others, disabled or injured/Being helped

6 = Told to stay

99 = n/a

24. LPHONE - ‘Location when Evacuee Made Phone Call’

1 = Office

2 = Other floor

3 = Stairs

4 = Outside

5 = Multiple locations

99 = n/a

25. WPHONE - 'Recipient of Evacuee Phone Call'

COLUMN CHECKED OFF FOR EACH GROUP MENTIONED

1 = Family and friends (spouse, parents, home)

2 = Colleague or boss

3 = Authorities (building security, 9-1-1)

99 = n/a

26. REST - 'Rest during Evacuation'

1 = T1 Lower (basement-42) in stairs

2 = T1 Lower (basement-42) on a floor

3 = T1 Mid (43-76) in stairs

4 = T1 Mid (43-76) on a floor

5 = T1 Upper (77-110) in stairs

6 = T1 Upper (77-110) on a floor

7 = T1 in stairs, level not specified

8 = T1 location not specified

9 = T1 mezzanine, lobby, concourse

10 = T2 Lower (basement-42) in stairs

11 = T2 Lower (basement-42) on a floor

12 = T2 Mid (43-76) in stairs

13 = T2 Mid (43-76) on a floor

14 = T2 Upper (77-110) in stairs

15 = T2 Upper (77-110) on a floor

16 = T2 in stairs, level not specified

17 = T2 location not specified

18 = T2 mezzanine, lobby, concourse

19 = Outside

99 = n/a

27. OBSTCN - 'Obstructions Encountered During Evacuation'

COLUMN CHECKED OFF FOR EACH OBSTRUCTION MENTIONED

1 = Door Jam (locked or jammed)

2 = Debris (wall falling, floor collapse, material damaged)

3 = Smoke

4 = No power

5 = Smell (of fuel)

6 = Water

7 = Fire

8 = Crowd, disabled, injured

9 = Trapped by building rubble

99 = n/a

28. TMOUT - 'Time Evacuee Exited Building'

1 = T1: 8:48-9:02

2 = T1: 9:03-9:30

3 = T1: 9:31-9:58

4 = T1: 9:59-10:27

5 = T1/T2: 10:28+

6 = T2: 8:48-9:02

7 = T2: 9:03-9:30

8 = T2: 9:31-9:58

99 = n/a

29. KNWSIT - 'Evacuee's Knowledge of the Situation in the Initial Moment'

1 = High (terrorism/plane attack/ T2 collapsed/saw plane approaching/hitting building)

2 = Moderate (fire/bomb/earth quake/serious emergency/speculated plane/rumors)

3 = Low (reason for evacuation unknown or limited)

99 = n/a

30. SRSNSS - 'Level of Seriousness to Themselves in the Initial Moment'

1 = Very serious (fear, scared, want to get out ASAP)

2 = Somewhat serious (worried, did not know what was happening)

3 = Not serious (not concerned)

99 = n/a

31. SOINFL - ‘Social Influence on Evacuee’s Decisions’

1 = Authority figure (boss, supervisor, manager)

2 = Coworkers/Group influence

3 = Survivor took leadership role

4 = Boss and group influence

99 = n/a

32. TCINFL - ‘Technological Influence on Knowledge during Evacuation’

1 = Cell phone

2 = Blackberry, Text pager (deaf)

3 = TV, radio

4 = Walkie Talkie

99 = n/a

33. PERCEP - ‘Perception of Others During Evacuation’

COLUMN CHECKED OFF FOR EACH PERCEPTION MENTIONED

1 = Calm/Orderly (civil, supportive, chatty, composed)

2 = Momentarily Panicked (running, pushing, shoving)

3 = Upset (crying, shouting, fearful, anxious)

4 = Helpful (assisting others)

99 = n/a

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Appendix O
INTERIM REPORT ON TELEPHONE INTERVIEWS

O.1 SCOPE AND OBJECTIVE

Interviews with survivors of the World Trade Center (WTC) attacks were conducted using three methods: telephone interviews, face-to-face interviews, and focus groups. This appendix will review four aspects of the telephone interviews: methodology, sample disposition, telephone questionnaire, and preliminary results of the telephone interviews for the pre-September 11, 2001, data. Significant additional analysis will be completed over the next several months to develop as clear an understanding as possible of the evacuation of WTC 1 and WTC 2 on September 11, 2001. These findings will be enhanced and compared with findings from many other sources, including face-to-face interviews, focus groups, published accounts (see Appendix N of this report for a discussion of published accounts analysis), 9-1-1 records, and other materials.

The multimethod approach was selected for several reasons. First, multiple methods increase confidence in the conclusions and findings when more than one method arrives at the same conclusions. Second, the multiple objectives of the investigation mandated complementary approaches to accomplish all the goals. In other words, it is difficult to establish a scientific foundation for general findings while also broadly investigating and establishing new facts and discovering unique events using only one method. Finally, concerns associated with the time latency since September 11, 2001, suggest the use of different approaches and techniques in order to increase memory recall and accuracy.

The telephone interview questions and protocols met all Federal requirements regarding the Common Rule for the Protection of Human Subjects, including Institutional Review Board (IRB) and National Institute of Standards and Technology (NIST) approvals. Further, the telephone interview questions met the requirements of the Paperwork Reduction Act, subject to Office of Management and Budget (OMB) approval number 0693-0044.

O.2 METHODOLOGY

The survey objectives of the telephone interviews called for collecting 800 computer assisted telephone interviews (CATI) of persons occupying either of the two WTC towers (WTC 1 and WTC 2) at the time of the terrorist attacks on September 11, 2001. The sample size of 800 and allocation of n=400 to each tower were determined to simultaneously maximize the statistical precision within each tower. Primary statistical analyses are in the form of tabulations and linear statistics (e.g., reporting of percentages and average/means). Estimates of percentages from tower-specific survey data (at n=400) exhibit sampling errors no greater than 2.5 percentage points, and 95 percent confidence intervals of percentages are no greater than +/- 5 percentage points. This level of precision is more than adequate for examining characteristics of occupants and egress attributes.[1]

Attempts were made to equally divide the respondents among WTC 1 and WTC 2 occupants (i.e., n=400 occupant interviews from each tower). Within each of the WTC buildings, independent proportionate stratified samples of survivors were drawn. In other words, each occupant of a particular tower had an equal probability of being selected.

O.2.1 Sampling Frame

The sampling frame (i.e., the list from which the sample was drawn) consisted of the names of occupants from badge lists of WTC 1 and WTC 2. All occupants of the WTC were required to provide personal data in support of issuing badges to clear through the security station at the entrance of each tower. The badge lists were provided to NIST by the Port Authority of New York and New Jersey. The lists provide name, floor of occupancy, employer, and social security number, the only available means of uniquely identifying individuals.

O.2.2 Tracking and Screening the Sample

The badge lists contained September 11, 2001, occupants, occupants who were absent on the day of the attacks, decedents, former occupants, and nonperson listings (false names used in sample testing). This means that a screening effort was needed to identify "eligible" badge list members—namely, those who were inside WTC 1 or WTC 2 during the attacks. Moreover, the absence of telephone numbers for the badge holders on the list necessitated a tracking/locating effort. The primary tracking mechanism was to search public databases using commercially available batch matching and Web-based search utilities. This necessitated a large sample to generate the 800 completed interviews.

O.2.3 Design Parameters

The number of occupant selections drawn into the sample was contingent on four key design parameters:

- The percentage of individuals from badge listings for whom a working telephone number could be found (initial estimate: 80 percent tracking success)
- The percentage of badge listings that corresponded to a surviving WTC 1 or WTC 2 occupant on September 11, 2001 (initial estimate: 14 percent)
- The cooperation rate for screening the occupants (initial estimate: 65 percent)

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[1] Multivariate modeling such as correlation analyses, multiple linear regressions, and path analyses, are also a prominent part of the survey analyses. Like the tabulations, these analyses are being conducted independently by tower. A sample size of n=400 per tower provides more than ample statistical power for the F tests used to determine the significance of the regression models (i.e., testing the null hypothesis that the ratio of explained variance to error/residual variance is equal to zero). For instance, in a multiple regression analysis featuring 20 independent variables, the sample size of 400, and 0.05 level of significance (Type I error), the power of the F test to detect an r^2 statistic (i.e., proportion of explained variance) of 0.06 is just over 81 percent. See also Chapter 9 of Cohen, J., 1988, Statistical Power Analysis for the Behavioral Science, Lawrence Erlbaum Associates, Inc., Hillsdale, N.J. Multivariate modeling results will be presented at a later date.

- The interview response rate among September 11, 2001 survivors (initial estimate: 50 percent)

O.2.4 Expected Dispositions

In planning the CATI survey, a number of design parameters needed to be quantified in order to determine the number of persons to draw from the badge list. The expected disposition of the sample was developed using the parameters defined in the aforementioned paragraph. A total sample of 22,735 persons from the badge list was needed to generate the desired 800 completed interviews. The expected disposition by tracking efforts, screening, and interviewing are discussed later.

O.2.5 Reserve Sample

A reserve sample of about 14 percent (or about n=3,265) was added in the event additional sample size was needed due to unanticipated circumstances (e.g., the eligibility rate is lower than anticipated). This brought the total sample size to 26,000. The reserve was held "in reserve" while the main sample was worked. Working the main sample allowed preliminary estimates of all design parameters to be monitored so that an informed decision could be made on the necessity of releasing none, some, or all of the reserve.

O.2.6 Disproportionate Allocation

The badge list contained different counts of persons from each tower, yet our sample design called for equal samples to be drawn from the collections of badge holders in WTC 1 and WTC 2. Thus, a disproportionate design (across tower strata) was employed. But within a tower, independent proportionate samples were drawn using stratification by floor (within tower), employer (within floor) and last name (within employer). This served to increase the statistical precision of the tower-specific samples.

Thus, equal-sized samples of 13,000 selections were drawn from each of WTC 1 and WTC 2 badge lists. Each tower-specific sample was partitioned into 20 random replicates (comprising 5 percent of the total), and the reserve sample was determined by the last several random replicates for each tower. It is important to note that all badge holders from WTC 1 floors 92 and above were omitted from sampling because there were no survivors from those floors.

O.2.7 Final Sample Disposition Analysis

A total sample of 26,000 was drawn, comprising 13,000 names for each tower. Table O–1 summarizes the final disposition of the CATI sample and the total (locating) sample. The table is comprised of two sets of rows. The top set pertains to the CATI sample and represents those sample persons for whom an initial telephone number was identified prior to commencing the CATI survey operations. The bottom set of rows with the heading "Total Sample Disposition" represents the results of our locating/tracking effort used to identify usable telephone numbers associated with the sample subjects. (Recall that only name, SSN and employer were available; no other contact information was readily available).

Table O–1. Disposition of the CATI sample and the total sample by tower.

CATI Disposition	WTC 1[a]	WTC 2[a]	Total	% Distn
Interview	427	376	803	4.0 %
Partial interview	47	37	84	0.4 %
9/11 decedent	20	40	60	0.3 %
Other decedent	49	39	88	0.4 %
Not eligible	3,712	3,752	7,464	37.5 %
Language barrier	135	129	264	1.3 %
Eligible refused to interview	138	139	277	1.4 %
Other refusal	224	181	405	2.0 %
Respondent not interviewed	247	168	415	2.1 %
Can't contact/locate respondent	4,987	5,076	10,063	50.5 %
CATI total	9,986	9,937	19,923	100.0 %
Total sample disposition:	WTC 1	WTC 2	Total	% Distn
Found initial telephone number	9,986	9,937	19,923	76.6 %
Unable to find a telephone number	3,014	3,063	6,077	23.4 %
Sample total	13,000	13,000	26,000	100.0 %

a. Table data are unweighted. Tower location as indicated in the badge list and may differ from reported tower location.

The bottom set of rows shows that telephone numbers were identified for just over three quarters (76.6 percent) of the sampled subjects. Moreover, this rate was fairly uniform across towers. The 19,923 individuals with an initial telephone number were then loaded into the CATI sample management system for calling. Ultimately, all reserve samples were used in the telephone survey. In the initial design parameters, it was assumed that 82 percent of the subjects would be locatable. While 76.7 percent is close, many of the numbers were obsolete (e.g., disconnect, wrong number) and necessitated additional tracking during CATI operations. Ultimately, by the end of data collection, only half the sample (49.5 percent) represented confirmed contacts with subjects.

The top set of rows in Table O–1 presents the final disposition of the sample by tower as well as for the overall sample. Several statistics in the percentage distribution (rightmost) column are notable. First, we were unable to contact subjects for half the sample (50.5 percent), due either to failures to answer the phone, answering machines, unusable numbers (e.g., wrong number, disconnected, business), etc. Most of these telephone numbers represent “unlocatable” subjects—subjects for whom the initial telephone number was incorrect. It bears reiterating that substantial additional research during CATI operations was conducted using powerful subscription-based Web-based search engines. Unfortunately, little information was available for these individuals.

A second result of interest is the prevalence of ineligible subjects—those not in the building on the morning of September 11, 2001. An assessment of eligibility rates appears later in this appendix. A third result is the existence of decedents—some from the September 11 attack and others from causes not necessarily related to September 11, 2001 (e.g., cause unknown, natural causes). Most of the September 11, 2001, decedents were encountered due to a difference in the full (formal) name of the subject and the name that appeared on the badge list (e.g., the badge list sometimes contained maiden names, middle names, nicknames, misspelled first or last names, out-of-sequence names, titles, and so on). This impeded the ability to remove known decedents prior to calling.

The outcome of CATI operations on the final outcome rates is presented by tower in Table O–2. The table shows screening rates, interview rates, and rates of eligible occupants (among those who responded to the screening questions). The first row shows that screening response rates were relatively uniform across towers at about 46 percent. A screening response rate of 65 percent had been planned. Similarly, interview response rates (among screened eligible subjects) were relatively stable across towers at about 49 percent. This is consistent with the planned interview response rate of 50 percent.

Table O–2. Summary disposition rates by tower.

Disposition Rate	WTC 1	WTC 2	Total
Screen	46.5 %	45.8 %	46.1 %
Interview	48.6 %	49.5 %	49.0 %
Eligibility	18.9 %	16.7 %	17.8 %
Overall	22.6 %	22.7 %	22.6 %

Note: Definitions for "Rates" consistent with American Association of Public Opinion Research (AAPOR) Standards, which may be found at http://www.aapor.org/pdfs/standarddefs2004.pdf.

The eligibility rates were higher than expected – about 18 percent overall compared to the 14 percent expected. The eligibility rate among WTC 1 subjects was slightly higher than those of WTC 2. However, the overall response rates are essentially uniform across towers, at 22.6 percent.

O.3 TELEPHONE INTERVIEW SCHEDULE

The telephone interview was conducted by a trained interviewer using a computer program which provides questions and answer categories for the interviewer. Prior to calling, subjects received a letter that outlined the scope and purpose of the investigation, the purpose of the interview, and the telephone call that came several days later. A full informed consent statement appeared in the letter, as well. A copy of the letter can be found in Attachment 1 of this appendix.

When interviewers reached the subjects by telephone, the respondents were provided a description of the survey, the confidentiality of responses, the length of the interview, and the voluntary nature of participation. They were then asked if they wished to participate, thereby obtaining oral informed consent. The full text of the informed consent statement appears after the advance letter as Attachment 2.

The telephone interview instrument, Attachment 3 at the end of this appendix, includes the questions, variable names, response options, and skip patterns directly from the computer program used by the interviewers. Variable names are used as shorthand for subsequent data analysis. Questions had a variety of response option categories: multiple choice, interval, Likert scale, or open-ended. Open-ended responses were minimized where possible due to the analysis burden and the fact that face–to–face interviews are also being conducted. Skip patterns reduce burden on the respondent by skipping questions that would not apply to a particular respondent. For example, a respondent would not be further questioned about fire drills if they did not receive fire drill training. Subsequent discussion of the questions indicates whether a respondent was read a list of choices or was expected to give a free response.

The interview was designed with five primary groups of questions, covering emergency training and preparedness, three stages of evacuation experience, and background information about the respondent.

O.3.1 Preparedness and Training

The first group of questions served to measure the extent to which an occupant had any special level of knowledge about the building, other than what would be obtained by performing their job. The most prevalent special knowledge would be formal evacuation training, or fire drills. If a respondent indicated that they participated in evacuation training during the 12 months prior to September 11, 2001, further questions were asked about the content of the training. The occupant's understanding of the emergency procedures, or the way it was 'supposed to go,' was also measured. Next, a Likert Scale[2] measured the usefulness of the evacuation training in the context of their egress experience on September 11, ranging from very helpful to very unhelpful. Finally, the respondent was asked whether he or she knew that there was a floor warden for their floor.

O.3.2 Initial Experience on September 11, 2001

The second group of questions covered the first moments of September 11, 2001, as experienced by the respondent, also known as the initial awareness period. How a person first became aware that something was not normal, whether in their building or the neighboring building, may have influenced subsequent decisions. Examples of awareness channels may include sensory perception, such as feeling, hearing, or seeing the building shake, seeing or smelling fire or smoke, or may include a conversation with a person inside or outside the WTC complex. Next, the respondent was asked to provide context to the initial moment of awareness. Context was first created by identifying what activity the respondent was performing. Activities may include, but are not limited to, working, conversing with coworker(s), eating, or participating in a meeting. The respondent was then asked to recall the number of other people they were with at the first moment of awareness. People in groups often defer to group decisions rather than making their own evacuation decisions. Next, a list of observations was read aloud and the respondent is asked to indicate whether they noticed the event during the period of initial awareness. These events included smoke, fire, fireballs, collapsed walls, jet fuel, severely or fatally injured people, sprinklers going on, fire alarm sounding, power outage or flickering lights, fallen ceiling tiles, and extreme heat. The event proximity was probed for every affirmative response to determine whether the observed event was in the immediate area or outside the building. If no affirmative responses were indicated, the respondent was asked whether they observed any disaster related events not previously mentioned. Finally, the extent of any injuries to the respondent or those in the immediate area was ascertained, as well as whether the respondent felt that their life or the lives of other people were in danger.

[2] A Likert Scale measures the degree to which the respondent agrees or disagrees with a statement. In this case, the scale measured helpfulness, including very helpful, helpful, unhelpful, and very unhelpful. A neutral response was not included.

O.3.3 Interim Experience on September 11, 2001

The format of the interim experience group of questions mirrored the format of the initial awareness questions. The interim time period was defined as the time after initial awareness, but before the person entered a stairwell or elevator to leave the building. This time period may range from moments to tens of minutes. The objective of the interim period questions was to determine what motivated/forced people to either immediately evacuate or delay their evacuation by some period of time.

Information about the nature of the event often forms the basis for decision-making during the interim period. Many people may have found the environmental cues from the initial awareness period sufficient to initiate an immediate evacuation. Others may have required additional information in order to feel comfortable leaving the workplace. Occupants could have obtained information in two ways: passively and actively. Passive information is information received without seeking it out. In other words, the information was received regardless of whether the person felt it was needed. Active information is information which the respondent actively seeks and considers important with respect to their decision to evacuate. The respondent was first asked whether they received any additional information about the event during the interim period. If so, the source (who), the nature (what), and the channel (how) of the information was probed. Next, additional information sought out by the respondent was probed, including the source, nature, channel, and whether the process was successful in gathering additional information.

The perception of risk to the respondent's life, as well as the lives of others was asked in the same way as during the initial period, in order to determine whether the sense of risk was increasing or decreasing over time. The interviewer probed about the activities of other people in the proximity of the respondent, which may influence the respondent's subsequent choices. Whether people began evacuating prior to the respondent was specifically asked. Next, the respondent was asked about the activities they undertook during the interim period, as well as activities that they wanted to do but could not. These activities included work-related actions, such as saving files or shutting machines down; personal actions, such as gathering belongings or calling people; or emergency-related actions, such as fighting fires/smoke, and searching for or helping others. If a respondent was unable to accomplish an action, the action and the reason for being prevented from accomplishing the action was gathered.

As with the initial period, any observations of building damage or distress were collected. If the respondent received help in any way before initiating evacuation, the nature and source of the assistance was determined. The respondent was asked what the primary cue was which initiated their evacuation on September 11 and how many minutes passed before they started evacuating. Finally, the respondent was asked whether anything prevented them from evacuating sooner than they reported.

O.3.4 Evacuation Experience on September 11, 2001

The next group of respondents completed the questions about the September 11, 2001, evacuation experience and focused on time spent in the stairwell and/or elevator(s). The respondent was first asked whether they began their evacuation alone or with other people. Which stairwell (or elevator) the respondent entered was collected as either the stair identification letter (A, B, or C) or the geographic location, if known. Knowing where the stairwell emptied out at the bottom may also narrow down which stairwell was used, which was collected near the end of this group of questions, [Stairs A/C (44 in. wide) emptied out to the upper, Mezzanine level, while Stair B (54 in. wide) went to the lower, Concourse level]. Next, the respondent's rationale for using a particular stairwell was probed. The respondent was then asked whether they left the stairwell or turned back for any reason during the evacuation and, if so, why?

