Executive Summary
Extracted from

NIST NCSTAR 1-6 (Draft)

Federal Building and Fire Safety Investigation of the World Trade Center Disaster

Structural Fire Response and Probable Collapse Sequence of the World Trade Center Towers (Draft)

EXECUTIVE SUMMARY

E.1 PURPOSE AND SCOPE

One of the four objectives of the National Institute of Standards and Technology (NIST) investigation of the collapse of the World Trade Center (WTC) towers was to determine why and how the two towers (WTC 1 and WTC 2) collapsed following the initial impacts of the aircraft. Both the north and south towers of the World Trade Center were severely damaged by the impact of Boeing 767 aircraft, yet they remained standing for some time. The ensuing fires were observed to move through both buildings and eventually, both buildings collapsed. The probable collapse sequence for each of the WTC towers as well as the extent and relative importance of the damage caused by the aircraft impact and subsequent weakening by fires were investigated under this project, *Structural Response and Collapse Analysis of WTC Towers to Aircraft Impact Damage and Fire Conditions*.

Events that played a significant role in the structural performance of the towers were the aircraft impact, rapid ignition of fire on multiple floors, and the growth and spread of fire in each tower. Detailed information was required on the condition of the structural system and its passive fire protection system, both before and after the aircraft impact, and during the ensuing fires that elevated temperatures in the structural members. The purpose of this project, then, was to analyze the response of the WTC towers to fires—both with and without aircraft damage—and to determine the probable sequence of structural collapse for each tower. Specifically, the *Structural Response and Collapse Analysis* project intended to:

- Determine the pre- and post-aircraft impact condition of the passive fire protection used to thermally insulate the structural members and provide resistance to fire damage,
- Conduct tests of structural components and systems under fire conditions to quantify their behavior,
- Evaluate the response of floor and column systems under impact and fire conditions to understand their response,
- Evaluate the response of the WTC towers under impact and fire conditions, with and without aircraft impact damage, and
- Develop and evaluate failure hypotheses, resulting in the probable sequence of structural events leading to collapse for each WTC tower.

The unprecedented complexity and sophistication of these analyses required the use of various strategies for managing the computational demands while adequately capturing the essential physics. The overall approach—from impact analysis to collapse initiation—combined mathematical modeling, statistical and probability-based analysis, laboratory testing, and analysis of photographic and videographic records.

Data were collected from a number of sources and included structural plans and specifications; thermal and mechanical (adhesion/cohesion) properties of fire resistant materials; the thickness and condition of the passive fire protection in the towers; and recorded observations of structural events subsequent to aircraft impact and prior to collapse. Information about tower construction was obtained from original drawings, design and construction specifications, project documents including correspondence and reports, and records provided by the Port Authority of New York and New Jersey (PANYNJ), Leslie E. Robertson Associates (LERA), Silverstein Properties, and a number of contractors that had worked on the design, construction, or modifications of the towers. Information about the events that occurred in each tower on September 11, 2001, was obtained from analysis of available photographic and videographic records, eyewitness accounts, and mechanical and metallurgical analysis of recovered structural steel.

Computer simulations were used to model the complete sequence of events leading to the initiation of collapse of the WTC towers. The analyses simulated the damage to the towers resulting from aircraft impact, the spread of multi-floor fires, the heating and thermal weakening of structural components, and the progression of local structural failures that led to the collapse of the buildings. The structural response analyses relied upon the following information:

- Reference global structural models of the WTC 1 and WTC 2 towers, and typical floor and exterior wall subsystem models (NIST NCSTAR 1-2)
- Extent of damage to the structural systems and interior contents of the WTC 1 and WTC 2 towers resulting from aircraft impact (NIST NCSTAR 1-2)
- Temperature-dependent mechanical properties of the steels, welds, and bolts used in the construction of the towers, including elastic, plastic, and creep properties from 20 ºC to 700 ºC (NIST NCSTAR 1-3)
- Time-temperature histories for structural components and connections for standard fires (e.g., ASTM E 119) and actual fires based on fire dynamics simulations (NIST NCSTAR 1-5).
- Photographic and videographic records with time stamps that documented the observed sequence of events (NIST NCSTAR 1-5).

E.2 METHODOLOGY AND ANALYSIS RESULTS

E.2.1 Overview and Approach

The interdependence of the analyses of significant events is illustrated in Fig. E–1. Reference structural models were first developed and used to determine the baseline performance of each tower prior to September 11, 2001. The reference models were then used as a basis for the aircraft impact damage models and the structural response models to ensure consistency between structural models. The aircraft impact analysis determined damage to the interior of the building including the structural system, fireproofing, partition walls, and furnishings for each tower. The analysis also provided an estimate of the fuel dispersion in the towers. These results provided initial conditions to the fire dynamics analysis, thermal analysis, and structural analysis. The fire dynamics analysis simulated the growth and spread of fires and produced gas temperature histories for each floor involved in fire. The fire dynamics model accounted for window breakage and damage to interior partition walls and floors (both affecting ventilation conditions), and the distribution of debris and fuel. The thermal analysis used the heat transfer model to determine temperature histories for the various structural components. The thermal analysis required input from the structural analysis model, fire dynamics analysis results, damage to fireproofing, and temperature-dependent thermal material properties. The structural temperature histories, also referred to as thermal loads, were input to the structural analysis, along with the structural impact damage and temperature-dependent material properties, to determine the structural response of each tower.

