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Overview of the Structural Design of World Trade Center 1, 2, and 7 Buildings

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Abstract: This paper summarizes the primary structural systems that comprised World Trade Center 1, World Trade Center 2, and World Trade Center 7, which were destroyed in the terrorist attacks of September 11, 2001. There were four major structural subsystems in the towers: the exterior walls, the core, the floor system, and the hat truss. The major structural systems within WTC 7 were the foundation, exterior moment frames, floor system, interior columns, and column transfer trusses and girders. At the time of design and construction, the World Trade Center (WTC) towers were innovative in many ways, and resulted in a tremendous increase of open-plan commercial office space in downtown Manhattan. As the first of four papers, this paper summarizes the structural and passive fire protection features of each building, and focuses on the structural systems which played a critical role in the outcome of the attacks of September 11, 2001. Three companion papers address the effects of aircraft impact damage on the WTC towers and debris damage on WTC 7, the effects of fire on the three buildings, and how these events contributed to building collapse by describing the contribution of key structural systems to the overall building behavior and collapse, such as the floor systems and hat trusses in WTC 1 and WTC 2 and the floor connections around Column 79 in WTC 7.

Keywords: World Trade Center; Failure Investigations; Building Collapse; Structural Design;

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1. Introduction

On September 11, 2011, two airplanes struck the World Trade Center (WTC) towers in New York, U.S.A. Within hours, the three tallest buildings on the site were destroyed, along with damage to or destruction of numerous nearby buildings. Subsequently, the National Institute of Standards and Technology (NIST) conducted a technical investigation of the WTC towers and building 7 collapses and issued final reports in 2005 and 2008 [1,2]. This paper, part of a series in this Special Issue, describes the structural systems and passive fire protection of the 110 story twin towers (WTC 1 and WTC 2) [3] and the 47 story WTC 7 building [4].

This paper focuses on aspects of the structural design of the WTC buildings which played a critical role in the outcome of the attacks of September 11, 2001, such as the floor system in WTC 1 and WTC 2 and the floor connections around Column 79 in WTC 7. The structural systems of each building are described to provide a foundation for understanding the effects of the aircraft impact, growth and spread of fires, and ultimately, the collapse of each building. These events are described in the three companion papers [5,6,7].

2. The Site

By 2001, the WTC complex had become an integral part of lower Manhattan. It was composed of seven buildings (here referred to as WTC 1 through WTC 7) on a site toward the southwest tip of Manhattan Island (Figure 1 and Figure 2). The two towers, WTC 1 (North Tower) and WTC 2 (South Tower), completed by early 1970's, were each 110 stories high, dwarfing the other skyscrapers in lower Manhattan. WTC 7 was a 47-story office building on land owned by the Port Authority of New York and New Jersey (PANYNJ) across Vesey Street on the north side of the Plaza complex. Built over the Con Edison substation serving the WTC complex, it was completed in 1987.

Below the 11 western acres of the site, underneath a large portion of the Plaza and WTC 1, WTC 2, WTC 3, and WTC 6, was a 6-story underground structure. The structure was surrounded by a wall that extended from ground level down 21.3 m (70 ft) to bedrock. Holding back the waters of the Hudson River, this wall enabled rapid excavation for the foundation and continued to keep groundwater from flooding the underground levels.

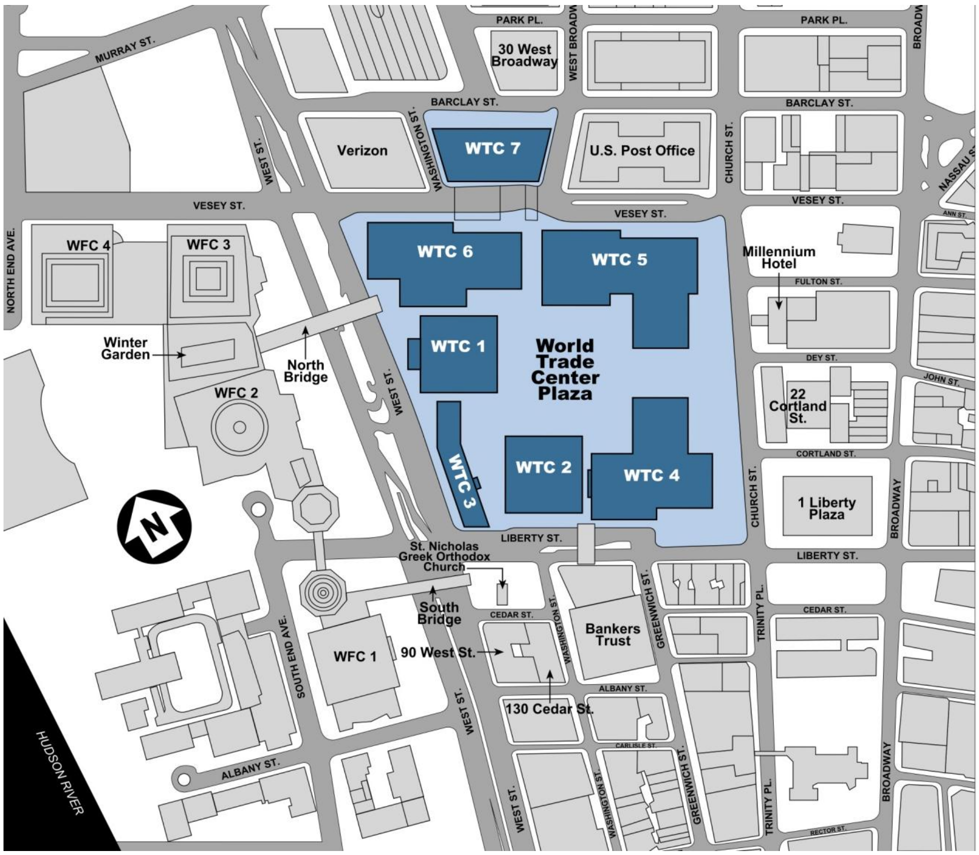

Figure 1. The World Trade Center site in lower Manhattan [1].

Figure 2. Lower Manhattan and the World Trade Center site viewed from the west [8].

3. World Trade Center 1 and 2 Structural System

3.1. Overview of WTC 1 and 2 (WTC Towers)

In any collapse investigation, the greater the degree of damage, the more difficult it is to reconstruct the condition of the building at the time of the attacks. Review of records not destroyed in the building collapse, interviews with hundreds of people involved in the design, operation, tenancy, and maintenance of the building, and analysis of photographs taken prior to and during the events of September 11, 2001 aided the investigation team.[†] To the best of our knowledge, the material presented herein provides the best description of the structural systems of the towers prior to the attacks on September 11, 2001. The following paragraphs are a summary of the historical context for the architectural and structural design, as well as a description of the primary structural systems in each tower.

In 1962, the firm of Minoru Yamasaki & Associates was hired to perform an architectural design for a grand economic engine for lower Manhattan, which was first unveiled in 1964. The team involved Emory Roth & Sons, P.C., as the architect of record. The structural engineers were Worthington, Skilling, Helle, and Jackson (WSHJ). (Some time after completion of the construction, Skilling, Helle, Christiansen, and Robertson, and then Leslie E. Robertson Associates (LERA) assumed that role.) Jaros, Baum & Bolles were retained as the mechanical engineers, and Joseph R. Loring & Associates were the electrical engineers. Tishman Construction Corporation was the general contractor. In 1966, the formal groundbreaking for the towers took place. Construction began in 1968, with the first occupancy in 1970. These dates establish the historical context for the building codes and the state of practice under which the complex was designed and constructed.

The two towers were the focus of the WTC complex, and were taller than any other building in the world at that time. The roof of WTC 1 was 417 m (1368 ft) above the Concourse Level, 1.8 m (6 ft) taller than WTC 2, and supported a 110 m (360 ft) tall antenna mast for

[†] Sources included The Port Authority of New York and New Jersey and its contractors and consultants; Silverstein Properties and its contractors and consultants; the City of New York and its departments; the manufacturers and fabricators of the building components; the companies that insured the WTC towers; and building tenants.

television and radio transmission. WTC 2 was also designed to support an antenna which was never built. The footprint of each tower was a square, about 64 m (210 ft) on a side (approximately an acre), with beveled corners. Internally, each floor was a square, about 63 m (206 ft) on a side.

Tenant floors in the towers were intended to offer an open-plan office space, virtually uninterrupted by columns or walls. This called for an innovative structural design, to minimize the total mass of 110 stories and yet be strong enough to support the huge building with all its furnishings and people. Structural engineers refer to the building weight as the dead load; the people and furnishings are called the live load. Collectively, these are referred to as gravity loads.

The buildings also needed to resist lateral loads and excessive swaying, principally from hurricane force winds that periodically strike the eastern seaboard of the United States. Extensive and detailed studies were conducted in wind tunnels, instead of relying on prescriptive building code requirements, to estimate the wind loads for the design of these buildings. This approach was based on the allowance by most state and local building codes for alternative designs and construction if evidence were presented that ensured equivalent performance.

An additional load, not required by any building codes, but stated by PANYNJ to have been considered in the design of the towers, was the impact of a commercial airliner. Documents obtained from PANYNJ indicated that the impact of a Boeing 707 or DC 8 aircraft flying at a speed of 268 m/s (600 mph) was analyzed during the design stage of the WTC towers. The life safety considerations following such impact were also addressed. One document stated that "…Analysis indicates that such collision would result in only local damage which could not cause collapse or substantial damage to the building and would not endanger the lives and safety of occupants not in the immediate area of impact." No other documentary evidence on the aircraft impact analysis was available to review the criteria and methods used in the analysis of the aircraft impact into the WTC towers or to provide details on the ability of the WTC towers to withstand such impacts [9].

