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Structural Response of World Trade Center Buildings 1, 2 and 7 to Impact and Fire Damage*

Therese P. McAllister**, John L. Gross, Fahim Sadek
National Institute of Standards and Technology
100 Bureau Drive
Gaithersburg, MD 20899

Steven Kirkpatrick, Robert A. MacNeill
Applied Research Associates
2672 Bayshore Pkwy, Suite 1035
Mountain View, CA 94043

Mehdi Zarghamee, Omer O. Erbay, Andrew T. Sarawit
Simpson Gumpertz & Heger Inc.
41 Seyon Street, Bldg. 1, Suite 500
Waltham, MA 02453

Abstract. The National Institute of Standards and Technology (NIST) conducted an extensive investigation of the collapse of World Trade Center towers (WTC 1 and WTC 2) and the WTC 7 building. This paper describes the component, subsystem, and global analyses performed for the reconstruction of the structural response of WTC buildings 1, 2, and 7 to impact and fire damage. To illustrate the component and subsystem analyses, the approach taken for simulating the performance of concrete slabs and shear stud connectors in composite floors subject to fire conditions are presented, as well as steel floor framing connections for beams and girders. The development of the global models from the component and subsystem analyses is briefly described, including the sets of input data used to bound the probable conditions of impact and fire damage. The final analysis results that were used to develop the probable collapse hypotheses, and a comparison of the results against observed events, are presented for each building. A review of research activities focused on improving understanding of structural system response to multi-floor fires following the WTC disaster is also provided.

Keywords: *World Trade Center; Structural fire effects; Impact damage; Structural analysis; Failure analysis; Global collapse*

** Correspondence should be addressed to Therese McAllister at therese.mcallister@nist.gov, phone 301-975-6078, fax 301-869-6275

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1.0 Introduction

In 2002, the National Institute of Standards and Technology (NIST) was charged by the U.S. Congress to investigate the building construction, materials, and technical conditions that contributed to the collapse of World Trade Center (WTC) buildings 1, 2, and 7. To support determination of why and how the buildings collapsed, a sequence of analyses was performed: 1) aircraft impact analyses to estimate the initial damage to buildings, 2) fire dynamics simulations to model the spread and growth of the fires, 3) thermal analyses to predict the temporal and spatial distribution of temperature in the structure, and 4) structural analyses to simulate the response of the structure to impact and fire events and the sequence of structural failures that led to the collapse of the buildings. This paper[1] presents key steps of the reconstruction of the structural response of the WTC buildings to impact and fire damage, and identifies structural features and events that were common to all three buildings. Examples of model development for components and structural systems are presented, as well as comparisons to other WTC studies. There are three companion papers: [1] describes the buildings, [2] presents the structural analysis approach and reconstruction of impact damage, and [3] presents the reconstruction of the fires and thermal environment during the event. The four papers provide an overview of the complex and extensive investigation undertaken by NIST at a level of detail that has scientific merit but presents key aspects from the voluminous official reports at a level suitable for the technical literature.

In planning the approach for the structural analyses of the WTC towers, a review of the technical literature found other collapse theories and supporting analyses for the WTC towers. However, other than the truss design calculations [4], no other studies or papers on long-span composite floors with trusses, similar to those in the WTC towers, were found in the literature.

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Technical papers on the WTC towers collapse mechanism were reviewed (see [2] for full discussion). The analyses by others generally did not include structural damage from the aircraft impact, used assumed time-temperature curves, and limited their analyses to components or subsystems (i.e., floor trusses and exterior columns). The exception was finite element analyses by Abboud et al. [5], which included structural damage from the aircraft impact and global response of each tower. The global analyses included subsystem and global responses that occurred during the fire exposure, such as the heating of core columns with dislodged SFRM and the role of the hat truss in transferring loads between the core and exterior columns.

NIST conducted four furnace tests of the composite floor truss systems to determine their performance in a standard fire test [4]. The tests were allowed to run past the typical termination points for standard fire tests. Failure mechanisms that were observed included buckling of web diagonals near the end of the main truss and spalling of the concrete slab near the slab ends (which were thermally restrained). A detailed finite element model of a composite truss section showed similar web buckling near the end of the truss, but did not show slab failure, as the slab was not restrained (see Section 2.1).

For the WTC 7 analyses, no other analyses of the global collapse or supporting analyses by others were found. However, since the WTC 7 composite floors had typical wide flange beam construction with a slab on a metal deck and shear studs, a number of technical papers provided guidance to the development of the finite element model and the analysis approach. However, the only source of data for uncontrolled multi-floor fires and their effect on structural systems was found in reports of building fires.

Available building fire data from World Trade Center Building 5 (WTC 5) [6, 7], One Meridian Plaza [8, 9, 10], First Interstate Bank Building [11, 12], and One New York Plaza [13] were used to support the development of collapse hypotheses and failure mechanisms for WTC 7. For example, One New York Plaza, a 50 story office building in New York City had reported damage of steel beams that twisted or deflected several inches and connecting bolts that sheared off or failed. One Meridian Plaza, a 38 story office building, had significant structural damage to floor sections, with some sagging as much as three feet, and cracks through the reinforced concrete floors in many places. The WTC 5 experienced failure of large sections of floor, in areas not damaged by falling debris from WTC 1, following bolt tear-out as a result of the uncontrolled fires.

The Cardington Laboratory fire tests, conducted in 1995 and 1996, provided full scale tests of a structural system subject to compartment fires and realistic boundary conditions [14]. An eight story test building with a steel braced-frame was built by the British Research Establishment within its Cardington Laboratory to conduct large compartment fires. The typical bay size was 6 m by 9 m (20 ft by 30 ft), the floor beams and girders were not thermally protected, and the steel floor framing had end plate and fin connections. The connections and columns were thermally protected during some tests to limit the area of heat exposure. The failure mechanisms that occurred during the six tests were primarily local buckling of the lower beam flange near connections and lateral buckling of a beam during heating, and failure of connections by bolt shear or plate fracture during cooling. The local buckling of the beam lower flange occurred at locations where cooler framing on the opposite side of the connection restrained the thermal expansion. The floor framing on the east side of WTC 7, where the collapse was observed to initiate, had long span floor beams (16 m or 52 ft) that framed into the girders on one side only; no significant restraint of thermal expansion by the floor beams was provided by the interior girders [22]. The failure mechanisms of lateral buckling of long span floor beams and bolt shear at their support were included in the WTC analysis models. The finite element models for simulating structural response to fire used beam elements; local buckling of beams was not simulated.

Review of the literature prior to 2002 found that analytical models of structural response to fire included steel and concrete stress-strain relationships at elevated temperatures, thermal expansion effects, creep strains for steel and concrete, and uniform and/or non-uniform temperature distributions across steel and concrete sections. The representation of floor framing connections beyond pinned or fixed nodes when modeling structural response to fire was recognized in [15] as a novel feature in the 1990s. A connection element was developed to represent semi-rigid connection behavior with a moment-rotation relationship.

The authors noted that there was no data to quantify the connection performance at elevated temperatures.

A detailed analytical study of support conditions on the fire performance of steel and composite beams that were exposed to a range of heating scenarios [16] found that the behavior at elevated temperatures is more complex than at ambient temperature, with continuous interrelated changes in the deflected shape, axial force, bending moments and internal stresses that vary with the type of support condition.

Based on simulations of the Cardington fire tests [17], the authors observed that the actual global behavior was different, and typically far better, than that shown in standard small-scale fire tests. As the beams heated and lost much of their load carrying capacity, the composite slab supported the gravity loads through the tensile membrane capacity of the slab. Another analysis of the Cardington fire tests [18] found that the effects of restrained thermal expansion dominated the behavior of the framing and composite floor slab, with degradation of stiffness and strength a secondary factor. They also noted that the tensile action provided by the reinforcement mesh provided significant load carrying capacity at elevated temperatures.

The inclusion of shear stud connectors in models of composite floors was addressed in [19]. Shear stud connectors transfer shear forces between a steel beam and composite slab, resulting in a composite beam section (steel beam and concrete slab) that acts as a single unit responding to loads. When the shear studs fail, the beam and slab revert to acting independently. A nonlinear procedure for modeling partial shear connection was incorporated into the computer program VULCAN, which had been validated against ambient composite beam tests and two of the Cardington tests. Simulation of the Cardington test results was acceptable, although it was noted that data was needed on shear connector degradation at high temperatures.

Given the state of research and data for structural response to fire, the noted analysis features and failure mechanisms were included in the development of the finite element analysis (FEA) models for the WTC towers and WTC 7. The models were carefully constructed according to building geometry, materials, and connection details shown in available drawings, photographs, and material test results. Two features of the FEA models went beyond customary design practice: 1) the inclusion of detailed connections with multiple failure modes and 2) the extension of the analysis beyond initial member failure to investigate the sequence of failures leading to global collapse. To capture the local and global response of the WTC towers and WTC 7 structures to gravity loads and fire effects, the analyses needed to reasonably represent system behavior, while keeping the models at a feasible size for computations. Where test data was not available, the models were developed with a series of preliminary, detailed models verified by engineering analyses. Once global collapse started, the analyses were truncated because even detailed structural analyses cannot reliably simulate the complex, chaotic nature of a structural collapse.

What follows is a brief summary of an extensive reconstruction of the structural response that accompanied and followed the aircraft impact and uncontrolled fire events. Numerous facts and data were obtained and combined with validated computer modeling to produce an account that is close to what actually occurred. The reader should keep in mind that the building and the records kept within it were destroyed, and the remains of the WTC buildings were disposed of before this investigation began. As a result, there are some facts that could not be discerned, and there are uncertainties in this accounting. Nonetheless, NIST was able to gather sufficient evidence and documentation to conduct a full investigation upon which to reach firm findings and recommendations.