Some events and features of the stairwells aided the progress of the evacuation, while other features constrained the progress of the evacuation. The following features or events were identified to the respondents, who were asked to indicate whether it acted as an aid to their egress: instructions or assistance from their floor warden, a police office, or fire fighter, support/encouragement from others, exit signage, and photoluminscent paint. The following items were identified to determine whether they served to constrain the evacuation: crowded stairwells, counterflow (people moving up the stairs, against the flow of occupants), disabled or injured people being taken down the stairwell, locked doors, poor lighting, confusing or missing signage, and lack of clear instructions.

As with the initial and interim time periods, environmental cues related to fire smoke, jet fuel, and other disaster-related observations were probed, as well as whether the observation was in the immediate area or outside the tower. The final question about the respondent's own evacuation estimated the elapsed time from entering the stairwell until they left the building. A concluding evacuation question determined whether they knew why someone on their floor did not survive the WTC attack, if applicable.

O.3.5 Respondent Background

The final group of questions explored the background of the respondent relevant to evacuation. The first question identified any preexisting disabilities or injuries which made evacuation more difficult. The respondent's age, gender, and primary language were collected. If the respondent was working in the building prior to 1993, they were asked whether they were present during the February 26, 1993 bombing. If so, respondents were asked questions about their evacuation experience.

The interview concluded with an open-ended opportunity for the respondent to say anything additional about their evacuation experience on September 11, 2001. Respondents who indicated that they had a disability, were near the floors of impact, observed fire, smoke, or fireballs in their immediate area, or had a role of building responsibility on September 11, 2001, were asked if they would be willing to participate in a follow-up face-to-face interview.

O.4 PRELIMINARY RESULTS

The following section is a preliminary analysis of the telephone interview data. For this interim report, only pre-September 11, 2001 questions, or occupant background, preparedness, and training data, are analyzed and presented. Data related to September 11, 2001, evacuation experiences are currently being analyzed in the context of other data, such as face-to-face interviews and 9-1-1 tapes.

O.4.1 Response Rate Analysis

The response rate analysis of the telephone interview sample indicated an inverse relationship between floor height and the rate of response in WTC 1, as shown in the last column of Table O–3. The nonresponse weight adjustment is the inverse of the overall response rate. For example, the inverse of

25.3 percent is 3.95. In general, the weight adjustment for WTC 1 indicates that representative results should reflect that a single interview with a respondent high in the building is representative of more occupants than a single interview with a person lower in the building.

Table O–3. Response rate analysis for WTC 1.

Floor Stratum	Number of Selections	Number of Interviews	Screen	Eligibility	Interview	Overall	Non-response Weight Adjustment
1 to 42	4,464	256	46.2 %	22.6 %	54.8 %	25.3 %	3.95
43 to 75	3,714	137	48.6 %	16.6 %	45.8 %	22.3 %	4.49
76 to 92	1,802	34	42.7 %	14.7 %	30.1 %	12.9 %	7.78
Floor missing	6	0	50.0 %	0.0 %	N/A	N/A	
Total	9,986	427	46.5 %	18.9 %	48.6 %	22.6 %	

While a similar analysis of telephone interview response rates for WTC 2 (shown below in Table O–4) does not indicate a significant need to weight the results, it is a conservative assumption to be consistent with WTC 1 analysis and the results will be weighted.

Table O–4. Response rate analysis for WTC 2.

Floor Stratum	Number of Selections	Number of Interviews	Screen	Eligibility	Interview	Overall	Non-response Weight Adjustment
1 to 42	4,339	143	44.8 %	14.8 %	49.7 %	22.3 %	4.49
43 to 75	3,187	134	45.0 %	17.7 %	52.8 %	23.8 %	4.21
76 to 110	2,203	94	48.3 %	19.5 %	45.2 %	21.8 %	4.58
Floor missing	208	5	50.5 %	9.5 %	50.0 %	25.2 %	3.96
Total	9,937	376	45.8 %	16.7 %	49.5 %	22.7 %	

All subsequent telephone interview data analysis will thus reflect weighting of the results in order to more accurately generalize the results. By convention, when a sample number is indicated (n =), the sample number will be the actual number of responses. Where percentages are indicated, however, the percentages were weighted to allow for generalization, unless otherwise indicated.

O.4.2 Initial Building Populations

The total building population is the sum of survivors and decedents. At the time of this report, the City of New York has officially determined 2,749 people to be killed at the WTC on September 11, 2001; no official breakdown of where people were killed presently exists. While an analysis of this issue by Dennis Cauchon,[3] a reporter for *USA Today*, in the months immediately following September 11, 2001, was remarkably complete, differences between his projections and the official numbers from the City of New York and other official sources exist. These differences are shown in Table O–5. For example, the number of first responders depends upon the definition of first responder. The City of New York published an occupational analysis of WTC decedents based upon a Census of Fatal Occupational Injuries

[3] Cauchon, Dennis. '*For many on September 11, survival was no accident.*' USA Today, December 20, 2001.

(U.S. Department of Labor, Bureau of Labor Statistics, in cooperation with the New York City Department of Health and Mental Hygiene and State and Federal agencies). Four hundred and thirty-three decedent's occupations were listed as firefighting, police, or security. This number exceeds by 30 the number of FDNY, NYPD, and PAPD reported killed. This may be attributable to private security forces present inside the towers on September 11, 2001, and/or first responders not employed by New York City or the Port Authority. NIST is attempting to resolve these differences in order to fully understand the initial building population.

Table O–5. Reports of WTC decedents.

Decedent	Official Numbers		USA Today[a]
WTC 1 occupants			1,434
At or above impact			1,360
Below impact			72
WTC 2 occupants			599
At or above impact			595
Below impact			4
First responders (total)	433 [b,c]		479
FDNY	403[d]	343[e]	
NYPD		23[f]	
PAPD		37[g]	
UA 175 and AA 11	157[d]		157
Uncertain location in towers			147
Bystanders			10
Total number of decedents	**2,749[b,h]**		**2,826**

a. Cauchon, Dennis. 'For many on Sept.11, survival was no accident.' USA Today, December 20, 2001.
b. Summary of Vital Statistics 2002: The City of New York. Bureau of Vital Statistics, New York City Department of Health and Mental Hygiene. December 2003.
c. Table WTC 8: Occupation of Decedents. All decedents classified as 'protective service' occupations, which includes firefighting, police, and guards.
d. World Trade Center Building Performance Study. FEMA 403. May 2002.
e. Increasing FDNY's Preparedness (McKinsey Report). Available at: http://www.ci.nyc.ny.us/html/fdny/html/mck_report/index.shtml
f. Available at http://www.ci.nyc.ny.us/html/nypd/html/memorial_01.html
g. Available at: http://www.panynj.gov/AboutthePortAuthority /PortAuthorityPolice/InMemorium/
h. Does not include 10 airplane hijackers for whom the City has not issued death certificates.

Using the known eligibility rates allows for a projection of the survivors of WTC 1 and WTC 2 present in the building at 8:46 a.m. on September 11, 2001. The analysis indicates that WTC 1 had approximately 7,470 ± 750 surviving occupants, while WTC 2 had approximately 7,940 ± 920 occupants. Thus, the total population of survivors from both towers was 15,410 ± 1,180. Table O–6 summarizes the projection of population of WTC 1 and 2 on September 11, 2001. Pending resolution of decedent locations, the total building population at the time of the first airplane impact was 17,440 ± 1,180, calculated using the building decedent locations reported by Cauchon.

Table O–6. Occupancy estimates on September 11, 2001, by tower.

	WTC 1	WTC 2	Total
Number in sampling frame	39,454[a]	47,608	87,062
Survivor occupancy rate	18.9 %	16.7 %	17.7 %
Estimated total population of survivors	7,470	7,940	15,410
Statistical Precision Calculations			
Sample n	427	376	803
Standard error (p)˜	1.90 %	1.92 %	1.36 %
Standard error (total)	750	920	1,180
Confidence limits at 5 %	±1,470	±1,790	±2,320
Number of Occupant/Decedents			
Decedents	1,434[b]	599[b]	2,033 - 2,192[c]
Total Building Population			
	8,900	8,540	17,440

a. Includes only occupants below floor 92.
b. Calculated from Cauchon as 1,434 + 599.
c. Calculated as 2,749 – 403 first responders – 157 airplane passengers.

O.4.3 Occupant Characteristics

The results of the background analysis of the average WTC occupant are identical to the precision presented whether the data was weighted or unweighted. Occupants of the WTC towers were twice as likely to be male as female (65 percent male [n=284]) for WTC 1 and 69 percent [n=250] for WTC 2). As shown in Table O–7 and Table O–8 below, the average age of the occupants was mid-forties, with a range of people from their early twenties to mid-seventies. The vast majority of respondents (92 percent (n=739)) spoke English as their primary language, although no attempt was made to account for the fact that some telephone contacts ended with a language barrier and no interviews were conducted in any language other than English.

Table O–7. Age for WTC 1 respondents.[a]

N	**Valid**	439
	Refuse	1
Mean		45
Median		46
Minimum		22
Maximum		73

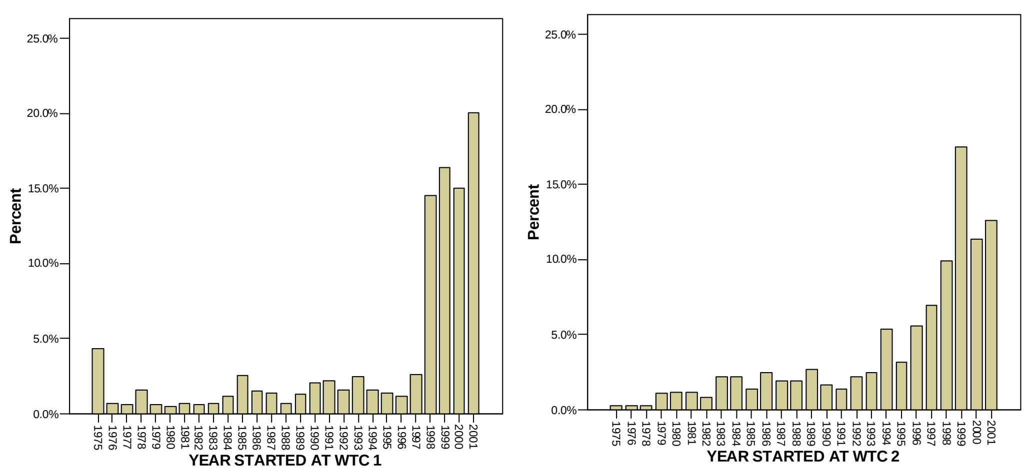
a. Mean and Median values are weighted. N, Min, and Max are unweighted.

Table O–8. Age for WTC 2 respondents.[a]

N	**Valid**	361
	Refuse	2
Mean		45
Median		44
Minimum		21
Maximum		74

a. Mean and Median values are weighted. N, Min, and Max are unweighted.

Tenant and employee turnover at the WTC was not uncommon. Figure O–1 shows the reported start dates for respondents in WTC 1 and WTC 2. In WTC 1, 4 percent (n=18) of the occupants had worked in the building since 1975. Further, 25 percent (n=110) had been working in the building prior to the 1993 bombing, although only 15 percent (n=64) of the WTC 1 respondents were present on February 26, 1993. For WTC 1, 67 percent (n=287) of the occupants had started working in the building in the last four years (1998–2001). The mean residence time in WTC 1 was over 5.6 years, while the median was 2 years.



Percentages are weighted.

Figure O–1. Employment start date at WTC.

Occupant tenure in WTC 2 demonstrated a similar trend. While only one respondent had worked in the building since 1975, 25 percent (n=91) of the respondents had been working in the building prior to the 1993 bombing (with 16 percent (n=59) present on the day of the bombing). Another 51 percent (n=185) started working in the building in the previous 4 years (1998–2001). The mean residence time in WTC 2 (n=360) was 5.9 years, while the median was 3 years.

Overall, 7 percent (n=56) had a formal responsibility or special knowledge about the building. These respondents were fire safety staff, floor wardens, searchers, building maintenance, or security staff. Approximately 13 percent (n=105) of the respondents were employed by the Port Authority, which may not imply a special knowledge of the building as some Port Authority employees had job duties related to functions outside the WTC.

Some 6 percent (n=52) reported having a limitation which impacted their ability to evacuate. These limitations included obesity, heart condition, needing assistance to walk, pregnancy, asthma, elderly, chronic condition, recent surgery or injury, and other.

O.4.4 Previous Experience

Whether an occupant had a previous evacuation experience may have affected the decisions an individual made during the September 11, 2001, evacuation. Further analysis will develop this hypothesis. Of the WTC 1 occupants present on September 11, 2001, 16 percent (n=64) were also present during the 1993 Bombing. In WTC 1, 60 percent (n=38) of evacuees in 1993 reported that they evacuated immediately, 30 percent (n=20) reported that they waited to evacuate, and 9 percent (n=6) did not recall. Most (95 percent [n=53]) who were able to recall their 1993 evacuation decision felt that they made the right decision, while 5 percent (n=3) did not believe they made the right decision.

Similarly, 16 percent (n=59) of WTC 2 evacuees on September 11, 2001, also evacuated in 1993. In WTC 2, however, only 75 percent (n=42) felt that they made the right decision in 1993, possibly due to the fact that many more waited to evacuate in 1993 in WTC 2 (69 percent (n=39)) than did so in WTC 1. Only 31 percent (n=17) who reported their decision evacuated immediately from WTC 2 in 1993, keeping in mind that the bomb had a more significant impact upon WTC 1 in 1993.

O.4.5 Preparedness and Training

Long a cornerstone of public policy on the emergency preparedness of office workers around the country, the Port Authority required tenants to conduct regular fire drills and appoint employee floor wardens and searchers. Overall, 66 percent (n=529) of WTC 1 and WTC 2 occupants reported participation in at least one fire drill in the 12 months immediately prior to September 11, 2001. Another 17 percent (n=139) reported that they did not participate in any fire drills in the 12 months prior to September 11, 2001, and 17 percent (n=135) did not know. Fire drill participation rates were similar between the two towers, as shown in Table O–9.

Table O–9. WTC fire drills in 12 months prior to September, 11, 2001.

Number of Drills	WTC 1[a]	WTC 2[a]
None	18 % (n=78)	17 % (n=61)
1	13 % (n=57)	8 % (n=29)
2	21 % (n=90)	24 % (n=88)
3	11 % (n=47)	15 % (n=53)
4	10 % (n=44)	9 % (n=32)
5 – 11	7 % (n=31)	9 % (n=32)
12 or more	3 % (n=13)	4 % (n=13)
Don’t know	18 % (n=80)	15 % (n=55)

a. Percentages are weighted, n values unweighted.

One of the goals of fire drill training is to make occupants aware of the location of the emergency exits. Of respondents who reported participation in a fire drill, 93 percent (n=490) were instructed about the location of the nearest stairwell. However, of the respondents who reported being shown a stairwell, 82 percent (n=432) did not enter or use the stairwell. Some 17 percent (n=92) reported that they did use the stairs during a drill, while approximately 1 percent (n=5) reported not knowing. Overall, more than half (51 percent (n=415)) of the occupants had never used a stairwell in WTC 1 or WTC 2 prior to September 11, while 48 percent (n=386) had used a stairwell. Two persons reported not knowing whether they had used the stairs previously.

Another goal of the fire drills was to introduce the floor warden system and evacuation procedures. Most occupants (82 percent (n=528)) with fire drill training were aware that there was a floor warden for their floor. Approximately 70 percent (n=557) of all occupants reported that they were aware of the evacuation procedures. When asked what those evacuation procedures comprised, however, answers varied significantly, including: wait in hallway for further instructions; do not use elevators, use stairs; meet at a designated site outside the building for a head count; or proceed down (varied number of) flights of stairs and wait. Further analysis of the understanding and implementation of the emergency procedures is under way.

O.5 SUMMARY

Eight hundred and three occupants of WTC 1 and WTC 2 were interviewed by telephone. Sample disposition analysis indicated differential nonresponse, particularly for WTC 1. In other words, the closer the occupant was to the impact area in WTC 1, the more likely it was that they would choose not to complete the telephone interview. Telephone interview percentages were then weighted to adjust for this effect.

On the morning of September 11, 2001, 17,440 people (± 1,180) were present at WTC 1 and WTC 2. This does not include first responders. The initial population of both towers was similar: 8,900 (± 750) in WTC 1 and 8,540 (± 920) in WTC 2.

The average age of an occupant of the WTC towers was mid-forties. Two-thirds of WTC 1 occupants had started working in the building during the previous 4 years (1998–2001), while half of WTC 2 occupants had begun working there during the same time period. Overall, 7 percent of occupants reported having special knowledge about the building, and 6 percent reported a preexisting limitation to their mobility.

Of those present on September 11, 2001, 16 percent were also present during the 1993 bombing. Two-thirds of occupants reported having participated in a fire drill in the 12 months immediately prior to September 11, while 17 percent reported that they received no training during that same period. Ninety-three percent of those participating in fire drills were instructed about the location of the nearest stairwell. Slightly over half of the occupants, however, had never used a stairwell at the WTC prior to September 11.

Significant additional analysis is presently under way. It is particularly important that results of questions related to the events, observations, and activities within the towers on September 11, 2001, be analyzed within the context of the findings coming from face-to-face interviews, focus groups, and other data collection activities.

Attachment 1
CATI ADVANCE LETTER TO OCCUPANTS

Dear [Name]:

You are being asked to voluntarily participate in the federal investigation of the collapse of World Trade Center structures on September 11, 2001. The National Institute of Standards and Technology (NIST) is investigating the cause of the collapse of the World Trade Center towers on September 11 in order to improve the way that building professionals, emergency responders, and regulatory authorities prepare for and respond to future emergency events.

Because you were an occupant of the WTC buildings, you have been identified as a person who can provide NIST with information critical to its investigation. Your cooperation with the investigation involves participating in a 20 minute telephone interview with a representative of our survey research contractor, Datasource. The purpose of the interview is to gather information about where you were in the WTC buildings at the time of the September 11 events, what you observed and experienced, and how you evacuated the building.

You may also be asked to participate in a voluntary face-to-face interview. Participating in the telephone survey does not obligate you to participate in the face-to-face interview.

NIST and its contractors NuStats and Datasource will keep the identity of all participating individuals as confidential as possible. To the extent permitted by law, no one other than NIST, authorized Federal officials, NIST contractors NuStats and Datasource, and Essex Institutional Review Board will have access to your identity. Access to identifying information will only be provided to staff members on an as-needed basis. Data will be reported in summary form.

NIST is a non-regulatory agency within the U.S. Department of Commerce and is conducting this investigation under the authority of the National Construction Safety Team Act (P.L. 107-231). The investigation involves strict fact-finding. No part of the NIST Investigation report can be used in any suit or action for damages. For more information, see http://wtc.nist.gov.

A representative of Datasource will phone you in the next week or two. Please be aware that he / she will want to conduct the interview at your convenience. If you agree to do the survey, you may choose not to answer any question. If you wish, you can choose to withdraw your responses at any time during the interview or at the end of the interview.

If you have any questions or comments regarding your participation in the NIST investigation, please feel free to contact Dr. Johanna Zmud, NuStats project director, at 800-447-8287, ext. 2225 or Jason Averill, NIST project director, at 301-975-2585. If you have any questions about your rights as a participant or if you have any concerns, you may contact the Essex Institutional Review Board, Inc. (IRB), 121 Main Street, Lebanon, NJ; Phone: 908-236-7735. The IRB is a committee that has reviewed this research investigational plan to help ensure that your rights and welfare are protected and that the investigation is carried out in an ethical manner.

Sincerely,

NIST OFFICIAL

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Attachment 2
ORAL CONSENT STATEMENT

THE TELEPHONE INTERVIEWER USES THE FOLLOWING SEQUENCE OF STATEMENTS AND QUESTIONS TO EFFECT ORAL CONSENT PRIOR TO BEGINNING THE INTERVIEW:

SUBJECT NAME: _______________________

Hi, may I please speak with <SUBJECT NAME>?

YES, CONTINUE	1	
NO	2	SET CALLBACK

Hi, my name is ______ and I am calling on behalf of the National Institute of Standards and Technology (NIST). NIST is conducting the federal investigation of the World Trade Center disaster. *Information about the investigation is available at the website "wtc.nist.gov" or we can provide you a toll-free number to call.*

We are interviewing people about their experiences on September 11. We sent you a letter about the study informing you of our call. Did you receive the letter?

YES	1	→ *ASK IF THERE ARE ANY QUESTIONS*
NO	2	→ *ASK IF THERE ARE ANY QUESTIONS*

SCREENER:

First, I need to ask you a few questions because we want to speak to people who had certain types of experiences that may be especially helpful to NIST. For this study, we are conducting interviews with people who were in WTC 1 or WTC 2 during the September 11, 2001 attacks.

(SCREEN) At the time of the attack, were you in WTC 1 or WTC 2 at the World Trade Center?

YES	01	
NO	02	[THANK AND TERMINATE]
RF	99	[THANK AND TERMINATE]

(SCREEN) Which tower were you in?

WTC1	01	
WTC2	02	
OTHER, specify	97	[THANK AND TERMINATE]
RF	99	[THANK AND TERMINATE]

(SCREEN) What floor were you on?

