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

Volume 3
Contains Appendices D, E, and F

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U.S. Department of Commerce
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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

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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

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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.

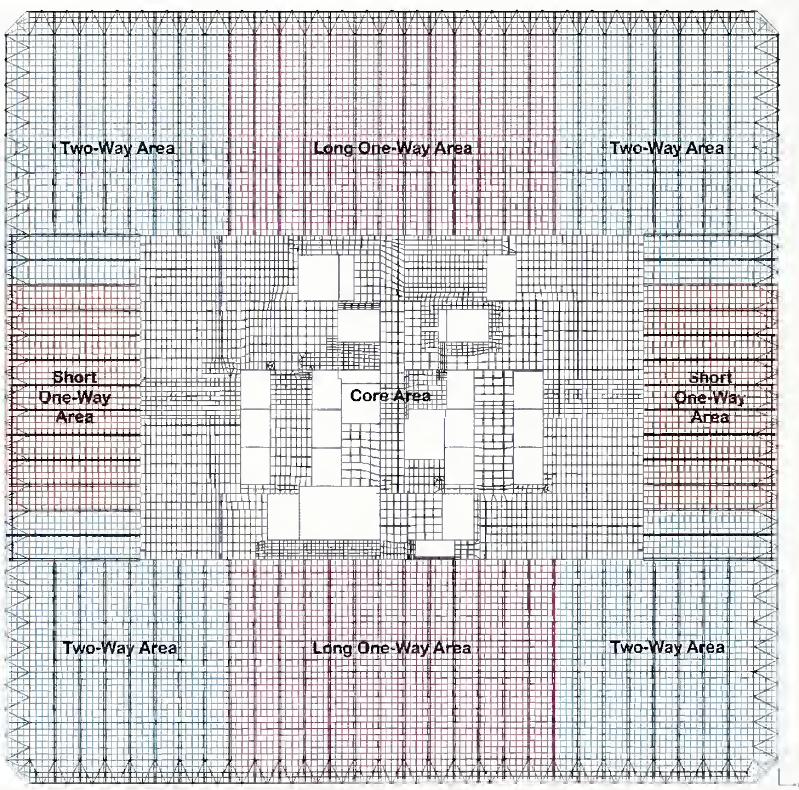


Figure D–1. Plan of typical truss-framed floor with loading areas indicated.

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/m²)]				
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/m²)]				
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/m²)]					
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 °C, 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

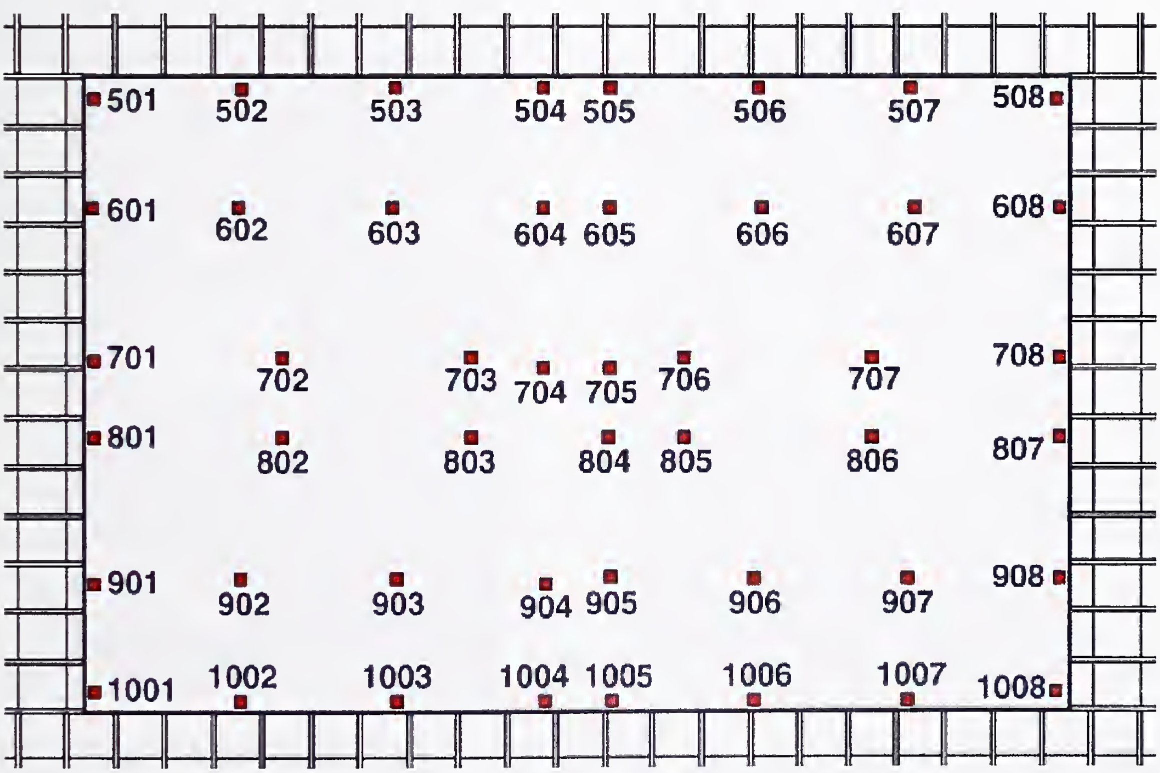


Figure D–2. Plan of typical core column layout (courtesy FEMA).

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.

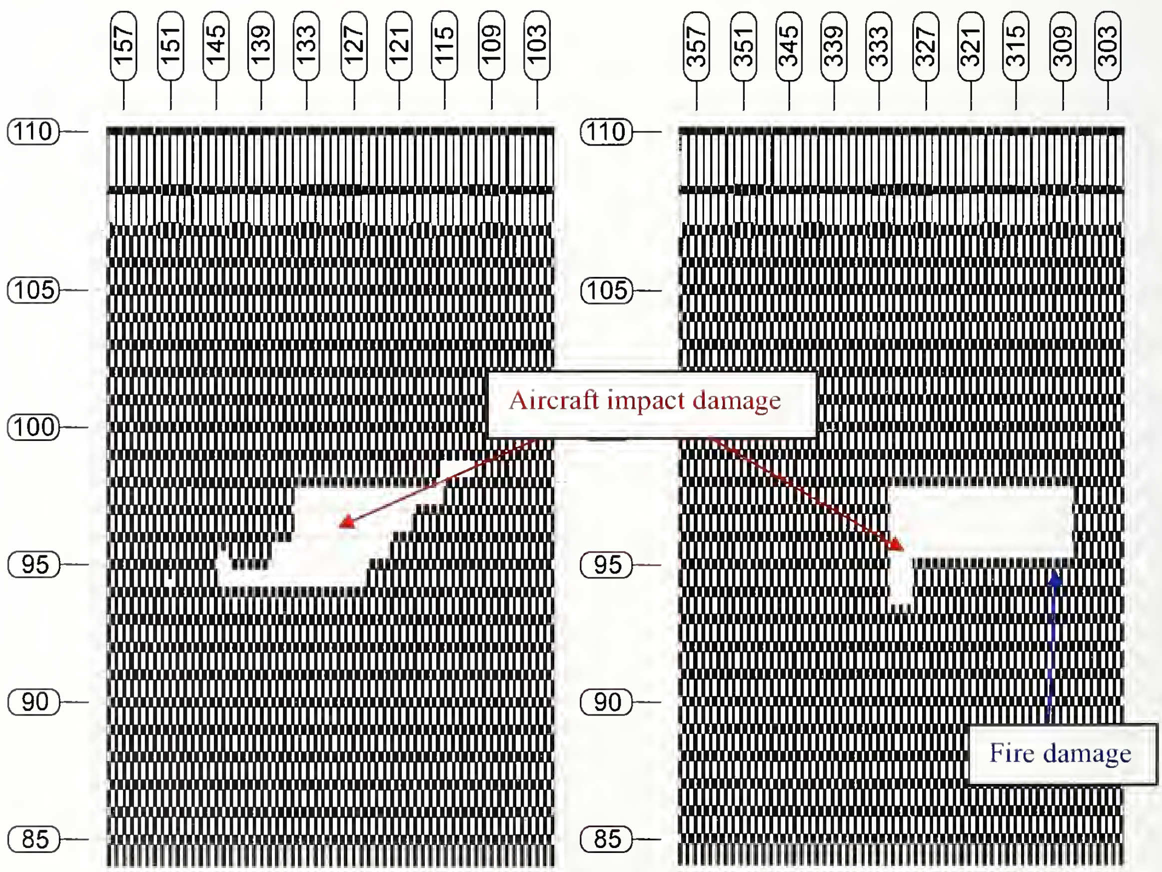


Figure D–3. North and south elevations of WTC 1 indicating columns and spandrels removed due to aircraft impact and fire effects.

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 °C 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 Column Load kip (kN)	Floor 98 Load to Capacity Ratio	Floor 101 Column Load kip (kN)	Floor 101 Load to Capacity Ratio	Floor 104 Column Load kip (kN)	Floor 104 Load to Capacity Ratio	Floor 106 Column Load kip (kN)	Floor 106 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.

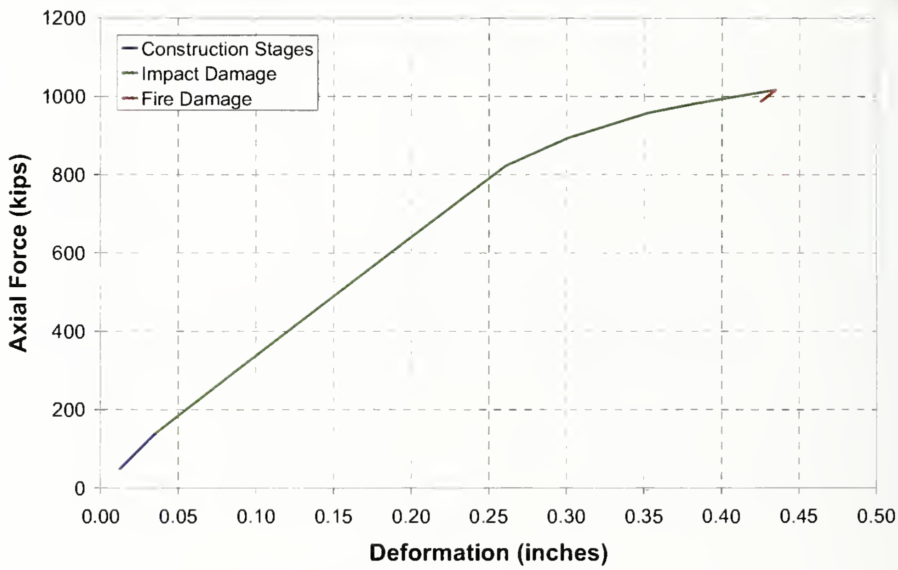


Figure D–4. Load vs. deformation in column 111 at floor 98 (north face, west side of damage).

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.

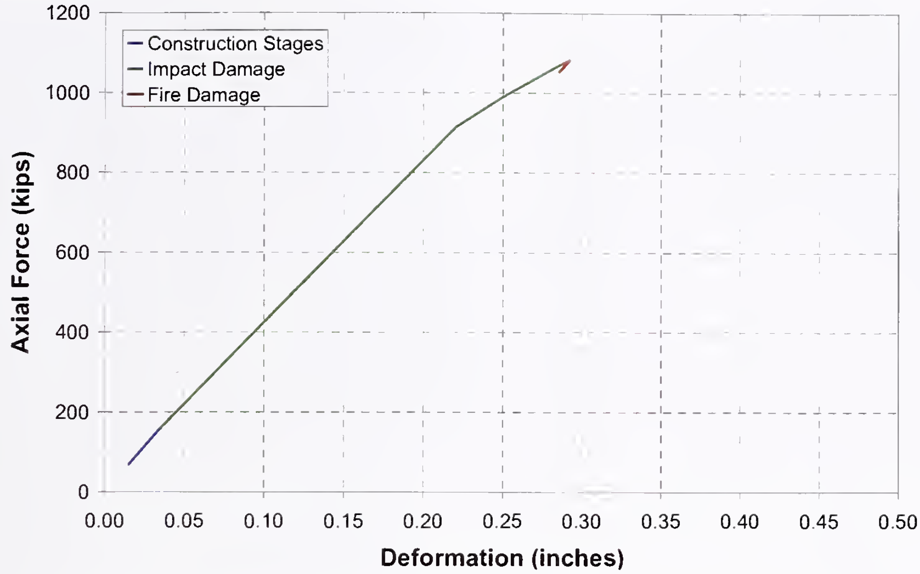


Figure D–5. Load vs. deformation in column 145 at floor 95 (north face, east side of damage).

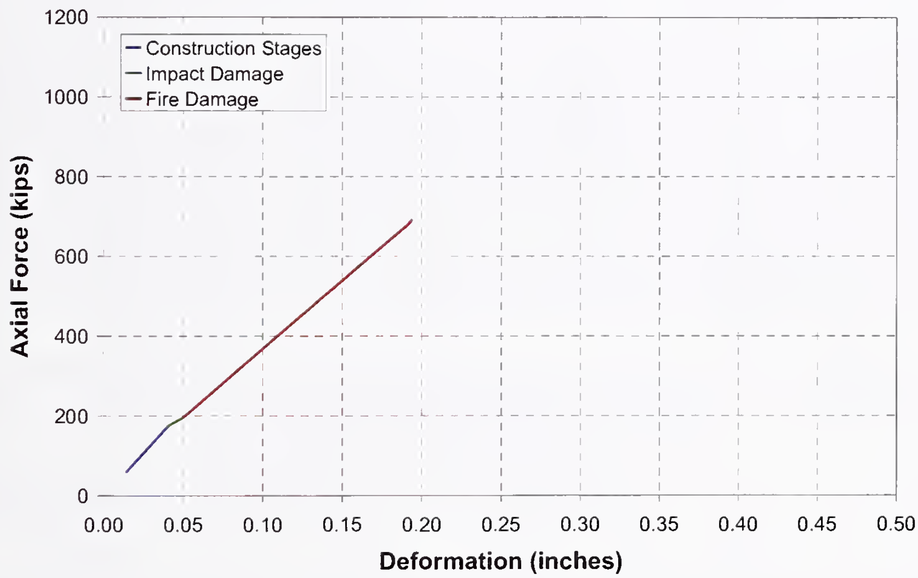


Figure D–6. Load vs. deformation in column 332 at floor 97 (south face, west side of damage).

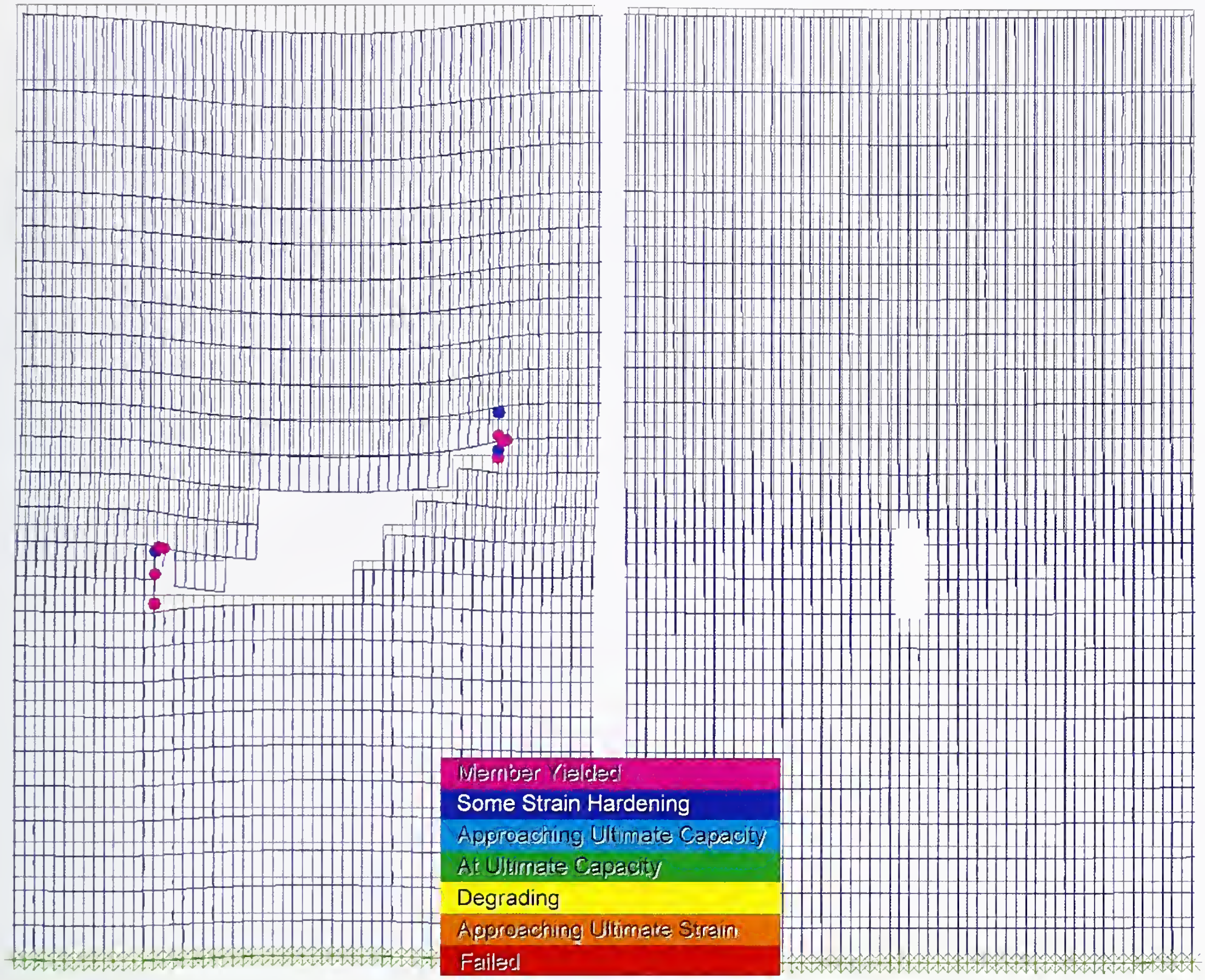
Table D–8. Distribution of loads on exterior walls and core columns at floor 99 for Core Damage Case 2.