There were four major structural subsystems in the towers: the exterior wall, the core, the floor system, and the hat truss. The structural design team incorporated a framed-tube concept for the exterior structural system. Columns supporting the building were located both along the external faces and within the core. The core also contained the elevators, stairwells, and utility shafts. The dense array of columns along the building perimeter resisted lateral wind loads, while also supporting the gravity loads about equally with the core columns. The floor system provided stiffness and stability to the framed-tube system in addition to supporting the floor loads.

The first major structural subsystem was the exterior framing, which was a vertical square tube that consisted of 236 narrow columns, 59 on each face from the 10th floor to the 107th floor (Figure 3). There were fewer, wider-spaced columns below the 7th floor to accommodate doorways. There were also columns on alternate stories at each of the beveled corners, but these did not carry gravity loads. Each column on floor 10 to 107 was fabricated by welding four steel plates to form a tall box, nominally 0.36 m (14 in) on a side. The space between the steel columns was 0.66 m (26 in), with a framed plate glass window in each gap. Adjacent columns were connected at each floor by steel spandrel plates, 1.3 m (52 in) high. The upper parts of the buildings had less wind load and building mass to support. Thus, on higher floors, the thickness of the steel plates making up the columns decreased, becoming as thin as 6 mm (¼ in) near the top down from as thick as 76 mm (3 in) at the lower floors. There were 10 grades of steel used for the columns and spandrels, with yield strengths ranging from 248 MPa (36 ksi) to 690 MPa (100 ksi). The grade of steel used in each location was dictated by the calculated stresses due to the gravity and wind loads. All the exterior columns and spandrels were prefabricated into welded panels, three stories tall and three columns wide. The panels, each numbered to identify its location in the tower, were then bolted to adjacent units to form the walls (Figure 4). Field panels were staggered so that every third panel was spliced at each floor level. The use of identically shaped prefabricated elements was an innovation that enabled rapid construction.

The second structural subsystem was a central service area, or core (Figure 3), measuring approximately 41 m by 26.5 m (135 ft by 87 ft), that extended virtually the full height of the building. The long axis of the core in WTC 1 was oriented in the east-west direction, while the long axis of the core in WTC 2 was oriented in the north-south direction. The 47 columns in this rectangular space were fabricated using primarily 248 MPa (36 ksi) and 290 MPa (42 ksi) steels and decreased in size at the higher stories. The four massive corner columns bore nearly one-fifth of the total gravity load on the core columns. The core columns were interconnected by a grid of conventional steel beams to support the core floors.

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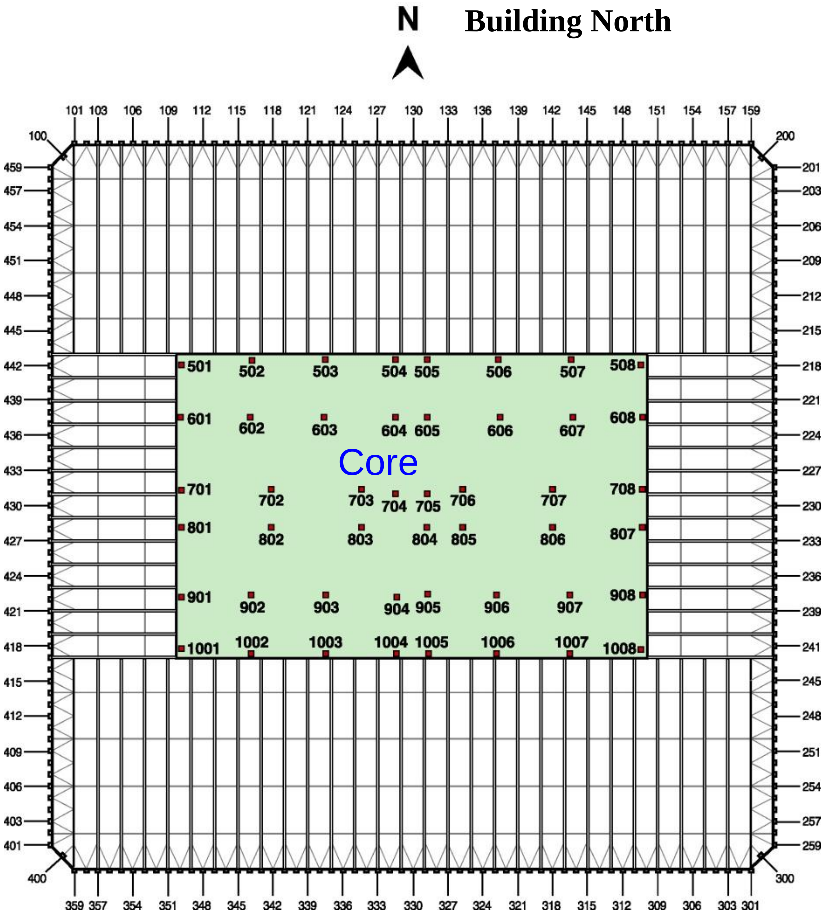

WTC 1

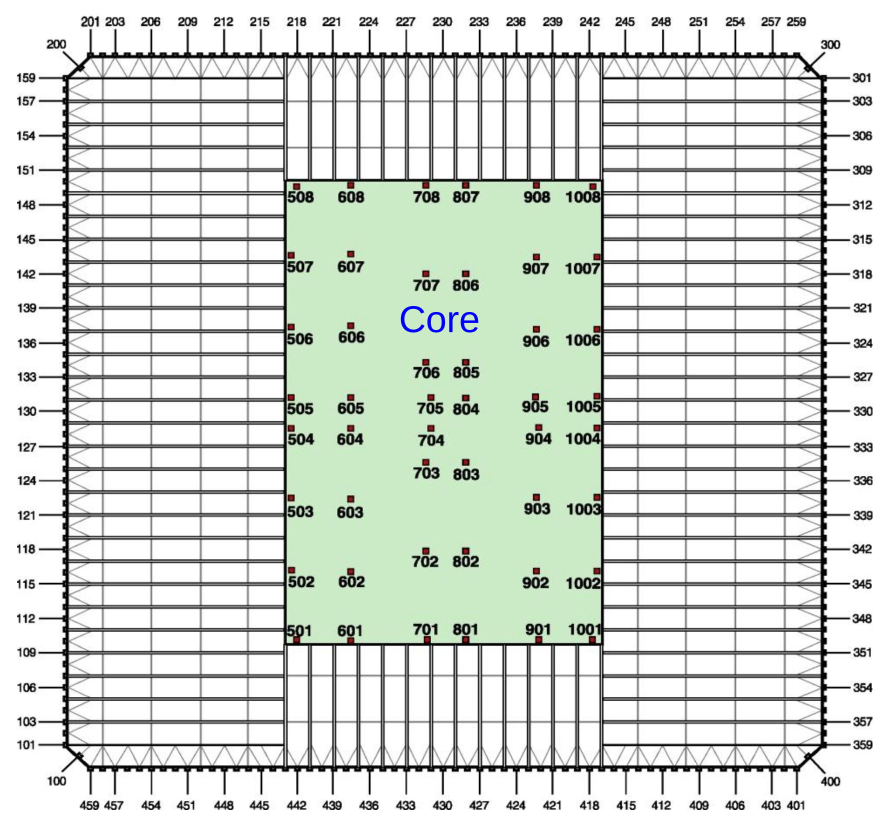

Figure 3. Typical tenant floor plans with column numbers [1].

WTC 2

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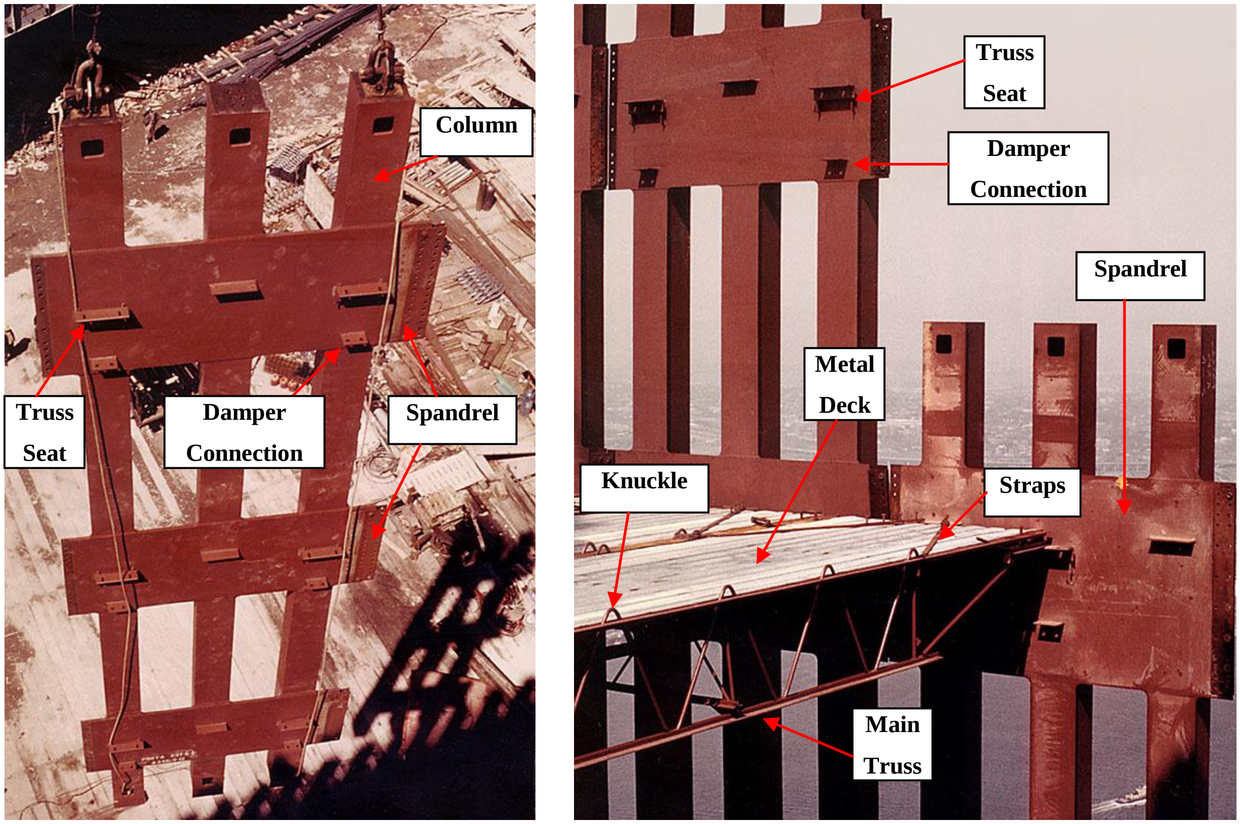

Figure 4. Perimeter column/spandrel assembly and floor structure [1].