<Enter floor number>

BASEMENT	990
CONCOURSE/LOBBY	991
PLAZA	992
OTHER, SPECIFY	997
DK	998
RF	999

PROGRAMMER NOTE: NEED CODED FLOOR NUMBER CATEGORIES FOR SAMPLE TRACKING:

LOWER FLOORS (T1: BASEMENT – 42)	01
LOWER FLOORS (T2: BASEMENT – 42)	02
MIDDLE FLOORS (T1: 43 – 76)	03
MIDDLE FLOORS (T2: 43 – 76)	04
UPPER FLOORS (T1: 77 – 91)	05
UPPER FLOORS (T2: 77 – 110)	06

We would like you to participate in our study. Before we start, I'd like to read a statement to you about this study to help you decide if you wish to participate:

In this study, we want to ask about when and how you left the tower you were in during the attack on 9/11. The information you provide will help engineers and emergency planners to improve the safety and evacuation procedures for high rise buildings. The interview length is about 20 minutes and your participation is voluntary. Because this interview involves recalling a traumatic event, you may experience emotional discomfort. You are free to skip over any question you do not wish to answer. You may take a short break or stop the questions at any time. We can also provide you counseling referrals if you like. *Your identity will* be kept as confidential as *possible. To* the extent permitted by *law, no* one other than NIST, authorized Federal officials, NIST contractors *NuStats and Datasource, and Essex Institutional Review Board* will have access to your identity. *There are no direct benefits to participants. If you have any questions or comments regarding your participation in the NIST investigation, you may contact Dr. Johanna Zmud, NuStats project director, at 800-447-8287, extension 2225. If you have any questions or concerns about your rights as a participant, you may contact the Essex Institutional Review Board at 908-236-7735.*

3a. Are you willing to participate?

YES, NOW	1	
YES, LATER	2	[SET CALLBACK APPOINTMENT]
NO	99	THANK AND TERMINATE

Attachment 3
TELEPHONE INTERVIEW INSTRUMENT

I would like to start by getting some background information. What year did you first start working at the World Trade Center? RANGE: 1975 – 2001

$E 1975 2001

DK	9998
RF	9999

«YRWRK»

On September 11, 2001, were you in any of the following positions with the World Trade Center?

PORT AUTHORITY STAFF	1	
FIRE SAFETY STAFF	2	
FLOOR WARDEN OR SEARCHER	3	
MAINTENANCE OR SECURITY STAFF	4	
NONE OF THESE	0	X
DK	8	
RF	9	

«ROLES_01»
«ROLES_02»
«ROLES_03»
«ROLES_04»

During the year from September 11, 2000 to September 11, 2001, how many fire drills did you take part in at the World Trade Center?
$E 0 99

NONE	00	=> SWLOC
DK	98	=> SWLOC
RF	99	=> SWLOC

«FIRED»

During these drills, were you ever instructed about the location of the emergency stairwell nearest to your office?

YES	1	
NO	2	=> SWLOC
DK	8	=> SWLOC
RF		=> SWLOC

«DEXIT»

How many emergency stairwells were you shown?

ONE	1
TWO	2

THREE		=> LVFSW
OTHER, SPECIFY	7	O
DK	8	
RF	9	

«HMEXT»
«O_HMEXT»

Before September 11, had you learned in other ways about the locations of the three emergency stairwells?

YES	1
NO	2
DK	8
RF	9

«SWLOC»

SKIP IF NO FIRE DRILLS

=> USESW

Else => +1

if FIRED=00,98-99

«SOUT1»

During any of the fire drills, did you leave your floor using one of the stairwells?

YES	1	
NO	2	=> USESW
DK	8	=> USESW
RF	9	=> USESW

«LVFSW»

Which stairwells did you use?

STAIRWELL A	1	
STAIRWELL B	2	
STAIRWELL C	3	
OTHER, SPECIFY	7	O
DK	8	
RF	9	

«WHSW1_01»
«WHSW1_02»
«WHSW1_03»
«WHSW1_04»
«O_WHSW1»

Which side of the building was the stairwell located on?
=> +1

if NOT WHSW1=8

NORTH	1	
SOUTH	2	
EAST	3	
WEST 4	4	
OTHER, SPECIFY	7	O
DK	8	
RF	9	

«WHSL1»
«O_WHSL1»

Had you ever used any of the emergency stairwells prior to September 11?

=> DHELP
if LVFSW=1

YES	1	
NO	2	=> DHELP
DK	8	=> DHELP
RF	9	=> DHELP

«USESW»

SKIP FOR NO DRILLS AND NO USE OF STAIRWELLS

=> AEVOF
Else => +1

if FIRED=00,98,99 AND USESW>1

«SOUT2»

Which stairwell did you use?

STAIRWELL A	1	
STAIRWELL B	2	
STAIRWELL C	3	
OTHER, SPECIFY	7	O
DK	8	
RF	9	

«WHSW2_01»
«WHSW2_02»
«WHSW2_03»
«WHSW2_04»
«O_WHSW2»

SKIP IF NO FIRE DRILLS

=> AEVOF
Else => +1

if FIRED=00,98-99

«SOUT3»

When you were evacuating on September 11, how helpful was your experience during these drills?

=> +1
if FIRED=00

VERY HELPFUL	1
SOMEWHAT HELPFUL	2
SOMEWHAT UNHELPFUL	3
VERY UNHELPFUL	4
DK	8
RF	9

«DHELP»

Prior to September 11, were you aware of the evacuation procedures for your floor?

YES	1	
NO	2	=> FLWAR
DK	8	=> FLWAR
RF	9	=> FLWAR

«AEVOF»

Prior to September 11, what was the evacuation procedure you were told to follow?

LEAVE BUILDING IMMEDIATELY	1	
GO TO ELEVATOR LOBBY	2	
GO TO FLOORS UP OR DOWN	3	
GO TO ROOF	4	
STAY WHERE YOU ARE	5	
OTHER, SPECIFY	7	O
DK	8	
RF	9	

«EVACP»
«O_EVACP»

Did you know that there was a Floor Warden for your floor?

=> +1
if ROLES=1-4

YES	1
NO	2
DK	8

RF	9	

«FLWAR»

The next questions ask about 3 different time periods. The first series of questions asks about when you first became aware that something had happened at the World Trade Center. This is a period of just a few seconds. The next series of questions asks about the time from when you first became aware that something had happened, to the time you first entered a stairwell or elevator to exit the building. The third series of questions asks about what happened during your evacuation, meaning the time from when you first entered a stairwell or elevator until you exited the tower. At the end of the interview, I will ask you if there is anything else about your experience on September 11 that you would like to contribute.

CONTINUE	1	D

«IFAWA»

Now thinking back to the morning of September 11, how did you first become aware that something had happened at the World Trade Center?

$E 1 9

HEARD SOMETHING (BOOM, CRASH, EXPLOSION, BLAST, ROAR, RUMBLING, ALARM)	01	
SAW SMOKE OR FLAMES	02	
SAW DEAD BODIES	03	
SAW A PLANE	04	
SAW DEBRIS	05	
FELT SOMETHING (BUILDING MOVING, IMPACT, SHAKING, SWAYING, ROCKING, JOLT, EARTHQUAKE)	06	
FELL DOWN/FELL OFF CHAIR	07	
WARNED BY SOMEONE AROUND ME	08	
CONTACTED VIA PHONE	09	
CONTACTED VIA EMAIL	10	
PUBLIC ADDRESS SYSTEM	11	
NEWS MEDIA (TELEVISION, RADIO)	12	
OFFICE FURNITURE OR FIXTURES FALLING	13	
FURNITURE OR OTHER ITEMS FALLING OVER/DOWN	14	
OTHER, SPECIFY	97	O
DK	98	
RF	99	

«FAWAR»
«O_FAWAR»

What were you doing when you first became aware that something had happened to the World Trade Center? PROBE: Anything else?

$E 1 9

WORKING INDEPENDENTLY	01
IN MEETING	02

ON PHONE	03	
CHECKING/WRITING EMAIL	04	
WAITING FOR ELEVATOR	05	
RIDING IN ELEVATOR	06	
CHATTING WITH COWORKERS	07	
EATING/HAVING COFFEE	08	
ENTERING BUILDING	09	
OTHER, SPECIFY	97	O
DK	98	X
RF	99	X

«ACTV1_01»
«ACTV1_02»
«ACTV1_03»
«ACTV1_04»
«ACTV1_05»
«ACTV1_06»
«ACTV1_07»
«ACTV1_08»
«ACTV1_09»
«ACTV1_10»
«O_ACTV1»

At the moment when you first became aware that something had happened at the World Trade Center, did you notice any of the following? FOLLOW UP: Was that in your immediate area or outside the Tower?

	Did Not Notice	Noticed in Immediate Area	Noticed Outside the Tower
Smoke			
Fire or Flames			
Fireballs			
Collapsed walls			
Jet Fuel			
Severely or fatally injured people			
Sprinklers going on			
A fire alarm sounding			
Power outage or flickering lights			
Fallen ceiling tiles			
Extreme heat			

«NOT01_01»
«NOT01_02»
TIME PERIOD: 1
Were there any disaster related events going on around you at this time?
=> WHTW2
if OR[NOT01-NOT11]=2-3

YES	1	
NO	2	=> WHTW2
DK	8	=> WHTW2
RF	9	=> WHTW2

«OEVEN»

TIME PERIOD: 1
What was going on?

ENTER RESPONSE	1	O
DK	8	
RF	9	

«GOING»
«O_GOING»
TIME PERIOD: 1

Were you still in<WHTOW>at this time? IF YES, SELECT APPROPRIATE CHOICE IF NO, ASK WHICH TOWER THEY WERE IN

WTC 1	1
WTC 2	2
DK	8
RF	9

«WHTW2»

TIME PERIOD: 1

And were you still on the<WHFLO>floor at this time? RANGE: 1st - 110th FLOOR IF YES, SELECT/ENTER FLOOR IF NO, ASK WHICH FLOOR THEY WERE ON AND SELECT/ENTER IT

$E 1 110

BASEMENT	990	
CONCOURSE/LOBBY	991	
PLAZA	992	
IN ELEVATOR	993	
OTHER, SPECIFY	997	O
DK	998	
RF	999	

«WHFL2»
«O_WHFL2»

TIME PERIOD: 1

At the moment when you first became aware that something had happened to the World Trade Center, approximately how many people were with you? RANGE: 0 - 999 PEOPLE WE WANT THE NUMBER OF PEOPLE THAT WERE IN THE SAME LOCATION AS THE RESPONDENT. (IN THEIR LINE OF SIGHT)

$E 0 999

NONE	00	=> YOUIN
DK	98	=> YOUIN
RF	99	=> YOUIN

«PEOP1»

TIME PERIOD: 1
Were any of these people injured at that time as a result of the event?

YES	1	
NO	2	
DK	8	
RF	9	

«PEOIN»

TIME PERIOD: 1

Were you injured at that time, as a result of the event?

YES	1	
NO	2	=> ORISK
DK	8	=> ORISK
RF	9	=> ORISK

«YOUIN»

TIME PERIOD: 1

Would you say your injury was a …

AN INJURY THAT DID NOT IMPACT YOUR ABILITY TO EVACUATE,	1	
AN INJURY THAT DID IMPACT YOUR ABILITY TO EVACUATE BUT WAS NOT LIFE THREATENING, OR	2	
A LIFE THREATENING INJURY	3	
OTHER, SPECIFY	7	O
DK	8	
RF	9	

«NATIN»
«O_NATIN»

TIME PERIOD: 1

Still thinking about the moment when you first became aware that something had happened at the World Trade Center, did you believe that other people were in danger of being killed?

YES	1
NO	2
DK	8
RF	9

«ORISK»

TIME PERIOD: 1

Did you believe you were in danger of being killed?

YES	1

NO	2	
DK	8	
RF	9	

«YRISK»

TIME PERIOD: 2

Now please think about the time period between when you first became aware that something had happened and when you first entered a stairwell or elevator to leave the tower. During this entire time period, were you given any additional information about what was going on? AFTER BECOMING AWARE OF THE EVENT, BUT BEFORE EVACUATION.

YES	1	
NO	2	=> SEEKI
DK	8	=> SEEKI
RF	9	=> SEEKI

«GETIN»

TIME PERIOD: 2

Who gave you this information? PROBE: Anyone else?

MANAGER/SUPERVISOR	1	
COWORKER INSIDE BUILDING	2	
FAMILY/FRIEND OUTSIDE BUILDING	3	
POLICE/FIREFIGHTER	4	
FLOOR WARDEN	5	
MEDIA PERSON (TV/RADIO)	6	
OTHER, SPECIFY	7	O
DK	8	X
RF	9	X

«WHINF_01»
«WHINF_02»
«WHINF_03»
«WHINF_04»
«WHINF_05»
«WHINF_06»
«WHINF_07»
«O_WHINF»

TIME PERIOD: 2

What information did you get? PROBE: Any other information?

INFORMATION ABOUT WHAT HAD HAPPENED	1	
INSTRUCTIONS TO LEAVE	2	
INSTRUCTIONS TO STAY	3	
OTHER, SPECIFY	7	O
DK	8	X

RF	9	X	

«WHATI_01»
«WHATI_02»
«WHATI_03»
«WHATI_04»
«O_WHATI»

TIME PERIOD: 2

How did you get this information?   PROBE: Any other way?

FACE TO FACE	1	
TELEPHONE	2	
EMAIL/BLACKBERRY	3	
PA ANNOUNCMENT	4	
TV/RADIO	5	
OTHER, SPECIFY	7	O
DK	8	X
RF	9	X

«HOWGT_01»
«HOWGT_02»
«HOWGT_03»
«HOWGT_04»
«HOWGT_05»
«HOWGT_06»
«O_HOWGT»

TIME PERIOD: 2

And during this same time period, did you try to get additional information about what was going on?
AFTER BECOMING AWARE OF THE EVENT, BUT BEFORE EVACUATION

YES	1	
NO	2	=> ORIS2
TRIED, BUT WAS UNABLE TO GET INFORMATION	3	=> ORIS2
DK	8	=> ORIS2
RF	9	=> ORIS2

«SEEKI»

TIME PERIOD: 2

Who did you go to for this information? PROBE: Anyone else?

MANAGER/SUPERVISOR	1
COWORKER INSIDE BUILDING	2
FAMILY/FRIEND OUTSIDE BUILDING	3
POLICE/FIREFIGHTER	4
FLOOR WARDEN	5
MEDIA PERSON (TV/RADIO)	6

OTHER, SPECIFY	7	O
DK	8	X
RF	9	X

«GOINF_01»
«GOINF_02»
«GOINF_03»
«GOINF_04»
«GOINF_05»
«GOINF_06»
«GOINF_07»
«O_GOINF»

TIME PERIOD: 2

What type of information did you try to find? PROBE: Anything else?

INFORMATION ABOUT WHAT HAD HAPPENED	1	
INSTRUCTIONS TO LEAVE	2	
INSTRUCTIONS TO STAY	3	
OTHER, SPECIFY	7	O
DK	8	X
RF	9	X

«WHAI2_01»
«WHAI2_02»
«WHAI2_03»
«WHAI2_04»
«O_WHAI2»

TIME PERIOD: 2

How did you get this information? PROBE: Any other way?

FACE TO FACE	1	
TELEPHONE	2	
EMAIL/BLACKBERRY	3	
PA ANNOUNCMENT	4	
TV/RADIO	5	
OTHER, SPECIFY	7	O
DK	8	X
RF	9	X

«HOWG2_01»
«HOWG2_02»
«HOWG2_03»
«HOWG2_04»
«HOWG2_05»
«HOWG2_06»
«O_HOWG2»

TIME PERIOD: 2

And during the time between when you first became aware that something had happened at the World Trade Center and when you first entered the stairwell or elevator to leave the tower, did you believe that other people were in danger of being killed? AFTER BECOMING AWARE OF THE EVENT, BUT BEFORE EVACUATION

=> YRIS2
if ORISK=1

YES	1
NO	2
DK	8
RF	9

«ORIS2»

TIME PERIOD: 2

During that time period, did you believe you were in danger of being killed?

=> PEODO
if YRISK=1

YES	1
NO	2
DK	8
RF	9

«YRIS2»

TIME PERIOD: 2

During this time period, what were the people around you doing? PROBE: Were they doing anything else? AFTER BECOMING AWARE OF THE EVENT, BUT BEFORE EVACUATION
$E 0 10

NOONE AROUND/WAS ALONE	00	X
TALKING TO OTHERS	01	
GATHERING PERSONAL/WORK ITEMS	02	
SEARCHING FOR OTHERS	03	
CALLING OTHERS	04	
FIGHTING FIRE/SMOKE	05	
LOCKING UP	06	
WORKING	07	
EVACUATING THE TOWER	08	
CRYING, RUNNING AROUND, IN SHOCK	09	
HELPING OTHERS	10	
OTHER, SPECIFY	97	O
DK	98	X
RF	99	X

«PEODO_01»

«PEODO_02»
«PEODO_03»
«PEODO_04»
«PEODO_05»
«PEODO_06»
«PEODO_07»
«PEODO_08»
«PEODO_09»
«PEODO_10»
«O_PEODO»

TIME PERIOD: 2

Did the people around you start evacuating before you did?

=> DOBEF
if PEODO=08

YES	1
NO	2
DK	8
RF	9

«EVACB»

TIME PERIOD: 2

Did you do any of the following before starting your evacuation?
$E 1 9

TALK TO ANOTHER PERSON FACE TO FACE	01	
GATHER PERSONAL ITEMS	02	
TELEPHONE OTHER PEOPLE	03	
CONTINUE WORKING	04	
SAVE OR TRANSFER COMPUTER FILES	05	
SEARCH FOR OTHERS	06	
FIGHT FIRE OR SMOKE	07	
MOVE TO ANOTHER FLOOR	08	
HELP OTHERS	09	
LOGGING OFF/SHUTTING DOWN COMPUTER	10	
NONE OF THESE	11	X

«DOBEF_01»
«DOBEF_02»
«DOBEF_03»
«DOBEF_04»
«DOBEF_05»
«DOBEF_06»
«DOBEF_07»
«DOBEF_08»

TIME PERIOD: 2

Did you do anything else during this time?

ENTER RESPONSE	1	O
NO OTHER ACTIVITIES	0	
DK	8	
RF	9	

«OACTI» «O_OACTI»

TIME PERIOD: 2

Before you began your evacuation, was there anything you wanted to do, but couldn't?

YES	1	
NO	2	=> SEE01
DK	8	=> SEE01
RF	9	=> SEE01

«WANTD»

TIME PERIOD: 2

What was that? PROBE: Anything else?

$E 1 7

GATHER WORK ITEMS	01	
GATHER PERSONAL BELONGINGS	02	
CALL FRIEND/FAMILY MEMBER	03	
FIND FRIEND/COWORKER	04	
HELP FRIEND/COWORKER	05	
LOCK UP	06	
EVACUATE IMMEDIATELY	07	
OTHER, SPECIFY	97	O
DK	98	X
RF	99	X

«WANAC_01»
«WANAC_02»
«WANAC_03»
«WANAC_04»
«WANAC_05»
«WANAC_06»
«WANAC_07»
«WANAC_08»
«O_WANAC»

TIME PERIOD: 2

Why couldn't you do that/those things?

$E 1 9

AFRAID	01	
LOCKED DOORS	02	
PHONE LINES DEAD	03	
INJURED	04	
EXIT BLOCKED	05	
TOO CROWDED	06	
TOLD TO STAY IN BUILDING	07	
TOLD TO LEAVE	08	
FATIGUE	09	
DISABLED	10	
SMOKE	11	
DAMAGE TO FLOOR	12	
WAS HELPING OTHERS	13	
OTHER, SPECIFY	97	O
DK	98	
RF	99	

«WHYNO_01»
«WHYNO_02»
«WHYNO_03»
«WHYNO_04»
«WHYNO_05»
«WHYNO_06»
«WHYNO_07»
«WHYNO_08»
«WHYNO_09»
«WHYNO_10»
«WHYNO_11»
«WHYNO_12»
«WHYNO_13»
«WHYNO_14»
«O_WHYNO»

Still thinking about the time between when you first became aware that something had happened at the World Trade Center and when you entered the stairwell or elevator to leave the tower, did you notice any of the following?
FOLLOW UP: Was that in your immediate area or outside the Tower?

	Did Not Notice	Noticed in Immediate Area	Noticed Outside the Tower
Smoke			
Fire or Flames			
Fireballs			
Collapsed walls			
Jet Fuel			
Severely or fatally injured people			
Sprinklers going on			
A fire alarm sounding			
Power outage or flickering lights			
Fallen ceiling tiles			
Extreme heat			

«SEE01_01»
«SEE01_02»

TIME PERIOD: 2

Were there any disaster related events going on around you at this time?

=> EVACF
if OR[SEE01-SEE11]=2-3

YES	1	
NO	2	=> HELPY
DK	8	=> HELPY
RF	9	=> HELPY

«ODISE»

TIME PERIOD: 2

What was going on?

ENTER RESPONSE	1	O
DK	8	
RF	9	

«GOIN2»
«O_GOIN2»

TIME PERIOD: 2

Were you still on the<WHFL2>floor at this time? RANGE: 1st - 110th FLOOR IF YES, SELECT/ENTER FLOOR IF NO, ASK WHICH FLOOR THEY WERE ON AND SELECT/ENTER IT

$E 1 110
=> +1
if (AND[SEE01-SEE11]=1) AND PEODO>0 AND PEODO<98

BASEMENT	990
CONCOURSE/LOBBY	991
PLAZA	992
ELEVATOR	993
OTHER, SPECIFY	997
DK	998
RF	999

«EVACF»
«O_EVACF»

TIME PERIOD: 2

Did anyone help you in any way before you started your evacuation?