Case	Const. Stage 1		Const. Stage 2		Const. Stage 3		Impact Damage		Fire Damage	
	(kip)	(%)	(kip)	(%)	(kip)	(%)	(kip)	(%)	(kip)	(%)
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



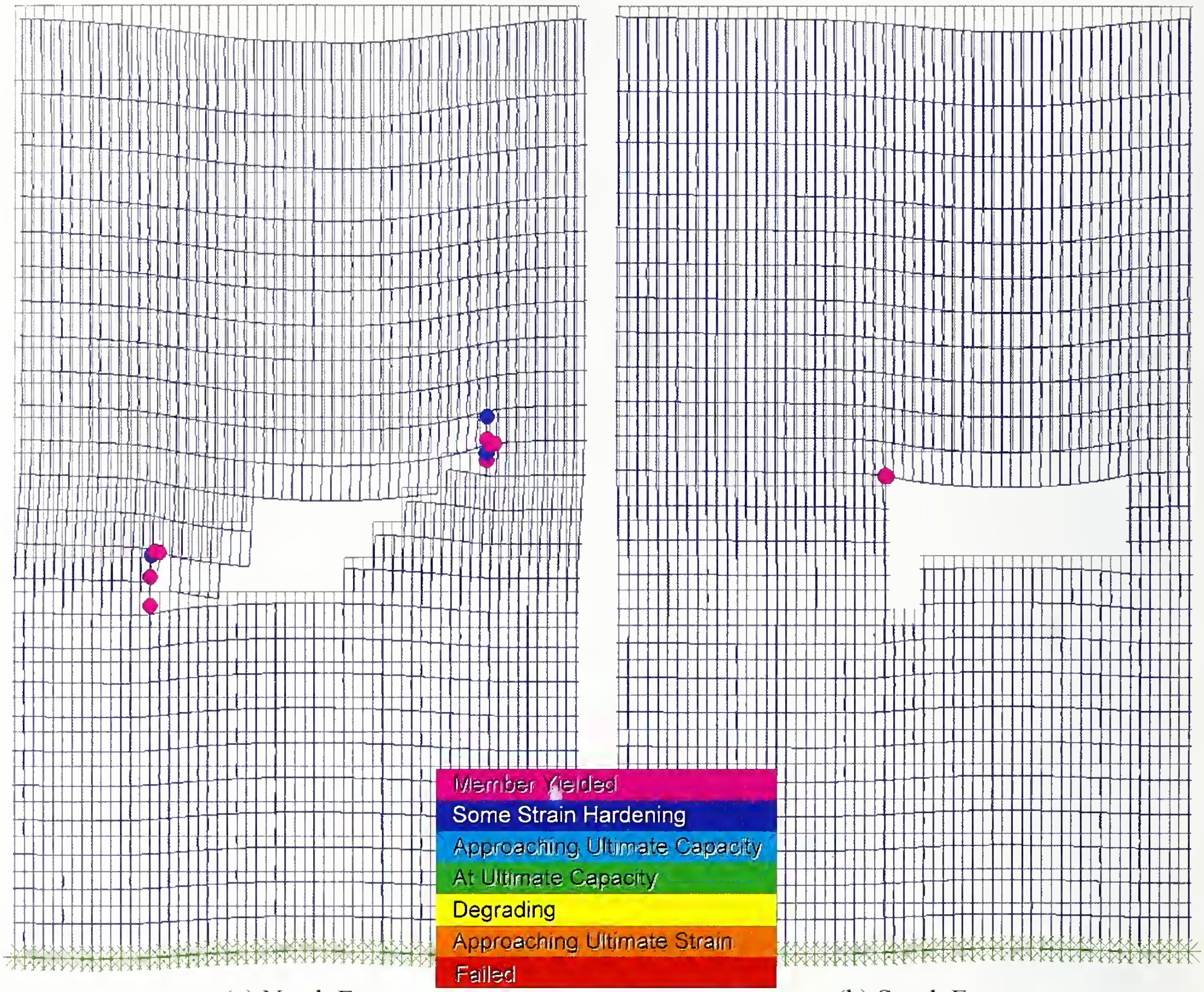
(a) North Face (b) South Face

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 °C (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.

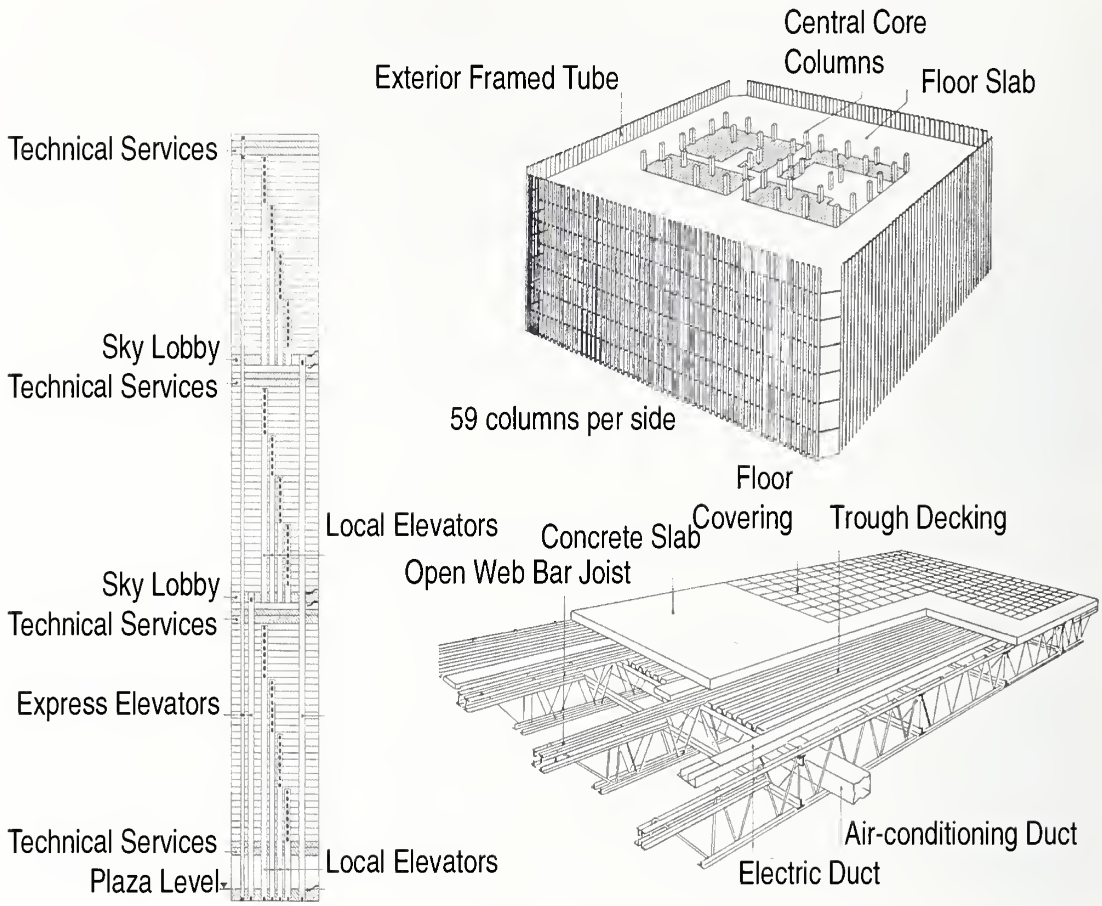
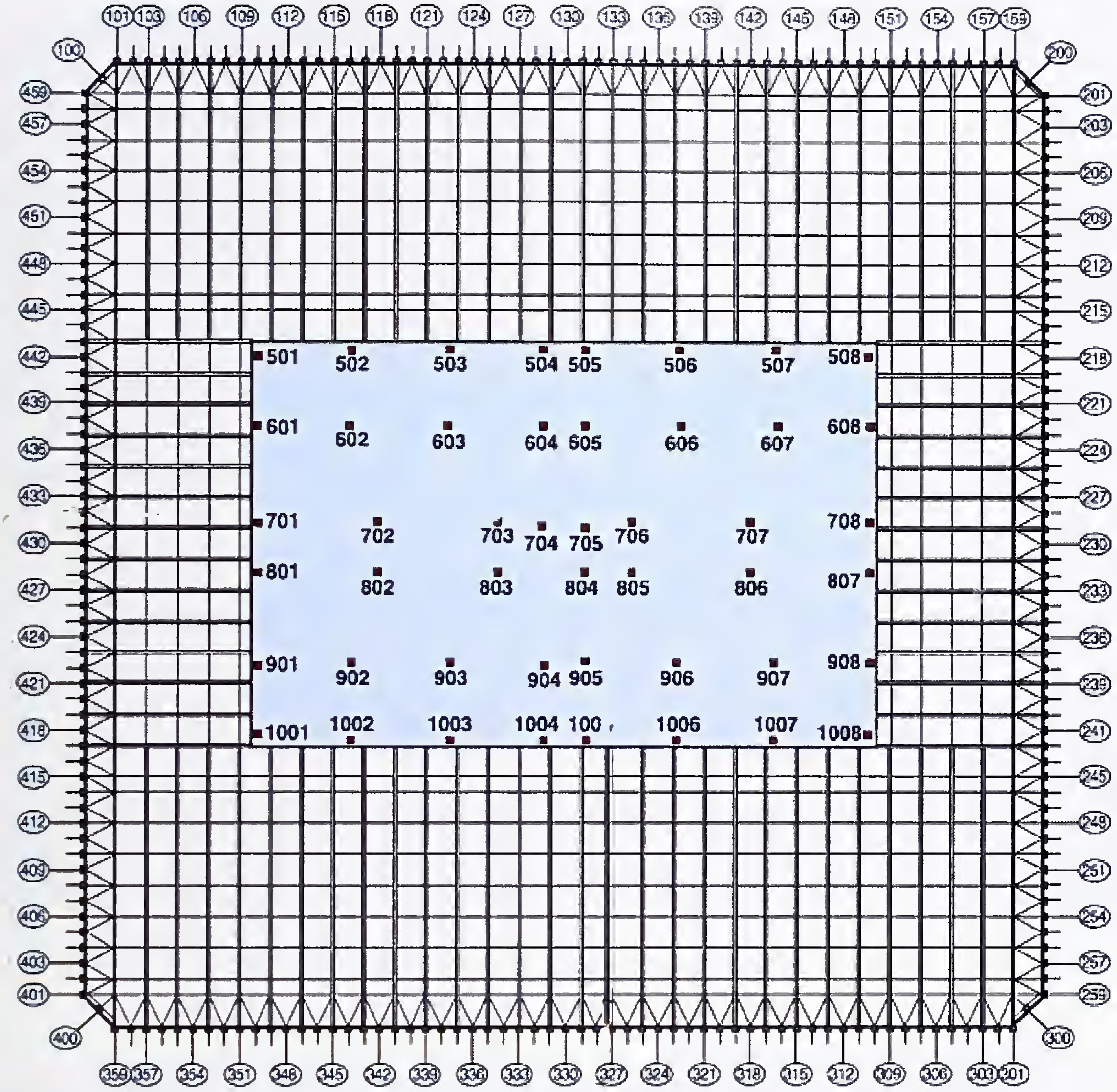


Figure E–1. Schematic diagram of the tower structure.

(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).

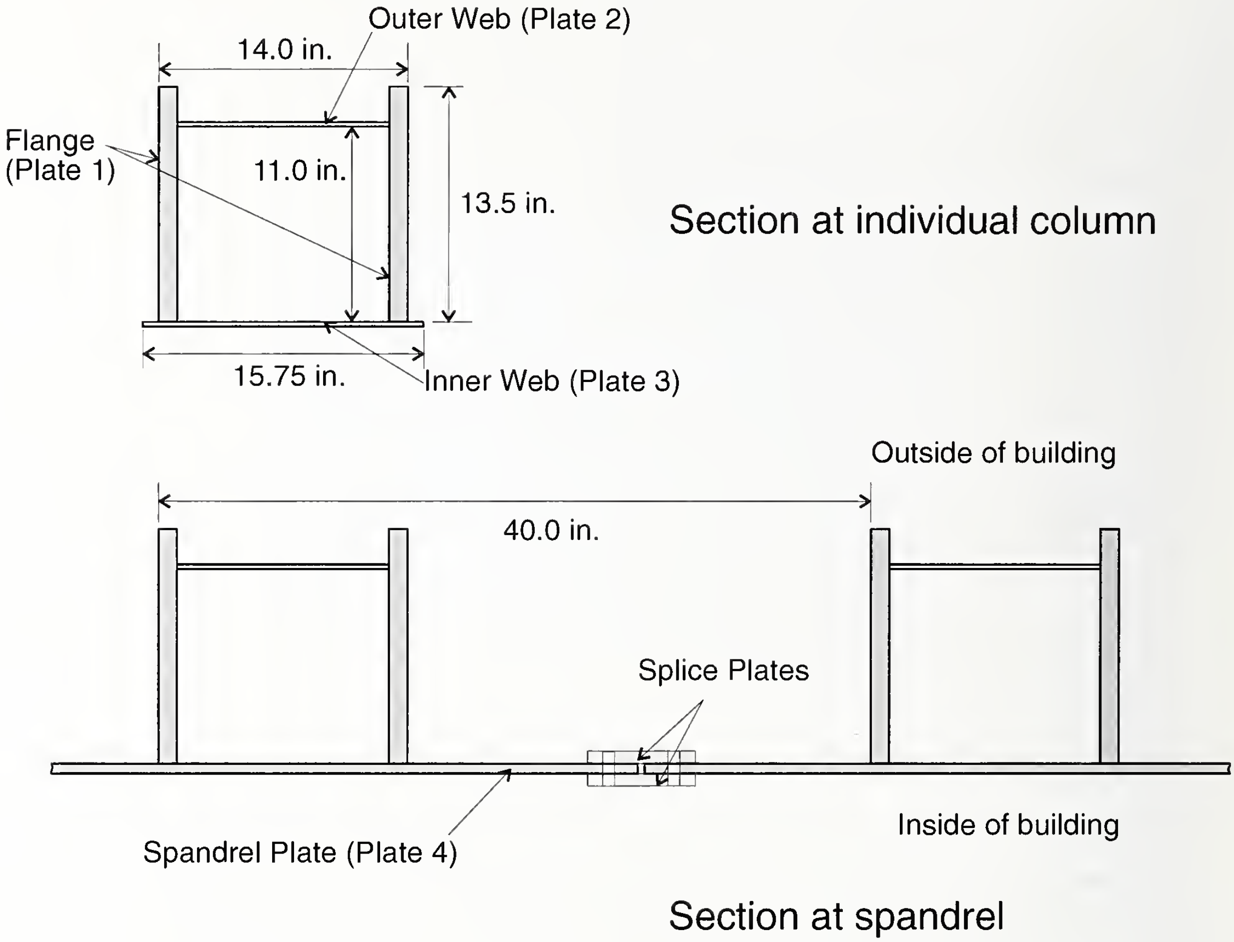


Figure E–3. Cross-section of perimeter columns; sections with and without spandrels.