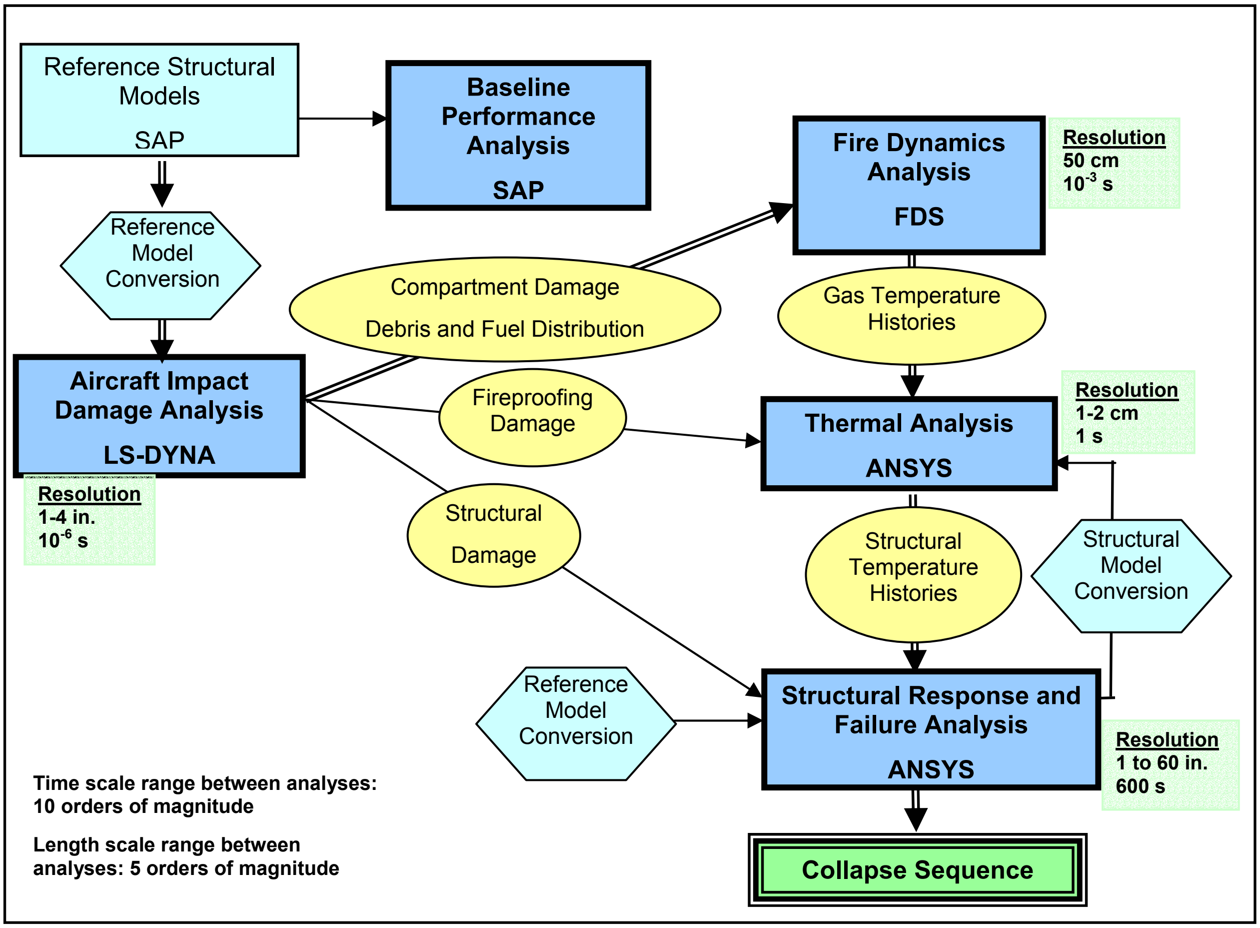


Figure E–1. Critical analysis inter-dependencies.

The WTC towers were large, complex structural systems. To include all of the structural components and connections and their associated behavior and failure mechanisms using refined finite element meshes would have been prohibitive. The analysis approach used was a variant of the well-established sub-structuring approach, adapted for the analysis of structures with highly nonlinear behavior that progressed from individual components to major subsystems to global systems, as shown in Fig. E–2. The component analyses were conducted to identify critical behavior and failure mechanisms that contributed to the global structural response of each tower. The subsystem analyses incorporated the behavior and failure mechanisms identified in the component studies, with modifications to reduce the model size and complexity, thereby enhancing computational performance, without adversely affecting the quality of the results. Whenever modeling modifications were used, they were validated against the detailed component model results. The global analyses incorporated critical behavior and failure mechanisms, determined from subsystem analyses, while making necessary modifications in the level of modeling detail.