The WTC investigation stimulated research of structural performance in fire, particularly for connections and composite floor systems in fire, and potential collapse due to fire effects.

2.0 Component and Subsystem Analyses

Component and subsystem analyses provided an understanding of the behavior of structural components and subsystems under gravity and thermal loading and were used to support development of global models. Floor components that were modeled in detail for the WTC towers included shear knuckles, truss seats, a single truss, and concrete slab, which are described in [1]. To determine the capacities of the floor connections in WTC 7, failure modes were evaluated for each connection type including weld failure, bolt failure (both

Fire Technology, DOI: 10.1007/s10694-012-0289-2

shear and tension), plate tear-out, block shear failure, and truss walk-off from the bearing support. Shear stud failure in a composite floor system was evaluated to include both types of failure: when the concrete slab is either crushed or cracked around the shear stud, or when the shear stud separates from the floor beam. The details of the failure modes and modeling approaches are given in [20, 21, 22]. The following examples illustrate how failure criteria were established and modeled for concrete slabs, shear stud connections, and steel framing connections.

2.1 Concrete Slabs

The WTC towers had reinforced, lightweight concrete floor slabs in the tenant areas and normal weight concrete in the core area. Welded wire fabric (WWF) was used for crack control. Component models of a single truss section in the tenant area (Figure 1) indicated that the lightweight concrete slab remained in compression as the slab heated and thermally expanded against the supporting columns and when the trusses buckled and the floor section sagged. WTC 7 had reinforced floor slabs and normal weight concrete with WWF that extended across the floor. The continuous floor slab was reinforced to prevent cracking over girders from reverse curvature effects. The reinforced concrete slab was modeled with von Mises yield criterion with temperature dependent material properties. Concrete failure by crushing in compression and cracking in tension were accounted for by checking element strains against limiting strain criteria. Once these strain criteria were met, concrete elements were softened (modulus of elasticity was greatly reduced) so that the gravity loads associated with the slab elements remained in the model.

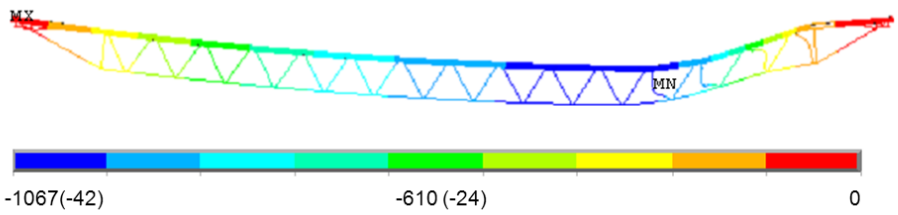


Figure 1. Vertical displacement of buckled composite floor truss section in mm(in) [20].

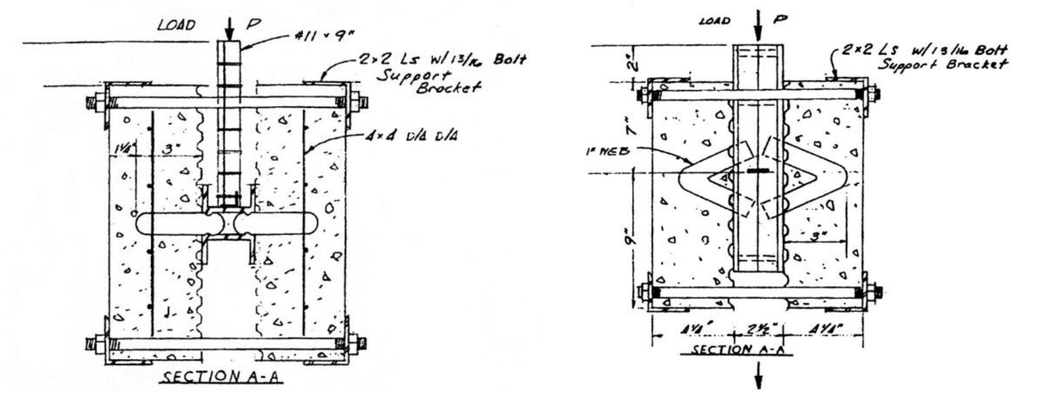
2.2 Shear Connectors

The tenant floors in the WTC towers were made composite through knuckles (truss web bars extended at the top of the truss and embedded in the slab as a shear transfer mechanism). As part of the original truss design, Laclede Steel Company in Saint Louis, Missouri, conducted experiments in 1967 to determine the transverse and longitudinal shear capacities of the knuckle. Knuckles were cast into two reinforced concrete blocks (Figure 2), and loaded to determine the knuckle shear capacity when the lightweight concrete was 6 and 27 days old. The average shear capacity measured was 75 kN (16.9 k) per knuckle when concrete shear failure occurred. A value of 156 kN (35 k) per pair of knuckles for the WTC floor system was determined after adjusting for the strength of in-place, mature, lightweight concrete.

Detailed finite element analyses by NIST that simulated the Laclede tests and incorporated the knuckles in a floor system model found that failure of the knuckles in the floor system was unlikely. This finding was also supported by the lack of any knuckle failures in the four standard fire resistance tests of the floor truss assemblies [4], which had twice the design floor load for the WTC floors. Therefore, the full floor system analyses did not include a failure mechanism for the knuckles.

The floors in WTC 7 had typical composite construction with shear stud connectors welded to wide flange beams and embedded in the concrete slab. Figure 3 shows typical shear stud placement relative to the metal deck rib, where the metal deck rib is perpendicular to the beam axis. Shear stud failure in a composite floor system occurs when the concrete slab is either crushed or cracked around the shear stud, or the shear stud weld to the floor beam fails. Shear stud failure in a composite floor system depends on a number of variables, including the depth and rib geometry of the metal deck, slab thickness, concrete properties, location of the studs relative to the beam axis, and the stud strength. Two sources for predicting shear stud strength in a composite floor system with a metal deck were used. Referring to Figure 3, when the shear load is parallel to the beam axis, the average strength of the shear stud connection was estimated using the procedure in [23]. The strength of the shear stud connection parallel to the beam axis for the strong and weak directions were also computed separately from the procedure in [24]. The average strength values calculated from [24] and [23] were essentially the same. Shear stud strength degradation was based on the NIST models for structural steel mechanical properties at elevated temperatures [25].

Figure 4 illustrates how shear studs were modeled in the floor system. Shell elements were located at the centerline elevation of the slab. Beam elements were located at the centerline of beams and girders. To model composite action between floor beams and slabs, user-defined "break" elements represented the temperature-dependent capacity of the shear stud connectors. Contact elements were used between the slab and girder to allow the slab to transfer gravity loads, but not lateral shear forces to the girder. Contact elements also allowed the girders to deflect independently of the floor slab, and prevented slab penetration through the girder.



Reproduced with permission of Laclede Steel.
(a) Transverse shear test.

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(b) Longitudinal shear test.

Figure 2. Laclede Steel Company shear tests of a knuckle [20].

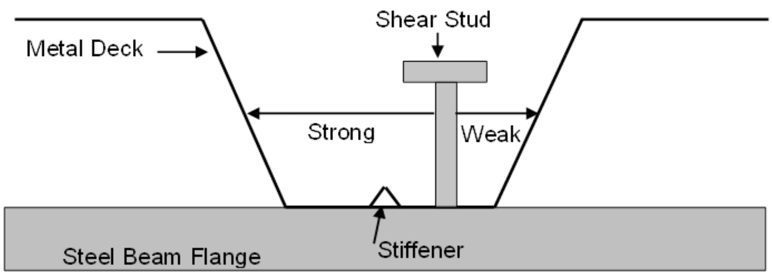


Figure 3. Schematic of shear stud placement relative to the metal deck [22].

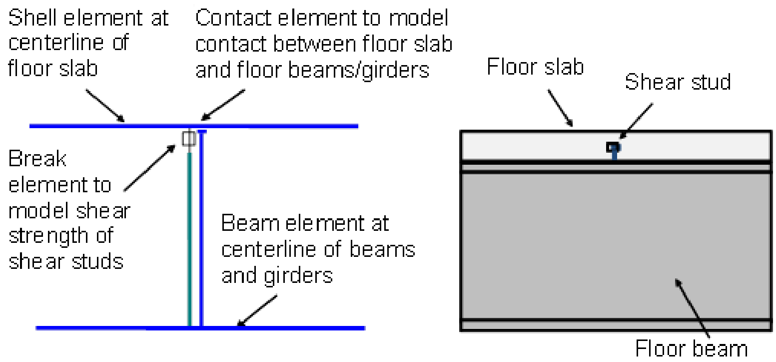


Figure 4. Analytical model for shear studs [22].

2.3 Framing Connections

The floor framing connections in the WTC towers included the truss seat connection and the knuckles (discussed in Section 2.2), which are illustrated in Figure 5. The vertical load on the truss seat was eccentric to the plane of connection between the seat and the spandrel. Because of this eccentricity, the truss seat resisted to combined effect of both shear and bending. The failure modes, and associated load and temperature conditions, for the truss seats were identified using a detailed finite element model (see Figure 6). The failure loads were computed for dead loads and service live loads. Possible failure modes for truss seats were identified for vertical force, horizontal tensile force, horizontal compressive force, and combined vertical and horizontal force for a range of temperatures.