The third major structural subsystem was the floors in the tenant spaces between the exterior walls and the core. These floors supported gravity loads, provided lateral stability to the exterior walls, and distributed wind loads among the exterior walls. With the exception of the mechanical floors (Floors 7, 8, 41, 42, 75, 76, 108, and 109) which had rolled structural steel shapes, tenant floors had truss systems. As shown in Figure 5, each tenant floor consisted of 102 mm (4 in) thick, lightweight cast-in-place concrete on a fluted steel deck. Supporting the slab was a grid of lightweight steel bar trusses. The top bends (or “knuckles”) of the main truss webs extended 76 mm (3 in) above the top chord and were embedded into the concrete floor slab. This concrete and steel assembly thus functioned as a composite unit, that is, the concrete slab acted integrally with the steel trusses to carry floor loads. Without the presence of the knuckles (or the shear studs in WTC 7), the floor slab and trusses (or beams) would have acted independently, resulting in reduced load capacity. The primary truss pairs were either 18.3 m (60 ft) or 10.7 m

(35 ft) long and were spaced at 2 m (6.7 ft). There were perpendicular bridging trusses every 4 m (13.3 ft). The floor trusses and fluted metal deck were prefabricated in panels that were typically 6.1 m (20 ft) wide and that were hoisted into position in a fashion similar to the exterior wall panels.

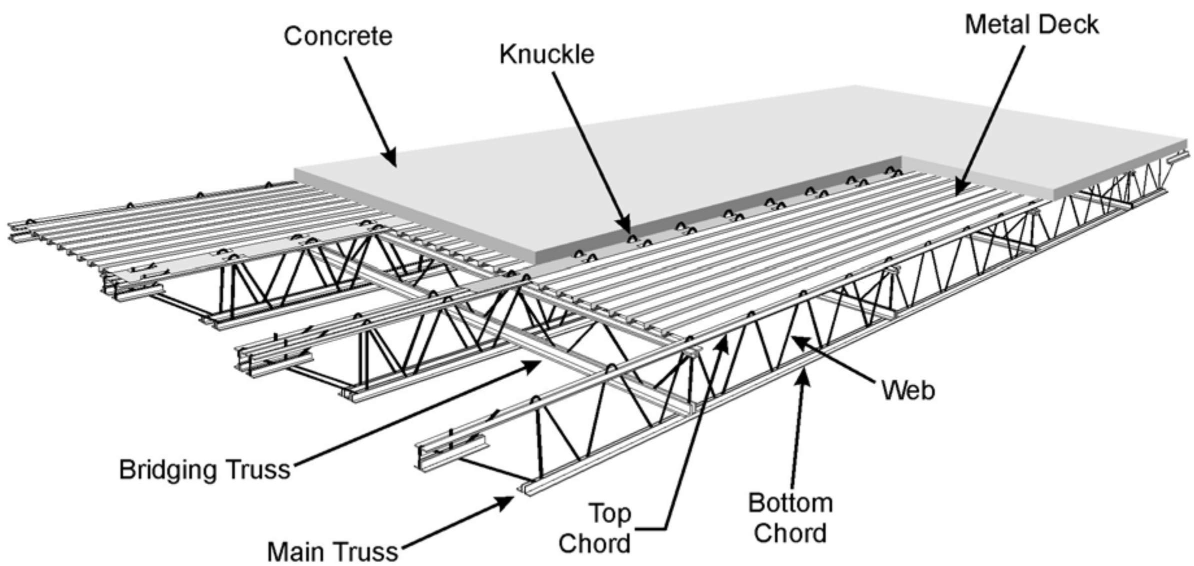

Figure 5. Schematic of composite floor truss system [1].

The bottom chords of the main trusses were connected to the spandrel plates of the exterior wall by viscoelastic dampers. Experiments on motion perception, conducted with human subjects, showed a potential for occupant discomfort when the building swayed in a strong wind. When the tower moved under wind loads, these dampers absorbed energy, reducing the sway and the vibration.

The fourth major structural subsystem was located from the 107th floor to the roof of each tower. It was a set of steel braces, collectively referred to as the “hat truss” (Figure 6). Its primary purpose was to support a tall antenna atop each tower, although only WTC 1 had a tall antenna installed. The hat truss provided additional connections among the core columns and between the core and perimeter columns, providing additional means for load redistribution.

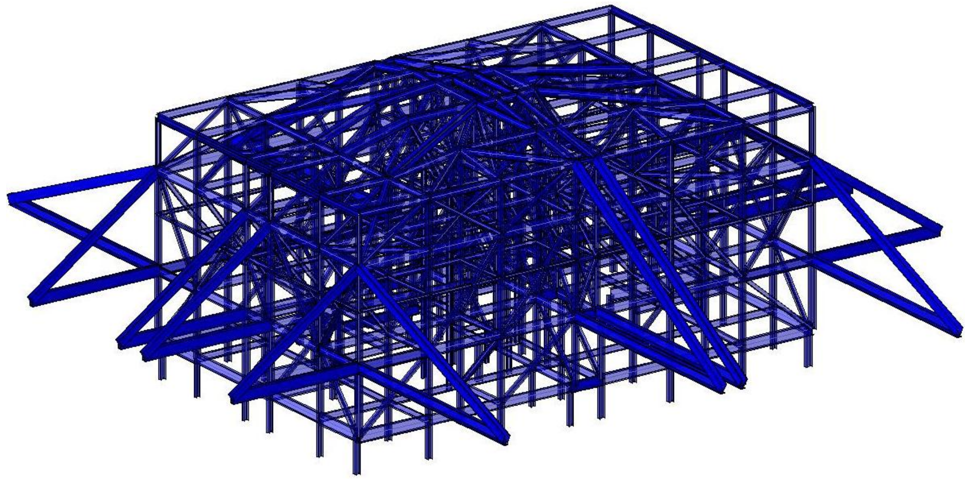

Figure 6. Computer model of hat truss framing at the roof level of the WTC towers [1].

3.2. Innovations in the World Trade Center Towers

The towers included a number of features that were considered innovative at the time of design and construction. These innovations are described in the following sections.

3.2.1 Framed Tube System

WTC 1 and WTC 2 were among the first high-rise buildings built using the framed-tube concept to provide resistance to lateral (wind) loads. A framed tube structure consists of closely spaced exterior columns tied together at each floor by deep spandrel beams, thereby creating a rigid wall-like structure around the building exterior. The behavior of the framed tube structure is hybrid, showing characteristics of both pure tube and pure frame systems.

In the framed-tube concept, the exterior frame system resists the force of the wind. The exterior columns carry a portion of the building gravity loads, and in the absence of wind, are all in compression. Under the effect of wind alone (not including gravity loads), columns on the windward side are in tension and the columns on the leeward side are in compression. The overturning moments of lateral wind loads are primarily resisted by tube action, i.e., axial shortening (compression) and elongation (tension) of the columns on all sides of the tube. The columns on the walls parallel to the wind direction are in tension on the windward side and in compression on the leeward side. The shear force from the wind loads is primarily resisted by frame action (in-plane bending of columns and spandrels) along the two faces parallel to the direction of the wind. In a framed tube system, the floor diaphragms play a key role since they carry lateral forces to the side walls of the building, thereby allowing tube action to take place. In addition, floor diaphragms provide lateral support for the stability of the columns.

3.2.2 Deep Spandrel Plates

The standard approach to construction of the framed tube would have used spandrel beams (a spandrel beam is an exterior beam that extends from column to column) to connect the columns. The towers used a band of deep plates as spandrel members to tie the perimeter columns together.

3.2.3 Uniform External Column Geometry

In a typical high-rise building, the columns would have been larger near the base of the building and would have become smaller toward the top as they bore less wind and gravity loads. However, the architectural design called for the appearance of tall, uniform columns (Figure 2). This was achieved by varying both the strength of the steels and the thickness of the plates that made up the perimeter columns.

3.2.4 Wind Tunnel and Human Perception Testing for Wind Loads

To determine the extreme wind speeds that could be expected at the top of the towers, WSHJ collected data on the wind speeds and directions recorded in the New York area over the prior 50 years. From these data, a design wind speed for the buildings was determined for a 50 year wind event, averaged over a 20-min duration at 457 m (1500 ft) above the ground. The estimated value was just under 44.7 m/s (100 mph) in all directions.