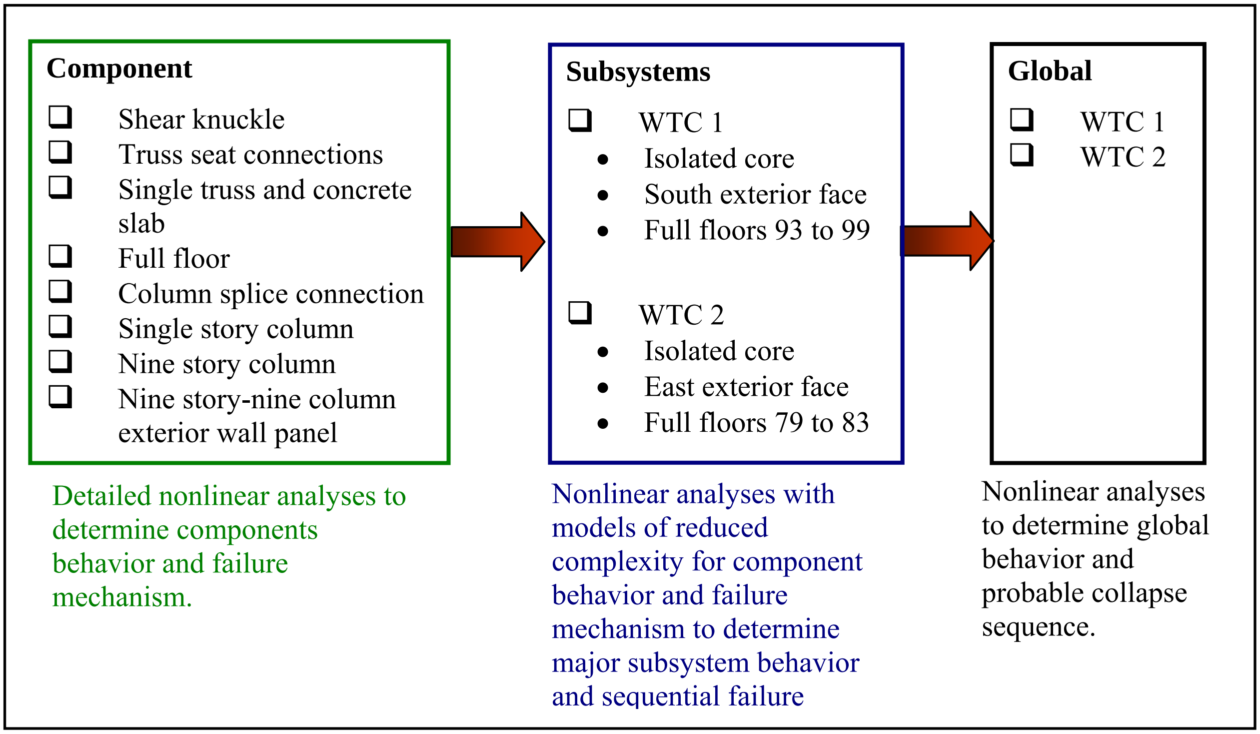


Figure E–2. Structural Analysis Sequence.

Analyses of the global behavior and determination of probable collapse sequences for both WTC 1 and WTC 2, which included work performed by other projects, was divided into the following tasks:

A. Develop finite element models based on reference models. Reference models faithfully represented the actual structures. These reference models became the basis for all subsequent finite element analyses.

B. Develop the constitutive relationships for the materials used in the construction of the towers. Mechanical and chemical properties were determined for steel specimens recovered from the WTC site to assure that the materials used were in conformance with properties specified in the original design. The mechanical properties at high loading rates for the aircraft impact analyses and at elevated temperatures (from room temperature to 800 °C) for the thermal and structural analyses were also determined from the steel specimens.

C. Characterize the passive fire protection applied to the structural steel. Neither the type of materials nor the required thicknesses of fire protection were identified in the contract documents or specifications. Estimates of the characteristics and condition of fireproofing materials were needed for the thermal and structural modeling of the towers.

D. Conduct standard fire resistance tests of composite truss floor system. Tests were conducted to: (1) establish the baseline fire resistance rating of the composite truss floor system used in the WTC towers, (2) understand the influence of thermal restraint by testing the floor system under both thermally unrestrained and restrained conditions, and (3) provide experimental data to validate and provide guidance to the development of the floor models and to interpret the analyses results.

E. Establish the damage to the structure, fireproofing, and partition walls as a result of aircraft impact. The aircraft impact resulted in significant damage to the exterior, floor, and core structures of the buildings. The jet fuel dispersed inside the towers ignited the building contents and furnishings as well as influenced the amount of oxygen reaching the fires. The passive fire protection of steel components was dislodged in areas of direct debris impact.

F. Document observations and data related to structural events. NIST validated analysis results with key observations obtained from its extensive collection of over 7,000 photographs and over 150 hours of videotape documenting the events at the World Trade Center on September 11, 2001. Key observations were used in the analyses in three ways: (1) to determine input parameters, (2) to impose time-related constraints upon an analysis, or (3) to validate analysis results.

G. Compute temperature histories for structural components subjected to fires. To determine how the towers were affected by the fires, estimates of the growth and spread of fires over time were developed using fire dynamics simulations. Temperature histories of the steel structural components and concrete floor slabs were predicted in thermal analyses.

H. Conduct component and subsystem analyses. These analyses provided understanding of the nonlinear behavior of structural components and subsystems under gravity and thermal loading and were used to develop reduced models for the global analyses. The components and subsystems considered included: (1) typical floor subsystem with (a) the shear knuckles, (b) truss seats, and (c) a single truss and concrete slab section; and (2) a nine-story by nine-column exterior wall subsystem with (a) bolted connection between exterior columns, (b) bolted connection between spandrels, (c) single exterior columns with spandrel sections, and (d) single exterior wall panel with three columns and three spandrels.

I. Conduct analyses of major subsystems. Analyses of three major subsystems - the isolated core framing subsystem, an exterior wall subsystem, and the composite floor subsystems - were analyzed to determine their ability to resist and redistribute loads after impact damage and response to elevated temperatures. The subsystem models used reduced models from the component analyses, which kept the analysis tractable while including nonlinear features and failure modes. These analyses were crucial for determining critical structural behaviors, including floor sagging under thermal loading, the resulting pull-in forces, and the inward bowing of the exterior walls.

J. Conduct a separate global analysis for each tower. These analyses determined the relative roles of impact damage and fires with respect to structural stability, sequential failures of components and subsystems, and probable collapse initiation sequences. Each global model was first evaluated for stability under gravity loads with structural impact damage. Temperature histories were applied in 10 min intervals and linearly ramped to the next temperature state. Pull-in forces from sagging floors were also applied during the appropriate 10 min intervals. The question of how the WTC towers would have responded to the same fires without the aircraft impact damage was considered to determine the vulnerability of the towers to collapse initiated by conventional large fires.

K. Determine the probable collapse sequence for each tower. A probable collapse sequence for each tower was determined. The collapse sequences were evaluated against key observables.

E.2.2 Structural Response

To conduct the global analysis of each tower, input data were collected from numerous sources, including fire dynamics, thermal, and impact analyses, as already described.

Thermal analyses to simulate the elevated temperatures of the structural components and consequent weakening required an assessment of the condition of the fireproofing, including its thicknesses and thermal properties. Additionally, tests of the WTC floor system under standard fire conditions provided insights into the dominant behavior of the floors at elevated temperatures and allowed validation of analytical results. Interpretation of the aircraft impact study results led to a determination of likely damage to load bearing structural elements and an estimation of damage to, and consequent loss of, passive fire protection of the floor trusses, core columns and beams, and exterior columns and spandrels. Properties of the materials of construction, including mechanical properties at room and elevated temperatures as well as thermal characteristics, were needed. The structural analyses of components, subsystems and, ultimately, the global systems could be accomplished with this information.