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.

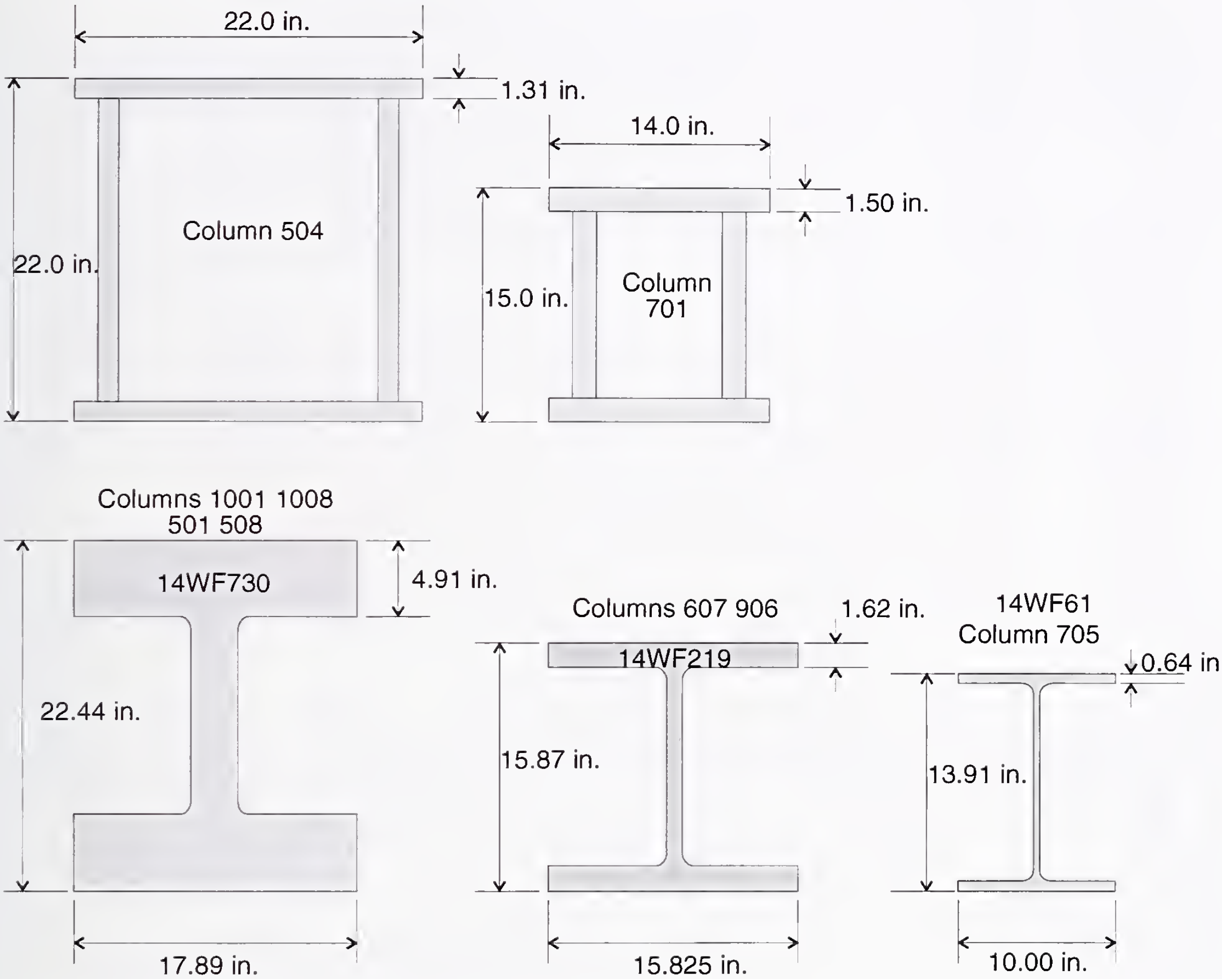


Figure E–6. Typical welded box members and rolled wide flange shapes used for core columns between floors 83 and 86 (to scale).

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.

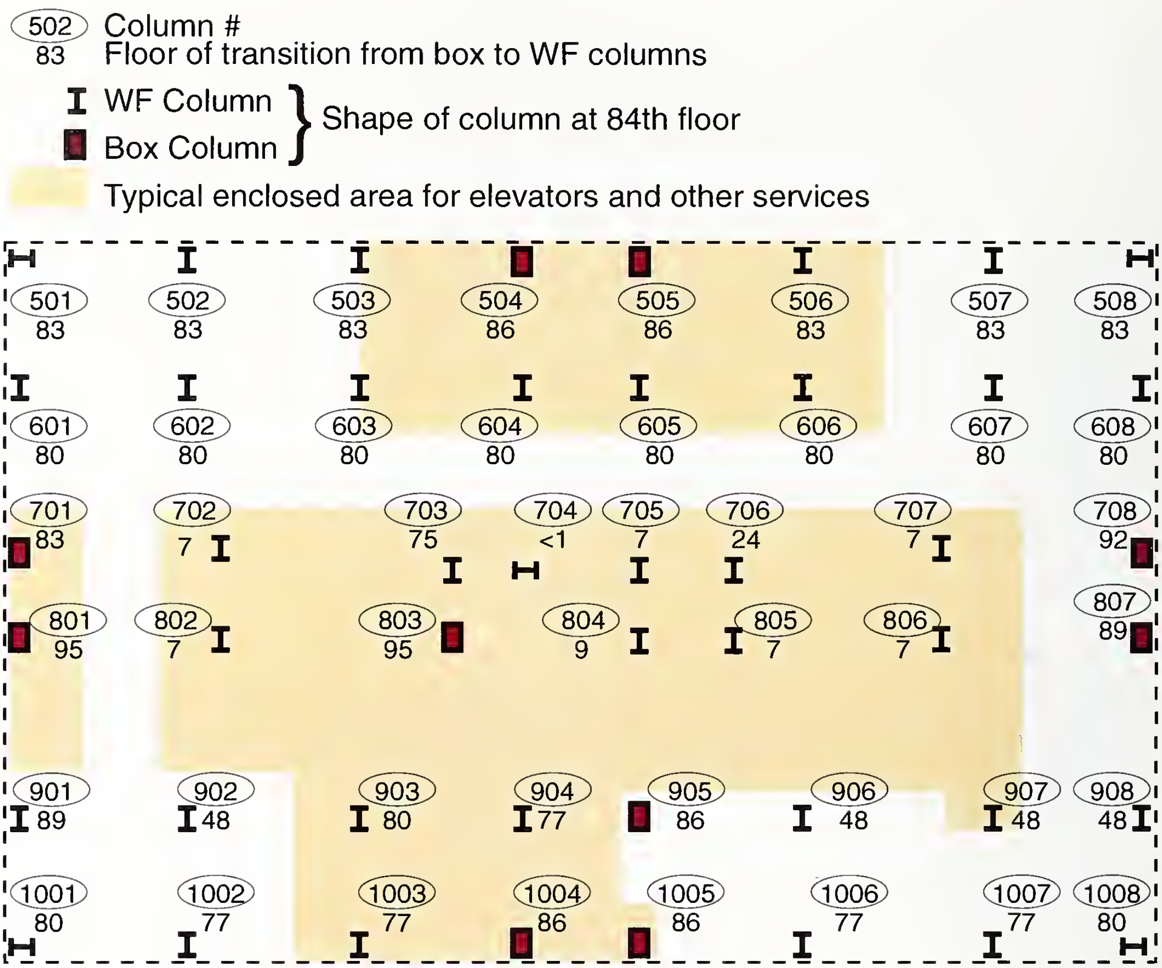
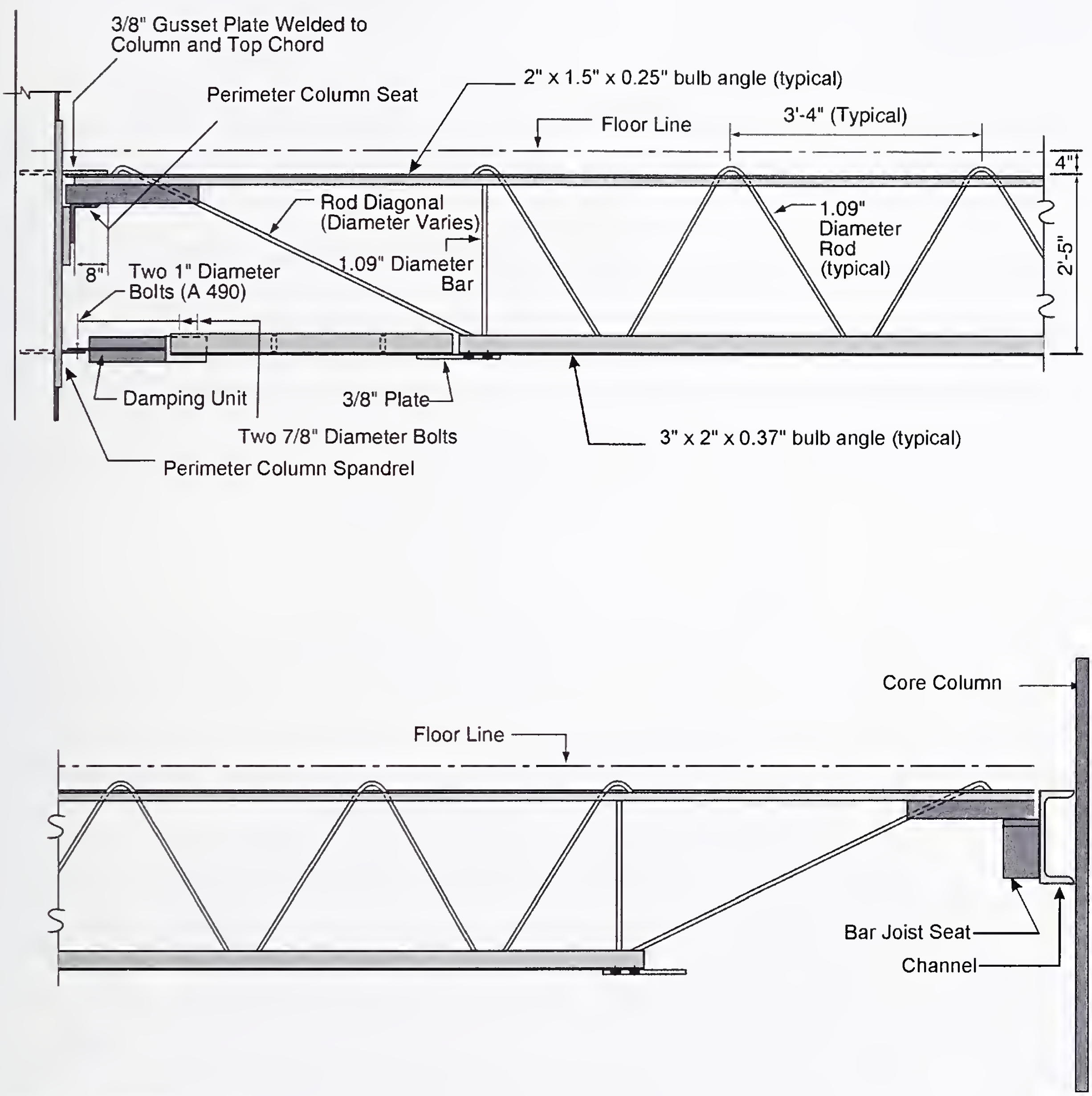


Figure E–7. Core column layout in WTC towers.

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.

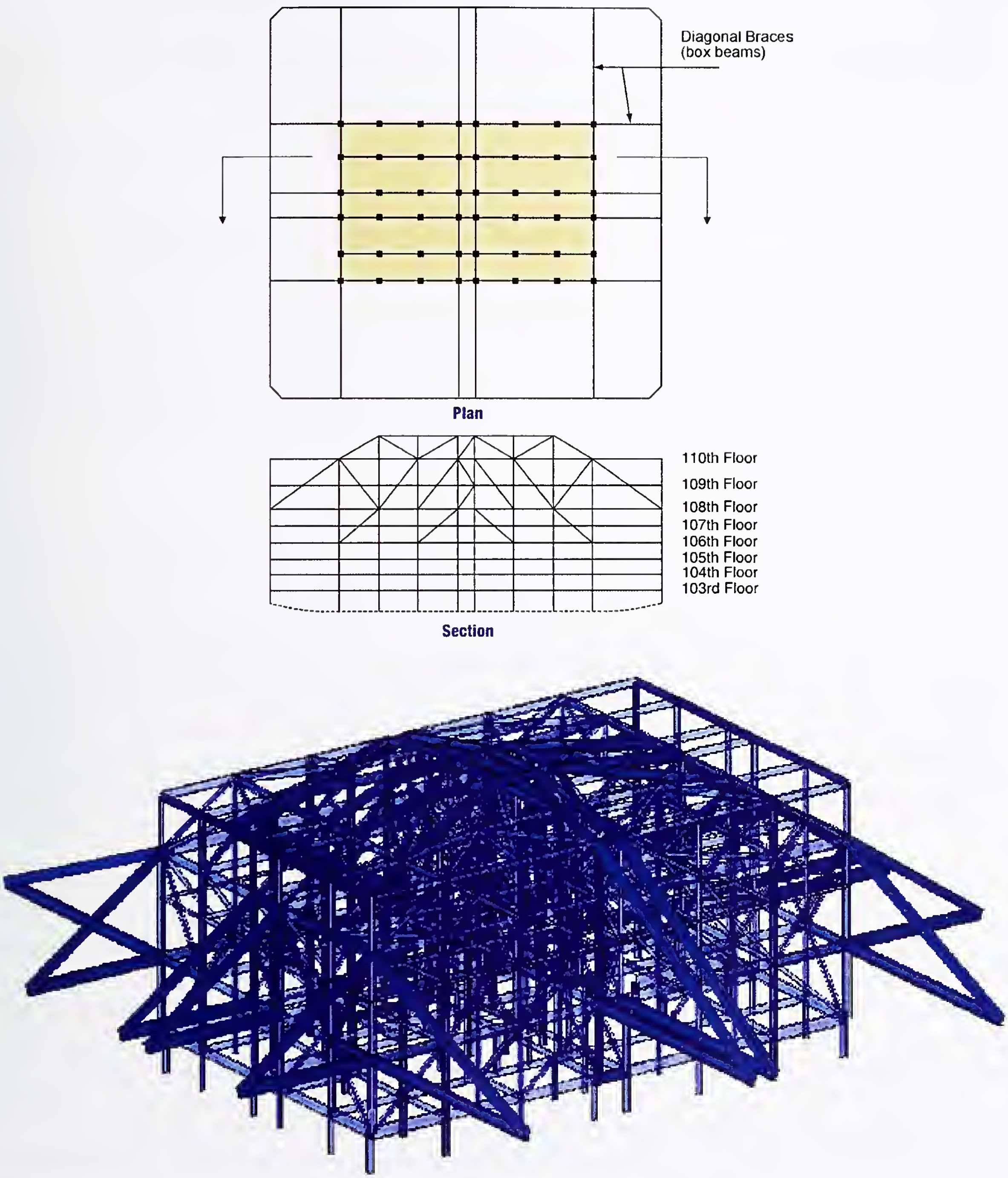
Perimeter columns for floors 107 to 110 also differed from the lower floors, and were alternating small tube columns or wide flange columns, with the wide flange columns supporting the floor system.

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.