To estimate how the buildings would perform under wind loads, both during construction and upon completion, WSHJ conducted a then unique wind tunnel testing program at Colorado State University and the National Physical Laboratory in the United Kingdom. In each wind tunnel, a physical model of Lower Manhattan, including the towers, was subjected to steady and turbulent winds consistent with the estimated design wind speeds. Tests on the two-tower models showed that the wind response of each tower was significantly affected by the presence of the other tower [9].

WSHJ also conducted experiments to determine the wind-induced conditions that would be tolerated by building occupants. Breaking new ground in human perception testing, it was found that surprisingly low building accelerations caused discomfort. The test results led to changes in the building design, including stiffer perimeter columns, and the addition of viscoelastic dampers described in the next section.

3.2.5 Viscoelastic Dampers

The tower design included the first application of damping units to supplement the framed-tube in limiting wind-induced oscillations in a tall building. Each tower had about 10,000 dampers. On most truss-framed floors (tenant floors), a damper connected the lower chord of a truss to a perimeter column. A depiction of the units is shown in Figure 7. On beam-framed floors (generally the mechanical floors with heavier gravity loads), a damper connected the lower flange of a wide-flange beam (that spanned between the core and the perimeter wall) to a spandrel plate.

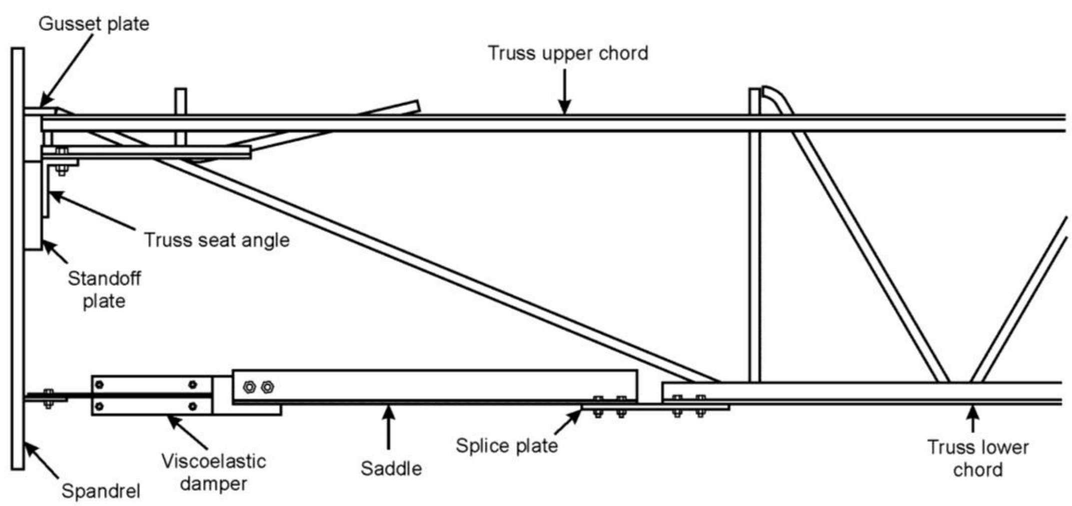

Figure 7. Diagram of floor truss showing viscoelastic damper [1].

3.2.6 Long-Span Composite Floor Assemblies

The floor system in the towers (as shown in Figure 5) was novel in two respects: composite floors constructed with open-web, lightweight steel trusses and lightweight concrete, and achieving composite action between the steel truss and concrete slab with “knuckles” (web bars that extended above the top chord and into the concrete).

Tests conducted in 1964 by Granco Steel Products and Laclede Steel Company (the manufacturer of the trusses for WTC 1 and WTC 2) determined the effectiveness of the knuckles in providing composite action. Another set of tests, performed by Laclede Steel Company, determined that any failure of the knuckles occurred well beyond the design capacity of the trusses. A third set of tests, performed at Washington University in 1968, confirmed the prior results and indicated that failure near the knuckles was due to crushing of the concrete [3].

3.2.7 Vertical Shaft Wall Panels

While similar to other gypsum shaft wall systems and firewalls, the compartmentation system used in the vertical shafts (e.g., for elevators, stairs, utilities and ventilation) was unique in that it eliminated the need for any framing. The walls consisted of gypsum planks placed into metal channels at the floor and ceiling slabs. The planks were 51 mm (2 in) thick and 406 mm (16 in) wide, with metal tongue and groove channels attached to the long sides that served as wall studs. An assembled wall was then covered with gypsum wallboard. The planks were likely custom fabricated for this job, as the investigation team found no mention of similar products in gypsum industry literature of the time or since.

3.3. Structural Steels

3.3.1 Types and Sources

Roughly 181,000 metric tons (200,000 U.S. tons) of steel were used in the construction of the two WTC towers. The building plans called for an unusually broad array of steel grades and multiple techniques for fabricating the structure. NIST obtained data to characterize the steels from structural drawings provided by PANYNJ, correspondence during the fabrication stages, steel mill test reports, interviews with fabrication company staff, search of the contemporary literature, and mechanical property measurements of recovered steel at NIST [10].

Fortunately, the potential for confusion led the building designers to develop a tracking system whereby the steel fabricators stamped and/or stenciled each structural element with a unique identifying number. The structural engineering drawings included these identifying numbers as well as the yield strengths of the individual steel components. Thus, when NIST found the identifying number on an element such as a perimeter column panel, the particular steel specified for each component of the element was known, as well as the intended location of the steel in the tower [10].

In all, 14 grades of steel were specified in the structural engineering plans, having yield strengths from 248 MPa to 690 MPa (36 ksi to 100 ksi). Twelve were actually used, as the fabricators were permitted to substitute 690 MPa (100 ksi) steel where yield strengths of 586 MPa (85 ksi) and 621 MPa (90 ksi) were specified. Table 1 indicates the elements for which the various grades were used. The higher yield strength steels were used to limit building weight and differential shortening between columns while providing adequate load-carrying capacity.

Table 1. Steel grades for various applications.

Application	Yield Strength MPa (ksi)											
	248 (36)	**290 (42)**	**310 (45)**	**317 (46)**	**345 (50)**	**379 (55)**	**414 (60)**	**448 (65)**	**483 (70)**	**517 (75)**	**552 (80)**	**690 (100)**
Perimeter columns	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
Spandrel plates	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
Core columns	✓	✓	(a)		(a)							
Floor trusses	✓				✓							

a. About 1 percent of the wide flange core columns were specified to be of these higher grades.

3.3.2 Specifications and Testing

NIST performed confirmatory tests on samples of the 236 pieces of recovered steel to determine if the steel met the structural specifications [10]. Making a definitive assessment was complicated by overlapping specifications from multiple suppliers, differences between the NIST test procedures and the test procedures that originally qualified the steel, the natural variability of steel properties, and damage to the steel from the collapse of the WTC towers. Nonetheless, the NIST investigation team was able to determine the following information [10].

Fourteen grades (strengths) of steel were specified. However, a total of 32 steels in the impact and fire floors were sufficiently different (grade, supplier, and gage) to require distinct models of mechanical properties. The steels in the perimeter and core columns generally met their intended specifications for chemistry and mechanical properties. Roughly 13 percent of the measured strength values for the perimeter and core columns were at, or below, the specified minimums. The strength variation was consistent with the historical variability of steel strength and with the effects from damage during the collapse of the towers. The measured values were within the typical design factor-of-safety. The yield strengths of many of the steels in the floor trusses were above 345 MPa (50 ksi), even when they were specified to be 248 MPa (36 ksi).

Tests on a limited number of recovered bolts showed they were much stronger than expected based on reports from the contemporary literature. The mechanical properties of steel are reduced at elevated temperatures. Based on measurements and examination of published data, NIST determined that a single representation of the elevated temperature effects on steel mechanical properties could be used for all WTC steels. Separate values were used for the yield and tensile strength reduction factors for bolt steels.

3.4. Concrete

Two types of concrete were used for the floors of the WTC towers: lightweight concrete in the tenant office areas and normal weight concrete in the core area and in the mechanical areas. Because of differences in composition and weight, the two types of concrete respond differently to elevated temperatures. While their tensile strengths degrade identically, lightweight concrete retains more of its compressive strength at higher temperatures.

The specified design strength for lightweight concrete was 20.7 MPa (3 ksi) and either 20.7 MPa (3 ksi) or 27.6 MPa (4 ksi) for normal-weight concrete, depending upon the floor location within the buildings. The actual strength of concrete at room temperature is greater than that measured from cylinders poured for testing during construction, referred to as 28-day cylinder strength, as concrete continues to strengthen with age. Methods for estimating changes in concrete strength with age are specified by the American Concrete Institute (ACI) 209 [11]. The actual compressive strength of WTC concrete slabs was estimated to be 38 percent greater than the specified design strengths: 37.9 MPa (5.5 ksi) for 27.6 MPa (4 ksi) normal-weight concrete and 28.3 MPa (4.1 ksi) for 20.7 MPa (3 ksi) normal-weight and lightweight concretes.

3.5. Passive Fire Protection

Building codes require that elements that support loads are to be protected to achieve a specified fire resistance rating, expressed in hours. The WTC towers were classified as Class 1B, which required the columns to have a 3 h fire endurance rating and the floor system to a have a 2 h rating when tested in accordance with ASTM E 119 [12]. To achieve these ratings, the structural steel was protected with sprayed fire-resistive material (SFRM) or rigid fire-rated gypsum panels. Since application of SFRM to floor trusses was an innovative fire protection method in the 1960s, PANYNJ arranged for demonstrations to establish its feasibility for the World Trade Center [13]. In 1969, The Port Authority directed that a 13 mm (0.5 in.) thick coating of SFRM be used to insulate the floor trusses. This was to achieve a Class 1A rating, even though the preponderance of evidence suggests that the towers were chosen to be Class 1B. NIST found no evidence of a technical basis for selection of the 13 mm (0.5 in.) thickness.