Passive Fire Protection for Structural Components

Passive fire protection delays the transfer of heat to structural components by providing an insulation barrier. Increasing thickness of passive fire protection materials, commonly referred to as fireproofing, correspondingly increases the time delay before the structural component temperature begins to rise. The amount of time delay for a given thickness of fireproofing is not predicted for design purposes because the actual fire conditions vary; instead, the relative performance is defined by comparative testing with the American Society of Testing and Materials (ASTM) Standard Fire Test.

The structural steel in the WTC towers was sprayed with fire resistive materials (SFRMs) or protected with rigid fire-rated gypsum panels. SFRMs are supplied as dry ingredients, and water is added at the time of application. The water mixes with the cementitious materials and allows the SFRM to adhere weakly to the steel. With time, the cementitious materials harden, and excess water evaporates resulting in a covering of insulation with some cohesive strength.

Three SFRM products that were used in the towers include:

- CAFCO BLAZE-SHIELD DC/F for floor trusses, core columns, and the exterior surfaces of the exterior columns and spandrels
- CAFCO BLAZE-SHIELD II for upgrades to floor trusses, which started in the 1990's
- W.R. Grace and Co., Monokote (sprayed cementitious vermiculite) for the interior surfaces of the exterior columns and spandrels

The gypsum panels were used to form fire-resistant enclosures around steel core columns, stairwells, mechanical shafts, and the core area in the towers. The core column fireproofing varied according to the column location and exposure to occupied spaces. Column surfaces in public access areas were protected with gypsum enclosures while the remaining surfaces were protected with SFRM.

The following information was required to determine the in-place condition of the passive protection before and after aircraft impact and to conduct thermal analysis of structural components:

- Thermophysical properties of the passive fire protection materials,
- Effect of gaps in thermal insulation and variability of insulation thickness,
- Effective thickness of thermal insulation for use in thermal-structural analyses that accounts for thickness variability effects,
- Adhesive and cohesive strengths of CAFCO SFRM products (vermiculite product is no longer available).

Thermophysical properties were determined with ASTM standard tests for CAFCO BLAZE-SHIELD DC/F, CAFCO BLAZE-SHIELD II, and Monokote MK-5 SFRM products and for gypsum board. The specify heat, thermal conductivity, and density of each material was determined for temperatures ranging from 25 °C to 1200 °C. The standard tests used for SFRM products were ASTM C 1113 (1999), ASTM E 1269 (2001), ASTM E 1131 (1998), and ASTM E 228 (1995). The standard tests used for the gypsum board products were ASTM D 5334 (2000b) and ASTM E 1269 (2001). Densities were calculated from the thermogravimetric analysis and linear thermal expansion measurements.

Analyses showed that when the SFRM thickness is variable, the isotherms in the steel depend upon the shape of the SFRM surface contour. Thus, the temperature history at any point in the steel depends on the local thickness of the insulation. It was shown that an increase in thickness variability reduced the time to reach a certain temperature. In addition to the effect of variation in thickness, the effect of gaps in the SFRM coating was studied. As expected, thermal analysis results indicated that the exposed steel heated quickly and transmitted heat to the adjacent interior steel. However, the temperature rise quickly dissipated as the distance from the gap increased. Review of available photographs showed that gaps were a relatively infrequent occurrence in most floor truss areas. Because there was insufficient information to determine the frequency of occurrence of these gaps or their typical locations, insulation gaps were not considered in the thermal modeling.

SFRM thickness measurements were determined from analysis and interpretation of photographs showing the condition of the originally applied material. Finite element simulations were used to determined a thermally equivalent uniform thickness of SFRM for the original variable-thickness fireproofing on the floor trusses. These values were used in the thermal analyses for determining temperature histories of structural components.

No information was available about the condition of fireproofing for the exterior columns and spandrel beams, and little information was available for the core beams and columns. For thermal analyses of the towers, the SFRM on these elements was taken to have uniform thicknesses equal to the specified thickness.

The adhesive strength of CAFCO BLAZE-SHIELD DC/F to steel coated with primer paint (average value of 171 psf to 185 psf) was found to be a third to a half of the adhesive strength to steel that had not been primed (average values of 450 to 666 psf). The SFRM products used in the WTC towers were applied to steel components with primer paint. Cohesive strengths varied from average values of 367 psf to 610 psf.

Tests of Truss Floor Components and Subsystem

Review of available documents indicated that the fire performance of the composite floor system of the WTC towers was an issue of concern to the Port Authority and its contractors during the original design and throughout the service life of the buildings. NIST conducted a series of four standard fire tests to establish the baseline performance of the floor system of the WTC towers as they were originally built, to differentiate the factors that most influenced the response of the floors, and to study the procedures and practices used to accept an innovative structural and fireproofing system. The ASTM E 119 furnace tests were performed on representative floor sections with spray-applied fire resistant materials (SFRM) for the as-specified thickness of 0.5 in. given in the design documents and the average as-built thickness of 0.75 in. that was applied before a program was established in the 1990's to upgrade the truss SFRM thickness to 1.5 in. The conditions in the standard test specified a prescribed temperature rise and duration until failure criteria were met; the estimated fire conditions in the WTC Towers imposed varied heating and cooling conditions as the fires grew and spread.

The tested floor assemblies were similar though not identical to steel-joist-supported concrete floors that are widely used in low rise construction. The test results provided valuable insight into the behavior of these widely used assemblies and also identified issues that require further study for other types of structural components such as beams, girders, columns, trusses, etc.

The tests showed that the floors were capable of considerable sagging without collapse. The tests also showed thermal damage to the bridging trusses and buckling of compression diagonals and the vertical strut near the supports. No evidence of knuckle failures was seen in the tests.

The NIST tests have identified areas where further study related to the standard test method is warranted. Among the issues related to the test method that NIST identified as requiring further study are:

- the scale of the test for prototype assemblies that are larger than the tested assemblies,
- the effect of restraint conditions on test results,
- the repeatability of test results (e.g., do multiple fire resistance tests conducted under the same conditions yield the same results?),
- effects of test scale, end restraint, and test repeatability on other types of structural components (beams, girders, columns, trusses, etc.), and
- the acceptance criteria to evaluate the load carrying capacity of the tested assemblies (currently tests are stopped before the load carrying capacity of the assembly is reached because other acceptance criteria are met or if the deflection becomes excessive and assembly failure could damage the furnace).