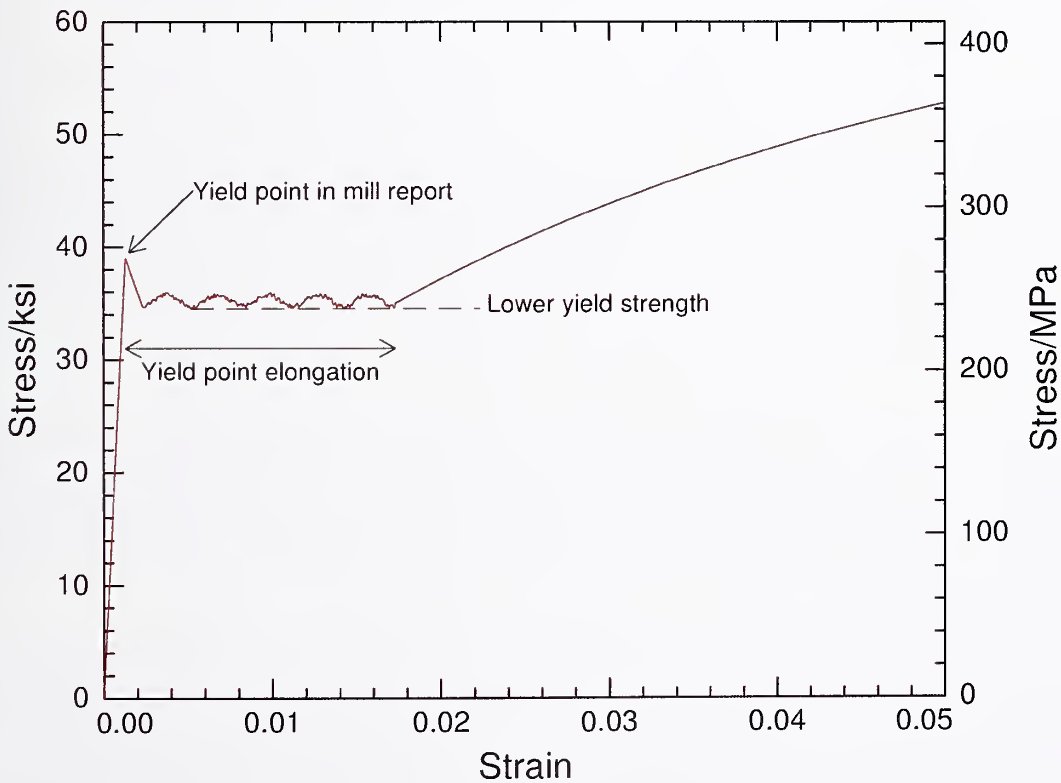


Figure E–10. Schematic diagram of the stress-strain behavior of a structural steel.

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 min (ksi)	TS min (ksi)	TS max (ksi)	Elong min %	C max %	Mn max %	Si max %	Ni %	Cr max %	V max %	Cu min %	P max %	S max %	Other
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	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		

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.
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

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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E.7.2 Private Communications

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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White, James. 1969a. Memo from SHCR to R. Monti, PONYA, documenting use of heat-treated steel above (PC&F) and below (PDM) the 9th floor splice. Contains statement that plate 3 (inside web) was fabricated from domestic steel, while plates 1, 2, and 4 (flange, outside web, and spandrel) are imported steel. Also contains table that shows where ASTM 441-modified and WEL-TEN grades were used, by thickness and yield strength. 28 pages. July 28.

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White, James. 1968b. Memo from SHCR to R. M. Monti, Port Authority, approving use of Bethlehem V60, V65, and V75 steels as specified for PC&F. January 4.

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White, James. 1967b. Memo from SHCR to R. M. Monti, PONYA, approving Ni-containing A 441-modified steel. Has two-page specification for Yawata A 441-modified. October 18.

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: $t \le 0.75$ 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: $t \le 0.75$ 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

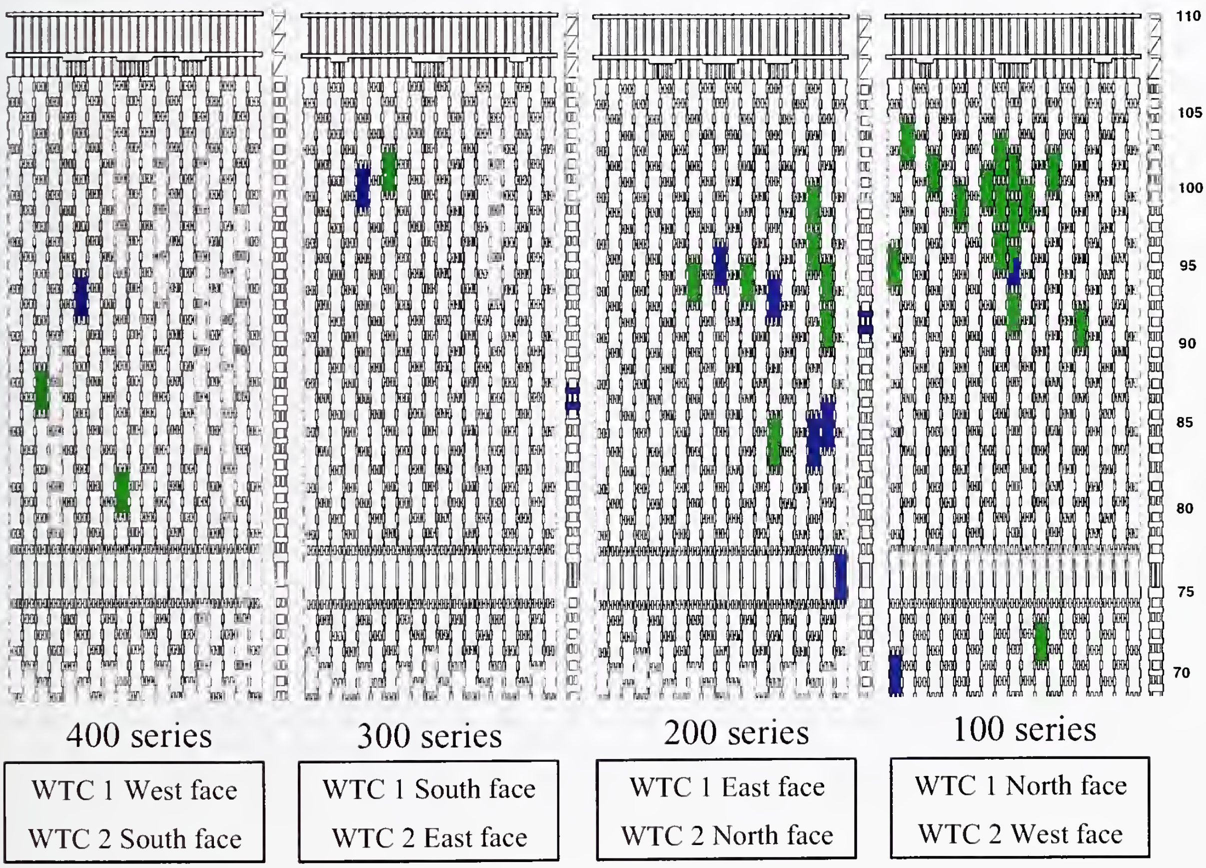


Figure F–2. Location of the exterior panels recovered from the top third of WTC 1 and 2.

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)

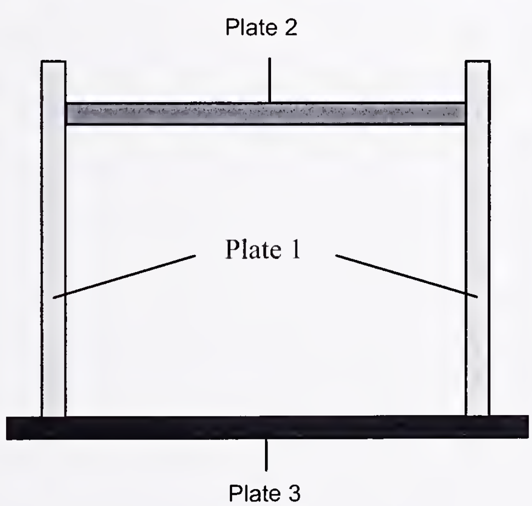


Figure F–5. (a) Example of stamping placed on flange indicating the column type (120), and (b) schematic indicating the various plates corresponding to Table F–5.

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

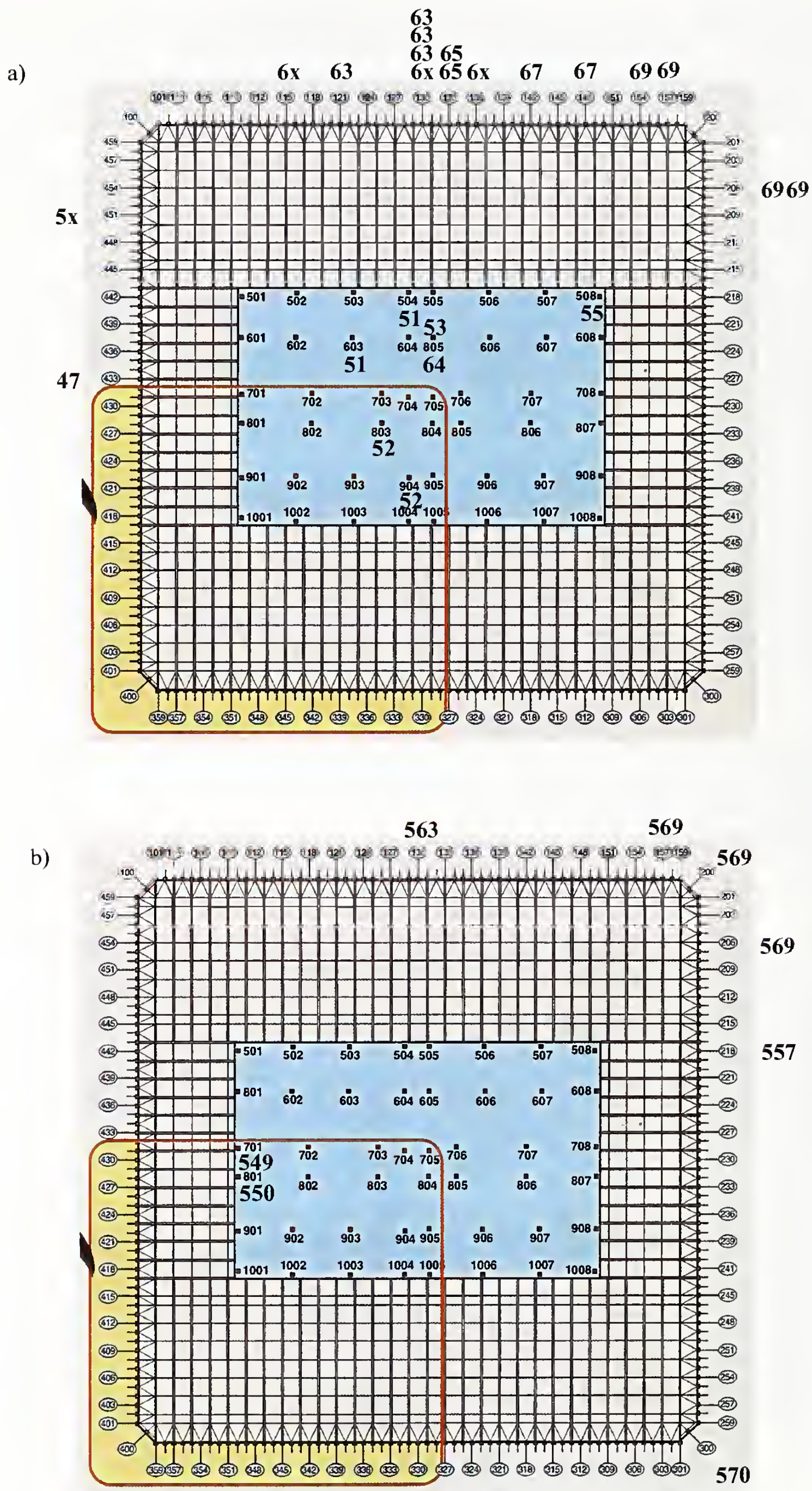
a)

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.

a)

b)



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			If WTC 2			Observed			Confirmed identification	Full Identification
					Column 1	Column 2	Column 3	Column 1	Column 2	Column 3	Column 1	Column 2	Column 3		
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 NAME	Markings	Column Line	Floors	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
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 eliminated
		233	94 - 97	120	120	120	120	120	120				
		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 eliminated
		206	92 - 95	120	120	120	120	120	120				
		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.

- <u>K-2</u>: 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.

- <u>M-2:</u> 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.

- <u>M-10a</u>: 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.

- <u>M-28</u>: 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.

- <u>M-30</u>: 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.

- <u>N-1</u>: 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.

- <u>N-7</u>: 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.

- <u>N-12</u>: 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.

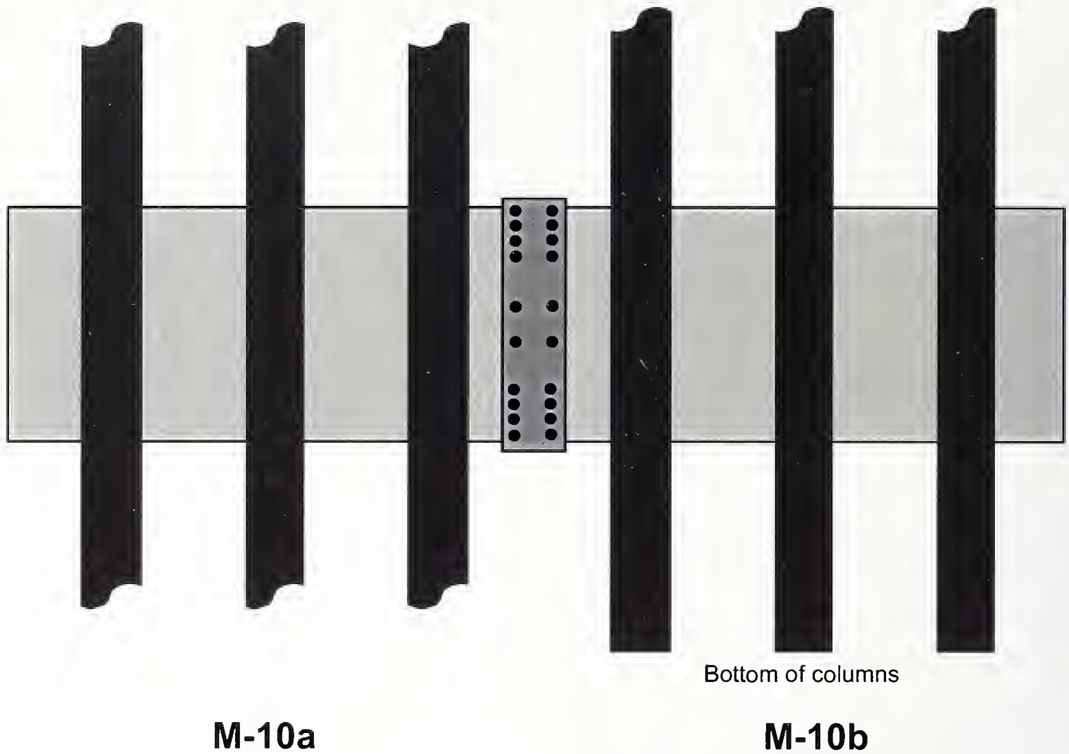


Figure F–8. Schematic showing the sample M-10 as two separate exterior column panels, M-10a and M-10b.

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

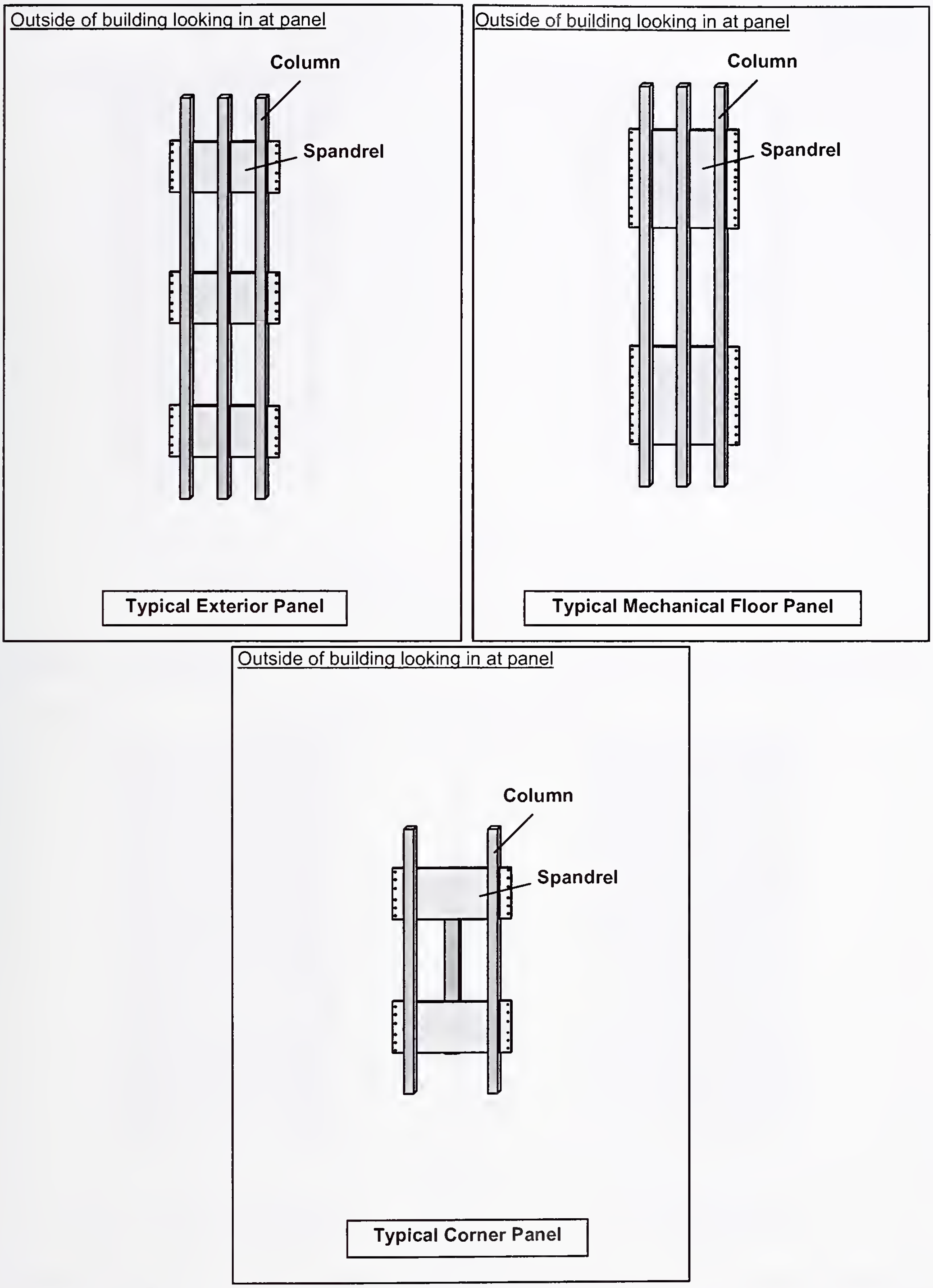


Figure F–9. Schematics displaying the various types of exterior column panels.