In 1995, The Port Authority performed a study to establish requirements for retrofit of sprayed insulation to the floor trusses during major alterations when tenants vacated spaces in the towers [13]. Based on design information for fire ratings of a similar, but not identical, composite floor truss system contained in the Fire Resistance Directory published by Underwriters Laboratories, Inc., the study concluded that a 38 mm (1.5 in.) thickness of sprayed mineral fiber material would provide a 2 h fire rating, consistent with the Class 1B requirements. In 1999, the removal of existing SFRM and the application of new material to this thickness became Port Authority policy for full floors undergoing new construction and renovation. In the years between 1995 and 2001, thermal protection was upgraded on 18 floors of WTC 1, including those on which the major fires occurred on September 11, 2001, and 13 floors of WTC 2 that did not include the fire floors.

Multiple approaches were used to insulate structural elements in the core. Those core columns located in rentable and public spaces, closets, and mechanical shafts were enclosed in boxes of gypsum wallboard. The amount of the gypsum enclosure in contact with the column varied depending on the location of the column within the core. SFRM was applied on those faces that were not protected by a gypsum enclosure. The thicknesses specified in the construction documents were 35 mm (1.375 in.) for the heavier columns and 60 mm (2.375 in.) for the lighter columns [13]. Columns located at the elevator shafts were protected using the same SFRM thicknesses.

The Port Authority specified partitions separating tenant spaces from exit access corridors to have a 2 h rating. Above the ceiling, penetrations for ducts or to allow for return airflow were fitted with rated fire dampers to preserve the fire rating. This 2 h rated construction was not used in the original design, but was specified later by The Port Authority as tenant spaces were altered [1].

For walls separating tenant spaces to achieve a 1 h rating, the tenant alteration guidelines required that tenant partitions have a continuous fire barrier from top of floor to bottom of slab. Enclosures for vertical shafts, including stairways and transfer corridors, elevator hoistways, and mechanical or utility shafts were required to be of 2 h fire rated construction. Following a 1975 fire, The Port Authority began installing sprinklers when a new tenant moved in. By September 11, 2001, the sprinkler installations had been completed throughout the towers, and, in general, the tenants on the impact floors had few internal partitions except for those surrounding conference rooms and executive offices [1].

Firestopping materials were used to fill gaps in walls and floors through which smoke and flames might pass. In the towers, unlike many buildings, the exterior wall was connected with the floors without gaps [1].

4. World Trade Center 7 Structural System

4.1 Overview of WTC 7

WTC 7 was a 47 story commercial office building located immediately to the north of the main WTC complex, see Figure 8. It occupied the block bounded by Vesey Street on the south, Barclay Street on the north, Washington Street on the west, and West Broadway on the east (see Figure 1). Located approximately 105 m (350 ft) from the north side of WTC 1, it was connected to the WTC complex by a 36.6 m (120 ft) wide elevated plaza and a 6.7 m (22 ft) wide pedestrian bridge at the 3rd floor level. Its location relative to the WTC Plaza is shown in Figure 1. WTC 7 contained approximately 186,000 m^2 (2 million ft^2) of floor area.

The architectural design was performed by Emory Roth & Sons, P.C. The structural engineer of record was the Office of Irwin G. Cantor, and the mechanical engineer was Syska & Hennessy, P.C. Tishman Construction Corporation was the general contractor.

WTC 7 was trapezoidal in plan with dimensions of approximately 100 m (329 ft) on the longer side, 75 m (247 ft) on the shorter side, 43 m (140 ft) wide, and 186 m (610 ft) high. The building was constructed over a pre-existing electrical substation owned by Consolidated Edison (Con Edison). The original plans for the Con Edison substation included a high-rise building, and the foundation was sized for the planned structure. However, the final design for WTC 7 had a larger footprint than originally envisioned and accommodations were made in the foundation [4]. Over the years, numerous structural modifications were made throughout the building, mainly to suit its largest tenant, Salomon Brothers Inc., later to become Salomon Smith Barney, and now Citigroup. One of the modifications was the addition of a roof penthouse, referred to as the east penthouse, which was used to house a chiller plant and cooling towers for Salomon Brothers. Also, large portions of Floors 41 and 43 were removed on the east side of the building to accommodate trading floors for Salomon Brothers. The removed floor areas were subsequently restored after the trading activity was moved to another venue. The following sections provide a brief overview of the primary structural systems of WTC 7.

Figure 8. WTC 7 viewed from the north [4].

4.2 Foundations

WTC 7 and the Con Edison (ConEd) electrical substation were supported on caisson foundations. When the substation was constructed in 1967, provision was made for a future office tower by including capacity to carry both the substation and the future building. Caissons were also installed in the property adjacent to the substation for the proposed future building. When WTC 7 was constructed approximately 20 years later, it was significantly larger than the originally proposed building, and additional caissons were installed.

4.3 Exterior Moment Frame

Column trees were fabricated for the east and west facades with bolted field splices at the spandrel beam mid-spans. On the north and south facades, the spandrel beams had moment resisting bolted flange and web connections. Exterior columns were nominally W shapes (W360 or W14) of ASTM A36 steel. Exterior column splices were similar to the interior column splices.

Above Floor 7, WTC 7 had an exterior moment frame that resisted lateral loads. At Floors 5 to 7 and Floors 22 to 24, there was a perimeter belt truss, shown in Figure 9. Below Floor 7 there was a combination of moment and braced frames around the exterior, and a series of braced frames in the core, as shown in Figure 10. The reinforced diaphragms of Floors 5 and 7 transferred in-plane loads from the exterior to the core.

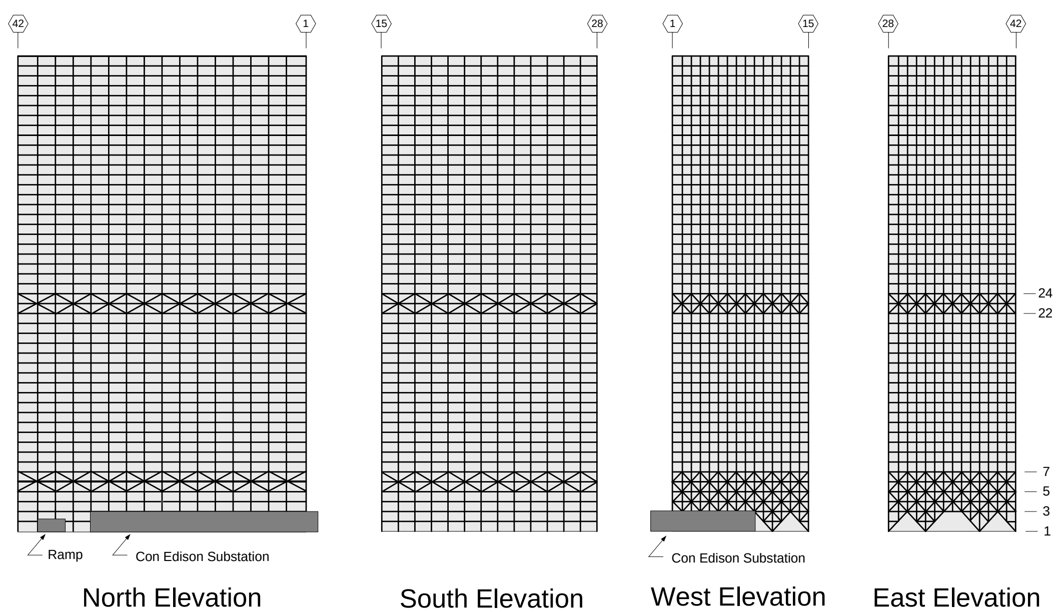

Figure 9. Exterior lateral load resisting system elevations [2]. Based on erection drawings [13].

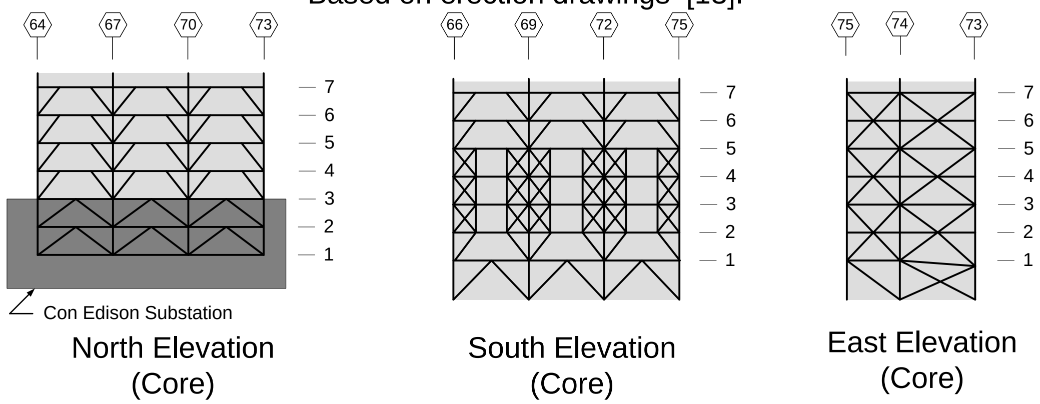

Figure 10. Interior lateral load resisting system [2]. Figure indicates column and floor numbers. Based on erection drawings [14].

4.4 Floor Systems

4.4.1 Typical Floor Systems: Floors 8 to 45

The typical floor framing system, shown in Figure 11 was composed of rolled steel wide-flange beams with composite metal decking and concrete slabs. Floors 8 through 45 had essentially the same framing plan, but the core layout varied over the height of the building. Figure 11 also shows the column numbering.