Structural Response of Components and Detailed Subsystems to Assumed Damage and Fire

Material Properties and Failure Criteria

The WTC towers were designed and constructed using 14 grades of steel and 2 types of concrete. Nominal properties for these materials were provided in the design documents. Additional information was required about the mechanical properties at room and elevated temperature for analysis of the towers' response to the impact and elevated temperature conditions.

The collapse analyses of the WTC towers concentrated on modeling failure mechanisms in steel rather than concrete components, since the WTC towers were essentially steel structures; concrete was used only for the floor slabs.

The two general types of steel that were used in the towers are typically described as carbon steels and high-strength steels. Carbon steels generally have lower strengths but are more ductile. The core columns, floor trusses, and beams and spandrel plates in the exterior wall were constructed with carbon steels, ranging from 36 to 50 ksi specified yield strengths. The exterior columns were designed with various grades of high strength steels, ranging from 55 ksi to 100 ksi yield strength.

Normal weight concrete (150 pcf) was used in the core and mechanical floors and lightweight concrete (110 pcf) was used in the floor system for the tenant spaces between the building core and exterior.

The mechanical properties of both steel and concrete are significantly affected by elevated temperatures. Steel and concrete properties that are temperature sensitive include modulus of elasticity, instantaneous coefficient of thermal expansion, tensile strength, and compressive strength. Additionally, creep strain rates for steel are also temperature dependent.

Mechanical properties of the various grades of steel used and normal and lightweight concrete, both at room temperature and throughout the expected temperature range, were determined. This information provided the bases for describing the material models used in the finite element analyses. In addition to material models, failure criteria were also developed for concrete and steel components. Failure criteria defined the necessary conditions to characterize and quantify the expected failure modes or mechanisms, including elastic or plastic buckling, yielding, or fracture. The state of component loads, material properties, and temperature also affected the mode of failure.

In addition, the following observations can be made:

- Modulus of elasticity is reduced by 25 percent at 600 °C for steel and by 50 percent to 75 percent for concrete.
- Steel yield strength reduces to 20 percent of its initial (room temperature) value and ultimate tensile strength is reduces to 40 percent of its initial value at 600 °C. Concrete compressive strength is reduced to between 30 percent and 50 percent of its initial value. Concrete tensile strength, which is already low, is also reduced to 30 percent.
- The instantaneous coefficient of thermal expansion for steel lies between that of lightweight and normal weight concrete for a given temperature. If steel truss and lightweight concrete

components are at the same temperature, the steel components will thermally expand more than the lightweight concrete. For steel beams and normal weight concrete in the core area, the normal weight concrete will expand more than the steel beams.

Floor Subsystem Analysis

The floors supported the occupants and furnishings and transferred these loads to the columns, acted as diaphragms to transfer loads between exterior faces when under wind loads, and provided lateral stability for columns. With damage to the fireproofing on the floor trusses, fires caused thermal expansion and sagging of the floors in the impact damage areas.

The analysis of floors progressed from individual components to major subsystems to global systems. Three truss components were studied with detailed models using ANSYS, a general purpose finite element software package, before developing a model of a full floor subsystem:

- Shear connector between the truss and concrete slab,
- Truss seat connection to the columns,
- Composite section of a single floor truss and concrete slab that included the truss seats, knuckles, and section of the supporting exterior and core channel beam.

Shear connector tests conducted by the truss manufacturer, Laclede Steel, in the early 1960s were reviewed and modeled. The shear connector between the truss and the concrete slab was referred to as a knuckle, due to the bent bar configuration that extended past the top chord of the truss, instead of the studs that are typically welded to the top chord. Detailed ANSYS models of the knuckle and concrete slab were analyzed and compared to the measured transverse and longitudinal shear capacities of a knuckle. A reduced model of the knuckle for use in the single truss and full floor models was developed that captured the dominant temperature-dependent behavior and failure modes.

Truss seats connected the trusses to the core and exterior columns. Truss seats were constructed with standoff plates, seat angles, bolts, and welded gusset plates; details varied for each truss seat depending upon its location within the floor plan. Truss seats were designed to carry floor gravity loads and small horizontal loads, typically a few percent of the column capacity to which it is attached. Typical truss seats were analyzed to determine their failure modes and associated loading and thermal conditions. A series of analyses were conducted to determine the truss seat response to thermal expansion of the floor slab, floor sagging or deformation, and heating of the truss seat. A model of reduced complexity was developed that captured the behavior and failure modes of the truss seats for use in the single truss and full floor models.

With reduced models of the knuckle and truss seat, a composite section of a full single truss and concrete slab was modeled to determine its behavior and failure modes for elevated temperatures and additional debris loads. Steel components with damaged fire resistant coatings heated and softened within 10 to 15 minutes. The bottom surface of the concrete slab heated quickly but the rate of heating through the slab depth was slower, so that the slab response to fire lagged the steel response. Concrete spalling was not included in the model. Analysis was conducted using uniform temperatures across the truss and an imposed linear thermal gradient across the slab depth to study the floor section response. These conditions were assumed prior to completion of the fire and heat transfer analyses used for the full floor subsystem analysis. Two failure modes of interest were (1) floor component failures leading to sagging (i.e. buckling of truss components or knuckle separation from the concrete slab) and the truss pulling inward on the columns and (2) failure of the truss seats. Analysis results were used to develop a model of reduced complexity with break elements that captured the behavior and failure modes of the floor section for use in the full floor model.

The full floor model included core columns and floor beams, exterior columns and spandrel beams, floor trusses and bridging trusses, and normal and lightweight concrete in the core and floor-truss areas, respectively. The columns were extended one floor level above and below the floor subsystem, and were required to include the interaction between the floor subsystem and the core and exterior columns. The full floor model contained a number of modifications from the model developed using the SAP2000 software of Floor 96 (NIST NSTAR 1-2) that reduced the number of finite elements and incorporated the features for analyzing the structural response to thermal conditions.