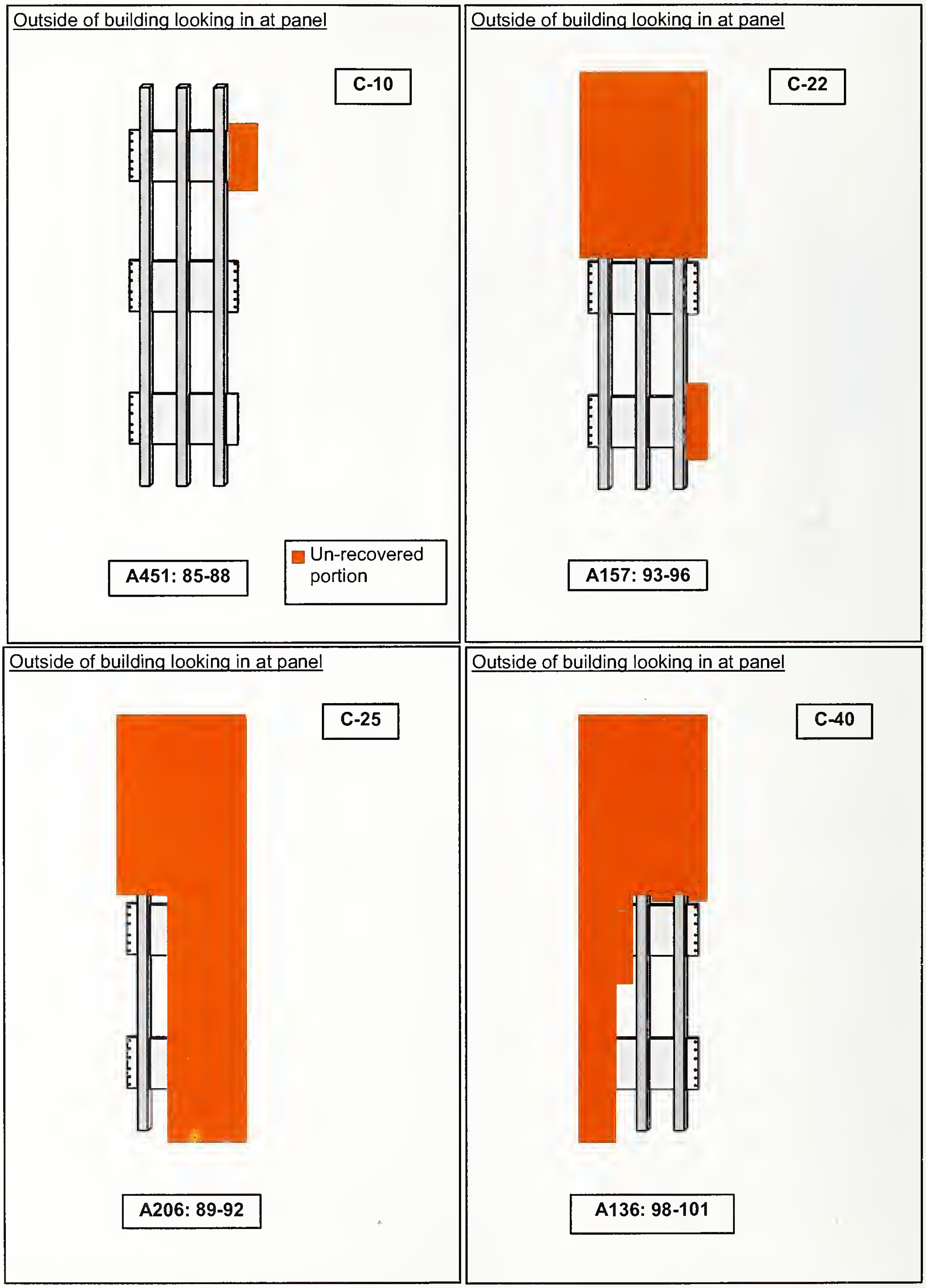


Figure F–10. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 1.

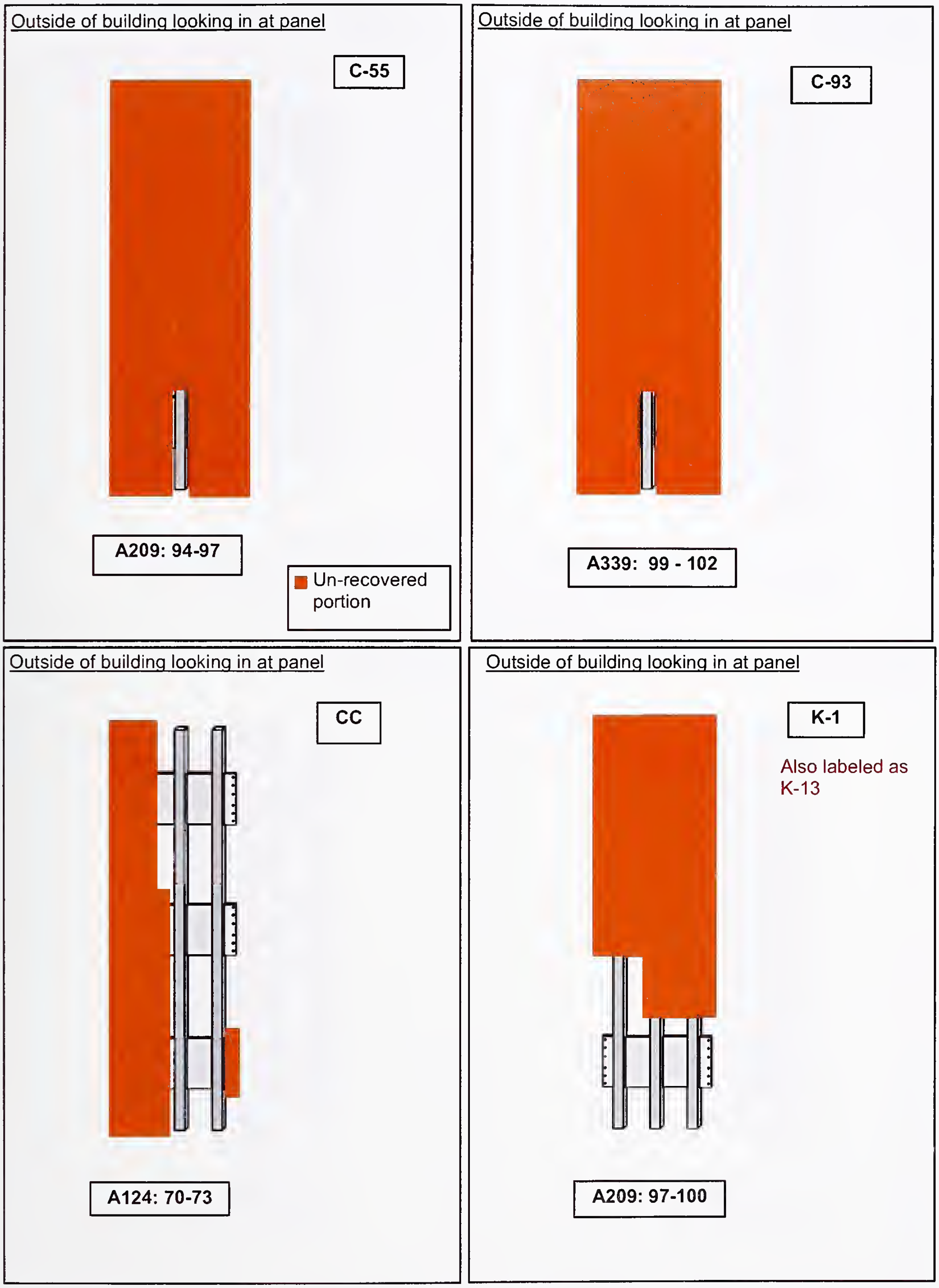


Figure F–10. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 1 (continued).

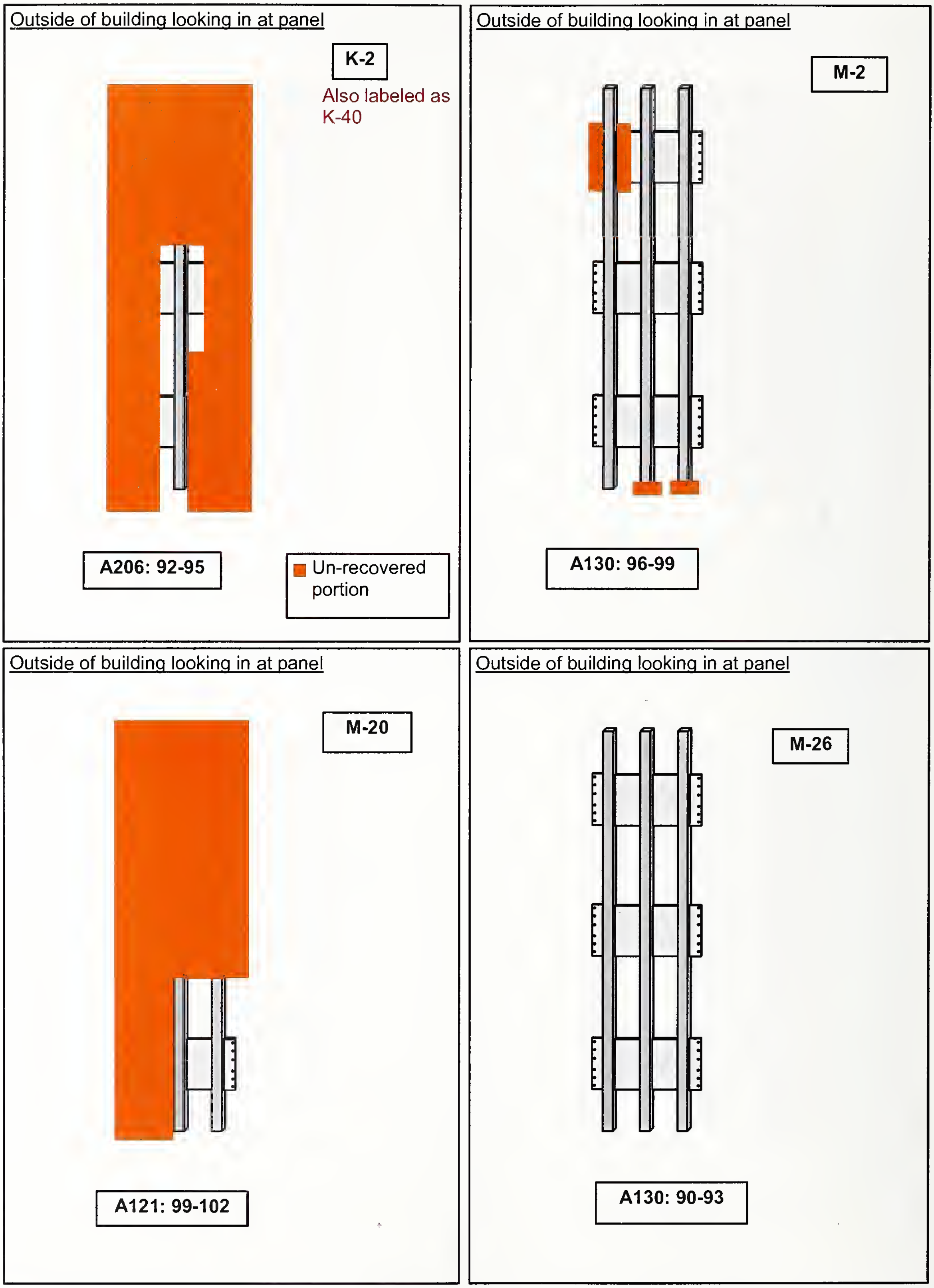


Figure F–10. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 1 (continued).

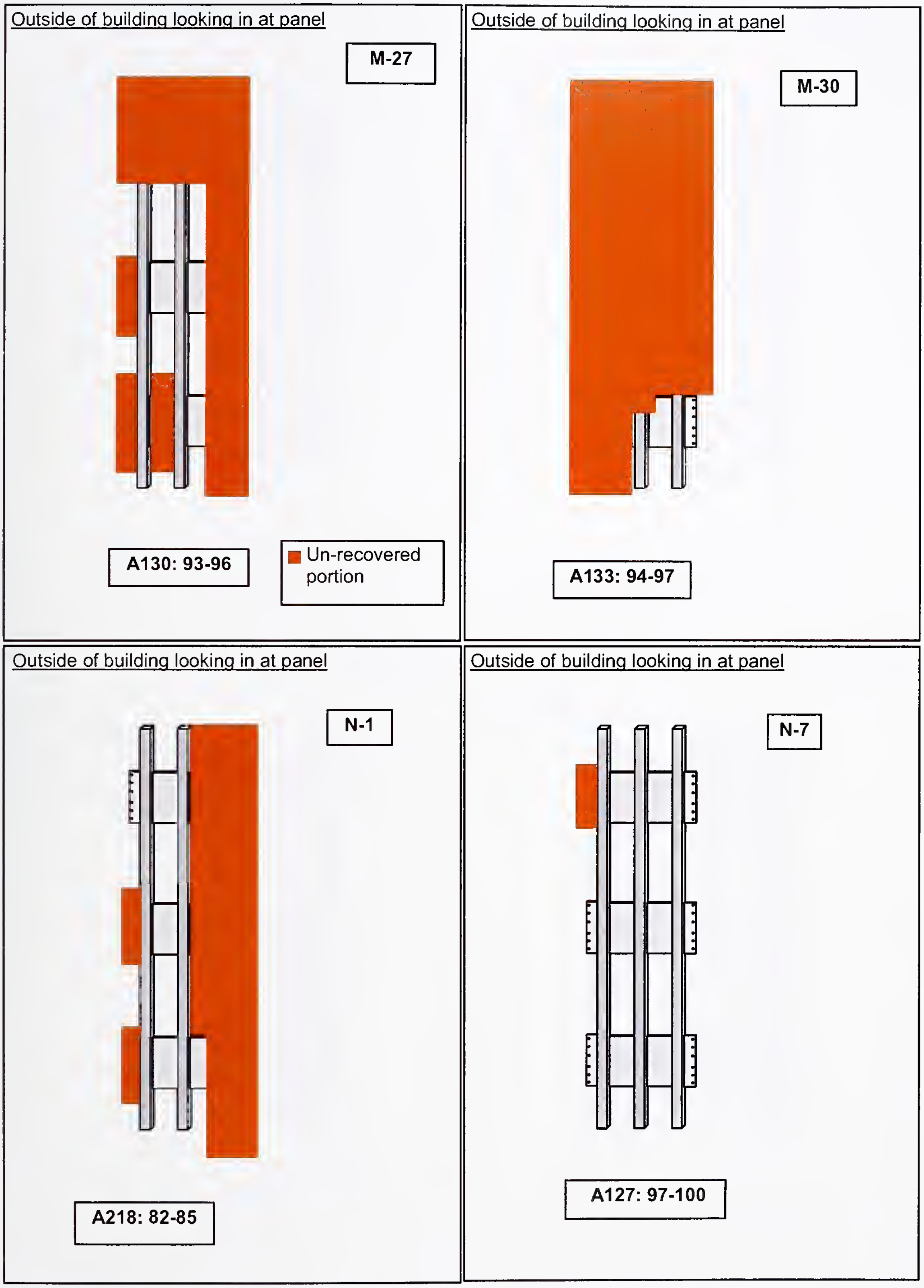


Figure F–10. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 1 (continued).