YES	1	
NO	2	=> DECID
DK	8	=> DECID
RF	9	=> DECID

«HELPY»

TIME PERIOD: 2

Who helped you? PROBE: Anyone else? WE WANT THEIR ROLE NOT THE NAME OF THE PERSON

POLICE OFFICER/FIREFIGHTER	1	
COWORKER	2	
STRANGER	3	
FLOOR WARDEN	4	
MANAGER/SUPERVISOR	5	
OTHER, SPECIFY	7	O
DK	8	X
RF	9	X

«WHOHE_01»
«WHOHE_02»
«WHOHE_03»
«WHOHE_04»
«WHOHE_05»
«WHOHE_06»
«O_WHOHE»

TIME PERIOD: 2

What did they help you with? PROBE: Anything else?
$E 1 7

LOCATING OTHERS	01	
HELPING OTHERS	02	
FINDING EXITS	03	
TREATING YOUR INJURIES	04	
PROVIDED INFORMATION/INSTRUCTIONS	05	
GATHER BELONGINGS	06	
CALM DOWN/EMOTIONAL ASSISTANCE	07	
OTHER, SPECIFY	97	O
DK	98	X
RF	99	X

«WHATD_01»
«WHATD_02»
«WHATD_03»
«WHATD_04»
«WHATD_05»
«WHATD_06»

«WHATD_07»
«WHATD_08»
«O_WHATD»

TIME PERIOD: 2

What was the one thing that made you decide to evacuate?

WAS TOLD TO EVACUATE	1	
FRIENDS CO-WORKERS EVACUATED	2	
AFRAID/FELT IN DANGER	3	
FIRE ALARM WAS GOING OFF	4	
SAW SMOKE	5	
SAW FIRE	6	
OTHER, SPECIFY	7	O
DK	8	
RF	9	

«DECID»
«O_DECID»

How many minutes had passed before you started to evacuate? IF NEEDED: How much time passed between when you first became aware that something had happened to the World Trade Center and when you entered the stairwell or elevator to leave the tower.   THIS IS NOT TIME TO EVACUATE. PLEASE CLARIFY WITH RESPONDENT IF TIME APPEARS TOO LONG. RESPONDENT WAS IN<WHTW2> RANGE FOR WTC 1: 1 - 103 MINUTES RANGE FOR WTC 2: 1 - 75 MINUTES
$E 1 103

DK	998
RF	999

«TIMEP»

SKIP FOR TOWERS

=> EVAC2
Else => +1
if WHTW2=2

«SKIP1»

Did you begin your evacuation… WE ARE INTERESTED IN WHAT THEY KNOW NOW. THEY MAY NOT HAVE KNOWN WHEN THEY WERE EVACUATING, BUT NOW THEY CAN TELL US WHEN IT WAS.

BEFORE THE PLANE HIT WTC 2	1
AFTER THE PLANE HIT WTC 2, BUT BEFORE THE WTC 2 COLLAPSE	2
AFTER THE WTC 2 COLLAPSE	3
DK	8
RF	9

«EVAC1»

SELECT1
$S NS=2 CO=1 IN=EVAC1<=1 ;CO=2 IN=EVAC1<=2 ;

BEFORE THE PLANE HIT WTC 2	1
AFTER THE PLANE HIT WTC 2, BUT BEFORE THE WTC 2 COLLAPSE	2
AFTER THE WTC 2 COLLAPSE	3
DK	8
RF	9

«SEL1»

SELECT2

BEFORE THE PLANE HIT WTC 2	1
AFTER THE PLANE HIT WTC 2, BUT BEFORE THE WTC 2 COLLAPSE	2
AFTER THE WTC 2 COLLAPSE	3
DK	8
RF	9

«SEL2»

Did you begin your evacuation…
=> EVCSO
if EVAC1>0

BEFORE THE PLANE HIT WTC 2	1
AFTER THE PLANE HIT WTC 2	2
DK	8
RF	9

«EVAC2»
SELECT4
$S CO=1 IN=EVAC2<=1 ;

BEFORE THE PLANE HIT WTC 2	1
AFTER THE PLANE HIT WTC 2	2
DK	8
RF	9

«SEL3»

Was there anything that kept you from evacuating sooner?

YES, RECORD RESPONSE	1	O
NO	2	
DK	8	
RF	9	

«EVCSO»
«O_EVCSO»

TIME PERIOD: 3

When you began your evacuation, were you alone or with other people? PEOPLE THAT THEY KNOW, PEOPLE THAT THEY WERE TALKING WITH

ALONE	1
WITH OTHER PEOPLE	2
DK	8
RF	9

«ALONE»

TIME PERIOD: 3

Which stairwell did you use for your evacuation?

STAIRWELL A	1	
STAIRWELL B	2	
STAIRWELL C	3	
USED ELEVATOR	4	=> FOLA1
OTHER, SPECIFY	7	O
DK	8	X
RF	9	X

«STAIR_01» «STAIR_02»
«STAIR_03»
«STAIR_04»
«STAIR_05»
«O_STAIR»

TIME PERIOD: 3

Which side of the building was the stairwell located on?
=> /WHYST
if NOT STAIR=8,7

NORTH	1	
SOUTH	2	
EAST	3	
WEST	4	
OTHER, SPECIFY	7	O
DK	8	
RF	9	

«WHISI»
«O_WHISI»

TIME PERIOD: 3

Why did you choose that/those stairwell(s) for your evacuation? PROBE: Any other reason?

CLOSEST ONE	1	
FOLLOWED OTHER PEOPLE TO IT	2	
OTHER EXITS WERE BLOCKED	3	
SAME AS I USED IN PREVIOUS EMERGENCY	4	
I WAS TOLD TO USE THIS STAIRWELL	5	
OTHER, SPECIFY	7	O
DK	8	X
RF	9	X

«WHYST_01»
«WHYST_02»
«WHYST_03»
«WHYST_04»
«WHYST_05»
«WHYST_06»
«O_WHYST»

TIME PERIOD: 3

At any time during your evacuation, did you leave that/those stairwell(s)? DO NOT INCLUDE PEOPLE WHO FOLLOWED THE PASSAGE WHERE THE STAIRWELLS START AND END.

YES	1	
NO	2	=> FOLA1
DK	8	=> FOLA1
RF	9	=> FOLA1

«LEVST»

TIME PERIOD: 3
Which floor were you on when you left the stairwell? IF RESPONDENT UNSURE, SELECT 997 AND RECORD RANGE OF FLOORS EXAMPLE: 34-40

$R 1 110

UNSURE, RECORD RESPONSE	997	O

«FLLST»
«O_FLLST»

TIME PERIOD: 3

Why did you leave the stairwell? PROBE: Any other reason?
$E 1 9

I GOT LOST	01
WAS TOLD TO LEAVE STAIRWELL	02
TO HELP SOMEONE	03
TO GO BACK AND GET SOMETHING	04
TOO CROWDED	05

SMOKE IN STAIRWELL	06	
PATH OBSTRUCTED	07	
A LOCKED DOOR	08	
STAIRWELL LED TO A FLOOR	09	
OTHER, SPECIFY	97	O
DK	98	
RF	99	

«WHYLS_01»
«WHYLS_02»
«WHYLS_03»
«WHYLS_04»
«WHYLS_05»
«WHYLS_06»
«WHYLS_07»
«WHYLS_08»
«WHYLS_09»
«WHYLS_10»
«O_WHYLS»

Screen [Template 3] -> FLOA5
=> +1
if FLWAR>1

Did any of the following help you evacuate while you were in the building?				
	Yes	No	DK	RF
Instructions or assistance from your floor warden				
Instructions or assistance from Police or Firefighters				
Support and encouragement from others				
Exit signs				
Photo luminescent paint in stairwells				

«FOLA1»

Screen [Template 3] -> EVCM7

=> +1

if NOT STAIR<4

Did any of the following make your evacuation more difficult while you were in the building?				
	Yes	No	DK	RF
Crowded stairwells				
Firefighters or Police moving up stairwell				
Disabled or injured people being taken down stairwell				
Locked doors				
Poor lighting				
Confusing or missing signs				
Lack of clear instructions				

«EVCM1»

Screen [Template 3] -> EXP11

Please tell me if you noticed any of the following at any time during your evacuation. FOLLOW UP: Was that in your immediate area or outside the Tower?			
	Did Not Notice	Noticed in Immediate Area	Noticed Outside the Tower
Smoke			
Fire or Flames			
Fireballs			
Collapsed walls			
Jet Fuel			
Severely or fatally injured people			
Sprinklers going on			
A fire alarm sounding			
Power outage or flickering lights			
Fallen ceiling tiles			
Extreme heat			

«EXP01_01»
«EXP01_02»

TIME PERIOD: 3

During your evacuation, did you turn back at any time? "TURN BACK" MEANS "GO BACK UP".

YES	1	
NO	2	=> EXITS
DK	8	=> EXITS
RF	9	=> EXITS

«TURNB»

TIME PERIOD: 3

Why did you turn back? PROBE: Any other reason?
$E 1 7

I GOT LOST	01	
I WAS TOLD TO TURN BACK	02	
TO HELP SOMEONE	03	
TO GET SOMETHING	04	
IT WAS TOO CROWDED	05	
SMOKE IN THE STAIRWELL	06	
MY PATH WAS OBSTRUCTED	07	
OTHER, SPECIFY	97	O
DK	98	X
RF	99	X

«WHYTB_01»
«WHYTB_02»
«WHYTB_03»
«WHYTB_04»
«WHYTB_05»
«WHYTB_06»
«WHYTB_07»
«WHYTB_08»
«O_WHYTB»

TIME PERIOD: 3

Did you exit the stairwell or elevator to the mezzanine or to the concourse?

MEZZANINE	1	
CONCOURSE	2	
OTHER, SPECIFY	7	O
DK	8	
RF	9	

«EXITS»
«O_EXITS»

TIME PERIOD: 3

How much time passed between the moment you first began your evacuation to when you exited the Tower? PLEASE CLARIFY WITH RESPONDENT IF TIME APPEARS TOO LONG. RESPONDENT WAS IN<WHTW2> RANGE FOR WTC 1: 1 - 103 MINUTES RANGE FOR WTC 2: 1 - 75 MINUTES
$E 1 103

DK	998
RF	999

«TIMP2»

SKIP FOR TOWERS

=> +2
Else => +1
if WHTW2=2

«SKIP2»

TIME PERIOD: 3

Did you exit the tower…

ELIMINATE ->	2	
ACCORDING TO NOT SEL1-SEL	2	
BEFORE THE PLANE HIT WTC	2	=> GETOU
AFTER THE PLANE HIT WTC 2 BUT BEFORE THE WTC 2 COLLAPSE, OR	2	=> GETOU
AFTER THE WTC 2 COLLAPSE	3	=> GETOU
DK	8	=> GETOU
RF	9	=> GETOU

«EXIT1»

TIME PERIOD: 3

Did you exit the tower…

Eliminate ->	1
According to NOT SEL	3
Before the plane hit WTC 2, or	1
After the plane hit WTC 2	2
DK	8
RF	9

«EXIT2»

Please remember that this study is intended as a fact finding mission and not a fault finding mission. It is crucial that we determine why some people were successful in their evacuation while others were not. Was there anyone on your floor that was not successful in their evacuation?

YES	1	
NO	2	=> PHYSI
DK	8	=> PHYSI
RF	9	=> PHYSI

«GETOU»

Why didn't they make it out? PROBE: Any other reason?
$E 1 8

WAS INJURED	01
WAS DISABLED	02
REFUSED TO LEAVE	03
DID NOT THINK IT WAS SERIOUS	04

STAYED BACK TO HELP SOMEONE	05	
WAS TOLD TO STAY	06	
STRUCTURAL DAMAGE	07	
SMOKE OR FIRE	08	
OTHER, SPECIFY	97	O
DK	98	X
RF	99	X

«WHYNG_01»
«WHYNG_02»
«WHYNG_03»
«WHYNG_04»
«WHYNG_05»
«WHYNG_06»
«WHYNG_07»
«WHYNG_08»
«WHYNG_09»
«O_WHYNG»

On September 11, 2001, did you have any physical problems that made it more difficult for you to leave the tower? Please do not include injuries caused by the incident or evacuation.

YES	1	
NO	2	=> AGE
DK	8	=> AGE
RF	9	=> AGE

«PHYSI»

What type of physical problem? PROBE: Anything else?
$E 1 9

BLIND/PARTIALLY BLIND	01	
DEAF	02	
IN WHEELCHAIR	03	
NEED WALKING ASSISTANCE	04	
OBESITY	05	
HEART CONDITION	06	
PREGNANT	07	
ASTHMA	08	
ELDERLY	09	
OTHER, SPECIFY	97	O
DK	98	X
RF	99X	

«LIMIT_01»
«LIMIT_02»
«LIMIT_03»
«LIMIT_04»
«LIMIT_05»
«LIMIT_06»

«LIMIT_07»
«LIMIT_08»
«LIMIT_09»
«LIMIT_10»
«O_LIMIT»

What is your age? RANGE: 1 - 98 YEARS
$E 1 99

RF	99

«AGE»

READ ONLY IF YOU CAN'T TELL. What is your gender?

MALE	1
FEMALE	2
RF	9

«GEND»

What language do you speak best?

ENGLISH	1	
SPANISH	2	
OTHER, SPECIFY	7	O
DK	8	
RF	9	

«PLANG»
«O_PLANG»

Were you working in WTC 1 or WTC 2 during the 1993 bombing?
=> SAY11
if YRWRK>1993

YES	1	
NO	2	=> CONCR
DK	8	=> CONCR
RF	9	=> CONCR

«WBOMB»

During the 1993 bombing, did you evacuate immediately or wait to evacuate?

EVACUATE IMMEDIATELY	1	
WAIT TO EVACUATE	2	
DK	8	=> +2
RF	9	=> +2

«EVBOM»

At the time of the 1993 bombing, did you feel you that your decision to<EVBOM>was the right decision?

YES	1
NO	2
DK	8
RF	9

«DEC93»

After the 1993 bombing how concerned were you that terrorists would attack the World Trade Center? Were you...

EXTREMELY CONCERNED	1
VERY CONCERNED	2
MODERATELY CONCERNED	3
SLIGHTLY CONCERNED	4
NOT AT ALL CONCERNED	5
DK	8
RF	9

«CONCR»

Is there anything else you would like to say about your experience on September 11?

YES, RECORD RESPONSE	1	O
NO	2	
DK	8	
RF	9	

«SAY11»
«O_SAY11»

IMPACT FLOOR FLAG
=> *
if IF(((WHTW2=1 AND WHFL2>91 AND WHFL2<99) OR (WHTW2=2 AND WHFL2>77 AND WHFL2<111)),1,0)

IMPACT FLOOR FLAG	1

«FFLAG»

163: LFLAG
Single
min = 1 max = 1 l = 1
2003/09/18 15:21
LOCATION FLAG
=> *
if IF((WHFL2>990 AND WHFL2<994),1,0)

LOCATION FLAG	1

«LFLAG»

EVENT FLAG
=> *
if IF(((AND[NOT02-NOT06]=2-3) OR (AND[SEE02-SEE06]=2-3) OR (AND[EXP02-EXP06]=2-3)),1,0)
EVENT FLAG 1

«EFLAG»

DISABILITY FLAG
=> *
if IF((PHYSI=1),1,0)

DISABILITY FLAG 1

«DFLAG»

ROLE FLAG
=> *
if IF((ROLES=1-4),1,0)

ROLE FLAG 1

«RFLAG»

We may be interested in learning more about your experience on September 11. Would it be okay if we follow up with you sometime in the future to get more detailed information on your evacuation experience?

=> +1
if FFLAG+LFLAG+EFLAG+DFLAG+RFLAG==0

YES 1
NO 2

«FOLUP»

PRESS ENTER TO CONTINUE

Those are all the questions we have. The valuable information you provided will help designers and engineers improve building safety, and help emergency planners improve building evacuation procedures. Thank you so much for taking the time to talk with me, and have a good day/evening. Good-bye.

END OF SURVEY 1 D => /INT99

«THANK»

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Appendix P
INTERIM REPORT ON EMERGENCY COMMUNICATIONS

P.1 INTRODUCTION

On September 11, 2001, radio and telephone communications played a significant role in the operations of emergency responders at the World Trade Center (WTC). Radio and telephone communications were a primary means of communicating information to emergency responders concerning the incident. These forms of communication were also used by emergency responders to communicate with people trapped in the WTC buildings and people attempting to evacuate from the buildings. They were used to communicate between members of the same emergency responder departments for planning and operations at the incident, and they were used to communicate between different departments or responding organizations at the incident.

Each of the governmental departments that had emergency responders at the WTC—New York City Fire Department (FDNY), New York City Police Department (NYPD), and the Port Authority Police Department (PAPD)—depended on their ability to communicate to accomplish their mission and to obtain information related to operations safety at the incident. Personnel from each of the departments used radios, cellular phones, and wired or landline telephones for communications during the incident. In addition, the emergency responders relied on the most basic form of communication, direct face-to-face communications.

As a normal practice during a typical emergency response many radio and telephone communications are recorded by the respective departments that respond to an incident. These recordings are normally made by the departments to provide an accurate record of operations during an incident. These records are often used by departments during review of incident operations. They are also used for investigative purposes and are sometimes used as evidence in legal cases. During the attack on the WTC many of the emergency responder communications were recorded and preserved. This study is based on these recordings. In addition, information gathered by personnel from the National Institute of Standards and Technology (NIST) during first-person interviews with more than 100 emergency responders has contributed to the report.

P.2 REPORT OBJECTIVES

The objective of this study is to develop a better understanding of the role that emergency communications played during the attack on the WTC, and to quantify information related to communications effectiveness. Although there have been numerous reports of radio equipment failures during the emergency response at the WTC, the only radio system examined in this study is that of the FDNY WTC site high-rise repeater that was installed by the Port Authority. This report does not address issues related to the technical capabilities of any other radio equipment. The analysis of handie-talkies and other radio equipment is in progress and will be addressed in a following report.

Many factors are associated with the ability of emergency communications to be successful. The following objectives were set in this report:

- To document radio and telephone communications operations
- To document radio communications readability or understandability
- To quantifying radio communications traffic volume
- To understand the impact of traffic volume on communications readability and the transfer of information
- To identify communications associated with dispatch and arrival of responders
- To identify communications related to evacuation and emergency response operations
- To identity communications related to building conditions at the WTC and the impact of this information on the emergency response

P.3 RADIO AND TELEPHONE COMMUNICATIONS

Both the Port Authority of New York and New Jersey (PANYNJ) and the NYPD supplied copies of audio recordings from the emergency response operations at the WTC. The PANYNJ provided digital copies of the audio communications tapes recorded by them during the incident. These recordings included communications from emergency response personnel, maintenance personnel, PAPD personnel, and a recording of the FDNY's Channel 30 radio repeater that was located at the WTC. Channel 30 was a City-wide channel designated by FDNY for use in high-rise building operations. The Port Authority had installed this radio repeater system at the WTC for use by FDNY after the 1993 bombing.

The NYPD submitted their communications to NIST in the form of audio tapes that were copies of the original tapes recorded on September 11, 2001. These tapes included radio communications from NYPD internal department operations.

FDNY communications recordings were not available from the incident location that day because the primary Field Communications truck was in the shop for repairs and a backup Field Communications van was used in its place. The backup Field Communications van did not have the capability to record the on-scene incident command or tactical communications; also, the backup van was destroyed when the towers collapsed. Therefore, the best record of radio communications available to NIST on FDNY operations came from the FDNY/PAPD Channel 30 tape and first-person accounts provided by FDNY personnel during their interviews. The Channel 30 tape provides a limited amount of information on FDNY communications and operations at the incident, but it does provide insight into FDNY operations inside WTC 2.

Each audio communications file was received from the source with the starting and ending times marked on the media jacket or the surface of the media. A list of all communications recordings acquired from the various departments is found in Attachment 1 at the end of the report.

P.3.1 Telephone Communications Recordings

Because telephone communication (both landline and cellular phone) was a contributing part of the emergency communications process during the incident, NIST received copies of telephone emergency response communications from the PANYNJ. Identification information for these recordings is also listed in Attachment 1. The City of New York provided NIST with opportunities to review their telephone recordings for 9-1-1 Emergency Operators and FDNY fire dispatchers in their New York City offices. At this time, the telephone recordings have been reviewed and documented, and the analysis work is still in progress. A detailed analysis of emergency telephone communications will be covered in a following report.

P.3.2 First-Person Accounts of Telephone Communications

As mentioned earlier, more than 100 first-person interviews were conducted with emergency responders that reported to the WTC incident. The following information was drawn from these interviews:

- Before the attack occurred on the WTC both the landline and cellular systems appeared to be working normally.
- Only moments after the first aircraft impacted WTC 1, the landline and cellular telephone systems were stressed by increased caller volume that made it difficult to get messages through. This condition continued for many hours following the attack.
- Telephone calls from the WTC to the 9-1-1 emergency operators and statements from various individuals being interviewed shows that even though WTC 1 and WTC 2 were severely damaged by the aircraft impact and fires, many of the landline telephones in the buildings continued to work up until the collapse of WTC 2.
- After the collapse of WTC 2, a number of cellular phone systems were not functional in the area of lower Manhattan.
- After the collapse of WTC 2, there were still some landline telephones working within the city block areas adjacent to the WTC site.