Based on the finite element analysis results, vertical shear force was carried primarily by the stand-off plates, while bending moments were resisted by tensile forces in the gusset plate and compressive force in the stand-off plates. The exterior seat restrained the moment until the horizontal force in the connection caused slip between the seat angle and bearing angle or plate stiffener. The controlling failure mode for the truss seats was fracture of the fillet welds at the stand-off plates to spandrel connection, which resulted in loss of vertical support. In the full floor model, the truss seat was represented by a sub-model that captured these failure modes.

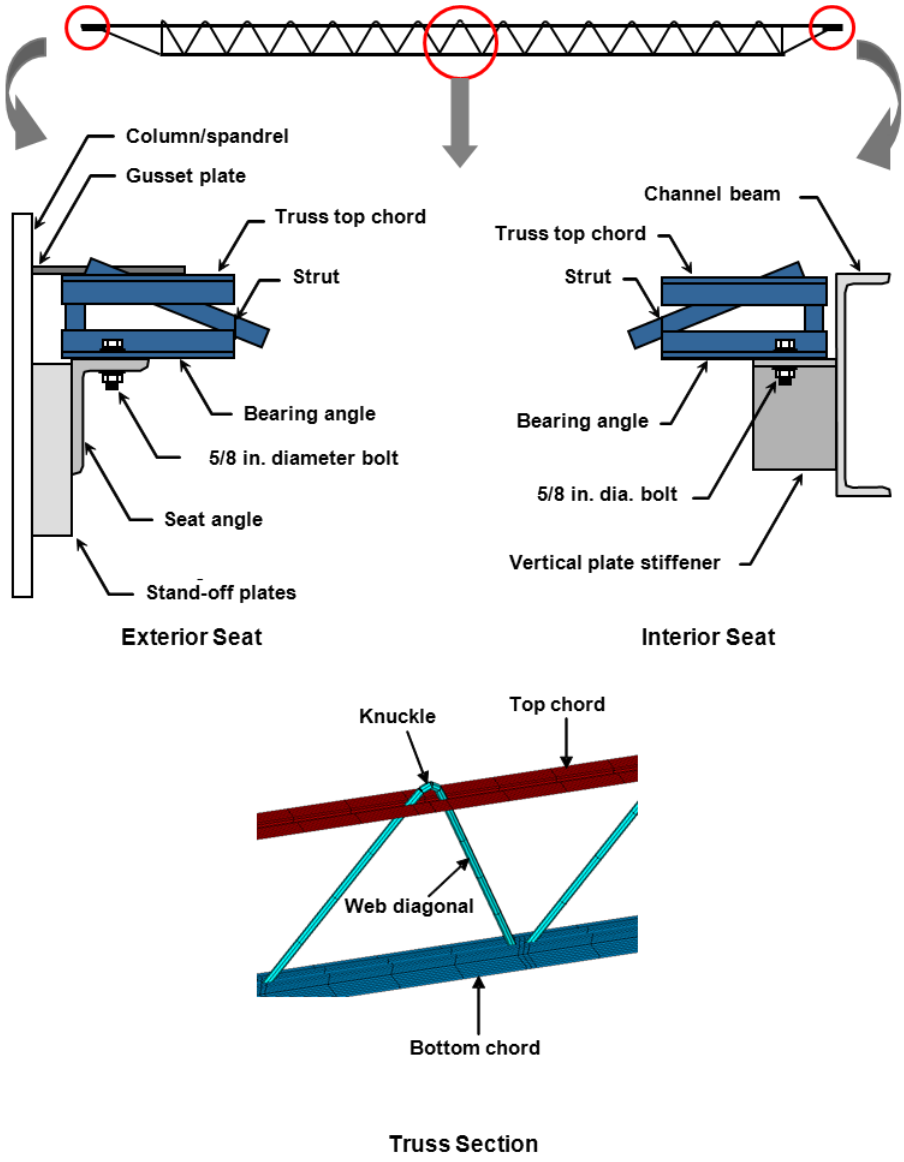


Figure 5. WTC floor truss components [20].

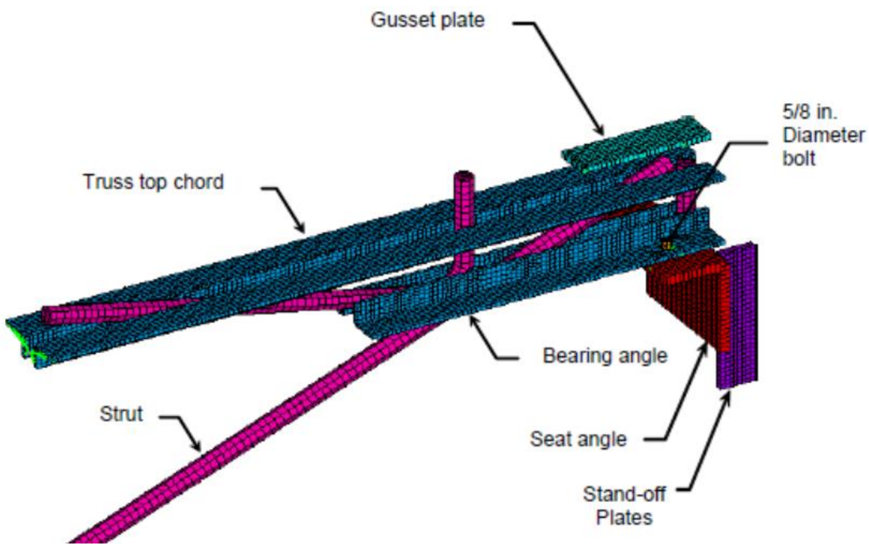


Figure 6. Finite element model of WTC floor truss at exterior seat [20].

The floor beams and girder framing connections in WTC 7 were constructed with plates, angles, welds, and bolts. To determine the capacities of the various types of floor connections, failure modes were evaluated for each connection type. Failure modes included weld failure [26], bolt failure (both shear and tension) [27], plate tear-out, and block shear failure. A model was developed for each connection type using beam elements, break elements, and contact elements. A model of single shear plate (fin) connection and its failure modes are illustrated in Figure 7. Component capacities were based on the AISC LRFD design provisions [24]. To determine failure loads, resistance factors were set to 1.0.

Bolt failure included temperature-dependent shear strength and tensile strength for high strength bolts [28]. For a conservative determination of bolt shear capacity, the threads were assumed to be excluded from the shear plane. Kulak [29] states that the values in AISC [24] were decreased by 20 percent to account for the uneven load distribution that occurs in splice connections, where a line of bolts are parallel to the applied load. However, in shear connection models, the loads are distributed evenly among all bolts under both vertical and horizontal loading. Thus, the nominal shear stress for bolts in floor framing connections was adjusted (divided by 0.8).

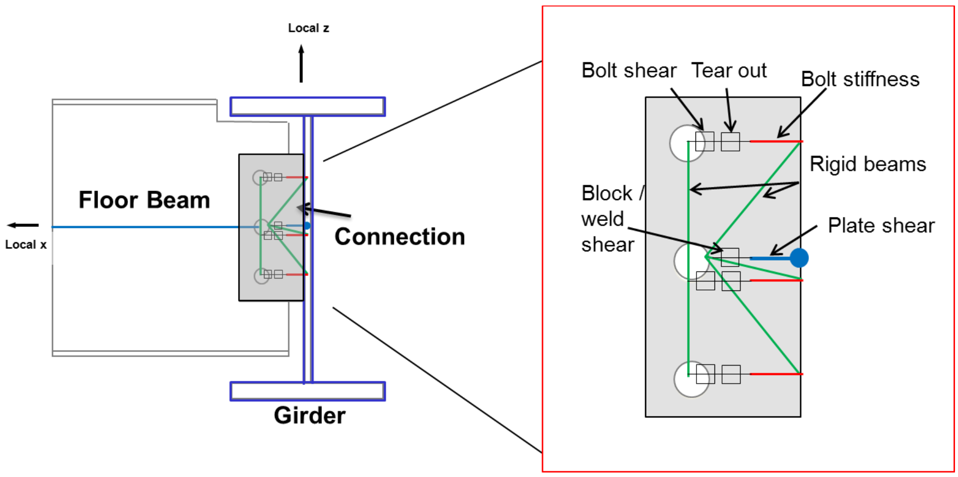


Figure 7. Model of a WTC 7 floor framing fin connection to include all possible failure modes.

2.4 Subsystems

Three major subsystems in the WTC towers - a core framing subsystem, an exterior wall subsystem, and a composite floor subsystem - were analyzed to determine their ability to resist and redistribute loads after impact damage and with elevated temperature. A full floor model accounted for each of the floor components and simulate component behavior and failure mechanisms, but with reduced level of modeling detail. A separate full floor model (see Figure 8) was developed and analyzed for each floor in WTC 1 and WTC 2 that was affected by fire in the aircraft impact zone. The model provided information on connection failures and reaction changes during the fire event. A single exterior column with spandrel sections and a single exterior wall panel with three columns and three spandrels were each modeled and incorporated into a larger exterior wall section with a nine-story by nine-column exterior wall subsystem.

Components and subsystems analyzed for WTC 7 focused on floor framing connections, floor subsystems, and possible failure modes for the connections, composite floor systems, and columns. In the global models, these failure modes were represented through user-defined elements that allowed modeling detail while maintaining the ability to simulate sequential failures, but with reduced degrees of freedom.