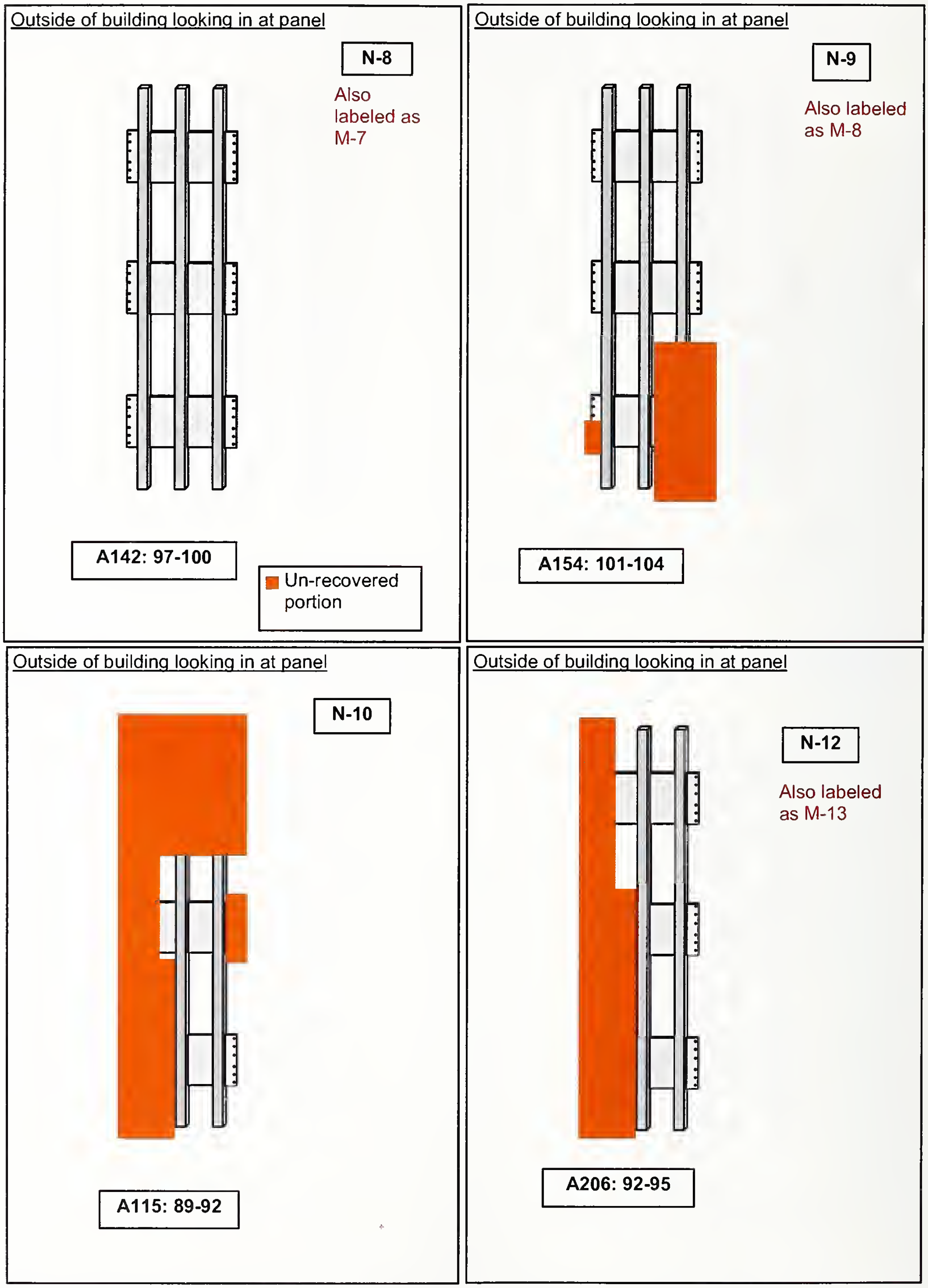


Figure F–10. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 1 (continued).

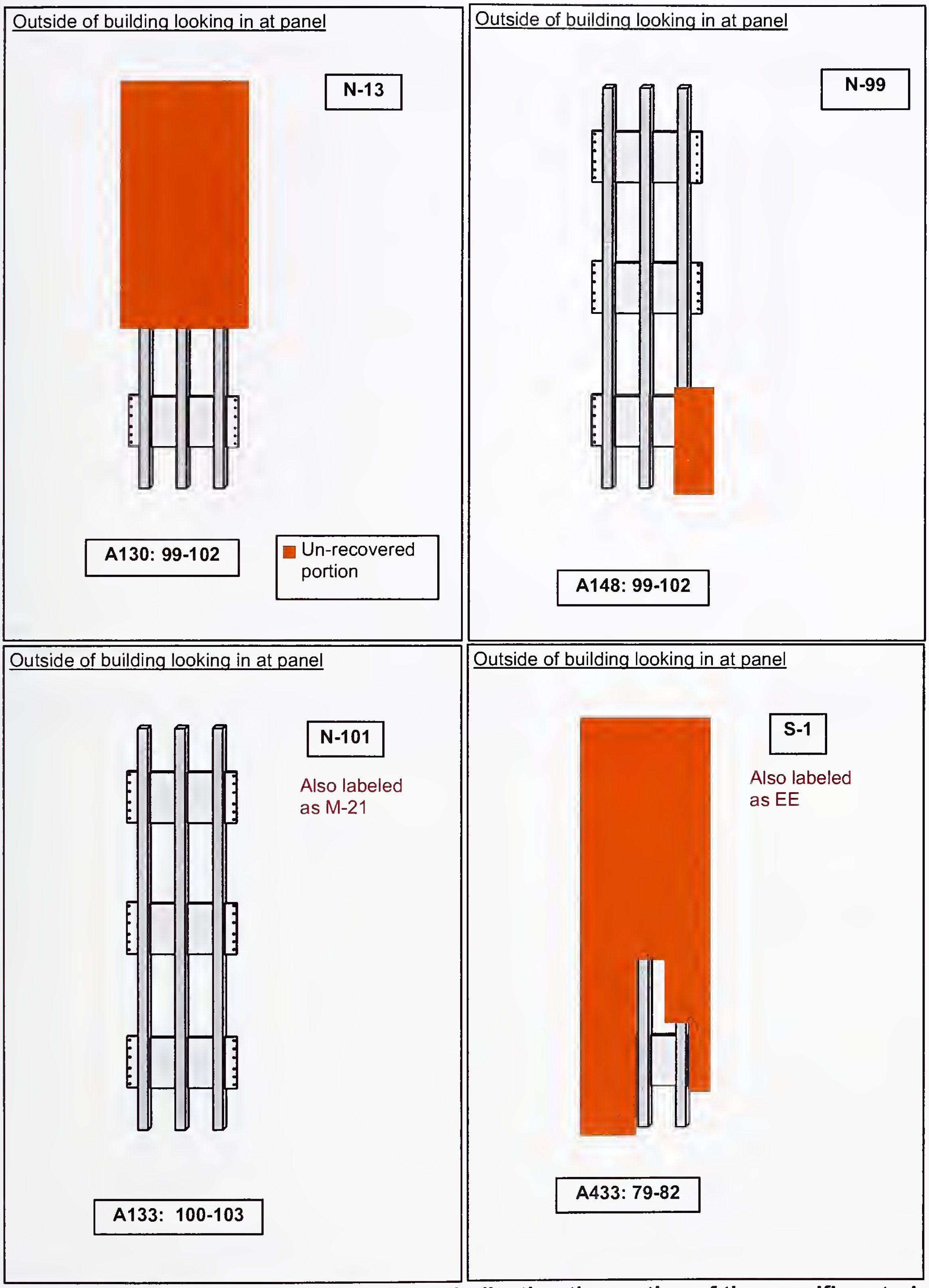


Figure F–10. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 1 (continued).

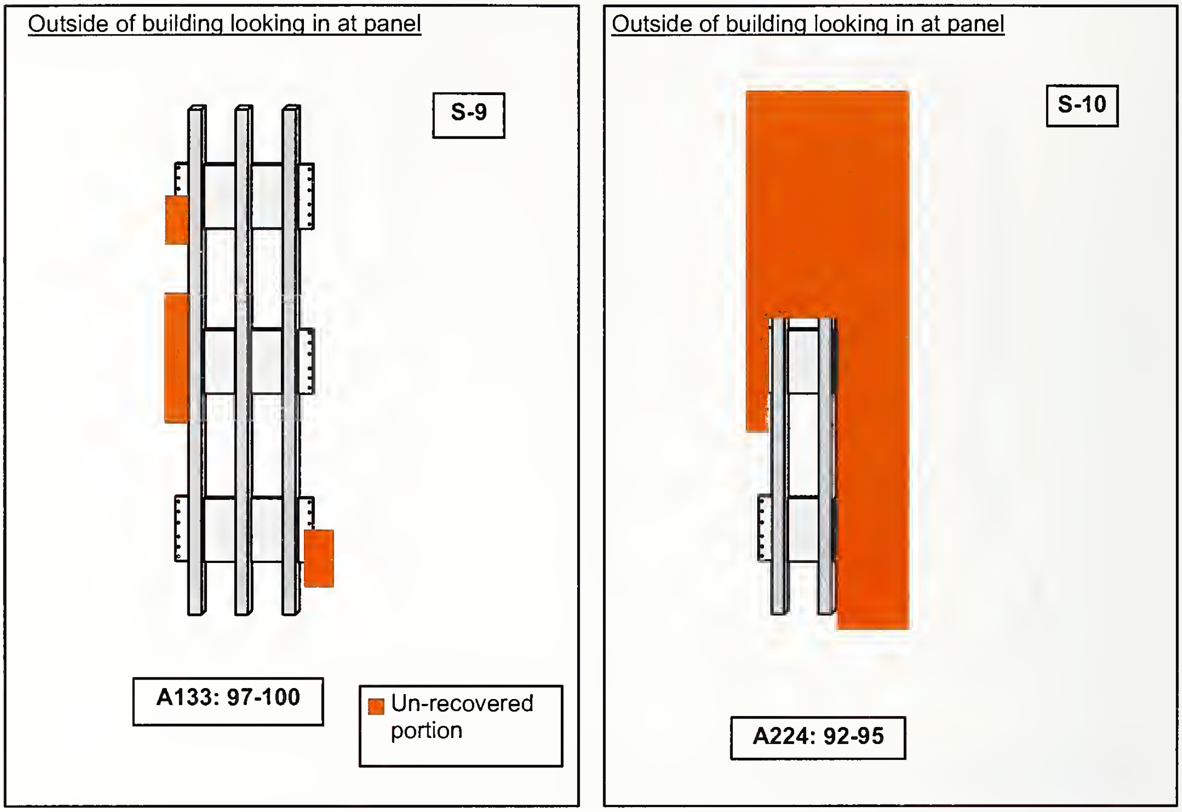


Figure F–10. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 1 (continued).

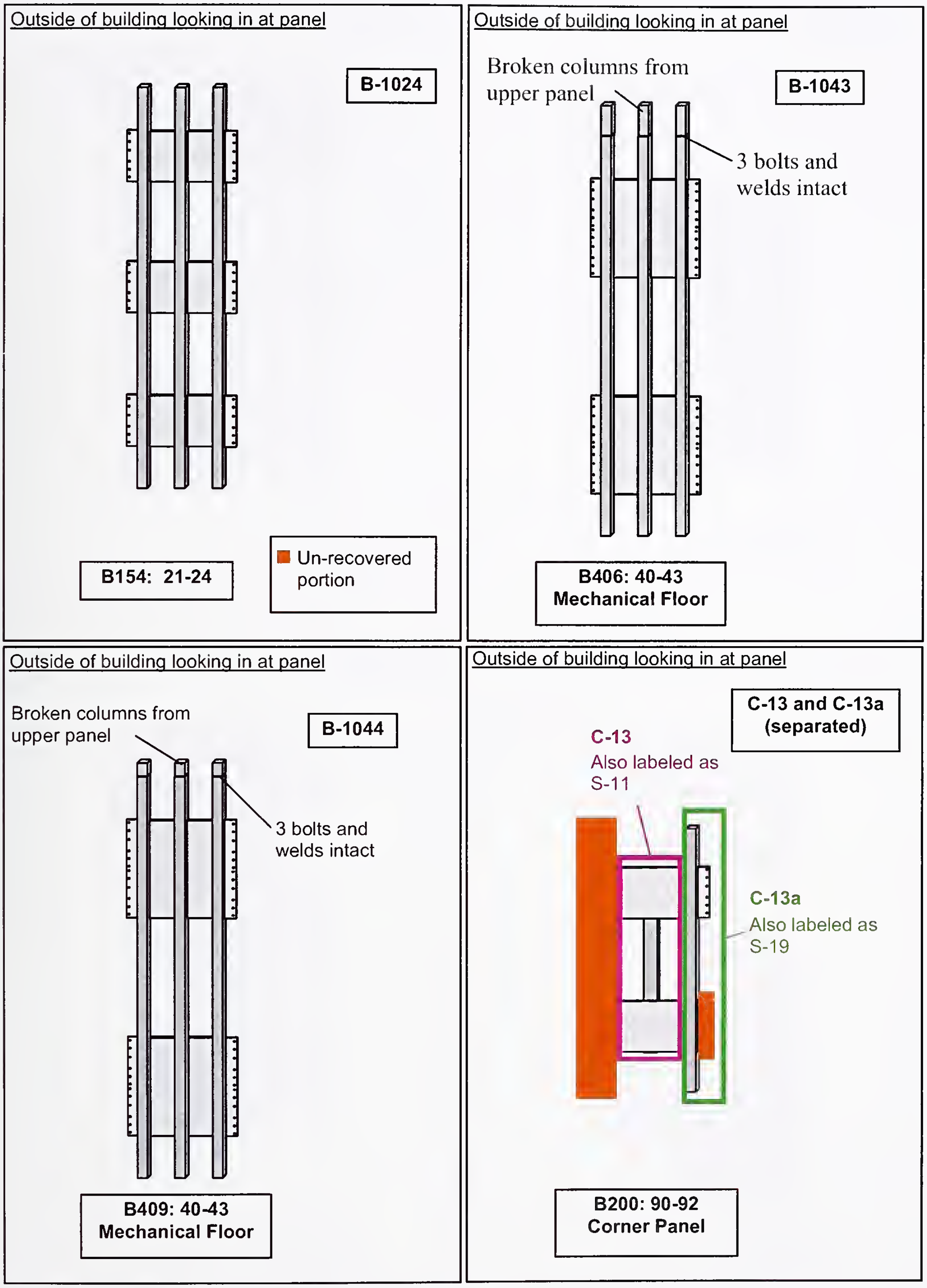


Figure F–11. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 2.