P.4 COMMUNICATIONS FILES PROCESSING AND PRELIMINARY EVALUATION

P.4.1 Audio Data Files and Processing

An evaluation of methods for listening to the recorded communications files was carried out. Comparisons were made between the functionality of using tape recorders versus that of using digital computer-based software for listening to the various emergency response communications files. It became apparent that the computer based listening system had advantages over the use of tape recorders. Some of the advantages of the computer based system are the ease of operation, ability to use the computer monitor for visually observing the beginning and end of communications periods, and the ability to easily and accurately reverse through a recording to a selected location so that a selected section of a communication could be listened to multiple times. As a result, it was decided to conduct the audio communications study using the computer based audio software system. This decision had a direct impact on the type of data format and media that would be needed for conducting the audio communications study. Therefore, NIST requested that audio communications be provided in a digital format on CD-ROM disks.

The communications recordings provided by the PANYNJ were digital files that were copied onto CD-ROMs, and they were in a format that could be played by computers while using audio player computer software. The audio recordings on each of the NYPD cassettes had to be converted to a digital format, and each file was then recorded onto a CD-ROM disk. In addition, some of the recordings that were received were recorded at very low amplitude that made it difficult to hear the communications. NIST used professional-quality computer audio software to increase the low audio volume recordings to a usable audio level.

P.4.2 Audio Data Computer Software

Three different types of software were used while conducting communications analysis on the audio recordings. Each of these software packages incorporated a clock for timing the audio recording and important communications during the incident.

The first, Sound Forge 6.0, a product of Sonic Foundry, Inc., of Madison, Wisconsin, is a professional digital audio editor (Sonic 2002). It possesses tools that can assist with increasing audio quality and volume of digital audio recordings. It has a graphic output to the computer monitor that allows for rapid evaluation of large audio files. It is also capable of operating as a tool for spectrum analysis. As a spectrum analyzer, it can be used to analyze waveforms by frequency, and it helps to identify noise problems in communications data. In addition, the audio waveforms can be expanded on various scales for detailed analysis. This software was used throughout the study for analysis of the recordings that required audio adjustments to improve quality.

The other two software packages were used as general purpose audio players. They both possess the same basic capabilities and were applied in this audio analysis process based on user preference.

Windows Media Player, a digital media player software package, is a product of Microsoft, Inc. (Microsoft 2003). This media player can be downloaded from the Internet. The player allows for viewing of audio wave forms from the digital audio files that are being listened to. It allows for easily changing a computer's audio volume, and it may be used effectively for locating specific points on an audio recording. The software also allows for movement through an audio file in a reverse direction so that selections of an audio file can be listened to multiple times.

WinAmp3 is a media playback software package for Windows that can be downloaded from the Internet, and it is a product of Nullsoft, Inc. (Nullsoft 2002). This player allows for viewing of audio wave forms from the digital audio files that are being listened to. It allows for easily changing a computers audio volume and it may be used effectively for locating specific points on an audio recording. The software also allows for movement through an audio file in a reverse direction so that selections of an audio file can be listened to multiple times.

P.5 ANALYSIS OF AUDIO COMMUNICATIONS FILES

Analysis of the communications recordings was a multistep process that began with sorting and cataloging the files. The initial sort separated radio communications files from telephone communications files. The files were also cataloged as it related to emergency response operations: PANYNJ, PAPD, FDNY, and NYPD. The respective files were then checked for content and primary emergency response channel files were selected for analysis first. Primary emergency response channels were channels specifically used by PAPD, NYPD, and FDNY for conducting emergency response operations at the WTC. The secondary channels relate to maintenance channels and other emergency responder channels that were not directly associated with operations at the WTC.

Analysis was carried out using the computer based software media players described above. The professional quality digital audio software, Sound Forge 6.0, was used for listening to and enhancing audio files that were difficult to hear. The two other media player software packages, Windows Media Player and WinAmp3, were used to listen to the majority of audio recordings.

P.5.1 Communications Transcription

Two different processes were used to transcribe the emergency communications. Data for the primary emergency communications files were put into a spreadsheet format so that a detailed analysis of results could be made. The overall analysis work is continuing; however, some of the data put into the spreadsheets was used to assist in quantifying communications quality and the radio traffic volume as related to time, as will be discussed in Section P.7. The second and simplest form of communications transcription was the verbatim transcription of the communications into a word processor data file, which was used to record the secondary communications files.

The primary communications audio files were selected for complete transcription to generate information concerning the quality of communications. The files selected were the FDNY Channel 7/PAPD Channel 30 radio repeater, the PAPD police desk radio channel, and the NYPD Special Operations Division channel and Division 1. These files included the following:

- Time of the radio transmission (radio transmission time was taken from the media player clock and was adjusted for the start time supplied with the communications file.)
- Type of radio transmission (voice or tone only for primary emergency response communications channels)
- Readability signal quality (done only for the primary emergency response communications radio channels)
- Content of the communication

As the communications transcripts were being prepared, the names of individuals identified during the communications were deleted to protect the identity of individuals and to adhere to the confidentiality agreements with the various organizations that supplied the communications data files.

P.5.2 Transcription Methods

As mentioned earlier, the communications transcripts were generated using three different computer based media players. The media players were installed on computer systems that were stand alone and isolated from the internet. The process for preparing a communications transcript was the following:

- The communications data file was loaded onto the computer.
- The media player was opened and the data file was selected.
- The spreadsheet on the computer was opened and prepared for data input.
- The transcriber would queue the communications recording to the beginning and check to be sure that the media player clock time was zeroed.
- The data file starting time was put into the spreadsheet.
- The communications recording was started, and the output was written into the spreadsheet.
- To improve accuracy of the transcripts, a second transcriber checked sections of the transcript against the audio recording.

For audio passages that were difficult to understand on the first pass, multiple passes of the section were used to improve comprehension.

P.5.3 Assessment of Radio Communications Quality

The Readability, Signal Strength, and Tone system for rating the quality of radio communications is used widely throughout the field of radio communications and is described in *The ARRL Handbook for Radio Communications* (ARRL 2003). This system is broken into three distinctive groups that can be rated: Readability, Signal Strength, and Tone. The rating for tone is only used to identify the quality of radio communications for “Continuous Wave” transmissions, and it does not apply to this analysis as “Tone” does not relate to voice communications. For voice radio communications, only “Readability” and “Signal Strength” are used. Signal Strength” is usually read from a signal strength meter at the time of the actual radio communication and is not available on the audio recordings.

Thus, in this study “Readability” only was used for rating the primary emergency responder radio communications channel recordings. It is recognized that this form of analysis is subjective, and it relates to the ability of an individual to hear and understand the radio communications. In an attempt to minimize the influence of the subjective rating system, individuals with extensive experience using radio communications and project staff trained by the experienced personnel were used to conduct the analysis. In addition, communication periods from the various recorded data sets were reviewed by more than one person where radio communications readability was difficult. The rating table for communications readability is listed below (ARRL 2003):

Readability (the term "readable" means "understandable")**:**
1 – Unreadable2 – Barely readable, occasional words distinguishable
3 – Readable with considerable difficulty
4 – Readable with practically no difficulty
5 – Perfectly readable

P.5.4 Training of Transcribers

Four NIST personnel were used to transcribe the emergency responder communications files. This included the Project Leader and three other staff personnel. The transcription protocol listed above was planned and tested by the two senior NIST personnel, including the project leader. When the protocol was found to be acceptable, the two other NIST personnel were trained by the senior members of the group. After the basic transcription training was completed, each of the new transcribers was given a communications file to transcribe. This communications file had previously been transcribed by the two senior personnel. After the file was transcribed by the new transcriber, their results were compared to that transcribed by the senior personnel. When it was demonstrated that the new transcriber had a full understanding of the transcription process, they were then assigned communications files to transcribe.

P.6 RADIO COMMUNICATIONS CONCEPTS

Currently, for most emergency responder radio systems the only way to produce a totally clear communication that can be received and understood is for only one communications signal to be transmitted at a time on a given radio frequency. This means that only one person can transmit a radio message at a time without creating communications interference on that radio frequency. With these systems, if two or more radio transmissions are made on the same radio frequency at the same time, signal mixing may occur and the communications may not be understandable. This difficulty with radio communications is often referred to as doubling. Under these conditions, usually the radio with the highest transmitting power will override transmissions from the lower-power radios and only the highest-power radio signal will be heard. This is often the case where an emergency response radio system uses a higher-power base station for dispatch communications or where a repeater is used to amplify a radio system's signal output. Where multiple radio communications are received by a radio repeater, signal mixing is likely to occur and the communications will not be understandable (ARRL 2003).

Over the last several years radio communications technology has undergone some significant advancements, particularly with cellular phones. These new systems can increase the effective use of the radio frequency/time factors related to radio communications (ARRL 2003), and are now beginning to be applied to emergency responder communications equipment.

P.7 COMMUNICATIONS DATA ANALYSIS

This analysis of communications addresses five major factors: (1) radio traffic volume, (2) communications duty cycle, (3) readability of communications, (4) operation of the FDNY site high-rise repeater at the WTC, and (5) the development of a chronology of radio communications from the incident.

The first two factors, radio traffic volume and communications duty cycle, are directly related, and each has an impact on readability, the ability to understand and also deal with the information being communicated. Generally, as radio traffic rate increases, the operations duty cycle approaches overloaded conditions. With very high traffic volumes it becomes more difficult for personnel at central communications facilities and personnel in the field to respond to the volume of traffic. Human operators of communications equipment become overloaded with work because not only do the operators have to verbally communicate with personnel over the radio, but they must often transfer the information gained to other locations. The transfer of information may also be done verbally using other communications systems or it may be done by hand through keyboard inputs or by both methods. Analysis of the radio traffic for each of the departments shows periods where radio traffic rates during the surge conditions potentially resulted in situations where base station radio operators were unable to relay important information.

P.7.1 PAPD Radio Communications

All radio communications evaluated for this report experienced traffic volume surge conditions as a result of the attack. The traffic volume surge greatly exceeded the traffic volume experienced under normal operating conditions.

PAPD Channel 26/W is used to demonstrate typical radio communications and operations conditions that occurred with the PAPD before and during the incident. This radio channel is used by PAPD police officers, NYPD supervisors and FDNY Engine 10 and Ladder 10 for communications at the WTC site. Tables P–1 and P–2 compare the number of transmissions and their length of time before and after the first aircraft impacted WTC 1. The percent of radio transmissions versus time are also shown on Fig. P–1. This percent of radio transmission, as well as others discussed in this report, was calculated based on the sum of transmission time and no transmission time over a given period of time. These data show that there was a 13 percent rate of radio transmissions on the PAPD police desk channel just prior to the aircraft impact. After the first aircraft impact on WTC 1 the radio communications were occurring 87 percent of the time. This surge in communications significantly impacted the functional capability of the radio system. After approximately 10 minutes, communications dropped to a steady operating level of 48 percent capacity.

P.7.2 FDNY Radio Communications

The communications for this FDNY, City-wide, high-rise building, radio Channel 7 was recorded by PAPD on their Channel 30. The Port Authority installed this high-rise repeater at the WTC for FDNY following the 1993 bombing. This FDNY channel was used primarily by FDNY personnel during operations in WTC 2. Personnel using this channel were FDNY Chief Officers, Company Officers, Aides, and firefighters.

Table P–1. Comparison of radio transmissions before and after the first aircraft impact.

Department	Number of transmissions before first aircraft impact (20 min period)	Number of transmissions after first aircraft impact (20 min period)
PAPD	42	176
FDNY	39	134
NYPD Division 1	7[a]	225
NYPD Special Operations Division	No data	192

a. Data only available for 2 min prior to first aircraft impact

Table P–2. Comparison of average and maximum radio transmission times before and after first aircraft impact.

Department	Average time per transmission before first aircraft impact and maximum (s)	Average time per transmission after first aircraft impact and maximum (s)
PAPD	3.8 (maximum 21.8)	3.3 (maximum 19.7)
FDNY	3.8 (maximum 50.9)	3.1 (maximum 19.5)
NYPD Division 1	1.9 (maximum 5.9)	3.4 (maximum 12.6)
NYPD Special Operations Division	No data	5.7 (maximum 31.5)

Note: All minimum transmission times were typically less than 1 s and were often related to the keying of a microphone.

While looking at these data it is important to keep in mind that several FDNY personnel at the incident did not think that the WTC site, high-rise channel, radio repeater was working. This is based on radio communications tests that were conducted by two Chief Officers working inside WTC 1 when the first Command Post was being set up in that lobby. A record of this radio communications test was recorded on the PAPD Channel 30 tape. Following this radio test, a Chief Officer involved in the test chose to use different channels for command and tactical communications during the incident. However, as FDNY operations increased in WTC 2, it was determined by members of the FDNY that the high-rise channel was functioning and use of the channel developed.

Preliminary analysis by NIST indicates that the repeater was operating at the WTC; however, there also appears to be some type of malfunction with the communications equipment. This malfunction was detected by the FDNY officers during the initial communications test, but it was not identified. As a result, this radio frequency was not primarily being used by many emergency responders. Two hypotheses are currently being studied related to the malfunction: (1) damage to the repeater antenna system and (2) failure of the radio hand set located at the Fire Command Desk in the lobby of WTC 1. Two failure modes are being considered, (1) the radio handset was broken, and (2) the volume on the handset was turned down. The evaluation of the repeater and its operation is still under way, and final conclusions have not yet been drawn concerning the repeater's performance. Additional information will be covered in the WTC Investigation final report.

Traffic load for this FDNY channel is summarized in Tables P–1 and P–2 and in Fig. P–2. Figures P–1 and P–2 shows that there was a significant peak in radio traffic that approached an 80 percent level which then dropped to a near steady high level of operations several minutes following the aircraft impact. The communications traffic level following the aircraft impact was four times greater than the level prior to impact.

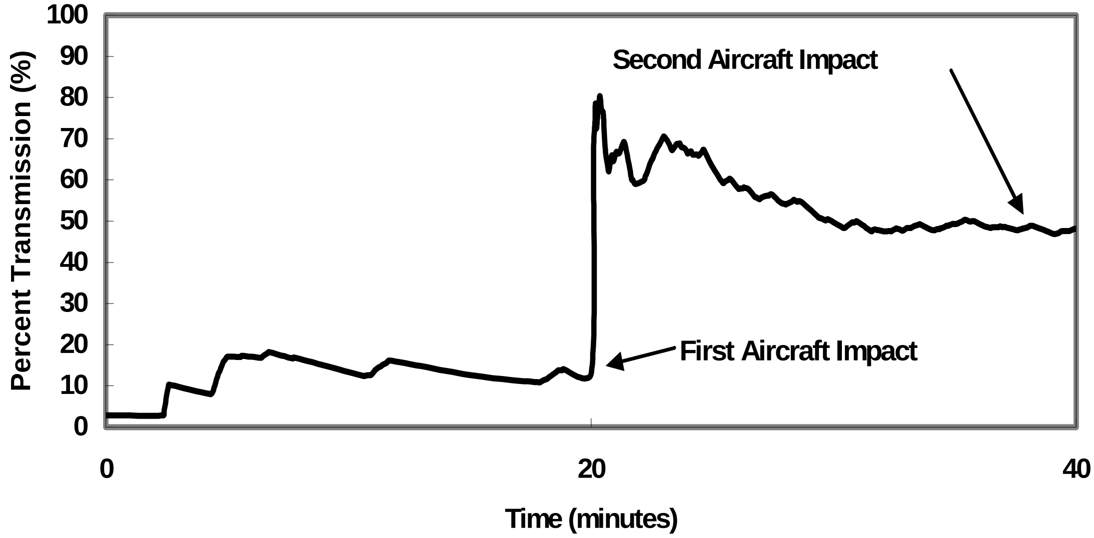


Figure P–1. PAPD police desk Channel 26/W plot of percent transmission versus time.

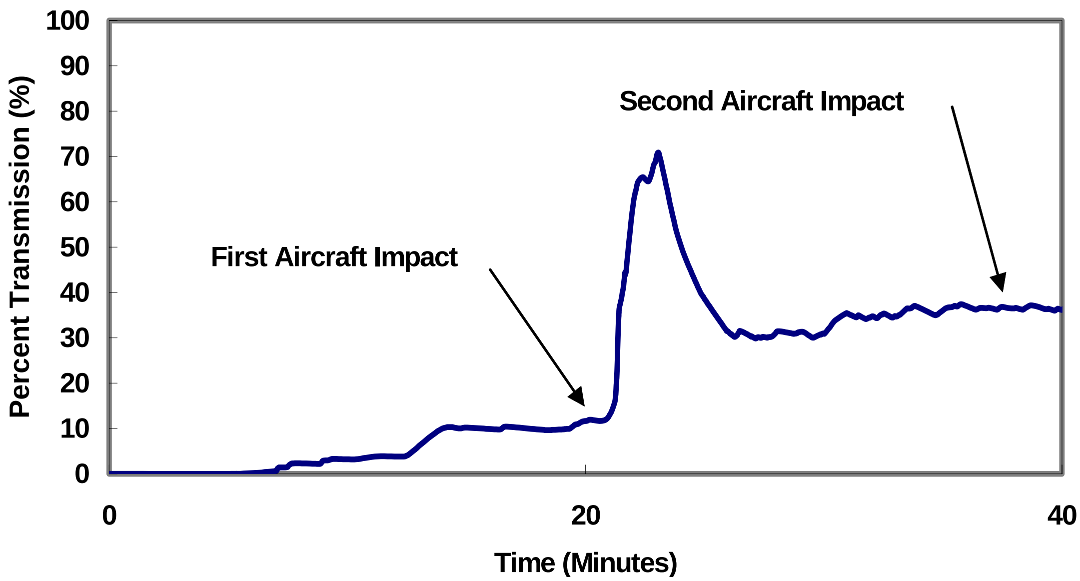


Figure P–2. FDNY City-wide high-rise Channel 7 (PAPD Channel 30) plot of percent transmission versus time.

P.7.3 NYPD Radio Communications

The third example illustrates radio communications for the NYPD Division 1 channel and the NYPD Special Operations Division channel. The Division 1 channel was used by police officers and supervisory police officers. The Special Operations Division channel was used by senior level NPYD management, supervisory police officers, Emergency Service Unit personnel, and aviation unit personnel.

The communications recordings provided by NYPD did not typically contain communications that preceded the attack. However, the Division 1 radio channel recording did start approximately 2 minutes before the first aircraft impacted WTC 1. This 2 minute period provides a limited sample of the level of radio communications prior to the attack. The volume of NYPD communications is shown in Tables P–1 and P–2. These data demonstrate that NYPD had a similar surge in radio traffic immediately following the attack. Figure P–3 shows that the level of transmissions before the attack was at approximately 15 percent. Following the attack the level of transmissions jumped to over 90 percent and then settled down to a level of 63 percent. Radio traffic on the Special Operations Division channel was even higher, as shown in Fig. P–4, peaking near the 95 percent level and staying in the 80 percent range over the remainder of the sample period.

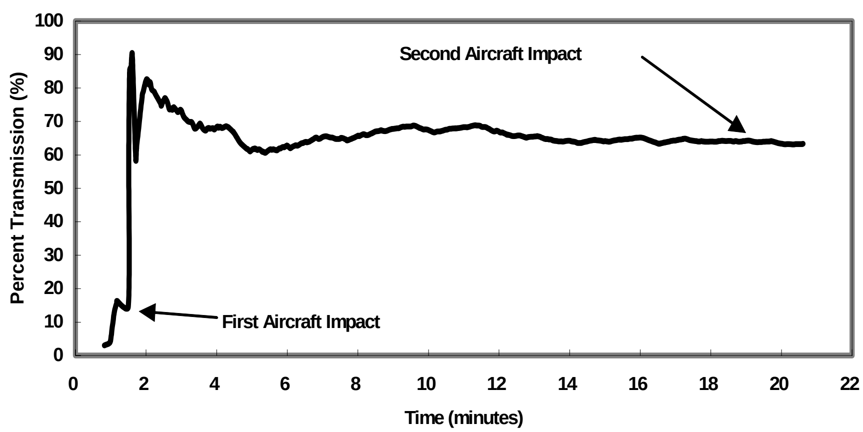


Figure P–3. NYPD Division 1 channel plot of percent transmission versus time.