Floors 8 through 45 had floor slabs composed of a 76 mm (3 in deep), 20 gage metal deck with 63.5 mm (2.5 in) of 24 MPa (3500 psi) normal weight concrete above the top of the metal deck, for a total floor thickness of 140 mm (5.5 in). There was one layer of welded wire fabric (WWF) within the concrete. The drawings showed a second layer of WWF placed over girders at the slab edges. The fastening requirements for the metal deck were not shown on the drawings, but standard practice provided puddle welds 305 mm (12 in) on-center at the beams and side lap welds, screws, or button-punching at 914 mm (36 in) on-center between adjacent panels of deck. The drawings contained a note calling for 38 mm (1.5 in) deep, 20 gage metal deck with 102 mm (4 in) of concrete topping (140 mm or 5.5 in total) in the elevator lobbies, and a 76 mm (3 in) floor finish.

Typical floor framing for Floors 8 through 20 and Floors 24 through 45 consisted of 345 MPa (50 ksi) wide-flange beams and girders. Between the core columns was a grid of beams and girders. Core girders ranged in size from W410x46 to W920x201 (W16x31 to W36x135[‡]), depending on the span and load. Beams spanned between the core and the exterior of the building, at approximately 2.7 m (9 ft) on-center spacing. On the north and east sides, the typical floor beam was a W610x82 (W24x55) with 28 shear studs[§], spanning approximately 16 m (53 ft). On the south side, the typical floor beam was a W410x39 (W16x26) with 24 shear studs spanning 11 m (36 ft). Between the exterior columns, moment connections were used to connect the spandrel beams to the columns as part of the lateral load resisting system of the building.

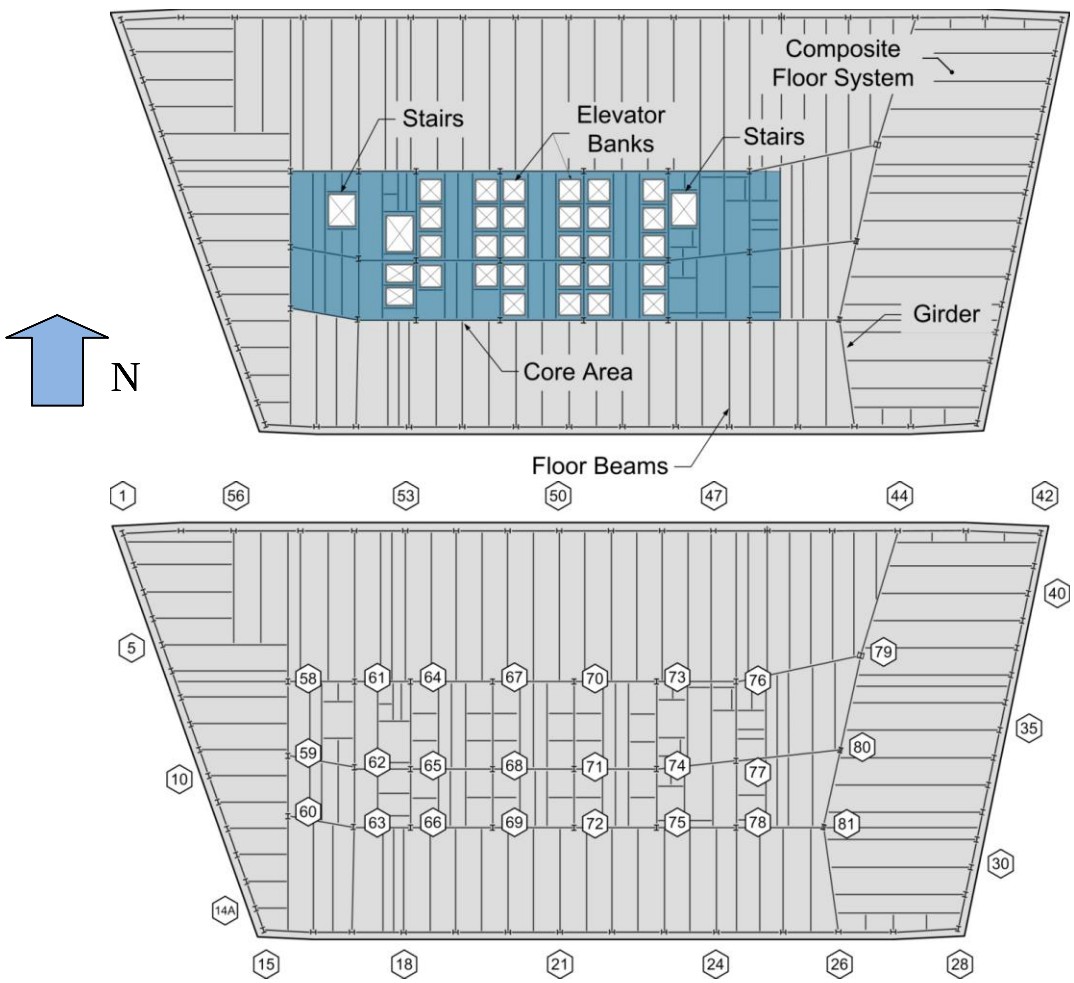

Figure 11. Typical floor framing plan (Floors 8 to 20 and 24 to 45) [2].

Based on structural drawings [15].

[‡] W920x201 (W36x135) refers to a wide flange section that is nominally 920 mm (36 in) deep and weighs 201 kg/m (135 lb/ft).

[§] Shear studs connect steel floor beams to the concrete slab so that they act compositely (together) to carry the floor loads.

Most of the beams acted compositely with the slabs through the use of shear studs. Typically, the shear studs were 19 mm (0.75 in) in diameter by 127 mm (5 in) long, spaced roughly 305 mm to 610 mm (1 ft to 2 ft) on center. The number of studs on a floor beam was indicated on the design drawings. Photographic records showing the demolition of a floor slab (tenant renovation) on the south side of a typical tenant floor confirms the number of studs shown on the drawings. Studs were not indicated on the design drawings for the girders, i.e., composite action did not develop between the girders and the slab.

Floor beam-to-girder and girder-to-interior column connections were generally a single shear plate or double angle connection, although in several instances, seated connections were used. The typical beam-to-exterior column connections were seated connections. The typical bolt used in the simple shear connections was a 22 mm (7/8 in) diameter ASTM A325, where ASTM A325 is a standard specification for a structural bolt. The bolt used for heavier brace and moment connections was a 25 mm (1 in) diameter ASTM A490.

4.4.2 Floor Systems: Floors 1 to 7

Floors 1 through 7 were atypical, due largely to the Con Edison substation. Floor 1 was built adjacent to the substation and included the truck ramp to the underground parking garage for the WTC complex. The floor was framed with steel beams that were encased in a formed concrete slab. The floor slab was 356 mm (14 in) thick, with typical No. 5 reinforcement bars (16 mm or 5/8 in rebar) and No. 6 rebar for the bottom reinforcement; No. 5 rebar was used for temperature reinforcement[**]. The southeast portion of the floor above the WTC truck ramp had a 152 mm (6 in) formed concrete slab with No. 4 rebar for top and bottom reinforcement; No. 4 rebar was used for temperature reinforcement.

The floor slabs for Floors 2, 3, 4, and 6 had a 76 mm (3 in) deep, 20 gage metal deck with 76 mm (3 in) of 24 MPa (3500 psi) normal weight concrete, for a total floor thickness of 152 mm

[**] Temperature reinforcement is designed to control cracking resulting from temperature differences through a concrete section due to heat of hydration and shrinkage of concrete as it cures.

(6 in). Floors 2 and 3 were also partial floors adjacent to the substation. In addition, they had a floor opening on the south side to form the atrium above the ground level lobby (shown in Figure 12). Floor 4 was above the substation and had a large opening over most of the south side of the building, to form a double-height space above the 3rd floor lobby. Floor 6 had two openings on the floor to form a double-height mechanical space, one on the east side and the other in the southwest corner. Truss 1, Truss 2 and Column 80 (refer to Figure 18) were located in the east double-height mechanical space.

The Floor 5 slab was 279 mm (11 in) of 24 MPa (3500 psi) normal weight concrete on top of a 76 mm (3 in) deep, 18 gage metal deck for a total slab thickness of 356 mm (14 in). The slab was heavily reinforced, with No. 7 rebar for top reinforcement in both directions and No. 9 rebar for bottom reinforcement that acted as additional diaphragm chord reinforcement in many areas. This floor also had 248 MPa (36 ksi) steel WT sections (W, or wide-flange, sections cut in half to look like a 'T' section) embedded in the 279 mm (11 in) concrete slab above the deck. The WT sections were designed to act as a horizontal diaphragm within the plane of the floor to transfer lateral loads from the exterior to the core columns.

The 7th floor slab consisted of 127 mm (5 in) of 24 MPa (3500 psi) normal weight concrete on top of a 76 mm (3 in) deep, 18 gage metal deck, for a total floor thickness of 203 mm (8 in). The slab was reinforced with No. 5 rebar in both directions. Regions of the slab on the south side of the building had 203 mm (8 in) of formed concrete without any metal deck. In these regions, two layers of steel reinforcement were provided.

4.4.3 Floors 46 and Above

The 46th floor had areas of heavier framing to support the cooling towers on the north side, and the setback roof on the south side. There was a 152 mm (6 in) reinforced concrete slab in a portion of the core and under the cooling towers (see Figure 13). Floor 47 had a double height space starting from the 46th floor to allow for the cooling towers on the north side. There was also a setback roof on the south side at Floor 46 (see Figure 14).