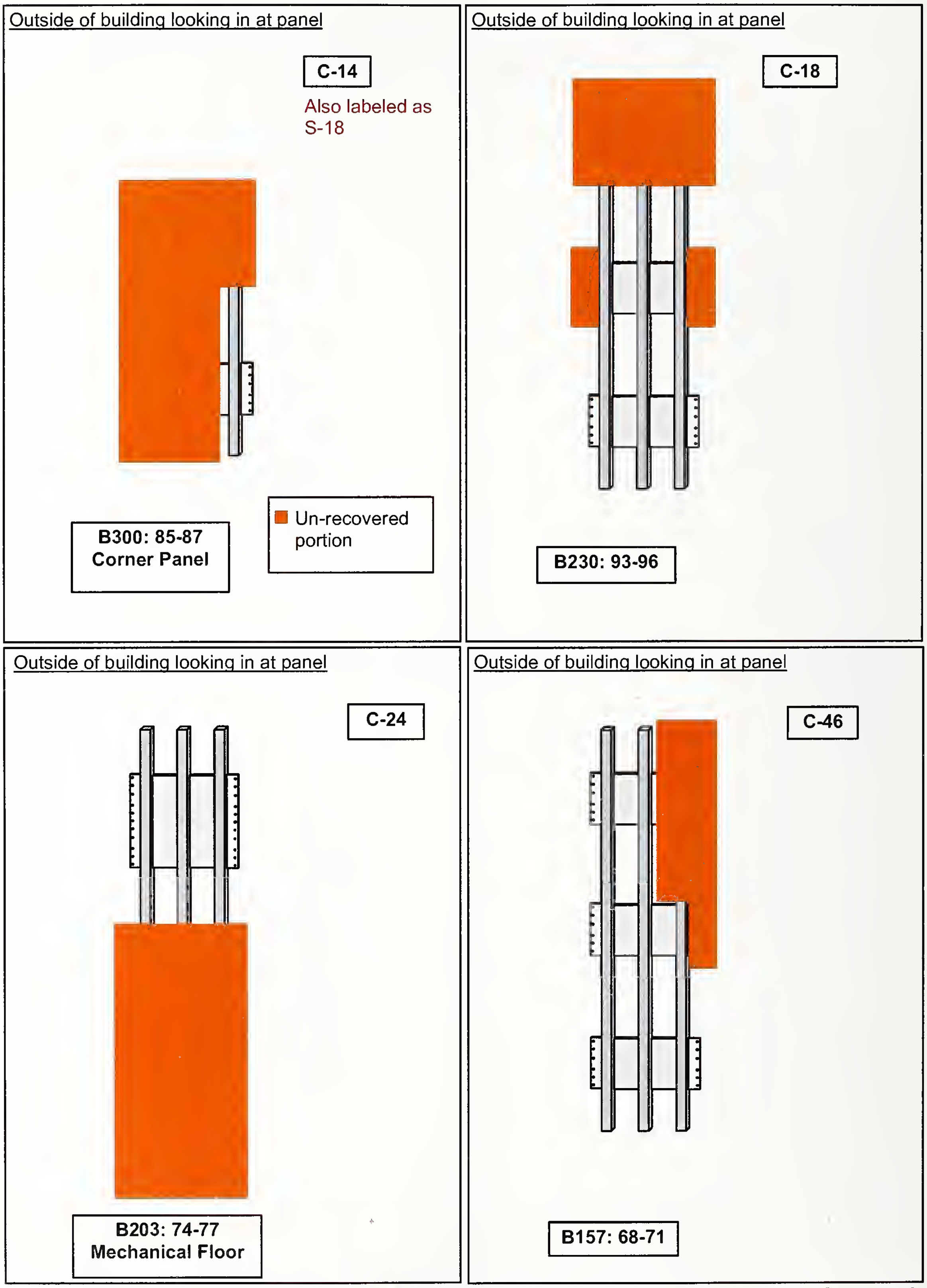


Figure F–11. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 2 (continued).

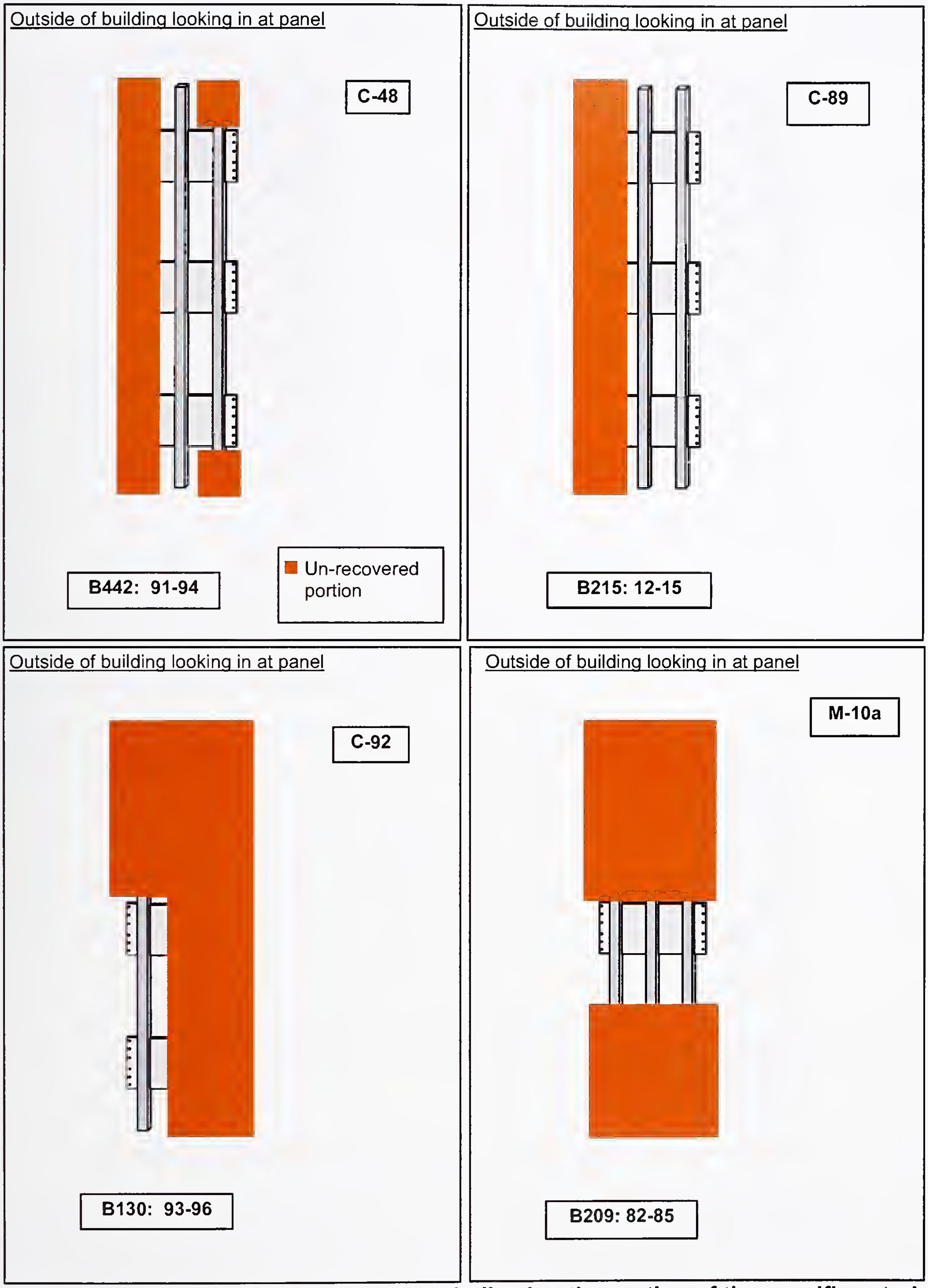


Figure F–11. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 2 (continued).

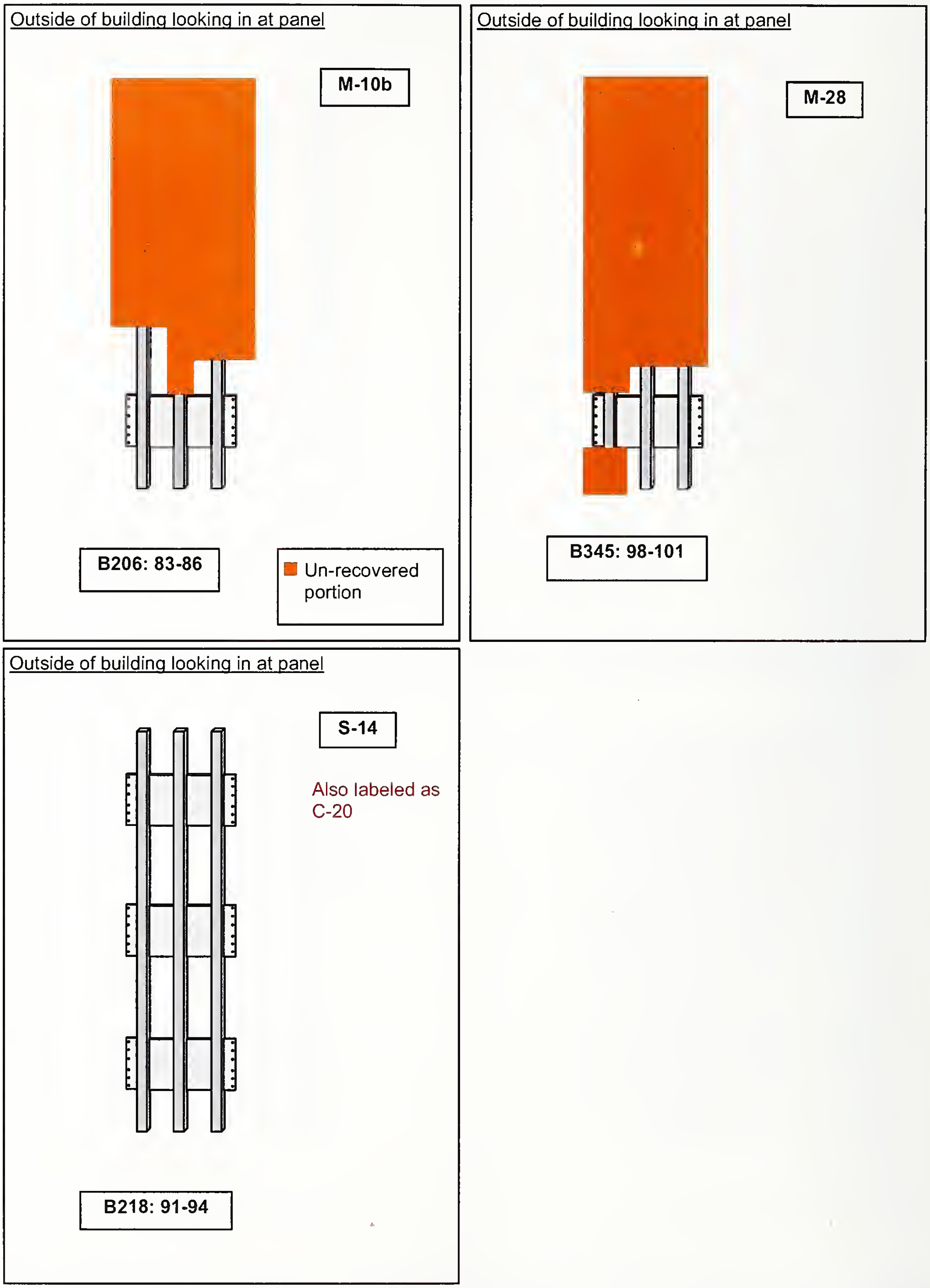


Figure F–11. Exterior column panel maps indicating the portion of the specific exterior column panel section recovered from WTC 2 (continued).

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.

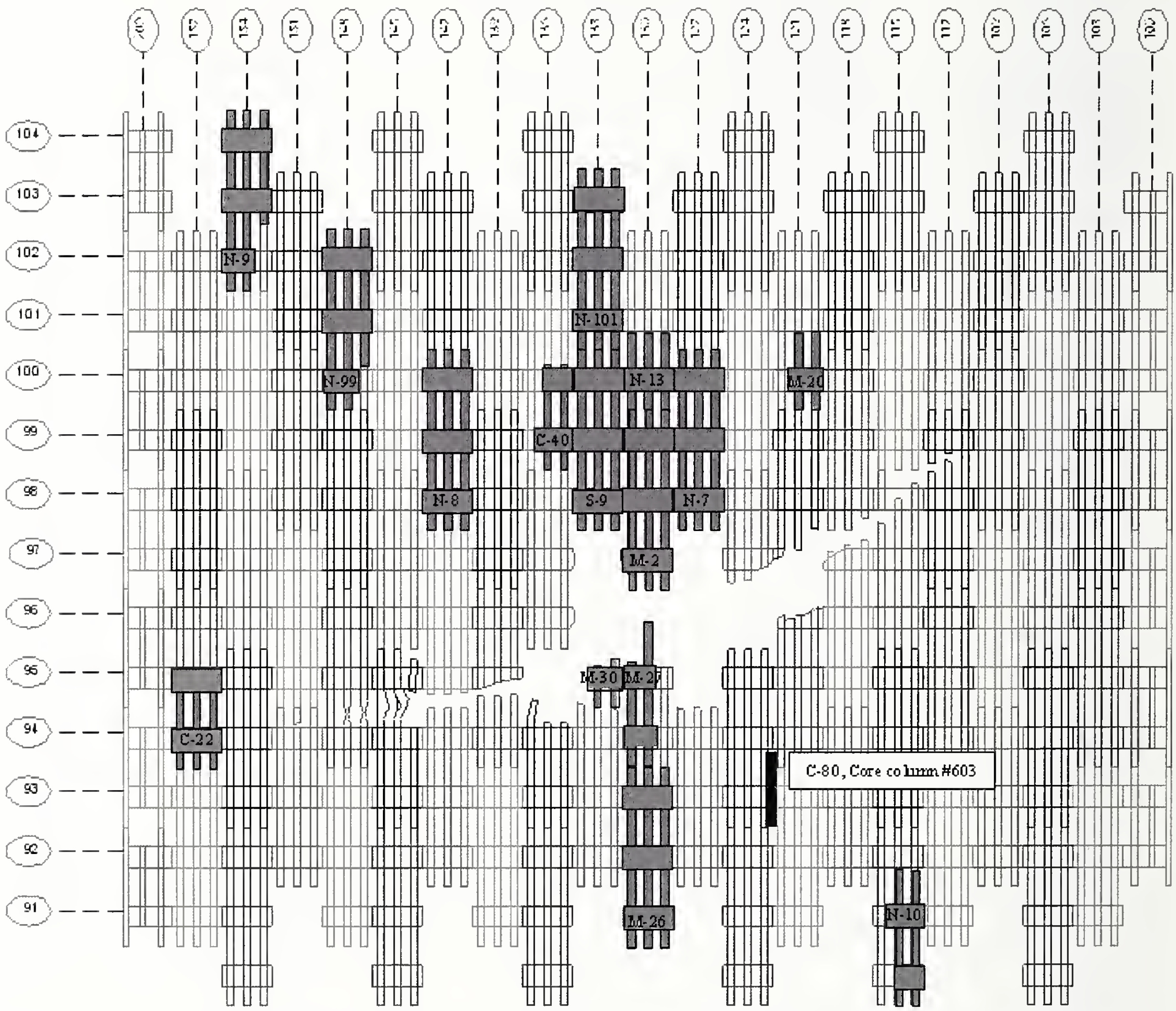


Figure F–12. Interpreted column damage, from photographic evidence, to WTC 1, with overlay of samples in NIST's possession.

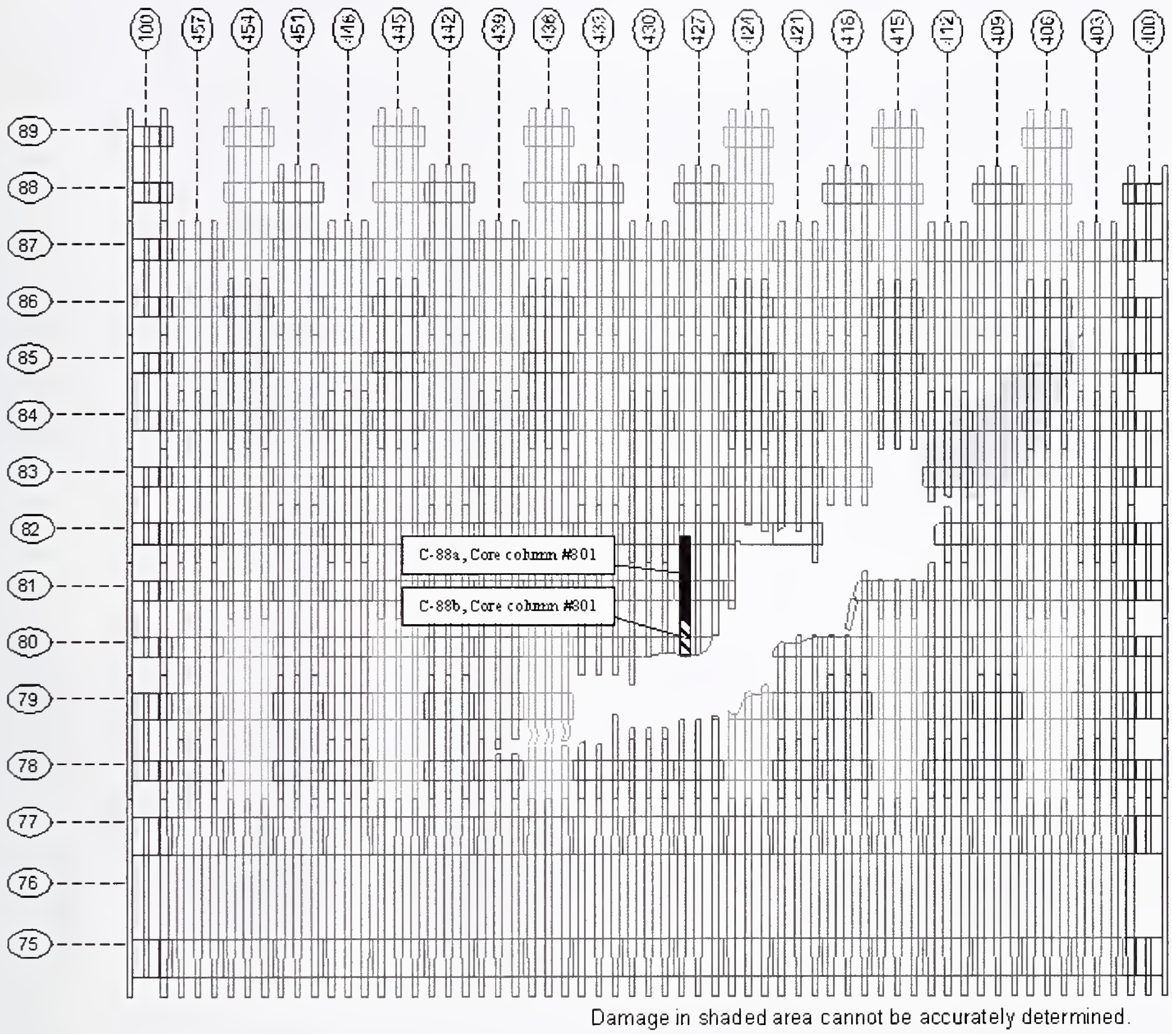


Figure F–13. Interpreted column damage, from photographic evidence, to WTC 2, with overlay of samples in NIST's possession.