Results of the floor system analyses showed that:

- Knuckle failures did not occur under gravity loading and elevated temperatures anticipated.
- Truss web diagonals buckled at loads and temperatures expected and, as a consequence, the floor system sagged.
- Sagging of the floor system resulted in possible inward pull on the exterior columns although the magnitude of the force depended on fire conditions on surrounding floors.
- Truss seat connections could fail under elevated temperature conditions and their behavior was included to accurately capture the overall performance of the floor system to impact and fire conditions.
- Essential floor behavior, including buckling of web diagonals and connection failures, could be achieved with reduced models.

Core Column and Exterior Column and Panel Analysis

The primary function of the core columns was to carry the building gravity loads. The exterior columns resisted wind loads and, in addition, carried approximately half of the gravity loads.

Preliminary analysis of the core and exterior columns considered their individual buckling behavior and how it varied for uniform elevated temperatures. The columns were found to have sufficient capacity for tower gravity loads, even under elevated temperatures and a loss of lateral support at several floors. This was also found in more detailed finite element models of the columns.

The core columns were studied to determine the most efficient way to reduce the complexity of the model while still capturing buckling behavior at room and elevated temperatures.

Four exterior wall components were studied with detailed ANSYS models before developing a model of a nine-story by nine-column wall area:

- Bolted connection for exterior columns
- Bolted connections for spandrels
- Single exterior columns with spandrel sections
- Single exterior wall panel, fabricated as a single unit for construction purposes with 3 columns and 3 spandrels

The column and spandrel connections were analyzed to determine their failure modes and associated loading and thermal conditions. A reduced model was developed that captured the connector behavior and failure modes for use in exterior wall models.

The single column model with spandrel sections was loaded axially to determine its buckling load and post-buckling behavior at room and elevated temperatures for one, two, three, and nine story column heights.

The computer model of a single wall panel was validated against the reference structural models for the towers. The models were subjected to vertical and horizontal forces in the plane of the wall, representing intended design behavior, and a horizontal force transverse to the wall, representing a possible floor load.

The exterior wall had three connections: the column splice, the spandrel splice and the truss seat (for the floors). The column splice had four bolts that connected columns through their end plates. The spandrel connection had a splice plate to connect the two spandrel plates using high strength bolts. The spandrel and column splices were represented in the nine by nine wall subsystem model and captured the spandrel failure modes of bolt shear, tearing of the spandrel plate, and tearout of the spandrel plate at the bolt holes.

The nine by nine wall model had a coarser mesh that used beam elements for the columns, shell elements for the spandrels, and break elements for the connections. The wall model was subjected to axial loads from above, lateral out-of-plane loads at the floor levels, and elevated temperature representative of fire conditions. The effect of missing floor supports was also evaluated.

Several analyses were run for a variety of temperature load cases and for various combinations of axial load, disconnected floors simulating floor failure and loss of lateral column support, and inward pull applied at one or more floor levels modeling floor sag due to elevated temperatures. Results showed that:

- Although spandrel plates experienced large distortions and high strains, column buckling did not occur under the various temperature loadings applied when floors remained in place and able to provide lateral support to the columns.
- Column buckling did not occur when lateral support was lost at three floors under the expected gravity load that included dead plus service live loads.
- Column buckling did occur when lateral support was lost at three floors and the gravity load was increased to 150 percent of the expected gravity load simulating redistribution of load to the exterior wall.

- Column buckling was found to occur when an inward lateral load (pull-in) of approximately 12 kips was applied to three adjacent floor levels. The inward deflection of the exterior wall when it could no longer support the gravity load (i.e., at the buckling load) was approximately 10 in.

Aircraft Impact Damage

The aircraft impact of the WTC towers caused extensive damage to the buildings' exterior, penetrated into the interior causing further damage to the structural system, dislodged fireproofing, and ignited multi-floor fires. The structural damage to each tower resulting from the aircraft impact was estimated using a transient finite element analysis. Results of this analysis were used to predict damage to the structure, fireproofing, and partition walls in the path of the debris field.

The fire dynamics, thermal, and structural analyses all required input data derived from the aircraft impact analyses. The fire dynamics analyses used estimates of damage to the floors and partition walls to describe ventilation paths, and to identify the distribution of fuel and debris immediately following impact. The thermal analysis required estimation of the areas that had dislodged fireproofing on the structural components of the towers. For the structural analyses, elements that represented severed or heavily damaged floors and columns were removed from the structural models of the towers.

The aircraft impact analyses considered three cases for each tower, where each case had a different set of input parameter values, based upon sensitivity studies and detailed component analyses. The results for the three cases were compared to observations from photographs and videos. Damage to the exterior walls predicted by the impact simulations matched reasonably well the exterior damage in photographic and video records. The observed exterior damage was used in the structural analyses. The analysis results from two cases for each tower were found to match observations reasonably well, and were selected for continued analysis by the fire dynamics, thermal, and structural analyses. The cases for each tower were referred to as Case A and Case B for WTC 1 and Case C and Case D for WTC 2. However, prior to determining the final aircraft impact analysis results, earlier aircraft impact analyses produced an initial set of aircraft impact cases for each tower. These initial cases, referred to as Case A_i and Case B_i for WTC 1 and Case C_i and Case D_i for WTC 2, were used to develop experience and gain understanding of the fire spread and growth, the rate of structural component heating, and the structural response to damage and elevated temperatures.

The final set of impact damage data for fire dynamics, thermal, and structural analyses was Cases A, B, C, and D, with the exception of the full floor subsystem analyses which used initial damage Cases A_i to D_i. The use of the aircraft impact data in the sequence of structural analyses was as follows:

1. Full floor subsystem models were analyzed for all initial damage Cases A_i to D_i before the final damage cases were available.

2. Full floor subsystem models were evaluated for changes in damage between final Cases A to D and initial Cases A_i to D_i. Changes in impact damage to the structural components and fireproofing reflected in the two sets of Cases were found to have little effect on the floor subsystem structural response. The full floor subsystem structural response for Cases A_i to D_i and Cases A to D were found to be equivalent.