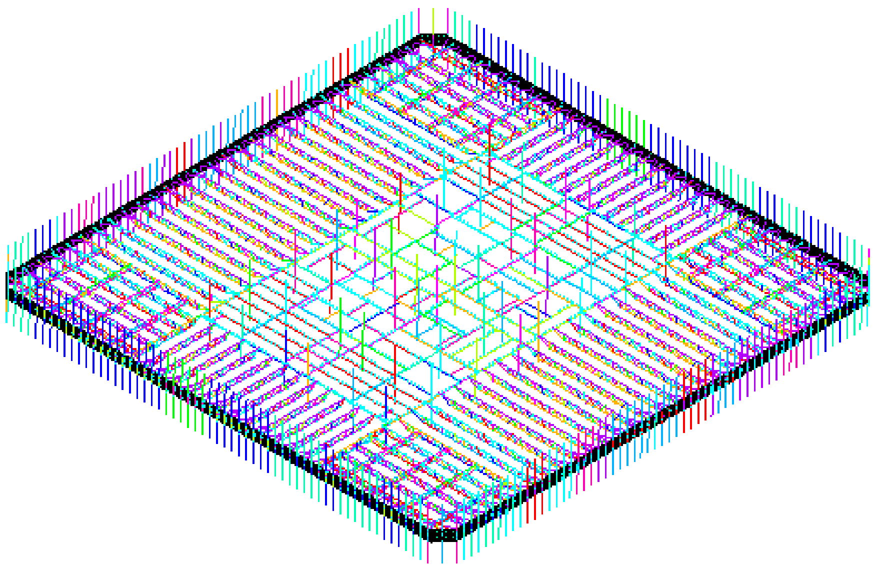


Figure 8. Floor framing for a full floor subsystem model for the WTC towers [20].

3.0 The Global Structural Response of the WTC Towers

The models of the WTC towers for the analysis of the structural response to aircraft impact damage and the subsequent fires up to collapse initiation extended several stories below the impact area to the top of the structure. The model for WTC 1 extended from floor 91 to the roof and the model for WTC 2 extended from floor 77 to the roof.

The structural models were developed in ANSYS [30], and included the core, the exterior walls, the floors, and hat truss. These global models used elements similar to those used in the subsystem analyses. However, the floor trusses could not be modeled individually due to model size and computational limitations. As a result, the floors were modeled using shell elements with a membrane stiffness equal to that of the full floor system. The equivalent floors functioned as diaphragms and transferred loads between the exterior walls and the core. Since out-of-plane displacement (sagging) was not included in the global floor model, gravity loads and loads from floor sagging were applied directly to the columns, based on the results of the individual full floor subsystem analyses. The results from each floor subsystem analysis and events identified in photographic and video evidence were used to determine failure of floor truss connections and pull-in forces from sagging floors for use in the global model. The nodal couplings between the exterior columns and the floors were removed at locations of where floor truss connections failed, based on the full floor analyses and observations. Similarly, pull-in forces were applied to the node couplings in the global models based on the full floor analyses and observed inward deflection of the exterior walls [31].

The structural models of WTC 1 and WTC 2 included the aircraft impact damage for each analysis by removing the severed and heavily damaged columns and floor areas, as shown in [2]. The aircraft impact damage was included in the thermal models by removing SFRM from the various structural elements as described in [3]. Two analyses were run for each tower: Cases A and B for WTC 1 and Cases C and D for WTC 2; see [2]. Cases B and D generally had more severe impact damage and fire conditions than Cases A and C. The results of the two cases for each tower provided some understanding of the uncertainties in the predictions. Each of the four cases was analyzed and the results were compared to observed events; no modifications or adjustments were made to parameter values during the analyses.

Temperature time-histories were input to the structural analyses in 10 min intervals with linear interpolation between these temperature states. Structural temperatures at 1 min and 10 min intervals were evaluated in the thermal analysis [3]. Since steel and concrete sections heat at a slower rate than gases, no significant difference was shown by applying temperatures at either interval. The analysis of fire growth and spread across each floor simulated direct heating by fire, as well as 'preheating' structural members as hot gases spread across the ceiling area. In the thermal analyses, columns with intact SFRM did not exceed 300 °C (572 °F). Only a few isolated truss members with intact SFRM reached temperatures over 400 °C (752 °F) in the WTC 1 simulations and temperatures over 500 °C (932 °F) in the WTC 2 simulations [20].

Pull-in forces from sagging floors were also applied during the appropriate 10 min intervals. To allow for sequential failures, elements were softened or removed. The results were compared to observed events. The global analysis results simulated a sequence of component and subsystem failures that led to the onset of global instability and collapse initiation.

Structural analyses of the towers with impact damage indicated that, in the absence of weakening by fire, the buildings would have continued to stand indefinitely [20]. The WTC 1 and WTC 2 global models were subjected to Case B and Case D aircraft damage and fires, respectively. The results of the isolated wall, core, and full floor analyses indicated that structural responses to Case B and Case D more closely matched observed structural than did Case A or Case C. Thus, Case B and Case D were chosen for the global analysis of WTC 1 and WTC 2, respectively. The application of the impact damage and fire scenarios in Cases B and D to the aircraft-damaged towers resulted in collapse.

3.1 WTC 1 Structural Response to Aircraft Impact and Fire

Based on the final aircraft impact, fire, thermal, and structural analyses, and their consistency with the collected evidence, the following events describe the probable collapse sequence of WTC 1.

Aircraft Impact Damage. The gravity loads carried by severed columns were redistributed mostly to columns adjacent to the impact zone. As the north wall section above the impact zone moved downward, the hat truss resisted the movement and redistributed the gravity loads from impacted walls to the other walls and core columns. At Floor 98, the load on the north and south walls decreased by about 7 percent and the load on the east and west walls increased by about 7 percent.

Core Weakening. Temperatures in the core area rose quickly and resulted in high plastic and creep strains in the core columns that continued to increase until collapse initiated. After 30 min (9:16 a.m. EDT), the plastic and creep strains exceeded thermal expansion strains. Due to high strains and plastic buckling of some core columns, at 100 min (10:26 a.m. EDT) the core structure at Floor 99 had displaced downward 50 mm (2.0 in) on average. The shortening of the core columns was resisted by the hat truss, which redistributed loads to the exterior walls. At Floor 98, about 80 min after impact, the exterior wall loads increased by about 12 to 27 percent and the core loads decreased by about 20 percent.

Sagging of Floors. The floors thermally expanded in the early stages of the fires. Due to the continued heating of the floors and the lack of SFRM, significant sagging of the floors occurred. The north floor areas sagged and then cooled as the fires moved toward the south side. When the fires reached the south side, the long-span trusses of Floors 95 to 99 sagged by as much as 584 mm (23 in.), as indicated in Figure 9a (view shows the concrete slab supported by the trusses), due to the damaged SFRM. The sagging floors induced pull-in forces on the south wall.

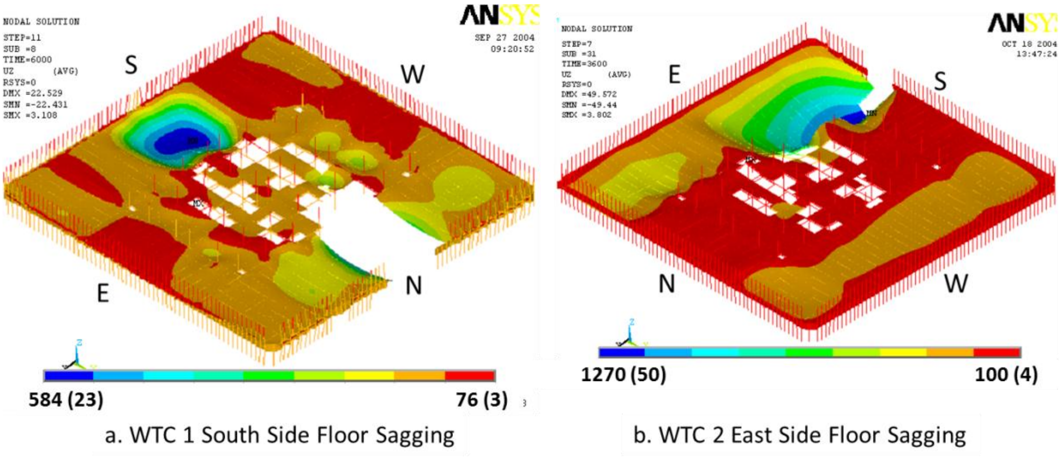
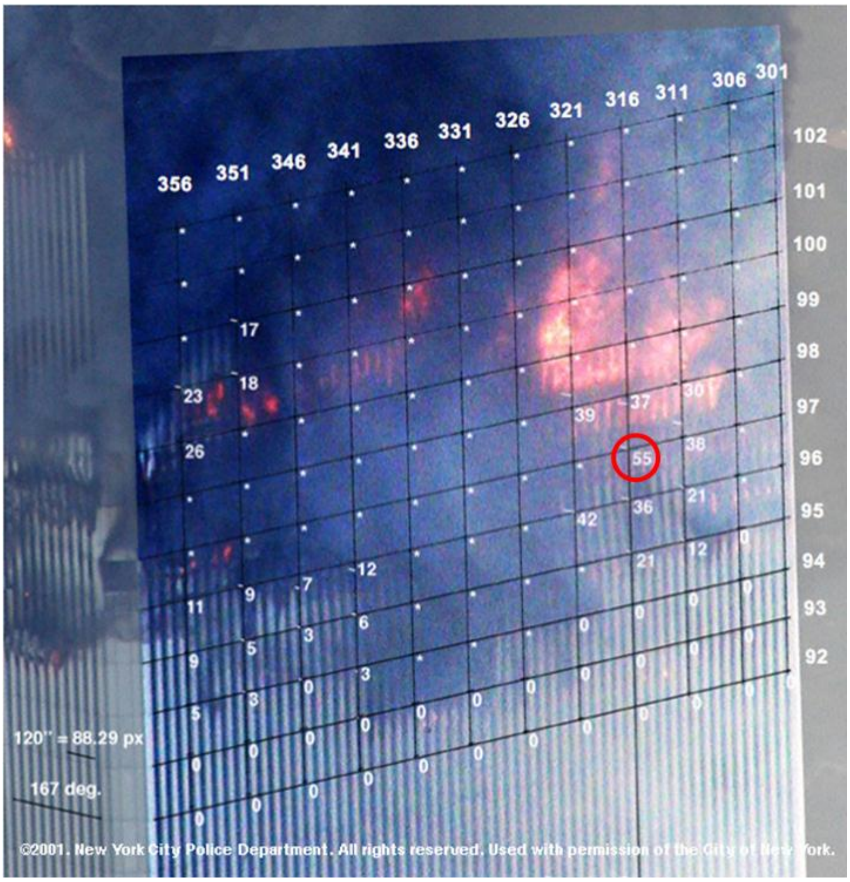


Figure 9. Single floor subsystem analysis results for floor sagging in response to fire exposure [20]. Vertical displacement shown in mm (in).