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		Panel Type	COLUMN 1		COLUMN 2		COLUMN 3		LOWER SPANDREL		MIDDLE SPANDREL		UPPER SPANDREL	
			Lower	Upper		Column Type	FY (ksi)	Column Type	FY (ksi)	Column Type	FY (ksi)	Thickness (in)	FY (ksi)	Thickness (in)	FY (ksi)	Thickness (in)	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	504 & 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.

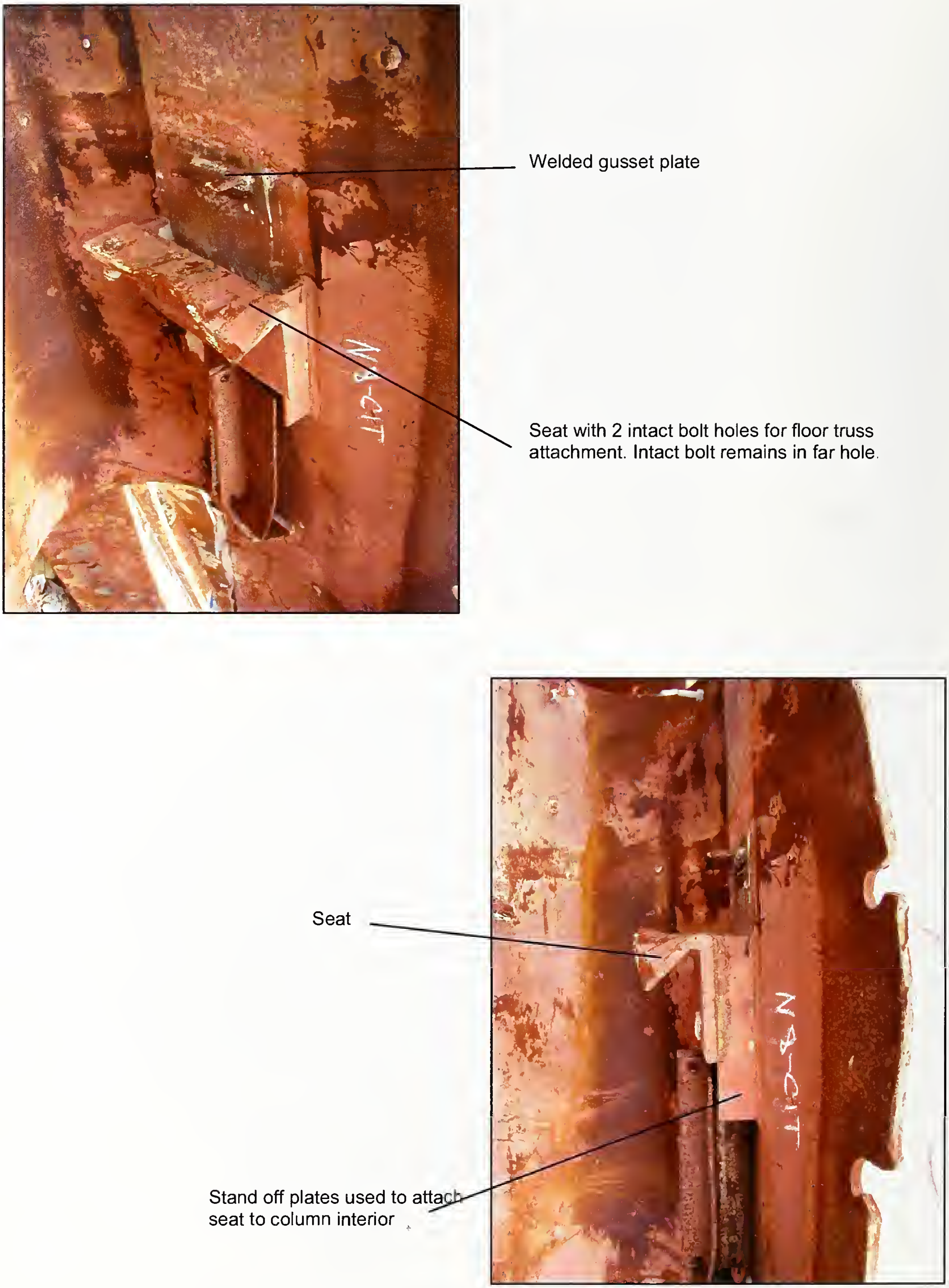

Figure 1–2. Floor truss seats shown from sample N-8.

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.

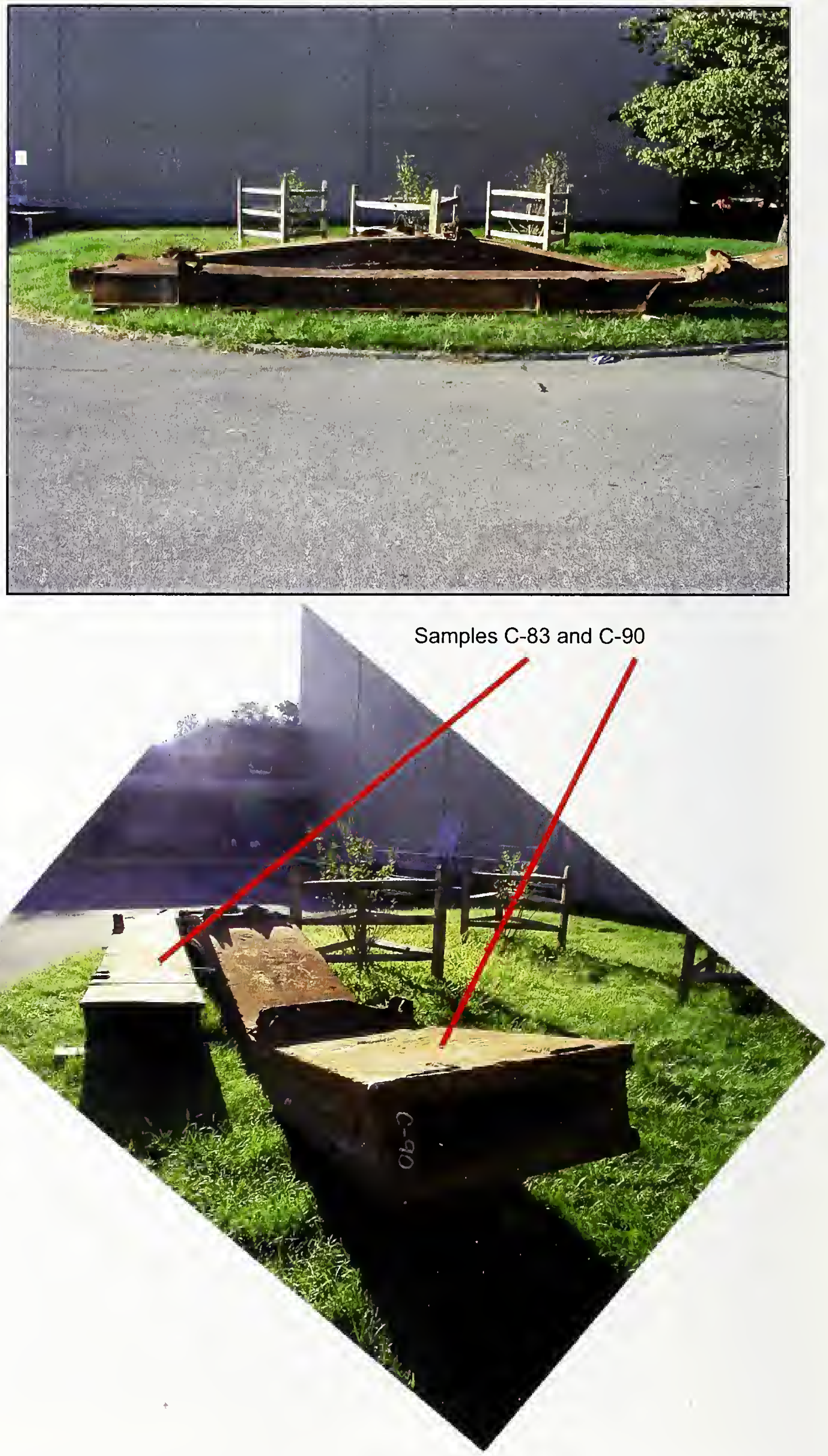

Figure 1–7. Recovered rectangular built up box sections used as core columns.

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.

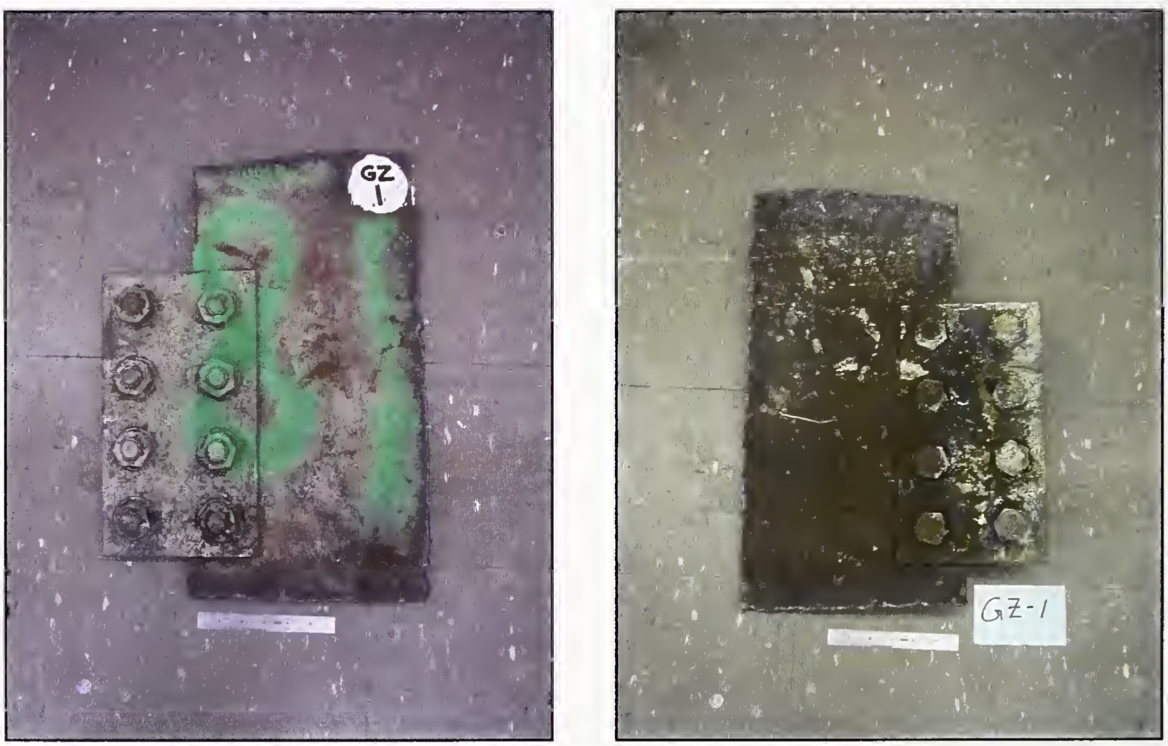

Figure 1–12. Coupons removed in the field from WTC 5; shown is sample GZ-1.

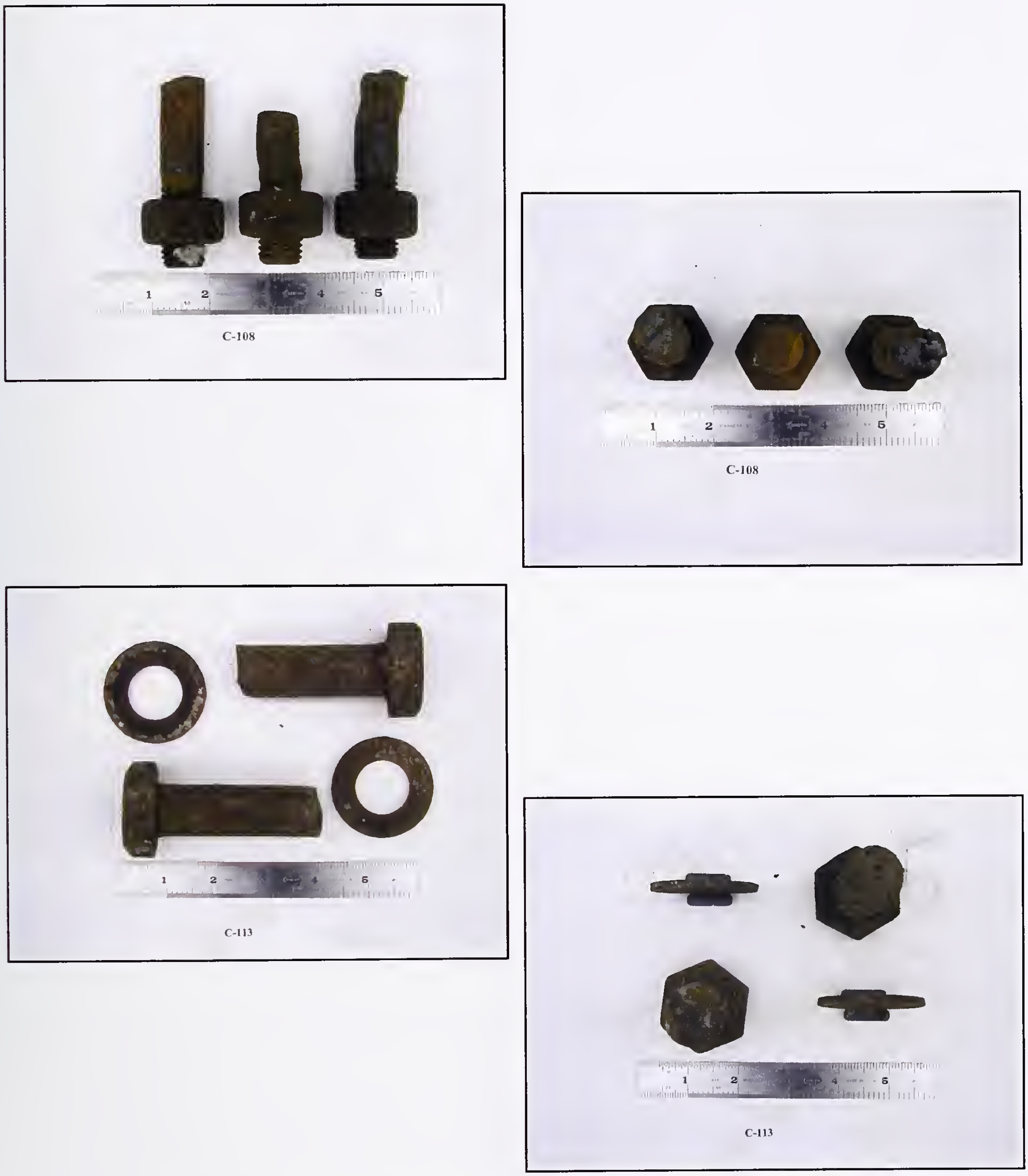

Figure 1–13. Examples of recovered bolts from various samples.

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.