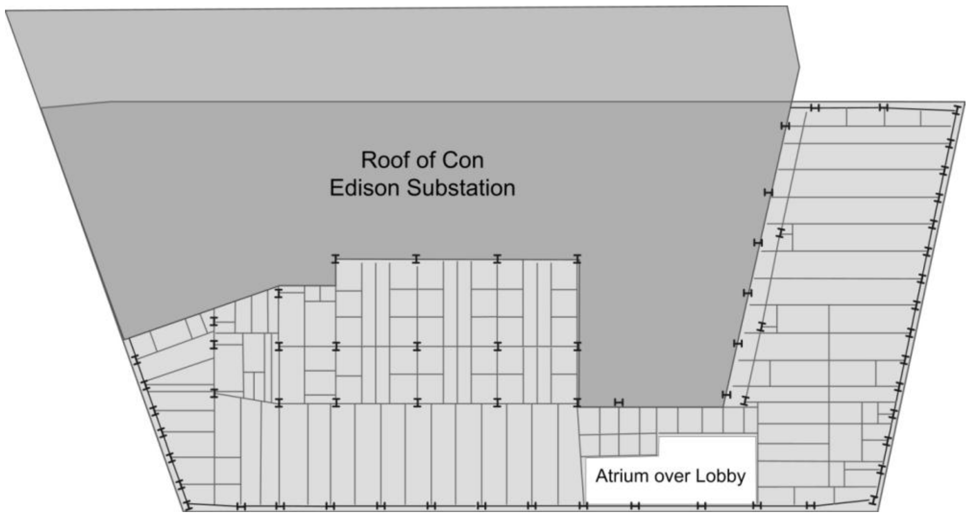

Figure 12. Floor 3 framing plan [2]. Based on structural drawings [15].

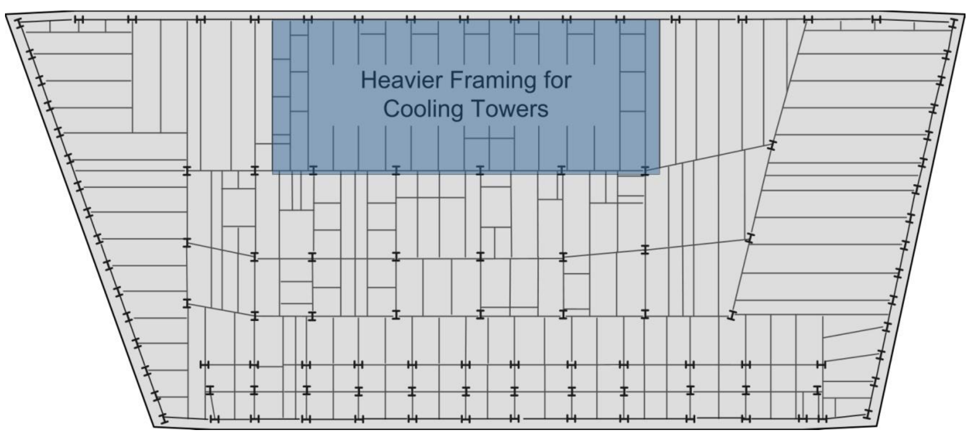

Figure 13. Floor 46 framing plan [2]. Based on structural drawings [15].

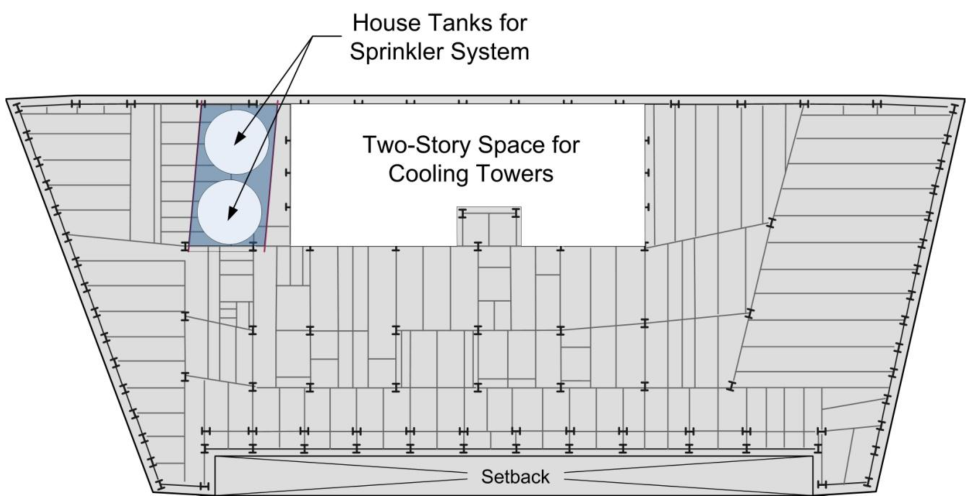

Figure 14. Floor 47 framing plan [2]. Based on structural drawings [15].

4.4.4 Roof and Penthouses

The roof had a concrete slab on metal deck. The top of the slab was sloped, from a 216 mm (8.5 in) thickness to a 140 mm (5.5 in) thickness, to provide drainage. The WWF in this slab was 70 percent heavier than at a typical tenant floor. There were slab openings for the cooling towers on the north side of the roof. The area above the cooling towers was framed in steel, with grating spanning between the beams. A series of diagonal WT150x14 (WT6x9) members under the grating provided diaphragm action in this area.

The east penthouse was added to the WTC 7 building in 1989. To accommodate the added load of equipment within the east penthouse, the roof beams and girders were reinforced using cover plates welded to their lower flanges, and a thick concrete pad was cast to support the air-conditioning equipment housed within the penthouse. Steel columns (or "posts" as they were termed on drawings) supported the new penthouse structure. A plan of the roof and penthouse framing is shown in Figure 15. The east penthouse posts (P17, P33, and P32 in Figure 15) framed into existing interior columns that fell within the east penthouse footprint (Columns 76, 77, 79, and 80). The other posts were supported by beams at the roof level.

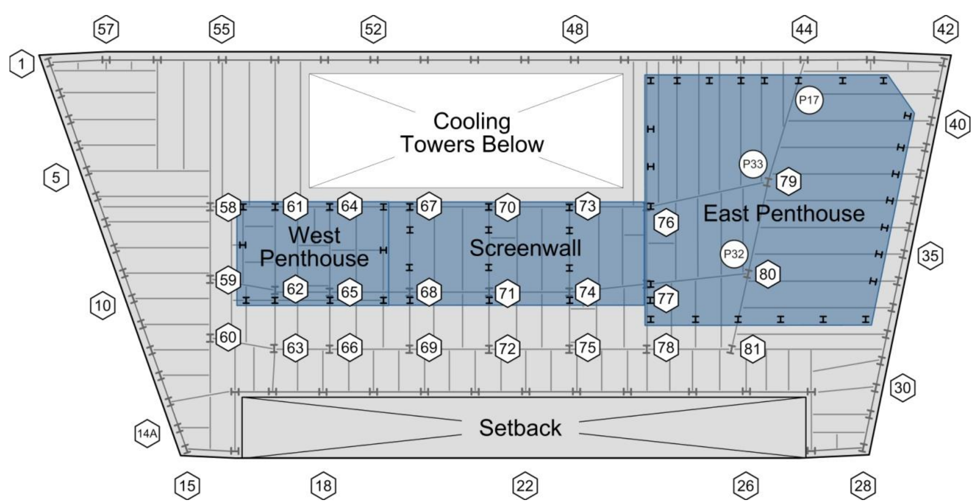

Figure 15. Roof layout, including penthouse locations [2].
Based on structural drawings [15, 16]

4.4.5 Connections in the Floor System

The structural design drawings indicated that the exterior framing (and portions of the core framing at Floors 5 and 7) had moment connections as part of the lateral load resisting system, and that the floor framing had simple shear connections. The structural design and fabrication shop drawings provided details of the connections used in the building. Details of all connections, including steel section sizes, plate thickness and dimensions, weld sizes, bolts sizes, bolt hole locations, and clearances were taken from approximately 2500 fabrication shop drawings. The fabrication shop drawings, as well as the structural design drawings and erection drawings, served as the source material for the floor plans and details.

Typical shear connections were used in the floor framing. All floor beams had simple shear framing connections with high strength bolts. Connections made with double angles were referred to as either header or knife connections. A header connection consisted of two angles shop fillet-welded to a beam web, and field bolted to either a girder or column. A knife connection was one in which the angles were shop fillet-welded to a girder web or column, and field bolted to a beam or girder web. A fin connection consisted of a flat plate that was shop welded to the web and underside of a girder flange using double fillet welds, and field bolted to a beam framing into the girder. Seated connections with web clips were used for core floor beams framing into girders.

Floor beams or girders that framed into exterior columns had a seated connection with a top plate or a top clip. Girders that framed into interior Columns 79 and 81 also had seated connections with a top clip. The seat was either a rolled angle or a seat plate welded to the column. The seat plate at Column 81 was stiffened while the seat plate at Column 79 was supported by a plate welded to the side cover plates on the lower stories. Figure 16 is a schematic of the seat arrangement for Column 79 where side plates were used. The beam or girder top flange was attached to the column with either a clip angle or plate to provide lateral restraint during construction.

4.5 Interior Columns

The interior columns were primarily rolled wide-flange shapes of Grade 36 or 50 steel. As the loads increased toward the base of the building, many of the column sizes were increased through the use of built-up shapes. Built-up columns had a W360x1086 (W14x730 section) with side cover plates welded to the flanges (to form a box), or web plates welded between the flanges, as shown in Figure 17. The plate welds were specified to be continuous fillet welds at the side cover plates and the web plates. Plate thickness ranged from 38 mm to 203 mm (1.5 in to 8 in).