Buckling of South Wall and Collapse Initiation. The south wall bowed inward as the columns were subjected to high temperatures, pull-in forces from sagging floors, and additional loads redistributed from the core columns. Inward bowing of the south wall of approximately 1.40 m (55 in.) was observed at 10:23 a.m. EDT, as shown in Figure 10. As the inward bowing increased and columns buckled, the column loads transferred to adjacent walls by the hat truss and shear transfer through the spandrel beams and to the thermally weakened core columns via the hat truss. Consequently, buckling progressed horizontally across the south wall and rapidly along the east and west walls. The section of the building above the impact zone tilted to the south, as indicated by the tilt of the antenna on the roof shown in Figure 11, as loads could no longer be redistributed. WTC 1 collapse began at 10:28:22 a.m. EDT.



Numbers on right give floor numbers. Numbers at the top give column numbers. Grid points numbers give inward bowing of the south wall in inches, scaled from photo.

Figure 10. Inward bowing of the WTC 1 south wall of WTC 1 at 10:23 a.m. [20].

Figure 11. Collapse initiation of WTC 1 [20].

Fire Technology, DOI: 10.1007/s10694-012-0289-2

3.2 WTC 1 Structural Response to Aircraft Impact and Fire

Based on the aircraft impact, fire, thermal, and structural analyses, and consistency with the collected evidence, the following events describe the probable collapse sequence of WTC 2.

Aircraft Impact Damage. Loads carried by severed columns in the south wall and southeast corner of the core were redistributed to adjacent columns and to the east wall. As the core leaned toward the east and south, the exterior walls restrained the core movement. The core column loads were reduced by about 6 percent, the north wall loads decreased by about 10 percent, but the east wall loads increased by about 24 percent.

Sagging of Floors. Thermal expansion of the floors occurred early during the fires. As east floor temperatures increased, Floors 79 to 83 sagged and began to pull inward on the east exterior columns shortly after impact by as much as 1270 mm (50 in.), as shown in Figure 9b.

Bowing of East Wall. The inward bowing in the east wall, shown in Figure 12, steadily increased with time due to the effects of increasing temperatures, pull-in forces, and redistributed loads. As the columns bowed, their loads were transferred to adjacent columns, but the total column load on the east wall remained more or less constant after aircraft impact.

Unloading and Tilting of Core. As temperatures increased over time, plastic and creep strains in the core columns started to exceed the thermal expansion strains approximately 30 min after the aircraft impact, resulting in unloading of the east core columns. The core tilt increased toward the southeast and the east wall loads increased by about 29 percent and the north wall loads decreased by about 12 percent.

Buckling of East Wall and Collapse Initiation. Column buckling on the east wall started at the center and spread rapidly along both sides. As the east wall buckled, loads redistributed to the weakened core through the hat truss and to the east side of the south and north walls through the spandrel beams. The building section above the aircraft impact continued to rotate to the east as it began to fall downward, as shown in Figures 13. When the gravity loads could no longer be redistributed, WTC 2 collapse began at 9:58:59 a.m. EDT.

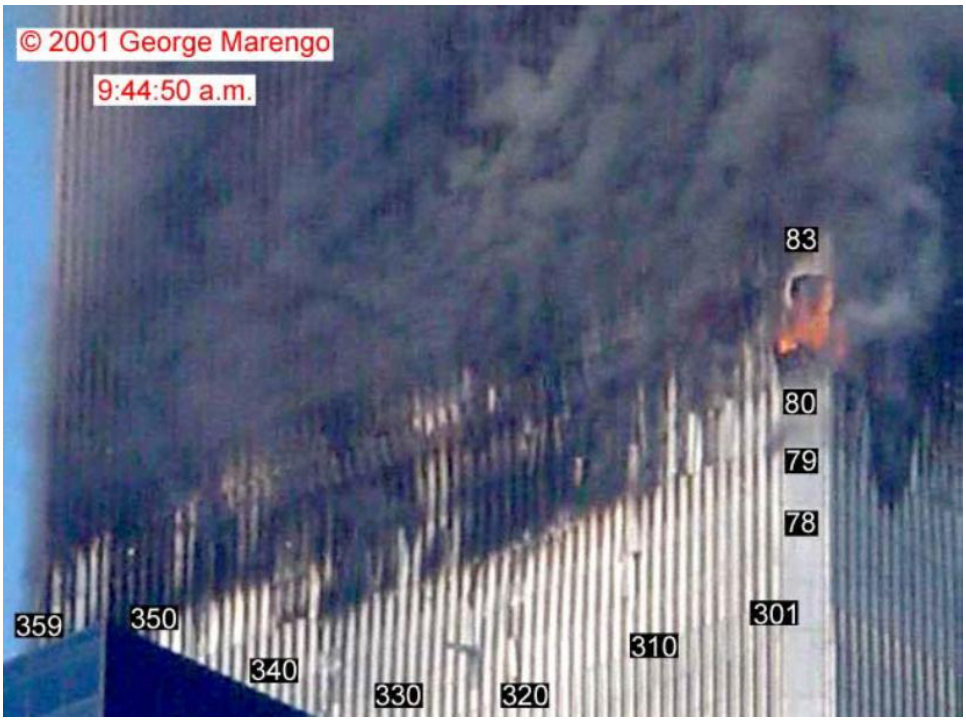


Figure 12. Inward bowing of east face of WTC 2 between Floors 79 and 83 at 9:44:50 a.m. [20].

Figure 13. Collapse initiation of WTC 2 [20].

3.3 Comparison of Observed and Simulated Events

The analysis results for WTC 1 and WTC 2 were compared with the visual evidence collected by NIST [32]. The WTC towers collapse sequence consisted of five main events: aircraft impact, core weakening, floor sagging and disconnection, inward bowing of exterior walls, and collapse initiation. The events that could be observed in collected visual evidence are listed in Tables 1 and 2. The simulations provide a rational method for determining the entire structural response, including events that could not be observed, such as core weakening.

The sequence of events simulated for the WTC 1 structural response to aircraft impact damage and fire effects matched the observed sequence of events, but the timeline lagged slightly (Table 1). Each structural analysis ran for a period of months due to the increasingly nonlinear response and accumulation of sequential component failures. The WTC 1 analysis was terminated after the core columns had weakened, shedding gravity loads through the hat truss to the south exterior wall columns, and the inward bowing of the south wall had reached about 1.1 m (43 in). As the fires were still heating the structure, the load shedding and inward bowing would have continued in the simulation.

The sequence of events simulated for the WTC 2 structural response to aircraft impact damage and fire effects matched the observed sequence of events, but the timeline events were somewhat earlier (Table 2). The WTC 2 analysis was terminated after the thermally weakened core columns shed gravity loads through the hat truss to the south and east exterior wall columns, and the building section above the impact area tilted to the south and east.

The level of agreement between observed and simulated events validates the sequential analysis approach, model development, and analysis results of each tower to the impact damage and fire events.

Table 1. Comparison of observed and simulated events for WTC 1.

Observed Events	Simulated Events
Following the aircraft impact, the tower still stood.	Following the aircraft impact, the tower was stable with significant reserve capacity.
The south wall first bowed inward at 10:23 a.m. from the 94th to 100th floors. The maximum visible bowing was 1.4 m (55 in).	The south wall bowed inward at 10:28 a.m. It extended from the 94th to the 100th floor, with a maximum inward bowing of about 1.1 m (43 in)
As the structural collapse began, the building section above the impact and fire zone tilted to the south and began to fall downward.	The south side continued to bow inward and weaken. The analysis was stopped as the initiation of global instability was imminent.
The time to collapse initiation was 102 min from the aircraft impact.	Instability was imminent at 100 min.

Table 2. Comparison of observed and simulated events for WTC 2.

Observed Events	Simulated Events
Following the aircraft impact, the tower still stood.	Following the aircraft impact, the tower was stable with significant reserve capacity.
The east wall bowed inward approximately 0.25 m (10 in) at Floor 80 at 9:21 a.m. and extended across most of the east face between the 78th and 83rd floors.	The inward bowing of the east wall had a maximum value of about 0.24 m (9.5 in) at 9:23 a.m. The bowing extended from the 78th floor to the 83rd floor.
The building section above the impact and fire area tilted to the east and south as the structural collapse initiated.	At the point of instability, there was tilting to the south and east.
The time to collapse initiation was 56 min after the aircraft impact.	The analysis predicted global instability after 43 min.