3. Isolated core and exterior wall subsystem models were analyzed for Cases A, B, C, and D.

4. The global model of each tower was analyzed for Cases B and D, based upon the results of the subsystem analyses.

Four classifications of core column structural damage were established: severed, heavy damage, moderate damage, and light damage. Classification criteria included plastic strain levels and lateral deformation from the column centerline. Columns that were severed or heavily damaged were removed to simulate impact damage in the global analysis of each tower. Two types of floor structural damage were identified from the impact analysis results: (1) missing floor areas and (2) severely damaged floor areas incapable of supporting loads.

Fireproofing was assumed to be dislodged from core columns only if the columns were subject to direct debris impact that failed wall partitions in the immediate vicinity of the column[1]. For exterior columns, the debris impact was required to be strong enough to damage or destroy room furnishings (modular office workstations) adjacent to the columns. For floor trusses, the debris impact was required to be strong enough to damage or destroy room furnishings (modular office workstations) in the same area of the affected floor.

The structural damage in WTC 1 extended from the north exterior wall into the north side of the core. An exterior panel was knocked out of the south wall by aircraft debris. Damage to the fireproofing from direct debris impact extended over a larger region, and extended to central regions of the south floor areas. Case B predicted more damage to core columns and a larger extent of fireproofing damage to the south floor area than Case A, including damage to the south exterior wall fireproofing on the inside face, as shown in Fig. E–3.

The structural damage in WTC 2 extended from the south exterior wall to southeast region of the core. Exterior columns were severed by debris near the northeast corner. Damage to the fireproofing from direct debris impact extended over a larger region, and extended to most of the east floor area to the north face. Case D predicted more damage to core columns than Case C, but the extent of the fireproofing damage was similar, as shown in Fig. E–4.

[1] The Pentagon was impacted by an aircraft of similar size and at a similar speed as the WTC towers. The observed stripping of the concrete cover from columns in similar circumstances provides an independent set of data that supports the criteria established for the removal of fireproofing materials subject to direct debris impact in the WTC towers.

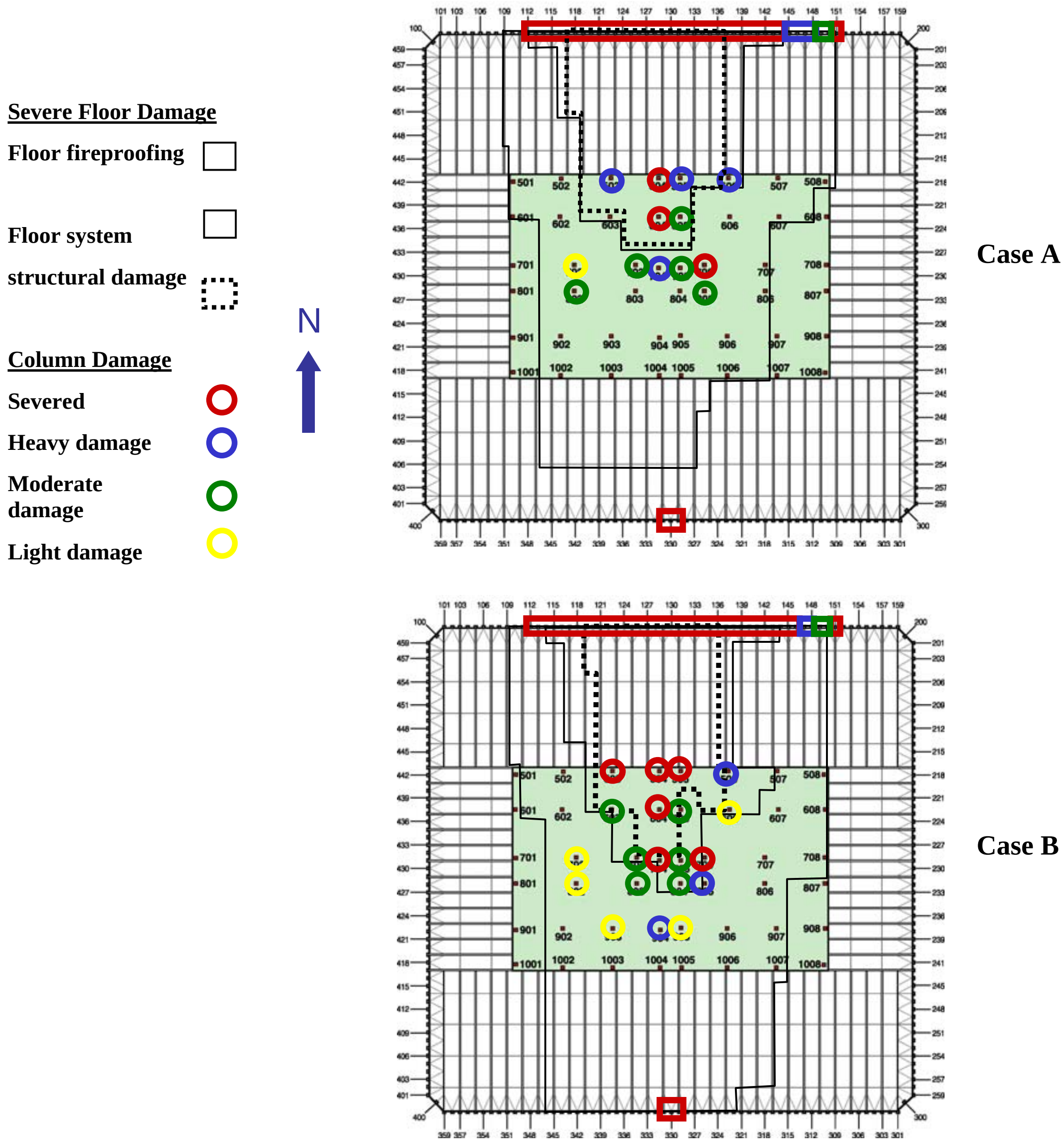


Figure E–3. Plan view of WTC 1 cumulative damage for Floors 93 to 99.