Each interior column was spliced every two stories. Typical interior column splices had milled ends and splice plates were welded or bolted to the outside of the column web and flanges. Built-up columns were also milled at their bearing ends but the splice plates were fillet welded to the cover plate or web plate.

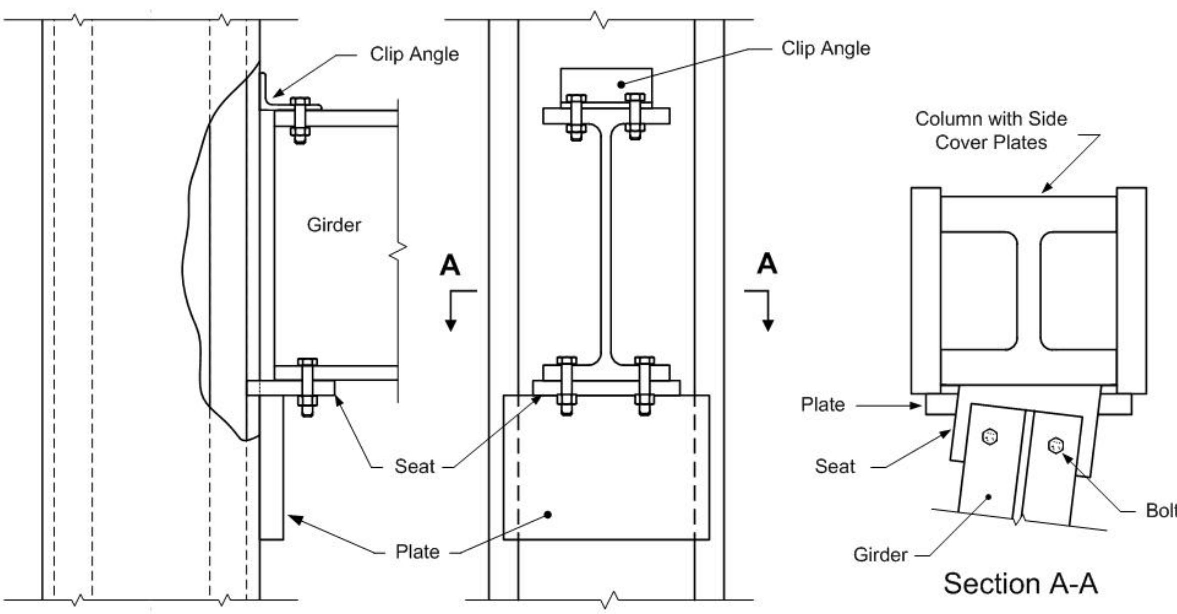

Figure 16. Schematic drawing of the seated connection at Column 79 [2].
Based on fabrication shop drawings [17].

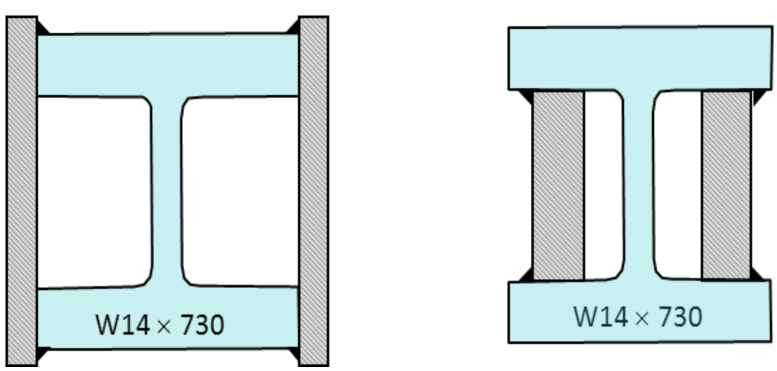

Figure 17. Typical built-up column details [2].

Based on structural drawings [15].

4.6 Column Transfer Trusses and Girders

As indicated earlier, the layout of the foundation substructure and Con Edison columns did not align with the column layout in the upper portion of WTC 7. Therefore, a series of column transfers were constructed. These transfers occurred primarily between Floors 5 and 7, and are shown in Figure 18.

Loads for columns 47 through 54, at the north face, were transferred at Floor 7 by cantilever girders to the substation columns, which were offset to the south. The cantilever girders were supported by the north side core columns. The easternmost cantilever girder was connected to Truss 1, and the westernmost cantilever girder was connected to Truss 3. Column 76 was supported at Floor 7 by Truss 1. The west side of Truss 1 was supported by Column 73, while the east side was supported by a transfer girder oriented in the north-south direction which was, in turn, supported by Columns E3 and E4 at Floor 5. Columns 58, 59, and 78 were transferred by girders at Floor 7. Column 77 and the transfer girder supporting Column 78 were supported by Truss 2. Truss 2 was supported by Column 74 at its west end and by Column 80 at its east end. Column 61 was supported by Truss 3. Truss 3 ran north-south and was supported by Columns 62 and 61A.

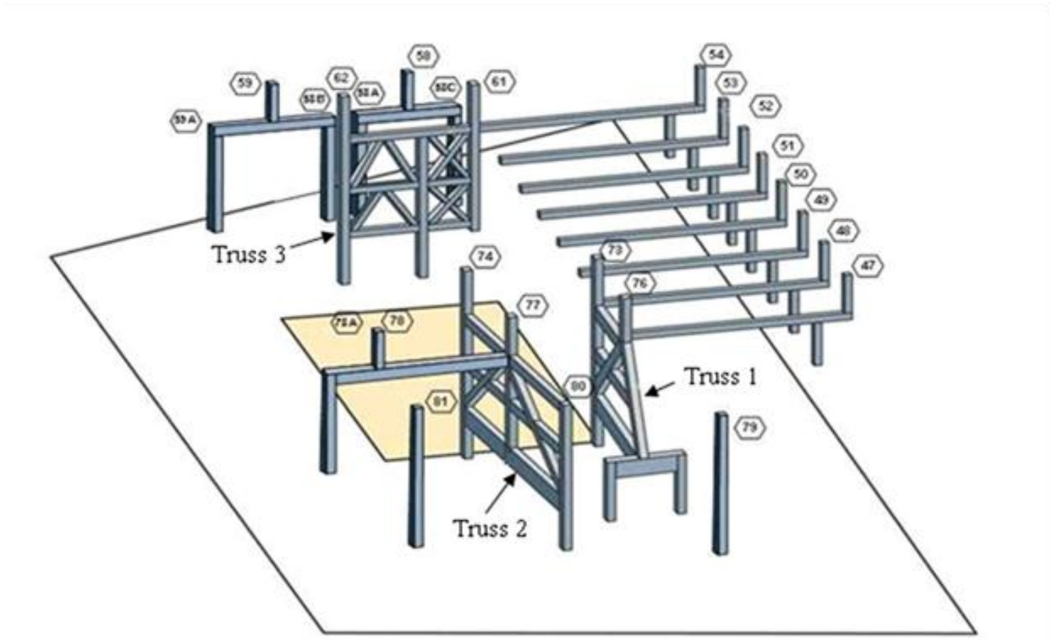

Figure 18. 3D schematic view of transfer trusses and girders between Floors 5 and 7 [18].

4.7 Passive Fire Protection

WTC 7 had sprayed fire-resistive material (SFRM) applied to the structural steel and to the underside of the metal floor decking [4]. According to the 1968 version of the NYCBC and Local Law 16 (1984), a fully sprinklered high-rise building could follow the fire resistance requirements for Type 1C construction. For this construction category, columns were required to have a 2 h rating as established by the Standard Fire Test (ASTM E 119 [12]); beams were required to have a 1.5 h rating. The instructions to the bidders for the WTC 7 job were to bid on a 3 h rating for the columns and a 2 h rating for the metal deck and floor support steel, which corresponded to the more stringent fire resistance requirements for Type 1B (unsprinklered) construction. According to the Underwriters Laboratories (UL) Fire Resistance Directory (1983), these ratings required a thickness of 22 mm (0.875 in.) of SFRM to be applied to the heavy columns, 48 mm (1.875 in.) to be applied to the lighter columns, 13 mm (0.5 in.) to be applied to the beams, and 10 mm (0.375 in.) to be applied to the bottom of the metal deck. Private inspectors found that the applied SFRM thicknesses were consistent with these values.

The architectural drawings indicated that there were fire-rated walls between tenant spaces on the same floor and between tenant spaces and the building core. Spaces housing mechanical equipment, power transformers, emergency power generators, and other such equipment were enclosed in fire-rated partitions. While the partitions between offices, conferences rooms, etc. were not required to be fire-rated, there was evidence from both the visual evidence and the fire simulations that some of these partitions retarded the fires.

There was no evidence of floor-to-floor fire spread until perhaps just before the WTC 7 collapse. Thus, the fire-rated floors were successful as fire penetration barriers.

5 Summary

The WTC towers were innovative in many ways, and their construction resulted in a tremendous increase of open-plan commercial office space in downtown Manhattan. This paper focused on aspects of structural design of the WTC towers and building 7 which played a critical role in the outcome of the attacks of September 11, 2001. The structural features of the World Trade Center 1, 2 and 7 buildings and the role of the structural systems as the buildings respond to gravity and lateral wind loads were described. Key structural systems of the WTC towers include the exterior framed-tube system, the core structure which (with the exterior framing) supported gravity loads, the composite floor truss systems supported by the exterior and core columns, and the hat truss that distributed loads between the core and exterior columns at the top of the buildings. Key structural features of WTC 7 included exterior moment frame for resisting lateral wind loads, the core structure which supported gravity load in conjunction with the exterior framing, the long-span composite floor system between the core and exterior columns, the floor framing connections, and the column transfer trusses and girders. Full reports of the NIST investigation into the collapse of the WTC buildings can be found at [19].

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