4.0 The Global Structural Response of the WTC 7

Two models of the WTC 7 building were developed for the analysis of the structural response to debris impact damage and fires, followed by a sequence of failures up to collapse initiation. The 16-story pseudo-static finite element model of WTC 7 determined the structural response to fire on Floors 7 to 9 and Floors 11 to 13. The 16-story model included the core, the exterior walls, and composite floors from the ground level to the 16th floor. The 47-story dynamic finite element model included the entire structure and determined the global response to the debris impact and fire damage when the initiation of collapse appeared imminent. The 16-story pseudo-static and 47-story dynamic analysis models used elements similar to those used in the component and subsystem analyses, see [2].

The nonlinear 16-story model was developed in ANSYS [30], and included connection models that captured failure of bolt shear, plate tear-out, or beam walk-off from the bearing seat, failure of shear studs within a composite floor system, buckling instability of beams and girders, and crushing and cracking of concrete floor slabs. Failure criteria identified when a structural component no longer contributed to the strength or stiffness of the structural system. Once a component failed, it was either softened (significantly reduced stiffness) or removed from the analysis to facilitate analysis convergence. For instance, concrete shell elements were softened if criteria for concrete crushing or cracking were met. By softening the shell element, loads applied to the shell elements remained active in the analysis. If a floor beam or girder met criteria for buckling, the beam elements were removed from the analysis. Component failures typically result in computational instability (ill-conditioned stiffness matrix); this approach allowed the analysis to progress beyond individual component failures.

The 47-story dynamic model, developed in LS-DYNA [33], included the following features: structural damage due to debris impact from the collapse of WTC 1; fire-induced damage from pseudo-static analysis; temperature-dependent mechanical properties for steel components; detailed modeling of connections and composite floor construction; component failures, including connections (e.g., bolt shear, plate tear-out, or walk-off of beam from its seat) and buckling of floor beams and columns; sequential failure of components and subsystems over the duration of collapse process; and dynamic effects of debris impact from falling components. The dynamic 47-story model was capable of explicitly modeling sequential failures, falling debris, and debris impact on other structural components. LS-DYNA was well suited for this type of analysis, since it can model dynamic failure processes, including nonlinear material properties, nonlinear geometry, material failures, contact between collapsing structural components, and element erosion based on a defined failure criterion. In addition, LS-DYNA can include thermal softening of materials and thermal expansion.

As indicated in [2], structural damage was observed between Floors 7 and 17 in the southwest quadrant of WTC 7 following the collapse of WTC 1. Debris impact damage was included in the dynamic model, but not in the pseudo-static model. The pseudo-static model could not simulate the load redistribution within the lower 16 stories, and inclusion of debris damage was not necessary for analyzing the fire-induced collapse initiating event that occurred in the northeast quadrant of the building.

Three different thermal cases were used in the heat transfer analyses and pseudo-static analyses. Case E used temperature data obtained from the fire dynamics simulation of the observed fires. Cases F and G increased and decreased the Case E gas temperature by 10 percent, respectively. These cases were within the range of realistic and reasonable fires in WTC 7 on September 11, 2001, and were judged to be within the range of uncertainty for the observed fires [22]. The analysis of fire growth and spread across each floor simulated direct heating by fire, as well as ‘preheating’ structural members as hot gases spread across the ceiling area. Analyses of three different thermal cases (E, F, and G) resulted in connection, beam, and girder failures occurring essentially at the same locations with similar failure mechanisms, but shifted in time between the three thermal cases [2].

Similar to the procedure for the WTC towers, ranges of temperature time intervals for WTC 7 structural elements were evaluated. For the WTC 7 analyses, which had fires burning for hours, temperature data for each node were input at 30 min intervals to the pseudo-static analysis for fires observed on Floors 7 to 9 and Floors 11 to 13. The temperatures were linearly ramped between the starting and ending temperature input for each time interval.

When the pseudo-static analysis reached a point where collapse initiation appeared imminent (failures in the floor systems around columns in the east floor area reached a state where column buckling was imminent), the accumulated damage and temperatures of structural components at that time were input to the dynamic model. The damage state of the connections was indicated by a numerical value ranging between 0.0 for no damage and 1.0 for full damage (i.e., no remaining capacity).

4.1 WTC 7 Structural Response to Debris Impact and Fire

Based on the final fire, thermal, and structural analyses, and their consistency with the collected evidence, the following events describe the probable collapse sequence for WTC 7. Only Case F (+10 percent) results are described.

Initial Local Failure for Collapse Initiation. Fires on the lower floors (Floors 7 to 9 and Floors 11 to 13) grew and spread since they were not extinguished either by the automatic sprinkler system or by firefighting because water was not available. By 3:00 p.m. EDT to 4:00 p.m. EDT, these fires were generally concentrated in the northeast region. Local fires on the upper floors (Floors 19, 22, 29, and 30) were not observed after approximately 1:00 p.m. EDT.

Even with intact SFRM, the fires heated the structural frame and slab. Prior to the fires reaching the northeast corner, the structural floor framing had been heated to 100 °C to 200 °C (212 °F to 392 °F), as shown in Figure 14a.

The long span floor framing on the east side of WTC 7 thermally expanded and failed the shear stud connections to the slab over time (see [1] for framing details). Drawings showed shear studs along floor beams but not along girders. The shear capacity of 28 shear studs on a floor beam in the northeast corner at ambient temperature was estimated to be 2.4 MN (546 kip), which is less than the force produced in a fully restrained floor beam with a 100 °C (212 °F) temperature increase. Therefore, shear stud failures between the lightly restrained beam and highly restrained slab were expected to occur.

As illustrated in Figure 15, the exterior framing was much stiffer laterally than the interior girder, so the thermal expansion of the floor beams pushed the girder laterally. As the concrete slab heated, cooler adjacent slab sections restrained its thermal expansion. Girder walk-off from the seat connection occurred when the beams pushed the girder laterally, sheared the bolts at the seated connection, and then continued to push the girder until it walked off the bearing seat. Failure of the Floor 13 system surrounding Column 79 triggered a cascade of floor failures. This, in turn, led to loss of lateral support for Column 79 over nine stories, which, in turn, led to the buckling of Column 79.

Progression of Failure. The buckling of Column 79 triggered a vertical progression of floor system failures up to the east penthouse and the subsequent failure of adjacent interior columns (specifically, Columns 80 and 81), as illustrated in Figures 16 and 17. Figure 16 shows the progression of floor failures until the interior columns on the east side of the building were unsupported and they buckled. Column 79 buckled first, followed by Columns 80 and 81. Figure 17 is a close-up view of the analysis state shown in Figure 16c. The floor system failures spread to include the entire east portion of the building. Interior columns then buckled in succession from east to west due to loss of lateral support from floor system failures, forces exerted by falling debris, and loads redistributed from other buckled columns, until all interior columns between Floors 9 and 14 had buckled.

Global Collapse. The exterior columns were left laterally unsupported in the east, south, and north faces (the west face floors remained intact above Floor 9 as no fires were observed above this floor on the west side). An exterior column adjacent to the debris impact zone buckled first. All the exterior columns buckled between Floors 7 and 14, as shown in Figure 18, as load redistributed during the downward movement of the building core. The building above the buckled-column region then moved downward in a single unit and began to collapse at 5:20:52 p.m. EDT.

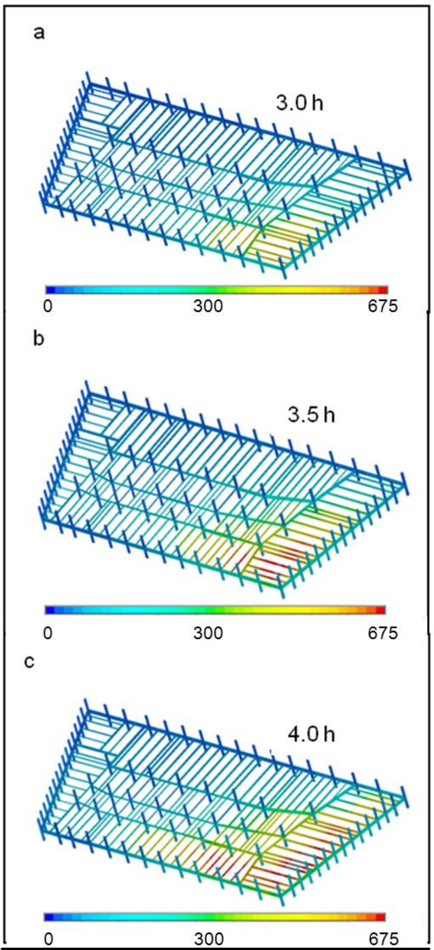


Figure 14. Temperatures (°C) of Floor 13 framing between 3.0 h and 4.0 h of heating in the thermal analysis.