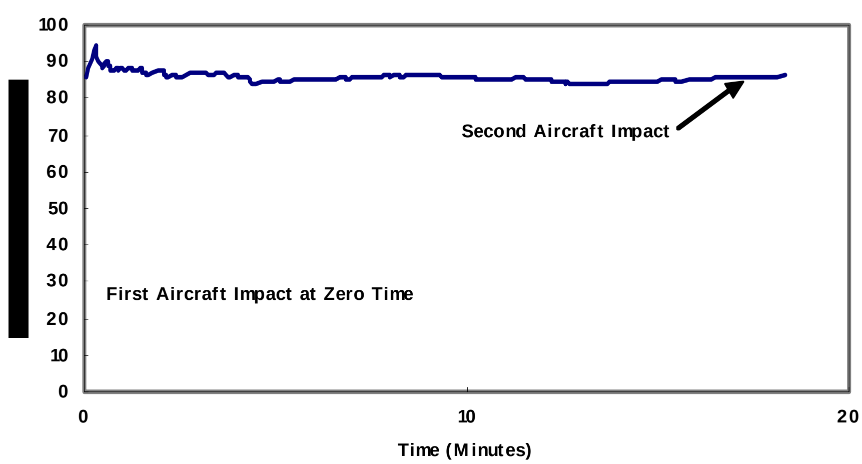
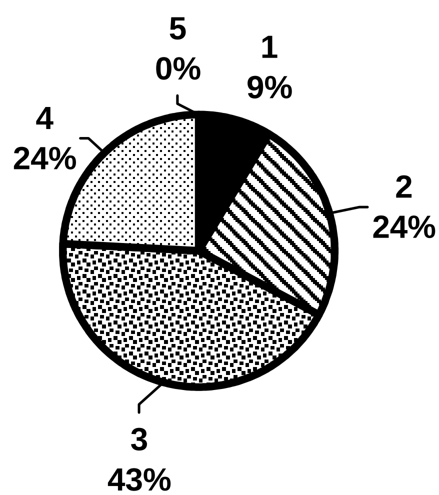


Figure P–4. NYPD Special Operations Division channel plot of percent transmission versus time.

P.7.4 Radio Communications Readability Analysis

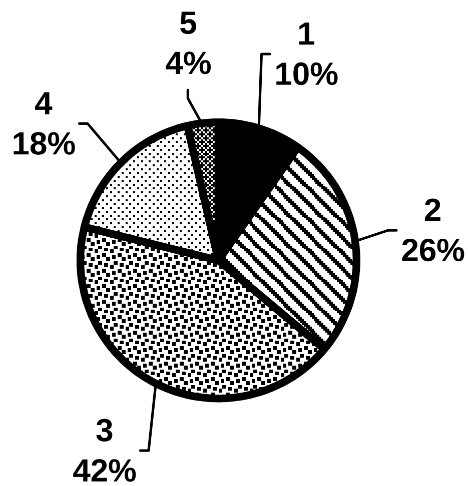
As each of the communications files was transcribed a readability value was assigned for each attempt to communicate. Results of this analysis are shown in Figs. P–5 through P–8. Analysis of these data showed that the ability to transmit a complete message was difficult during the communications surge. Data showed that approximately one-third to one-half of the radio communications for each of the three departments did not exceed a readability level of 2. These emergency communications were not complete and may have not been fully understood. The largest fraction of readability for all radio communications analyzed was recorded at a readability level of 3. This means that this fraction of communications was readable, but audio and radio transmission problems were being experienced. Some conditions that will cause poor communications quality are:

- Background noise either at the transmission point or receiving point or both,
- Operating health of transmitting and receiving radios and antenna systems,
- Doubling or crossing of radio signals caused by multiple transmissions at the same time on the same radio frequency, and
- Radio transmissions that may be affected by alternating materials or electromagnetic interference.



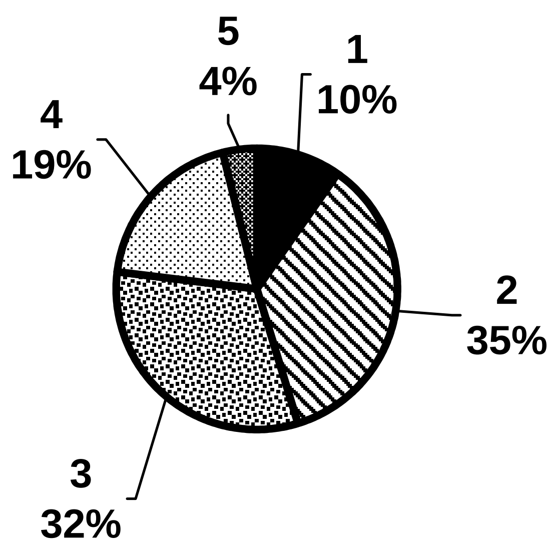
Note: Readability scale:
1 – Unreadable
2 – Barely readable, occasional words distinguishable
3 – Readable with considerable difficulty
4. – Readable with practically no difficulty
5. – Perfectly readable

Figure P–5. PAPD police desk Channel 26/W radio communications readability.



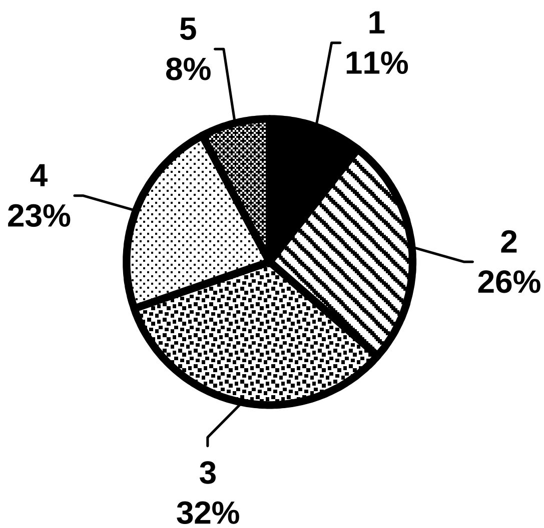
Note: Readability scale:
1 – Unreadable
2 – Barely readable, occasional words distinguishable
3 – Readable with considerable difficulty
4. – Readable with practically no difficulty
5. – Perfectly readable

Figure P–6. FDNY City-wide high-rise repeater Channel 7 at the WTC site (PAPD Channel 30), radio communications readability.



Note: Readability scale:
1 – Unreadable
2 – Barely readable, occasional words distinguishable
3 – Readable with considerable difficulty
4. – Readable with practically no difficulty
5. – Perfectly readable

Figure P–7. NYPD Special Operations Division channel radio communications readability.



Note: Readability scale:

1 – Unreadable
2 – Barely readable, occasional words distinguishable
3 – Readable with considerable difficulty
4 – Readable with practically no difficulty
5 – Perfectly readable

Figure P–8. NYPD Division 1 channel radio communications readability between FDNY personnel in the lobby of WTC 2 and FDNY personnel some 40 or more floors up inside the same building.

In addition, approximately 25 percent of the radio communications had readability levels 4 or above. Typically, the higher readability levels were produced by the various department base stations that operate at a higher radio transmission output power than the hand held radios. However, there is one exception: several of the radio communications on the FDNY City-wide high-rise radio channel, the PAPD recording of Channel 30. It appears that the repeater was operating at the WTC site. Several of the radio communications on this channel were assigned readability values of four and five as the FDNY began its operations in WTC 2. In addition, some of these 4 and 5 readability value radio communications were occurring between FDNY personnel in the lobby of WTC 2 and FDNY personnel some 40 or more floors up inside the same building.

P.7.5 Observations on Radio Communications

All of the radio systems analyzed appeared to work well during the period of normal operations before the attack on the WTC. It was noted that Channel W of the PAPD was experiencing some difficulty with a handie-talkie radio transmitting a carrier wave as a result of an open or keyed microphone, which disrupted communications on that channel. PAPD personnel recognized the problem and were busy trying to correct it just before the first plane struck WTC 1. The keyed microphone problem continued after the attack occurred. NYPD also had a problem with an open or keyed microphone after the incident began, occurring on the Special Operations Division channel. The problem was recognized and efforts were also made by the NYPD desk operator to get the problem cleared up. Initial attempts to correct the open microphone problem appeared to be successful.

Also, the data above for the various departments demonstrate the significant changes that occurred in radio communications traffic during the incident. It is evident that PAPD, FDNY, and NYPD all experienced similar surges in radio traffic volume following the first aircraft impact into WTC 1. It is also interesting to note that when the second aircraft struck WTC 2, 17 minutes later, there was no major surge in radio communications. This may be attributed to the fact that the initial emergency response assignments had already been made and that operations had already begun at the WTC, and an additional surge in radio communications was not needed. In addition, it is observed from the communications recordings and from first-person interviews that the emergency responders were trying to limit their use of the radios to reduce interference on their operating frequencies.

P.7.6 Preliminary Chronology of Emergency Communications

The following are lists of selected chronological communications messages that provide information concerning (1) dispatch and arrival of emergency response units, (2) evacuation, (3) emergency response operations, (4) emergency response communications, and (5) observations of building conditions.

Note: These chronologies are based on the best possible data provided to NIST for the analysis. The times are given to represent the exact event sequence. Based on the variations of recorded clock times for the data and times assigned for each recording provided by the departments, it is estimated that the error for time with these chronologies is likely to be on the order of ±2 minutes.

Dispatch and Arrival of Emergency Responders Chronology

This chronology clearly demonstrates that the emergency response to the World Trade Center was immediate. Within the first 3 minutes of the aircraft impact on WTC 1, PAPD was responding by providing information on the incident to the police desk, FDNY had dispatched 26 units to the incident, and NYPD had called of a department mobilization that included dispatching aviation units to the incident for visual assessment. In less than 10 minutes, PAPD had called a chemical mobilization; NYPD had dispatched five ESU teams and had two aviations units at the scene providing observations. In less than 30 minutes, 121 FDNY units had been dispatched to the scene and 30 units had signaled their arrival at the scene. This response combined with the activities undertaken demonstrates a high level of professionalism by the various departments.

8:46 a.m.	FDNY Chief makes report that an airplane has struck the upper floors of a WTC building and transmits a first and second alarm.
	PAPD officer reports to the police desk an explosion at the WTC.
8:48 a.m.	26 FDNY units dispatched. NYPD calls for a department mobilization.
8:49 a.m.	NYPD requests for aviation to get in the air and make a visual assessment.
8:50 a.m.	PAPD officer calls for a chemical mobilization.
8:52 a.m.	5 NYPD Emergency Service Units dispatched.
	NYPD aviation requests landing zone in the vicinity of the WTC.
	NYPD aviation unit arrives at the WTC and examines possibilities of roof rescue.
8:54 a.m.	NYPD aviation advises they have two units in the air to do aerial survey.
8:59 a.m.	FDNY Chief calls for all but one Rescue Squad to the WTC.
9 a.m.	66 FDNY units have been dispatched at this time.
9:03 a.m.	FDNY Marine unit reports that a second plane struck WTC 2.
9:15 a.m.	121 FDNY units dispatched and 30 FDNY units signal* their arrival.
9:29 a.m.	FDNY dispatcher relays that a department-wide recall has been instituted.

9:59 a.m. 171 FDNY units dispatched and 74 FDNY units signal their arrival.
10:29 a.m. 214 FDNY units dispatched and 103 FDNY units signal their arrival.
*Note: Arrival times are determined from 10–84 signals transmitted by units as they arrive at their assigned location. A 10-84 signal is sent by a firefighter from a fire department vehicle by pressing a button on the communications console.

Evacuation Chronology

The evacuation chronology exhibits a mix of responses to the incident. It provides insight into the successes and shortcomings of the evacuation from the WTC buildings and site. The first noteworthy event is that multiple orders were given by a senior PAPD police officer to evacuate the WTC buildings and the entire complex. There is no evidence that these orders were transmitted to appropriate personnel at the site to initiate the full evacuation of the complex. Data from these communications also show that the evacuation process was not always orderly and controlled. This is demonstrated by the fact that the first people jumped from WTC 1 at 8:52 a.m., only 6 minutes after the first aircraft struck WTC 1. In addition, it was reported that people were running from the PATH trains, and a report came in to the PAPD police desk from a police officer in WTC 5 stating that "I have people going crazy." However, it is a fact that most of the evacuation process from the WTC complex was orderly. This chronology also provides a view of the professionalism of the PA, PAPD, and building security personnel that held their posts in the face of life threatening conditions to assist people in the evacuation. In addition, these communications provide some basic information related to the status of people trapped in the buildings and the fact that the buildings elevators were not functioning or dangerous to use. Finally, several cases are listed where injured, elderly, or physically impaired people are not able to walk down the stairs in the building and need assistance to evacuate.

8:47 a.m. PAPD police desk receives a message to evacuate the building (WTC 1) and send people out towards WTC 5
PAPD police desk receives a message from a PAPD officer instructing employees to avoid the Concourse.
8:48 a.m. PAPD police desk receives two orders from a senior police officer calling for the evacuation of the building.
8:52 a.m. PAPD police desk receives report from police officer that people are jumping out of the windows from WTC 1.
8:53 a.m. PAPD police desk report indicates that people are running from the PATH trains.
8:55 a.m. PAPD police desk handles message calling for the evacuation of the Plaza
FDNY dispatcher relays information that people are trapped on floor 106 of WTC 1.
8:56 a.m. PATH trains are still bringing people into the WTC site.
PAPD police desk message attempts to assemble personnel at WTC 1 exits to the plaza to show people how to get out. One Port Authority person responds to the message that he cannot get over to the building exits because glass is falling all over the place.
PAPD police desk receives a radio message that they need assistance in WTC 4 because people are attempting to exit the building.
8:57 a.m. PAPD police desk: a message is sent stating, "Don't let anyone in the building evacuate to the Plaza at this time."
8:58 a.m. PAPD police desk instruction to security guards: hold your post and don't allow people into the Plaza or out onto the Courtyard.
PAPD police desk reports that people are trapped on floor 79 WTC 1.
8:59 a.m. PAPD police desk: a senior PAPD officer calls for the evacuation of WTC 1 and WTC 2.

PAPD police desk: a senior PAPD officer calls for the evacuation of the entire WTC complex, all buildings.

9 a.m. PAPD police desk: a police officer asks if building five should be evacuated and he was told to stand by.

PAPD police desk receives a report that there are people trapped inside suite 4711 of WTC 1 and can't get out.

PAPD police desk: orders were given to evacuate WTC 1, B4 level.

PAPD police desk: a Port Authority employee calls in that he is on floor 27 in the C staircase and has a man in a wheel chair and needs assistance.

9:01 a.m. PAPD police desk: a senior PAPD officer calls for the evacuation of all buildings in the WTC complex.

9:02 a.m. PAPD police desk: Port Authority person calls in reporting that he is stuck in an elevator on floor 78 of WTC 1 in car number 81A.

9:03 a.m. A second aircraft strikes WTC 2.

9:04 a.m. PAPD police desk: a call is made to evacuate everybody from the building now. Note: Building not identified.

9:05 a.m. PAPD police desk: a police officer indicates that WTC 4 is being evacuated. He is then going to WTC 5. He also reports that, "I have people going crazy."

PAPD police desk: a report comes in that Port Authority employees heard people stuck inside of some elevators and also report that they are getting them out. Note: Building and location not identified.

PAPD police desk: a call comes in to get everybody off the complex.

9:07 a.m. PAPD police desk: a report comes in that somebody is stuck in an elevator on floor 76. Note: Building not identified.

9:08 a.m. PAPD police desk report from an officer that debris is falling from WTC 2 by WTC 4 and Liberty Street, and to let the people out of WTC 4.

9:09 a.m. PAPD police desk report is received indicating that FDNY is entering elevator bank 11, 12. Note: Building not identified. Elevators 11 and 12 are shuttle elevators. Elevator 11 goes from the lobby to the 44th floor. Elevator 12 goes from the lobby to the 78th floor.

9:10 a.m. PAPD police desk receives a report that the express elevators could be in jeopardy of falling. Note: Building not identified.

9:12 a.m. PAPD police desk receives a radio report from the Command Desk in the lobby of WTC 2 that they cannot pick up the Warden phones and that they are making announcements telling people not to stay at the Warden phones. Note: This communication indicates that the Warden phones in WTC 2 were not working. Warden phones are located on each floor of the building for the use of floor wardens. They are wired for communications with the fire command desk in the building lobby.

9:14 a.m. PAPD police desk receives confirmation that no elevators are working. Note building not identified.

9:16 a.m. FDNY radio dispatcher advises a chief that there are people trapped in WTC 1 at the following locations: floor 82 east side; floor 83, room 8311; floor 103, room 103 near the corner, floor 104; and floor 106. WTC 2 at the following locations: floor 82 west side, floor 88, and floor 89.

9:17 a.m. PAPD police desk reports that four callers have made contact and need assistance on floor 106 of WTC 1.

9:20 a.m. PAPD police desk receives a message from an officer that no one is down on the B4 level of WTC 1.

9:23 a.m. FDNY radio dispatcher advises FDNY Field Communications Unit that 100 people are overcome in WTC 1 on the northwest and southwest corner of floor 103. The dispatcher also reports that Ladder 3 reports numerous injuries in the stairwell of floor 35 on up.

9:24 a.m. PAPD police desk receives a report from an officer that people from floor 64 are now coming down onto the courtyard level of WTC 1.

9:28 a.m. PAPD police desk receives a radio report of an injured person with burn injuries caused by a falling elevator. Note: Location of injured person was provided as A20. This may mean WTC 1 on floor 20 on the A stairway.

9:29 a.m. PAPD police desk reports that there is a medical emergency in the B stairway; there is a person that cannot walk down. The people are coming down from floor 51, and the person needing assistance has asthma. Note: Building not identified.

9:30 a.m. PAPD police desk receives a report that two elderly people on floor 51, B stairway, WTC 1, cannot walk down and need medical assistance.

9:37 a.m. PAPD police desk recorded the following message: “All World Trade Center units to the Command Post. All World Trade Center units escort everybody over the land bridge on West Street to the Financial Center. Do not, repeat, do not send people out into the Concourse on to south side.”

9:45 a.m. PAPD police desk receives a report that officers are sending people down, evacuating on the A stairway in WTC 1.

PAPD police desk copies a request for crowd control on Broadway. Answer to the request is that the City police should be responding.

9:56 a.m. PAPD police desk receives a report that WTC 1 is not completely evacuated and that people are still coming out of each stairway.

Emergency Response Operations Chronology

This section provides a view of the emergency response operations carried out by FDNY, NYPD, and PAPD at the WTC. The chronology highlights several communications that identify cases where emergency responders are assisting injured people, call for EMS assistance, and search for functioning elevators to help evacuate injured people. Some fires in the buildings are identified and some fire fighting operations in WTC 2 are identified. Communications from PAPD provide information on the locations of many people that were trapped in the buildings and in elevators. Several communications provide insight into FDNY operations in WTC 2 and show that some fire fighters actually reached the 78th floor in WTC 2. This assent to the 78th floor was assisted by the use of an elevator that operated up until just before the building collapsed. The elevator became stuck in the elevator shaft and the firefighter operating the elevator was chopping his way out when the building collapsed. Several radio communications provide insight into the difficulty that emergency responders had trying to climb the stairs of the WTC. Cases are noted where FDNY personnel had to stop and rest. Radio communications for the FDNY channel 7 repeater also point out the difficulty that some firefighters had with the identification of the two buildings. The exchange of communications by FDNY personnel at 9:29 a.m. clearly shows this difficulty. Several communications from NYPD aviation units show how the aircrews repeatedly accessed the possibility of landing on the roof of WTC 1 and reported that conditions were not safe for landing. However, at 9:38 and 9:40 a.m. an aviation unit calls in for permission to land on the roof of WTC 1. No evidence has been found that indicates that people were seen on the tower roof or that conditions had improved when these radio requests were made. Interviews with aviation personnel indicate that many of them were highly troubled by the number of occupants trapped in the buildings and the number of people jumping from the buildings, and they were distressed that they were unable to help them. At 9:43 a.m. the order came from a senior police department official that no one from the aviation units is to rappel on the building’s roofs. Communications in this section also provides information that many people were coming to the WTC to volunteer their assistance. This assistance was turned away as the emergency responders felt that they needed to get everybody away from the WTC complex.

8:46 a.m. An aircraft strikes WTC 1.

8:49 a.m. PAPD police desk a message is received that Emergency Medical Service (EMS) is needed because there is an injured security guard. The message was not complete; the location was not understandable.

8:50 a.m. FDNY establishes a command post in the lobby of WTC 1.
PAPD police desk message from an officer on the B2 level of WTC 1 that there are two workers injured on that level and that EMS is needed ASAP.
PAPD police desk receives a radio call from FDNY Ladder 10 requesting information from PAPD about which building was struck and the location of the fire, WTC 1 or WTC 2. FDNY uses a Port Authority Radio for the communication.

8:52 a.m. NYPD aviation unit arrives at the WTC to examine possibilities for roof rescue.

8:56 a.m. PAPD police desk recording: an officer calls for an ambulance at WTC 4 for an injured person.

8:58 a.m. NYPD aviation unit advises that they are unable to land on the roof due to heavy smoke conditions.

9 a.m. PAPD police desk receives a message that there is an injured person between floors 14 and 15 of WTC 2.

9:01 a.m. PAPD police desk receives a report of a fire in a parking lot.

9:02 a.m. PAPD police desk receives a report of a gas leak. (Incomplete message, location of leak not identified.)

9:03 a.m. An aircraft strikes WTC 2.

9:03 a.m. PAPD – by this time a PAPD senior officer has called three times for the evacuation of the World Trade Center, WTC 1 and WTC 2, and then “all buildings in the complex.”

9:03 a.m. PAPD police desk reports that another aircraft has stuck WTC 2.