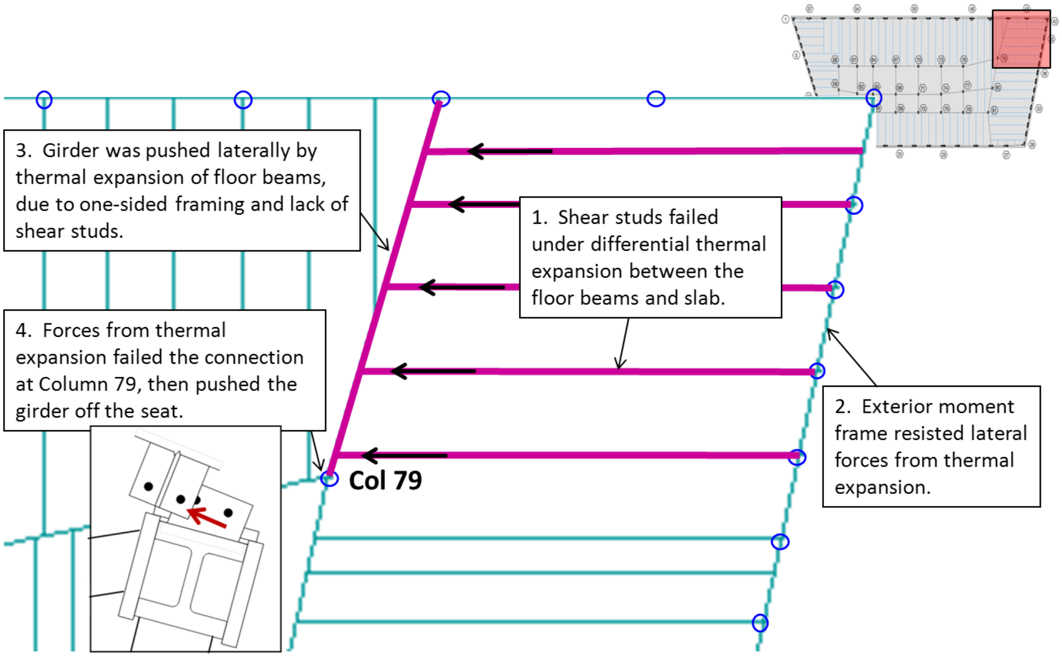
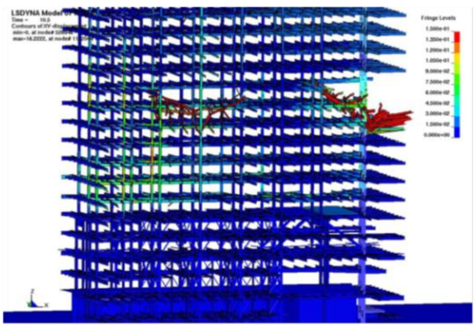
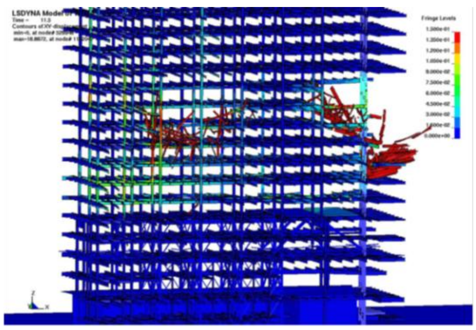


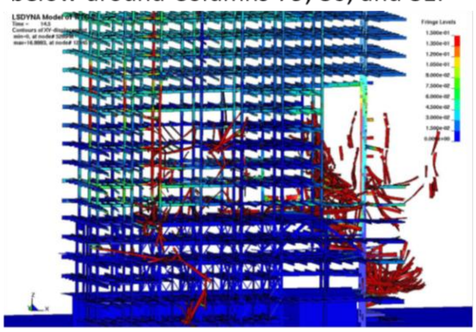
Figure 15. Thermal response of northeast floor framing.



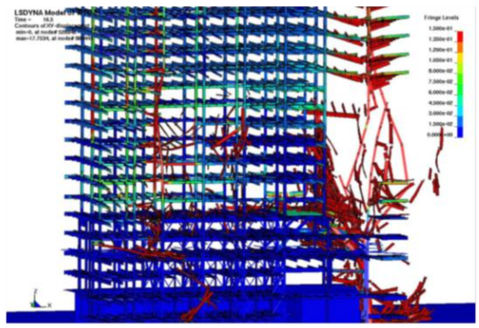
A. At -5.5 s, collapse of floors onto the floor below around Columns 79, 80, and 81.



B. At -4.5 s, floors continue collapsing around Columns 79, 80, and 81.



C. At -1.5 s, Column 79 buckles, quickly followed by the buckling of Columns 80 and 81.



D. At 0.5 s, columns above the buckled columns descend, followed by the pent-house structure on the roof of WTC 7 at 0 s.

Figure 16. Sequence from the dynamic structural analysis showing floor collapse to column buckling of interior framing at lower floors [26]. Times are relative to the downward movement of the east penthouse at 0 s.

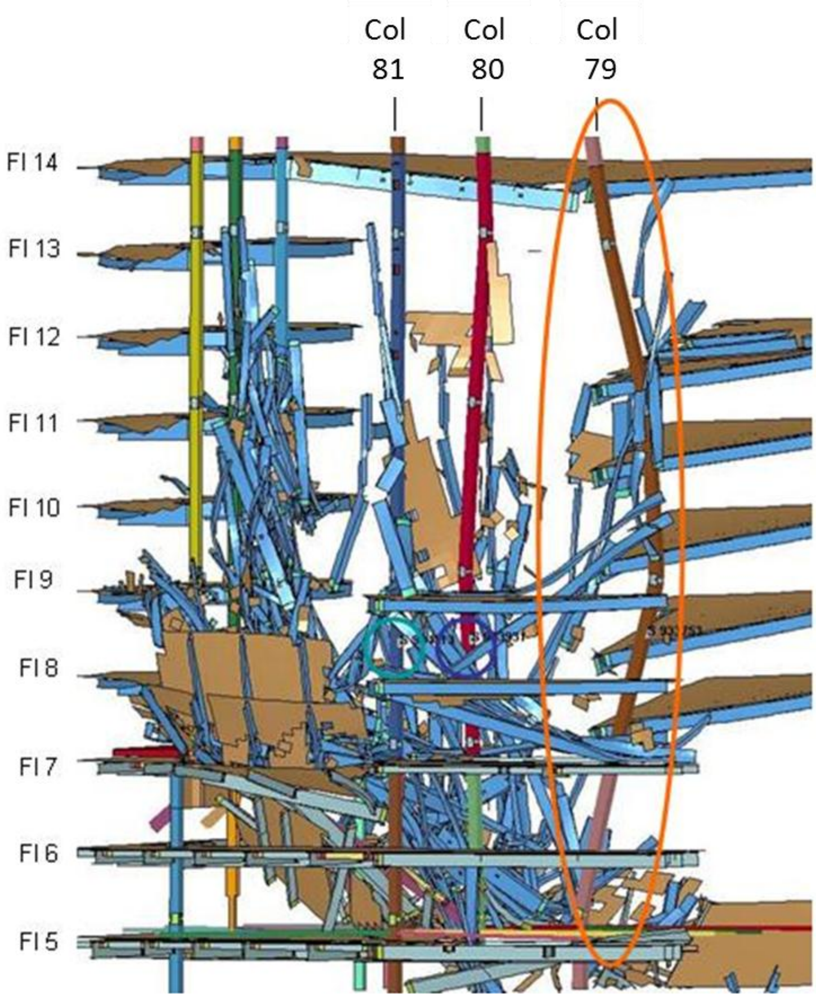


Figure 17. Close-up view of Column 79 buckling from Figure 16(c) [22].

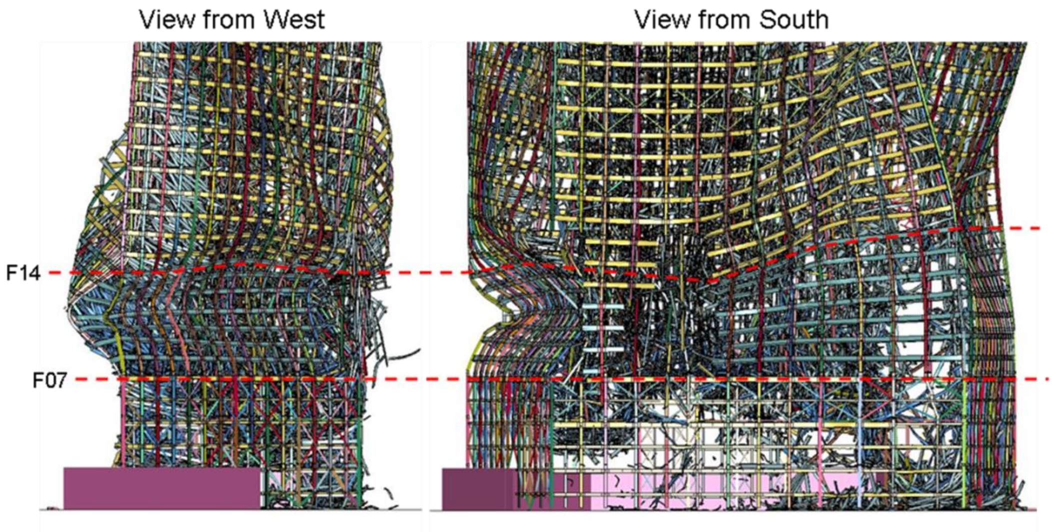


Figure 18. Buckling of exterior columns at lower floors from the dynamic structural analysis at 8.6 s [22].

4.2 Comparison of Observed and Simulated Events

Table 3 compares observed events from the visual evidence with the results from two global dynamic analyses (with and without debris impact damage). The event times are relative to the descent of the east penthouse. Photographs and videos of WTC 7 at the time of global collapse only showed the upper portion of the building (Figure 19). The simulations provide a rational method for determining the entire structural response, including events that could not be directly observed, such as the interior buckling of columns.

An east-west vibration of the building was observed in a video before the east penthouse began to move downward (Figure 18). The horizontal building motion started at nearly the same time as the cascading floor failures started in the LS-DYNA analysis (-6.5 s), which preceded the buckling failure of Column 79. The times for the first four events were quite similar between the visual evidence and the analysis results, and independent of the debris impact damage. The failure of floors surrounding Column 79 and the buckling of Column 79 could not be directly observed from any visual evidence. However, vibration analysis of video segments prior to collapse initiation [22] revealed horizontal motions 6 s before the east penthouse began to descend. The motion started at nearly the same time as the floor failures predicted in the analyses (6.6 s before the descent of the penthouse). The analyses indicated that Column 79 buckled approximately 1.3 s prior to the descent of the east penthouse.

The horizontal progression of interior column buckling also could not be directly observed in the videos. Comparing the results of the two analyses, with and without debris impact damage, the process took almost twice as long for the analysis without debris impact damage. For the analysis with debris impact damage at the southwest corner, some of the interior columns on the west side began to buckle at the same time as the columns near the middle of the core, thus shortening the total time for all interior columns to buckle. The lack of debris damage on the west side resulted in a more uniform sequence of column failures.

The initial downward movement of the north face roofline was observed at 6.9 s. The dynamic analyses straddled that value. The simulation results of the west penthouse descent also bracketed the event time.

Table 3. Comparison of observed and simulated events for WTC 7[a].

Event Time (s)[b]	Observed Events	Analysis Time with Debris Impact Damage (s)	Analysis Time without Debris Impact Damage (s)
≈ -6.0 s[c,d]	Start of cascading failure of floors surrounding Column 79	-6.6 s	-6.6 s
n.o.[d]	Buckling of Column 79, followed by buckling of Columns 80 and 81	-1.3	-1.4
≡ 0	Start of descent of east penthouse	≡ 0	≡ 0
2.0	Descent of east penthouse below roofline	2.4 - 2.7	2.3 - 2.6
n.o.[d]	Buckling of columns across core, starting with Column 76	3.5 - 6.1	3.2 - 13.5
6.9	Initial downward motion of the north face roofline on the east side	6.3	9.8
8.5	Descent of the east end of the screenwall below the roofline	7.3 - 7.7	8.7 - 9.2
9.3	Descent of the west penthouse below the roofline	6.9 - 7.3	10.6 - 10.9

a. The times cited relative to the start of the descent of the east penthouse.
b. Based on photographic and video analyses [22].
c. Based on vibration analysis of video prior to collapse initiation [22].
d. Not observable in the visual evidence since these columns were in the building interior.

As the global collapse was underway, the uncertainty in the progression of failures greatly increased, due to the random nature of the interaction, break up, and falling of debris. The uncertainty influenced the deterministic physics-based analyses, and the details of the progression of the horizontal failure and final global collapse were increasingly less precise. Thus, the mechanisms of building failure and collapse were quite different in the two analyses. In the analysis without debris impact damage, the exterior columns buckled near mid-height of the building. In the analysis with debris impact damage, the exterior columns buckled between Floors 7 to 14, due to the influence of the debris damage.

Figure 19. North face of WTC 7 approximately 1 s after the east penthouse began to move downward [22].

5.0 Progress After the NIST WTC Investigation

The WTC analyses by NIST and others led to renewed research in a number of topics related to structural system response in composite floor systems, steel framing connections, and structural response to fire. Recent studies on composite floor system behavior are developing improved design guidance for predicting the strength and performance of shear studs in composite floors [34, 35, 36, 37, 38] for room temperature conditions. A study on the collapse resistance of composite floor systems [39] evaluated the lateral strength of shear connections under tensile loading.

Research continues on the performance of composite beams and floor systems in fire conditions to determine their response during fire events for thermal restraint conditions and various thermal protection and fire scenarios [40, 41, 42]. The effect of thermal gradients on the performance of steel framing in fire conditions are studied in [43, 44]. The performance of floor framing connections in fire are being characterized for shear and moment connections [45, 46, 47]. Validated models of shear connections at a reduced level of detail, while capturing failure mechanisms, have been developed for ambient conditions [48]. Similarly, validated models of endplate connections have been developed for ambient and fire conditions [49].

Methods to evaluate the structural response of tall buildings to multi-floor fires continue to be developed. A study of fire effects on long span truss floor systems in tall buildings [50] with 2D models found that large displacements may occur in the floor systems without failure, and that load redistribution paths between the core and exterior columns have a significant impact on structural robustness. However, all beam-column connections were pinned so that connection failure or influence on the floor response was not considered.

A simplified method was developed to identify the limit state of collapse for multiple floor fires [51] without consideration of any particular design fire, and with calculations that can be performed in minutes. The procedure is based on the assumption that, for significant multi-floor fires, a number of floors will reach a state of catenary action that leads to destabilizing pull-in forces on the exterior columns.

Parametric studies for high-rise steel buildings subject to fire [52] considered the effects of 3D full frame models versus 2D plane-frame models, and uniform versus gradient temperature profile across a steel member cross-section. Results indicated that the 2D plane frame model can be reasonably used in some cases (e.g., a moment-resisting frame). Models with uniform beam temperature obtained reasonable estimates of the interaction between beams and columns. However, thermal gradients should be included when prediction of deflections or plastic limit state behavior are important.

This renewed and expanded interest in understanding the structural response to fire is encouraging and needed. Many commercial buildings now have floor spans on the order of 12 m to 15 m (40 ft to 50 ft), and thermal expansion effects within insulated floor framing can be significant during a fire. Current practice protects structures from fire effects using comparative performance data from standard fire tests. While this approach works well for many structures, designers and engineers are unable to predict if a structure so protected is susceptible to a fire-induced failure. Most structural-fire tests in the U.S. do not include connections for framing members and are limited to lengths of 5 m (17 ft) for floor systems.

6.0 Summary

Robust methods to evaluate the response of structures to multi-floor fires are needed to advance the design and performance of buildings in uncontrolled fire. The inclusion of the following factors made a significant impact on the structural response in the WTC analyses: full-floor fire simulations, the role of connections in the time-varying response of the floor system to fire, and structural models that account for local and global effects of heating as well as all possible failure mechanisms. However, frequently the global response to multi-floor fires are evaluated with tools developed for compartment fires, many failure mechanisms are ignored or excluded, and model connections are represented as fixed or pinned. The floor framing connections can greatly modify the response of the structural floor system and its interactions with the columns. If a connection should degrade or fail under either thermal expansion or contraction effects, and the modeling does not account for the degradation, a false conclusion about the system performance may result.

The WTC studies clearly illustrate the need for testing of structural systems, including connections, under realistic fire conditions. Due to the expense and difficulty of conducting such tests, most structural-fire testing is conducted on components or subsystems with furnaces or heating elements. However, the response of components or subsystems is inadequate for predicting the full structural system response to fire effects. Test results of full scale structural testing under real fire conditions is needed to validate and advance the design and analysis of structural system response to fire effects.

6.1 WTC Towers

Inward bowing of the exterior walls in both WTC 1 and WTC 2 was observed only on the face with the long-span floor system. In WTC 1, this was found to be the case even though equally extensive fires were observed on all faces. In WTC 2, fires primarily burned along the east face with a long-span floor. Fires were not observed on the long-span west face and were less intense on the short-span faces. Inward bowing of the exterior wall, due to the sagging of the long-span floors, was a necessary but not sufficient condition to initiate collapse. In both WTC 1 and WTC 2, significant weakening of the core due to aircraft impact damage and thermal effects was also necessary to initiate building collapse. The tower structures had significant capacity to redistribute loads (a) between the core and exterior walls via the hat truss, and (b) from bowed exterior walls to adjacent exterior walls via the spandrel beams.

Both WTC towers had sudden failures of a number of exterior and interior columns and floor sections following the aircraft impact but remained stable until the steel framing with dislodged SFRM was weakened by multi-floor uncontrolled fires. The following events were common to both WTC towers:

- Gravity loads redistributed between adjacent core columns and between the core and exterior walls through the hat truss.
- The core was weakened due to severed columns and elevated temperatures in columns with dislodged SFRM.
- Long span floors sagged due to the combined effects of dislodged SFRM, elevated temperatures, and buckling of floor truss web members.
- Exterior walls bowed inward, due to pull-in forces from sagging floors and redistributed loads from core columns, and buckled.
- Inward bowing of the exterior walls in both WTC 1 and WTC 2 was observed only on the face with the long-span floor system.

6.2 WTC 7

A two-phased simulation approach was used that included a 16-story pseudo-static analysis of the structural response to fire up to collapse initiation and a 47-story dynamic model for simulating the sequence of failures from the initiating event to the start of global collapse.

WTC 7 had uncontrolled multi-floor fires that slowly heated the insulated steel until local floor failures led to the buckling of Column 79 in the northeast corner, when the local column failure led to a fire-induced progressive collapse. The collapse of WTC 7 represents the first known instance of the total collapse of a tall building primarily due to fire.

The following events in WTC 7 led to the collapse initiation event.

Fire Technology, DOI: 10.1007/s10694-012-0289-2

- The long span floor beams heated more quickly than the floor slab and, therefore, experienced greater thermal expansion than the slab. The difference between thermal restraint conditions for the steel framing (one end of the beams framed into a girder without shear stud ties to the slab) and the concrete slab (the slab was continuous across the interior girder and any lateral movement was resisted) led to failure of shear stud connections between the floor beams and slab.
- The asymmetric floor framing exerted one-sided lateral forces on the girder, and the bolts at the column seat connection were sheared. Continued lateral forces from the floor beams as they heated pushed the girder off of its seat.
- The girder "walk-off" resulted in collapse of the floor onto floors below which had also been weakened by the uncontrolled fires. A cascade of floor failures occurred around Column 79, due to the effects of multi-floor fires and thermal weakening.

6.3 All Three Buildings

Analyses for the three buildings were unique in that they explicitly modeled connections and simulated a series of component and subsystem failures up to collapse initiation. Each analysis used a range of parameter values in multiple input files to account for uncertainties in the input data and its effect on the simulation results.

Features and events that were common to all three buildings included:

- Open floor plans with floor fires rather than compartment fires
- Uncontrolled multi-floor fires of normal building contents
- Preheating of structural framing across open floor plans by fire
- Long floor spans (nominally greater than 18 m or 50 ft)
- Thermal restraint effect on long floor spans.

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