9:05 a.m. PAPD police desk, an officer calls in and requests that every ambulance that can be spared be sent to the WTC.

9:10 a.m. FDNY dispatcher receives message that people are trapped on floor 86 of WTC 2.

9:11 a.m. FDNY reports that Engine 10 requests that all responding units stop short of the WTC buildings, either north or south of Liberty and West Street because of the large number of parked ambulances and debris falling from the buildings.

9:17 a.m. FDNY radio communications on the City-wide, high-rise Channel 7 (PAPD Channel 30) indicates that they are using an elevator for operations in WTC 2.

9:18 a.m. FDNY radio communication from WTC 2 indicates they have one elevator working to floor 40, and it is staffed by a firefighter from Ladder 15.
PAPD police desk receives a report that FDNY is abandoning its command post and going across the street.

9:22 a.m. FDNY radio communications on the City-wide, high-rise Channel 7 (PAPD Channel 30) states that a Battalion Chief is on floor 43 or WTC 2 in the B stairway.
FDNY Battalion Chief now located on floor 43 of WTC 2 receives a message from a FDNY member in the lobby that NYPD Emergency Service police officers (Emergency Service Unit) want to provide support for him. The Battalion Chief gives the Emergency Service Unit police officers direction to his location on floor 43 in the B stairway.

9:26 a.m. NYPD aviation unit advises that it is impossible to land on the roof at this time.

9:29 a.m. FDNY radio communications on the City-wide, high-rise Channel 7 (PAPD Channel 30): a Battalion Chief is communicating that he is located inside “Tower 2, the South Tower.” A firefighter follows the communication attempting to correct the Chief by saying that he was actually in the “North Tower, Tower 2.” This communication confused the actual location of the Battalion Chief, who later came back on the radio reporting that he was in the South Tower. Interviews with FDNY personnel conclusively show that the Battalion Chief was actually inside WTC 2, the South Tower.

9:30 a.m. PAPD police desk receives a report that EMS is setting up a triage station in the lobby of WTC 2.

9:32 a.m. FDNY radio communications on the City-wide, high-rise Channel 7 (PAPD Channel 30): firefighters have been able to get to floor 55 inside WTC 2.

9:38 a.m. NYPD aviation unit calls in to request a landing on the roof of the North Tower as soon as possible.

9:39 a.m. FDNY radio communications on the City-wide, high-rise Channel 7 (PAPD Channel 30): FDNY officer inside WTC 2 indicates that he is sending 10 to 15 injured people down to floor 40 and that the firefighter at that location should take the injured to the building's lobby in the elevator. The officer also requests that the firefighter operating the elevator bring an EMS crew back up with him.

9:40 a.m. NYPD officer advises that they need the aviation units on the roof as soon as possible.

9:41 a.m. FDNY radio communications on the City-wide, high-rise Channel 7 (PAPD Channel 30): Hazmat 1 reports that they are on floor 48 of WTC 2 in the B stairway.

9:42 a.m. FDNY radio communications on the City-wide, high-rise Channel 7 (PAPD Channel 30) a firefighter informs the Battalion Chief that he cannot find any elevator banks that are operating above floor 40. The Chief advises the firefighter that he should climb the B stairway from his location.

PAPD police desk receives a report that people have arrived and want to volunteer to help and where should they be sent. Answer: "Right now just send everybody away from the World Trade. We are not letting anybody come close to it."

PAPD police desk receives a radio report that a triage center has been set up at WTC 4 at Victoria's Secret.

9:43 a.m. NYPD officer advises that no one is to rappel onto the top of the buildings.

Note: The term "rappel" in this case refers to the process of emergency responders using ropes suspended from a helicopter to descend onto the roof of a building.

9:44 a.m. PAPD police desk receives a communication that "They haven't evacuated the Fire Command over here in building 2 or 1."

9:45 a.m. FDNY radio communications on the City-wide, high-rise Channel 7 (PAPD Channel 30): a firefighter calls the Battalion Chief and reports that they had to take their coats off.

9:49 a.m. FDNY radio communications on the City-wide, high-rise Channel 7 (PAPD Channel 30): Battalion Chief instructs firefighter that it is imperative that he get down to the lobby command post to get some people up to floor 40. Injured people are being sent down from floor 70. The firefighter is inside an operating elevator and is reporting that it is not operating properly and expresses concerns about the elevator becoming stuck in the shaft.

9:50 a.m. PAPD police desk receives a message that FDNY needs a resuscitator on floor 19, B corridor of WTC 1.

9:54 a.m. FDNY radio communications on the City-wide, high-rise Channel 7 (PAPD Channel 30): a Battalion Chief calls for a ladder company in the A stairway to extinguish two fires. They are attempting to stretch building hose lines on about floor 78.

FDNY radio communications on the City-wide, high-rise Channel 7 (PAPD Channel 30): a firefighter calls to the Battalion Chief that he is on floor 55 and must stop to rest.

PAPD police desk message indicates that an officer is located on floor 22, fire command center and that there is heavy traffic in the B stairway. The person indicates that they cannot release any emergency locked doors due to fire and the loss of electrical power.

Note: Communication appears to originate from WTC 1.

PAPD police desk receives a report that there are 18 passengers stuck in an elevator on floor 78 sky lobby of WTC 2 and that firefighters are working to get them out. They request EMS at the location on the double.

9:56 a.m. FDNY radio communications on the City-wide, high-rise Channel 7 (PAPD Channel 30): inside WTC 2, a firefighter states they are in the B stairway and that they will have to put some fire out in order to get to the A stairway.

9:57 a.m. PAPD police desk receives reports by radio on Channel X and by phone at 435-2131 from floor 78 of WTC 2 that people are coming out of the elevator banks.
At and below floor 79 of WTC 2, FDNY, NYPD, and PAPD personnel are evacuating occupants, assisting the injured, and fighting fires.
FDNY radio communications on the City-wide, high-rise Channel 7 (PAPD Channel 30): a firefighter in WTC 2 reports that he is trapped in an elevator in the elevator shaft and that they are chopping their way out.

9:59 a.m. FDNY Marine unit reports the collapse of WTC 2.

10:28 a.m. FDNY Marine unit advises that the second WTC tower collapsed.

Emergency Response Communications Chronology

This chronology provides information on communications difficulties experienced by NYPD and PAPD following the attack on the WTC. Much has been published concerning the communications difficulties experienced by FDNY during the incident, and first person interviews with FDNY personnel confirms some of these difficulties. However as shown by this chronology, FDNY was not the only emergency responder department that experienced radio equipment and communications difficulties. There are reports of radios not working well and communications showing that some personnel were not being heard or responded to. Also, some of the radio transmissions demonstrate the failure to communicate as a result of radio traffic surge conditions.

The chronology provides examples of numerous cases where radio transmissions were not understood because of "crossing or doubling" of radio signals when too many people are trying to talk at one time.

8:45 a.m. PAPD police desk Channel W: requests a radio check to locate an open microphone on a handie-talkie radio.

8:49 a.m. PAPD police desk requests, as a result of the surge in radio traffic volume, that police officers stay off the air.
PAPD police desk Channel W: extended period with an open microphone, lots of background noise and people talking.

8:50 a.m. PAPD police desk receives a message that the officer did not copy the previous transmission and asks what is going on.

8:51 a.m. NYPD Special Operations Division channel: a dispatcher advises a police lieutenant that his message was crossed and to repeat it. A message came through that he can't get ahold of someone on the cell phone.

8:53 a.m. NYPD Special Operations Division channel: a dispatcher advises a police department truck that their radio message is cutting off and all that the dispatcher got was something about the upper floors.

8:54 a.m. PAPD police desk is reporting that it is having trouble reading incoming radio transmissions.
PAPD police desk receives a message that an officer is having trouble reading radio messages because of so much commotion on the floor.

8:59 a.m. NYPD Special Operations Division channel: a dispatcher advises a police truck that their radio message is breaking up, and the dispatcher asks what units he wants to respond.

9 a.m. NYPD Special Operations Division channel: dispatcher advises that various units are crossing each other and that the dispatcher cannot understand them.

9:01 a.m. PAPD police desk Channel Y: a microphone is stuck open, interfering with communications.

9:02 a.m. NYPD Special Operations Division channel: a police officer asks the dispatcher if the last transmission was heard. The police office asks twice. There is no answer.

9:03 a.m. PAPD police desk receives a report that someone has found a supervisor's radio that has been lost.

NYPD Special Operations Division channel: an officer in an NYPD car requests that units give their messages slowly.

9:05 a.m. FDNY chief officers conduct tests of the City-wide, high-rise repeater located at the WTC.

9:07 a.m. NYPD Special Operations Division channel: a police officer requests that the air be cleared for emergency vehicles and personnel unimpeded.

9:08 a.m. NYPD Special Operations Division channel: a dispatcher advises officers directing traffic that they are coming over the air. Approximately 30 s later the dispatcher advises a second time that the officers directing traffic are coming over the air and requests that they stop.

9:09 a.m. NYPD Special Operations Division channel: a Special Operations Division officer requests that the dispatcher designate two channels for this emergency, one for units on the scene and one for units that are responding.

9:11 a.m. NYPD: a backup transmitter for City-wide communications is put into service in anticipation of potential problems with the primary transmitter.

9:12 a.m. NYPD City-wide channel: a dispatcher advises that "We need to keep this frequency clear unless it is in regards to the level four mobilization."

NYPD Special Operations Division channel: dispatcher state "Only emergency transmissions are to be made on this frequency."

FDNY Chief begins using the FDNY channel 7 repeater while working inside WTC 2.

PAPD police desk receives a radio report from the Fire Command Desk in the lobby of WTC 2 that they cannot pick up the Warden phones and that they are making announcements telling people not to stay at the Warden phones. Note: This communication indicates that the Warden phones in WTC 2 were not working.

9:15 a.m. PAPD police desk Channel W: a radio microphone is stuck open.

9:19 a.m. NYPD Special Operations Division channel: a dispatcher advises a police officer that his message was being cut off and that only part of the message was copied.

9:20 a.m. NYPD Special Operations Division channel: a dispatcher advises that there is an open carrier and the units should check their radios.

9:22 a.m. NYPD Special Operations Division channel: a dispatcher advises a police truck that his radio message was unreadable.

NYPD Special Operations Division channel: the dispatcher advises a second time that there is an open carrier and that messages are not being understood.

9:23 a.m. NYPD Special Operations Division channel: the dispatcher advises that the two frequencies are the Manhattan IO (Interoperability Channel) and the City-wide. The dispatcher also advises that the various units are crossing.

9:25 a.m. PAPD police desk Channel W: a radio microphone is stuck open and interfering with communications. Radio signals are garbled and broken, and there is a high level of background noise.

9:30 a.m. PAPD police desk Channel X has a communication indicating that everybody should turn their phone off.

9:31 a.m. PAPD police desk Channel X: a Port Authority officer is questioned as to whether they have brought any red bags with radios for the fire department. The answer is no, and is it safe to go into the building.

9:32 a.m. NYPD City-wide channel: a unit advises that he cannot communicate, his radio is going in and out and the cell phone is not working.

9:36 a.m. NYPD Special Operations Division channel: a police officer reports that the telephones at his location are not working. Note: Location not identified.

9:43 a.m. NYPD Special Operations Division channel: a police officer advises that he heard over an AM radio that a plane had crashed into the Pentagon.
9:49 a.m. PAPD police desk instructs officer that he was speaking too fast and that he must slow down so that he could be understood.
9:53 a.m. NYPD Special Operations Division channel: the dispatcher advises all units to check their portable radios for an open carrier.
9:54 a.m. NYPD City-wide channel: a dispatcher requests "Keep the air clear. We have problems in the City. Keep the air clear right now."
9:55 a.m. PAPD police desk Channel Z: police officers are having trouble reading communications over the radio and indicate that they will try to call on the telephone.
9:57 a.m. PAPD police desk Channel X: a report is received that an officer is responding to WTC 1 Fire Command and that he had been trying to contact the Command Center on floor 22, but they didn't know how to operate the other set of communications equipment.
9:59 a.m. NYPD Special Operations Division channel: an Emergency Service Unit police officer calls several times for the dispatcher. The dispatcher answers each time and apparently was not heard by the calling unit.
10:03 a.m. NYPD Special Operations Division channel: a dispatcher requests that some units standby while the needs of other units are addressed.
10:05 a.m. NYPD Special Operations Division channel: the dispatcher advises that all units need to talk one by one. The dispatcher further advises that units are cutting each other off.
10:09 a.m. NYPD Special Operations Division channel: an Emergency Service Unit advises the dispatcher that he can hear the dispatcher but is not sure if the dispatcher is hearing him.
10:10 a.m. NYPD Special Operations Division channel: the dispatcher advises that there are three units trying to talk at the same time and requests, "One at a time."

Condition of the WTC Towers Chronology

Information provided by this chronology partially describes the variable conditions found in WTC 1 and WTC 2 towers. It is shown that the impact of the first aircraft into WTC 1 produced an explosive condition all the way down to the building's basement. The impact of the aircraft into what appears to be WTC 2 produced jet fuel fires in the building on the 51st floor. Other communications indicate that there was no smoke or fire on the 68th, 73rd, or 74th floors, the walls in stairway B had been breached. A telephone call to a New York City Radio 9-1-1 telephone operator at 9:36 a.m. indicates that a floor in the 90's level of WTC 2 had collapsed. Information from this call concerning the floor collapse appears to be misstated by the NYPD Division 1 radio operator in the message transmitted at 9:41 a.m. and again at 9:51 a.m. Communications from the NYPD aviations units describe a steady deterioration of the two WTC towers before they collapsed.

8:47 a.m. PAPD police desk reports that there is a fire on floor 22 of WTC 1.
PAPD police desk receives a report that there is a lot of debris on floor 22 of WTC 1.
8:49 a.m. PAPD police desk reports that there is damage and a lot of debris on floor 22 of WTC 1.
8:51 a.m. PAPD police desk receives a call that an explosion was observed in the basement of the B1 level of WTC 1. The police desk informs the officer on the B1 level that what he saw resulted from an explosion on the upper floors of the building.
8:57 a.m. PAPD police desk receives report that water pipes are broken on the B4 level of WTC 1.
9:02 a.m. PAPD police desk receives message from a person trapped in an elevator on floor 78 of WTC 1 that the area has smoke, and water and debris are coming down from above.
9:10 a.m. PAPD police desk receives a report that there is burning jet fuel on floor 51 of one of the towers. Note: Communications suggest this is WTC 2.
9:13 a.m. PAPD police desk receives a report that WTC 1 is flooding.

9:32 a.m. PAPD police desk receives a message from an officer that the WTC Concourse is flooding.
9:36 a.m. New York City 9-1-1 telephone operator receives a message from an occupant of WTC 2 that a floor had collapsed below them in the 90's level.
9:41 a.m. NYPD dispatcher advises units that floor 106 in WTC 2 is collapsing and that the message comes from someone on that floor.
9:47 a.m. FDNY radio communications on the City-wide, high-rise Channel 7 (PAPD Channel 30): a firefighter inside WTC 2 reports that he is standing in the B stairway on floor 74 and there is no smoke or fire problem. He reports that the stairway walls have been breached on floors 73 and 74. Another FDNY unit in the same stairway reports that the walls were also breached on floor 68.
9:49 a.m. NYPD aviation unit gives a radio report stating that "large pieces" are falling from WTC 2.
9:51 a.m. NYPD dispatcher advises that at WTC 2, floor 106 is crumbling per communications with victims trapped on the floor.
9:58 a.m. NYPD aviation unit advises that the south tower is coming down.
10:06 a.m. NYPD officer advises that it isn't going to take much longer before the north tower comes down and to pull emergency vehicles back from the building.
10:20 a.m. NYPD aviation unit reports that the top of the tower might be leaning.
10:21 a.m. NYPD aviation unit reports that the north tower is buckling on the southwest corner and leaning to the south.
NYPD officer advises that all personnel close to the building pull back three blocks in every direction.
10:27 a.m. NYPD aviation unit reports that the roof is going to come down very shortly.
10:28 a.m. NYPD officer reports that the tower is collapsing.

P.8 FINDINGS

The following is a list of preliminary findings based on the current status of emergency responder communications analysis:

1. After the first aircraft struck the WTC, there was a peak increase in emergency responder radio communications by approximately a factor of 5, followed by an approximate factor of 3 steady level of radio communications.

2. A surge in communications traffic volume made it more difficult to handle the flow of communications and delivery of information.

3. Analysis of the radio communications records received by NIST indicates that roughly one-third to one-half of the radio messages transmitted during these radio traffic surge conditions were not complete messages nor understandable.

4. Preliminary analysis of the FDNY City-wide, high-rise Channel 7 (PAPD Channel 30) recording indicates that the WTC site repeater was operating.

5. Communications records and interviews indicate that smoke and heat conditions on the top of the two WTC buildings prevented the NYPD helicopters from conducting safe roof evacuation operations.

6. NYPD aviation unit personnel reported critical information about the impending collapse of the WTC towers several minutes prior to their collapse. No evidence has been found to

suggest that the information was further communicated to all emergency responders at the scene.

P.9 REFERENCES

ARRL. (American Radio Relay League). 2003. *The ARRL Handbook for Radio Communications*. Newington, CT.

Microsoft (Microsoft, Inc.). 2003. Windows Media Player. http://www.microsoft.com.

Nullsoft (Nullsoft, Inc.). 2002. WinAmp 3. http://www.winamp.com.

Sonic (Sonic Foundry, Inc.). 2002. Sound Forge 6.0. Madison, WI.

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Attachment 1
COMMUNICATIONS OF THE PORT AUTHORITY OF NEW YORK AND NEW JERSEY

The National Institute of Standards and Technology received duplicates of many radio and telephone channels. The tapes listed below cover a wide range of times. All recordings cover at least from 0845 to 0958. The remainder of the communications can be placed in two categories as follows: 0705 to 0958 and 0705 to 1800.

Central Police Desk (CPD): Police Command Channel 2 to 39 – Each one of these recordings is 198 minutes long.

CPD Ch. 002 – CPD.wav – Not assigned
CPD Ch. 003 – CPD.wav – Not assigned
CPD Ch. 004 – CPD.wav – Not assigned
CPD Ch. 005 – CPD.wav – Not assigned
CPD Ch. 006 – CPD.wav – Not assigned
CPD Ch. 007 – CPD.wav – Not assigned
CPD Ch. 008 – CPD.wav – Not assigned
CPD Ch. 009 – CPD.wav – Not assigned
CPD Ch. 010 – CPD.wav – Not assigned
CPD Ch. 011 – CPD.wav – Not assigned
CPD Ch. 012 – CPD.wav – Not assigned
CPD Ch. 013 – CPD.wav – Not assigned
CPD Ch. 014 – CPD.wav – Not assigned
CPD Ch. 015 – SPEN 1 State Police Emergency Network.wma
CPD Ch. 016 – Radio SPEN 2 State Police Emergency Network.wma
CPD Ch. 017 – Radio (Ch. A) PA Area Wide.wma
CPD Ch. 018 – Radio (Ch. W) LT Police.wma
CPD Ch. 019 – CPD.wav – Not assigned
CPD Ch. 020 – CPD.wav – Not assigned
CPD Ch. 021 – Phone 9-1-1 Emergency.wma (Note: Recording was blank.)
CPD Ch. 022 – Phone 9-1-1 Emergency.wma (Note: Recording was blank.)
CPD Ch. 023 – Phone SGT's Desk - 201-216-6800.wma
CPD Ch. 024 – Phone Clerk - 201-216-6800.wma
CPD Ch. 025 – Phone TTY NY- 201-216-6800.wma
CPD Ch. 026 – Phone Clerk Extra - 201-216-6800.wma
CPD Ch. 027 – Phone TTY NJ - 201-216-6800.wma
CPD Ch. 028 – Phone Absence Control Line 1 - 201-216-6988.wma
CPD Ch. 029 – Phone Absence Control Line 2 - 201-133-6988.wma
CPD Ch. 030 – Phone 800 number SGT's Desk - 201-216-6858.wma
CPD Ch. 031 – Desk TTY number 3.wma
CPD Ch. 032 – CPD.wav – Not assigned
CPD Ch. 033 – CPD.wav – Not assigned
CPD Ch. 034 – CPD.wav – Not assigned
CPD Ch. 035 – Phone 201-963-7247 Assignment Line 800-776-8580.wma

CPD Ch. 036 – Phone 201-963-7248 Assignment Line 800-776-8580.wma
CPD Ch. 037 – Phone 201-963-7249 Assignment Line 800-776-8580.wma
CPD Ch. 038 – Phone 201-659-3028 Toll Rob 800-TOLL-ROB.wma
CPD Ch. 039 – Phone 201-216-6794 Drug Tip 800-828-PAPD.wma

PATH Police Command: Ch. 02 to 31 – Recordings vary in length from 106 minutes to 193 minutes.

PATH Ch. 02 – Phone Desk Right.wma
PATH Ch. 03
PATH Ch. 04
PATH Ch. 05
PATH Ch. 06 – SGT. desk.wma
PATH Ch. 07 – Tour Commander.wma
PATH Ch. 08 – Report Room.wma
PATH Ch. 09 – Juvenile Room.wma
PATH Ch. 10 – Reserve Room 216-6078.wma
PATH Ch. 11 – Phone Desk Left.wma
PATH Ch. 12 – Jersey City Fire Department.wma
PATH Ch. 13 – Jersey City Medical Center.wma
PATH Ch. 14 – Jersey City Police.wma
PATH Ch. 15 – NYPD.wma
PATH Ch. 16
PATH Ch. 17
PATH Ch. 18
PATH Ch. 19 – Conference Room 1.wma
PATH Ch. 20 – Conference Room 2.wma
PATH Ch. 21 – Radio (R2) Train Master.wma
PATH Ch. 22 – PD Wall (Desk Area).wma
PATH Ch. 23 – Court Office 1.wma
PATH Ch. 24 – Court Office 2.wma
PATH Ch. 25 – Court Sgt.wma
PATH Ch. 26 – Radio (R1) Train Master.wma
PATH Ch. 27 – Radio (R30) Communications.wma
PATH Ch. 28
PATH Ch. 29
PATH Ch. 30
PATH Ch. 31

WTC Police Desk 1: Ch. 002 to 039 – Each one of these recordings is 171 min.

Ch. 002 WTC.wav
Ch. 003 WTC.wav
Ch. 004 WTC.wav
Ch. 005 WTC phone 435-8456 clerk.wav
Ch. 006 WTC phone 435-8462 clerk.wav
Ch. 007 WTC phone 435-2135 TC.wav
Ch. 008 WTC phone 435-3541 desk left.wav
Ch. 009 WTC phone 435-3541 desk center.wav
Ch. 010 WTC phone 435-3541 desk right.wav
Ch. 011WTC phone 435-8460 conf. room.wav
Ch. 012WTC .wav

Ch. 013 WTC phone 435-3519 office.wav
Ch. 014 WTC direct line FDNY.wav
Ch. 015 WTC direct line NYC EMS.wav
Ch. 016 WTC phone 435-7666 floor warden.wav
Ch. 017 WTC direct line fire command WTC 1.wav
Ch. 018 WTC direct line fire command WTC 2.wav
Ch. 019 WTC.wav
Ch. 020 WTC.wav
Ch. 021 WTC phone 435-2133 police reserve rm.wav
Ch. 022 WTC phone 435-2131 SHO desk.wav
Ch. 023 WTC phone 435-2948 desk.wav
Ch. 024 WTC radio Ch. A.wav
Ch. 025 WTC radio Ch. B.wav
Ch. 026 WTC radio Ch. W.wav
Ch. 027 WTC radio Ch. X.wav
Ch. 028 WTC radio Ch. Y.wav
Ch. 029 WTC radio Ch. Z.wav
Ch. 030 WTC FDNY radio.wav
Ch. 031 WTC.wav
Ch. 032 WTC.wav
Ch. 033 WTC.wav
Ch. 034 WTC.wav
Ch. 035 WTC.wav
Ch. 036 WTC.wav
Ch. 037 WTC.wav
Ch. 038 WTC.wav
Ch. 039 WTC.wav

Newark International Airport: Police Command – Ch. 02 to 39

EWR Ch. 002
EWR Ch. 003
EWR Ch. 004
EWR Ch. 005
EWR Ch. 006
EWR Ch. 007
EWR Ch. 008 – Phone 733-7525 – Newark PD.wma
EWR Ch. 009 – Phone PL234846 – Eliz. PD.wma
EWR Ch. 010 – Phone PL92866- Newark FD.wma
EWR Ch. 011 – PL234881 – Eliz. FD.wma
EWR Ch. 012 – Phone PL230333 – AFA.wma
EWR Ch. 013
EWR Ch. 014 – Phone PL234979 – REMCS.wma
EWR Ch. 015 – FAA Tower Crash Alarm.wma
EWR Ch. 016
EWR Ch. 017
EWR Ch. 018 – PNPD PVL - OSNA660-650.wma
EWR Ch. 019 – Phone 589-6321 - PNPD.wma
EWR Ch. 020 – Phone 589-0292 - PNPD.wma
EWR Ch. 021 – Phone 961-6666 - Line 3.wma
EWR Ch. 022 – Phone 961-6666 - Line 4.wma

EWR Ch. 023 – Radio – EWR Command – 800Mhz.wma
EWR Ch. 024 – Radio – EWR ARFF – 800Mhz.wma
EWR Ch. 025 – Radio – EWR TAC 1 - 800Mhz.wma
EWR Ch. 026 – Radio – Central police desk - 800Mhz.wma
EWR Ch. 027 – Radio – EWR Detectives.wma
EWR Ch. 028 – Police desk left phone – 961-6230.wma
EWR Ch. 029 – Police desk phone center – 961-6230.wma
EWR Ch. 030 – Police CAD desk phone – 961-6230.wma
EWR Ch. 031 – Police desk right phone – 961-6230.wma
EWR Ch. 032 – Phone 961-6666 – Line 2.wma
EWR Ch. 033 – Phone 961-6666 – Line 1.wma
EWR Ch. 034
EWR Ch. 035
EWR Ch. 036 – Radio Ch. Z – Operations & Terminals.wma
EWR Ch. 037
EWR Ch. 038 – Radio – Ch. X – Facility maintenance.wma
EWR Ch. 039 – Radio – Ch. B – Maintenance.wma

NYPD WTC Communications:

NYPD Special Operations Division
Tape 1, 08:46 – 09:33
Tape 2, 09:32 – 10:18
Tape 3, 10:18 – 11:04

NYPD City-wide 1 radio, Tape 4, 08:40 – 09:27
Tape 4b, 09:27 – 10:12
Tape 5c, 10:12 – 11.59
Tape 5d, 10:59 – 11:46

NYPD Division 1, Tape 6, 08:45 – 9:30
Tape 7, 09:29 – 10:15
Tape 8, 10:14 – 11:00

TABLE OF CONTENTS - Appendix Q

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Appendix Q
NIST'S WORKING HYPOTHESIS FOR COLLAPSE OF THE WTC TOWERS

In response to the terrorist attacks of September 11, 2001, and the resulting collapse of the World Trade Center (WTC) buildings, the National Institute of Standards and Technology (NIST) has been investigating possible collapse scenarios. Establishing the sequence of events that led to the towers collapsing is important in determining which factors allowed the buildings to hold up for as long as they did without collapsing, and which factors, if any, could have delayed or prevented the collapse of the WTC towers. Understanding these factors will provide valuable information on which to base recommendations for improvements to buildings practices, standards, and codes that may be warranted.

Background

NIST has estimated that 17,400 occupants (± 1,200) were present in the WTC towers on the morning of September 11, 2001, about equally divided between the two buildings (8,900 in WTC 1 and 8,540 in WTC 2). 2,159 building occupants and an additional 433 first responders, including security guards, were reported to have lost their lives that day. This does not include aircraft passengers and crew or bystanders.

Approximately 87percent of the WTC tower occupants were able to evacuate successfully. More than 99 percent of occupants below the crash impact areas had sufficient time prior to collapse of the buildings to safely evacuate. WTC 1 stood for one hour and 43 min after impact; WTC 2 collapsed 56 min after it was struck.

Preliminary estimates indicate that about 20 percent or more of those in the WTC towers who lost their lives may have been alive in the buildings just prior to their collapse. This includes nearly all of the first responders and 76 building occupants below the floors of impact. There were 72 fatalities reported in WTC 1 and 4 fatalities reported in WTC 2, not including first responders, below the floors of impact.

Buildings are not designed to withstand the impact of fuel-laden commercial airliners. However, Port Authority documents indicate that the impact of a Boeing 707 flying at 600 mph and possibly crashing into the 80th floor had been analyzed during the design of the WTC towers in February/March 1964. While NIST has not found evidence of the analysis, the documents state that such a collision would result in localized damage only, and that it would not cause collapse or substantial damage to the WTC towers. The effect of fires due to jet fuel dispersion and ignition of building contents was not considered in the 1964 analysis. Loss of life in the immediate area of aircraft impact was anticipated, but loss of life from fire and smoke was not considered.

Q.1 OBJECTIVES

The objectives of the NIST analysis are:

- Establish how and why the WTC towers collapsed after the aircraft impact, i.e., the 'triggering event'
- Determine the most probable collapse sequence
- Identify the factors that have the strongest influence on the most probable sequence

Q.2 APPROACH

To identify the most probable of the technically possible collapse sequences, NIST is adopting an approach that combines mathematical modeling, well-established statistical and probability based analysis methods, laboratory experiments, and analysis of photographic and video evidence. The approach accounts for variations in models, input parameters, analyses, and observed events. It allows for evaluation and comparison of possible collapse hypotheses based on different damage states, fire paths, and structural responses to determine the following:

1. The most probable sequence of events from the moment of aircraft impact until the initiation of global building collapse;

2. How and why WTC 1 stood nearly twice as long as WTC 2 before collapsing (103 min versus 56 min), though they were hit by virtually identical aircraft;

3. What factors, if any, could have delayed or prevented the collapse of the WTC towers.

Q.3 FACTORS TO EVALUATE THE WORKING HYPOTHESIS

In further evaluating the working hypothesis for the collapse of the WTC towers, NIST is considering the following factors:

- The relative contributions of aircraft impact damage and subsequent fires

- How safe each building was immediately after aircraft impact but before fire weakened the structures, i.e., to what extent the capacity of the buildings to carry design loads[1] was reduced

- Relative roles of the perimeter and core columns[2] and the composite floor system,[3] including connections

- The role played by fireproofing, especially the extent to which fireproofing may have been damaged due to aircraft impact

- Whether the undamaged towers would have remained standing in a “maximum credible fire”[4]

- The role compartmentation (i.e., areas divided by fire-rated walls) may have played, i.e., what would have happened if the floors had been separated into 7,500 or 10,000 ft^2 compartments with 1 h fire-rated partition walls or separations

Q.4 THE WORKING HYPOTHESIS

NIST has developed a working hypothesis to explain the sequence of events from aircraft impact until the initiation of global structural collapse. This hypothesis will be further refined based on the results of

[1] The design of the WTC towers was governed by gravity and lateral wind loads.

[2] The perimeter columns were designed to carry both gravity and wind forces and acted together as a framed-tube system. The core columns were designed to carry only gravity loads and not required to provide frame action.

[3] The composite floor truss system, which included long-span open-web bar joist elements, was designed to carry floor loads to the supporting core and perimeter columns. It also acted as a diaphragm that distributed wind forces to the perimeter columns of the framed-tube system and provided lateral stability to the perimeter columns.

[4] A maximum credible fire for the WTC towers is assumed to have the following characteristics: the sprinkler system is compromised, overwhelmed, or not present; there is no active firefighting; combustible building contents averaging 10 psf (in the range of about 5 to 20 psf for conventional office buildings); floor-to-floor fire spread to next upper floor at 30 or 60 min; and ventilation from windows broken by fire and a total of 50 ft^2 of air leakage between floors.

NIST's continuing comprehensive analyses to identify specific load redistribution paths and damage scenarios that are possible for each building, from which the most probable collapse sequence will be identified. NIST welcomes comments from technical experts and the public on the working hypothesis.

NIST's working hypothesis is based on analysis of the available evidence and data, consideration of a range of hypotheses (including those postulated publicly by experts), and a newly enhanced understanding of structural and fire behavior. It is consistent with all current evidence held by NIST, including photographs and videos, eyewitness accounts, and emergency communication records. NIST's analysis allows for different sequences of events and different possible event paths for each building.

To accommodate the aircraft impact and subsequent fire damage, the structure redistributed loads from structural element to structural element via redundant load paths and maintained overall structural stability. Structural collapse began when the structure was not able to redistribute loads any further. The working hypothesis addresses the following chronological sequence of major events; specific load redistribution paths and damage scenarios are currently under analysis:

1. Aircraft impact damage to perimeter columns with redistribution of column loads to adjacent perimeter columns and to the core columns via the hat truss;

2. After breaching the building's exterior, the aircraft continued to penetrate into the buildings, damaging core columns with redistribution of column loads to other intact core and perimeter columns via the hat truss and floor systems;

3. The subsequent fires, influenced by post-impact condition of the fireproofing, further weakened columns and floor systems (including those that had been damaged by aircraft impact), triggering additional local failures that ultimately led to column instability;

4. Initiation and horizontal progression of column instability ensued when redistributing loads could not be accommodated any further. The collapses then ensued.

Role of the Hat Truss System

The purpose of the hat truss was to support gravity and wind loads on the antenna. It was not designed to resist lateral forces on the towers, and, in an undamaged state, it did not have a significant role in carrying gravity loads. Lateral loads due to wind were distributed to the framed-tube system via diaphragm action of the floor system. The hat truss was connected to each perimeter face at only four points, all at the same level (at the 108th floor just below the concrete floor slab). The 47 core columns were connected to diagonal elements, heavier transfer beams, or smaller beam elements in the hat truss. Most of the core columns extended to the roof level, but four core columns, which were only minimally connected to the hat truss, terminated at floor 110. The hat truss provided minimal redistribution of loads (less than 10 percent) from perimeter columns to core columns. Most of the load redistributed due to aircraft impact damage occurred on the external face through vierendeel action.

Aircraft Impact Damage to Perimeter Columns

Initially, the WTC towers withstood the impact virtually identical aircraft. Based on video that NIST has obtained, it is known that WTC 2, which collapsed first and in about half the time as WTC 1, vibrated for over 4 min at an oscillation rate nearly equal to that measured for the undamaged building after it was struck, indicating that the buildings were continuing to respond normally. The lightly damped (about 1.2 percent of critical damping) oscillation had a maximum amplitude of approximately 20 in. at the roof level, where sway was about 3 ft to 4 ft under design wind conditions. Based on this information, structural damage to perimeter columns as a result of aircraft impact of the framed-tube system appears to have played a minimal role in initiating the collapse. Perimeter column bowing prior to collapse occurred on other faces (i.e., fire floors on the south face of WTC 1 and east face of WTC 2) that were not severed by the aircraft.

Aircraft Impact Damage to Core Columns

The core columns were designed to carry only gravity loads and not required to provide frame action. The aircraft trajectory at impact suggests damage to the core columns occurred as follows:

WTC 1—The aircraft was traveling about 450 mph and hit the tower near the center of the north face damaging floors 93 to 99. The aircraft fully entered the core area and severed or damaged central core columns in the north-south direction. Aircraft and building debris accumulated in the remaining core area and south-side floor areas as contents were displaced from the point of impact.

WTC 2—The aircraft was traveling about 550 mph and hit the tower near the southeast corner of the building damaging floors 77 to 85. Core columns to the south and east were severed or damaged. Aircraft and building debris accumulated in the core area and floor areas to the east and north.

Severed core columns redistributed their loads in three ways, depending on how many and which core columns were severed.

1. Isolated core columns were severed. Severed column and tributary floor loads at and above the point of impact were redistributed locally at each floor to adjacent intact core columns via core floor framing. This was limited by shear/bending capacity of floor-framing connections to adjacent columns.

2. Critical (e.g., corner) core columns and/or several other core columns were severed. The severed column and tributary floor loads, at and above impact floors, redistributed to intact core columns via the hat truss. Significant hat truss deflections may have occurred if there was adequate connection capacity since the severed core columns and the associated floors were hanging from the hat truss which was not designed to carry such loads. This was limited by the tensile capacity of bolted splices in the severed core columns, tensile/compression capacity of hat truss members, and tensile capacity of column connections to the hat truss.

3. Extent of core column failures precluded redistribution through the hat truss and/or exceeded redistribution capacity of the hat truss: severed column and associated floor loads, at and above floors of impact, redistributed to intact core and perimeter columns via the core and composite truss floor system. Floors were subjected to combined bending and diaphragm action (e.g., consider the scenario of no core columns in the floor span direction to visualize this action). The overall capacity of the floors was limited by shear capacity of floor-to-column connections (including perimeter columns) and tensile/bending capacity of composite truss floor connections to core or perimeter columns. Significant sagging of the hat truss system may have occurred if its capacity was exceeded.

Relative Roles of Fires and Aircraft Impact

Fires played a major role in collapse initiation. The tower structures withstood the initial aircraft impacts and remained stable. While aircraft impact damage did not, by itself, initiate building collapse, it had the following harmful effects that then contributed greatly to the subsequent fires:

- Compromised the sprinkler and water supply systems,
- Dispersed jet fuel and ignited building contents over large areas,
- Created large accumulations of combustible matter containing aircraft and building contents,
- Increased air supply into the buildings (through broken windows and holes in the sides of the buildings, and between floors due to damaged floors, vertical shafts, and columns) permitted significantly higher energy release rates than would normally be seen in ventilation limited building fires, allowing the fires to spread rapidly within and between floors, and
- Damaged ceilings enabling "unabated" heat transport over the floor-to-ceiling partition walls and to the floor trusses, spandrels, and tops of columns.

The jet fuel, which ignited the fires, was mostly consumed within the first few minutes after impact. The fires that burned for almost the entire time that the buildings remained standing were due mainly to burning building contents and, to a lesser extent, aircraft contents, not jet fuel.

Thermal Effects on Columns and Floors

Some floors in WTC 2 experienced partial collapse due to aircraft impact. For example, partially collapsed floor slabs were visible on the east and north faces. This included failures at the edges with perimeter columns causing floor edge sagging. There is no visible evidence of hanging floors in WTC 1.

- Fires may have had the following thermal effects: core columns and core floors may have been further weakened, with reduced ability to carry and/or redistribute load, causing such loads to be redistributed to other core and perimeter columns consistent with the residual reserve capacities of these columns and the transfer mechanisms (i.e., hat truss and floor system).
- The floor system may have been further weakened, either along the span of the floor system or localized at connections with columns. The weakening floor system may have pulled the perimeter columns inward (observed on the south face of WTC 1 and the east face of WTC 2 minutes prior to building collapse) and then initiated connection failures at perimeter or core columns.

Role of Fireproofing

The post-impact condition of the fireproofing played a key role in the structural response to fires. The post-impact condition of the fireproofing depends on the condition of the fireproofing prior to aircraft impact and the extent to which fireproofing was damaged due to aircraft impact. The fire-affected floors in WTC 1 had, in general, upgraded or thicker fireproofing (1.5 in. specified) while, in general, those in WTC 2 did not have upgraded fireproofing (0.5 in. specified].

- Perimeter columns may have been further weakened, with reduced ability to carry loads. Thermal effects could also cause inward bowing of perimeter columns due to differential temperatures between the inner and outer faces of the columns. The loads that could no longer be carried by the weakened columns would have been redistributed to adjacent perimeter columns.

Column Instability and Collapse Initiation

The perimeter columns were designed as part of a framed-tube system to carry both gravity and wind forces. Instability of perimeter columns resulted from a combination of (1) redistributed loads from the core columns via the floor system and possibly the hat truss, (2) inward bowing due to thermally weakened and sagging floors, (3) increased unsupported length due to failed floors, and (4) thermal effects directly on the perimeter columns.

The instability of a few perimeter columns spread instability across the entire face and around the corners just before or during collapse initiation. The initiation or spread of perimeter column instability also may have been facilitated by the hoop stress demand on the framed-tube system exceeding the capacity of the spandrels (horizontal steel plates) that tied the perimeter columns together (e.g., at the northeast corner of WTC 2).

The initiation of global collapse for both towers was first observed by the tilting of the sections above the impact regions of both WTC towers. The top section of WTC 1 rotated to the south (observed via antenna tilting in a video recording) and the top of WTC 2 rotated to the east and south and twisted in a counterclockwise motion. The primary direction of tilt of each tower was around the weaker axis of the core (north-south for WTC 1 and east-west for WTC 2). The rigid body rotation associated with the tilting and the propagation of column instability are synchronous processes that initiated global collapse. The rigid body rotation may have caused forces such as shear and torsion to spread the column instability laterally.

Q.5 ISSUES STILL BEING INVESTIGATED

Over the next few months, NIST will continue to investigate the following technical issues and modify its working hypothesis as needed. Findings on these issues will be included in the final report.

- Aircraft impact damage to structural components, fireproofing, and hat truss connections.
- Distribution of aircraft/building contents.
- Thermal effects on core columns and core floors, especially extent of fires and growth history.
- Thermal effects on welded perimeter columns, especially temperature gradients on columns.
- Extent of load redistribution to intact core columns and their reserve capacity to accommodate thermal loads.

- Capacity of hat truss connections to perimeter columns, especially to meet the demands of aircraft impact and any torsional effects.
- Capacity of hat truss to accommodate the load redistribution from severed columns.
- Capacity of bolted splices in the severed core columns to carry loads to the hat truss.
- Relative magnitude of the load redistribution provided by the local core floor, hat truss, and the core-truss floor system for each tower.
- Axial/shear/bending capacity of floor connections to core and perimeter columns.
- Effect of localized fires on floor truss connections.
- Mechanisms to propagate instability laterally in the perimeter columns (e.g., shear and torsion forces induced by a rigid body movement)
- Capacity of spandrels, including splices, to carry shear transfer in the framed-tube system, especially at the corners.
- Role of bolted splices on instability of perimeter columns.
- Outward bowing of perimeter columns due to thermal expansion of floors.
- Effect of uneven floor thermal expansion on perimeter column instability due to potential biaxial bending.
- Comparison and reconciliation of working hypothesis with observed facts (photographs and videos, eyewitness accounts, emergency communication records).
- Examination of other possible or probable hypotheses.

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