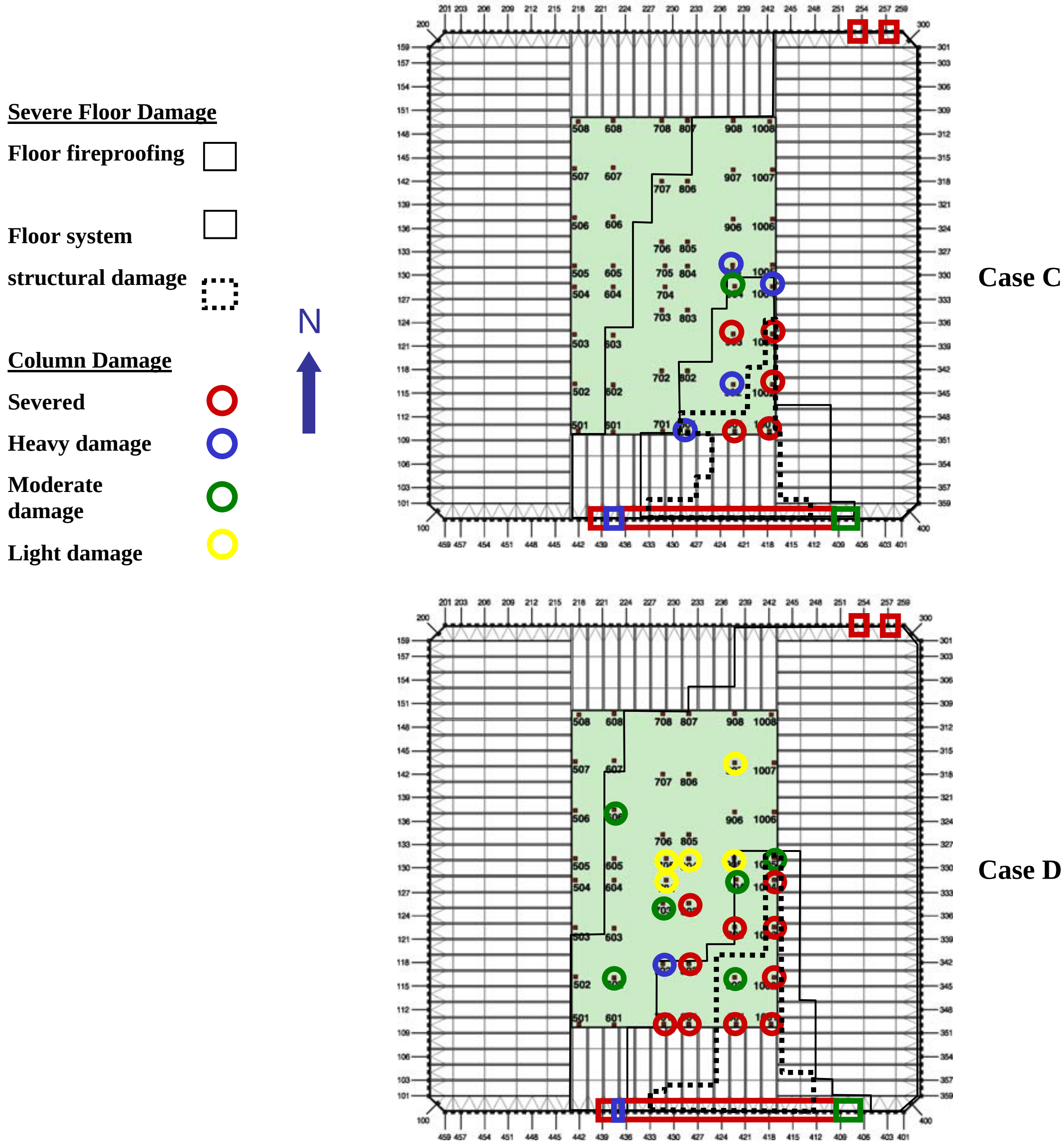


Figure E–4. Plan view of WTC 2 cumulative damage for Floors 78 to 84.

Observations and Timeline of Structural Events

NIST assembled a collection of nearly 150 hours of video footage and over 7000 photographs, which were reviewed for insights into the structural performance of the towers. A timeline of significant events that characterized the weakening and eventual collapse of the WTC towers was developed with the photographs and videos that were time-stamped. Quantitative information, such as the amount of inward bowing observed on the exterior walls of the buildings, was extracted from key photographs through image enhancement and scaled measurements. Key observations and the timelines were used to guide the global collapse analyses.

Development of the probable collapse sequence for each tower was shaped by evidence gathered in the investigation. Data about the events following the aircraft impact were primarily obtained from three sources:

- Photographic and videographic records that had been catalogued and time stamped for the NIST Investigation (NIST NCSTAR 1-5A)
- Interviews of individuals in the towers who survived and those who received telephone calls from individuals trapped in the tower (NIST NCSTAR 1-7)
- Interviews of emergency response personnel and emergency communication records (NIST NCSTAR 1-8)

Photographs and videos provided knowledge about aircraft impact damage to the exterior walls, fire growth and spread at the building exterior, inward bowing of an exterior wall in each tower, and the direction of tilt for the building section above the impact and fire zone as the towers collapsed.

Changes in structural performance are generally difficult, if not impossible, to perceive until significant deformation has taken place relative to the dimensions of the structure, and depend on the detail and resolution of the image being examined and the vantage point of the photographer. Observations of structural performance for the WTC towers included severed components, local deflections or buckling, possible sagging of floors, and relative alignment of columns or building sections.

Evidence was used in the analyses in three ways: (1) to determine input parameters, such as the aircraft speed and direction upon impact, (2) to impose time-related constraints upon an analysis, such as imposing observed broken windows over time to constrain the spread of fire, or (3) to validate analysis results, such as global stability after impact and during thermal loading.

Observations of structural behavior were broken into two groups: *key observations* and *noted observations*. Key observations were significant structural events that were explicitly addressed in or used to validate the structural analyses. Noted observations were events that may have been a structural response, but could not be conclusively identified as to their significance to the structural response.

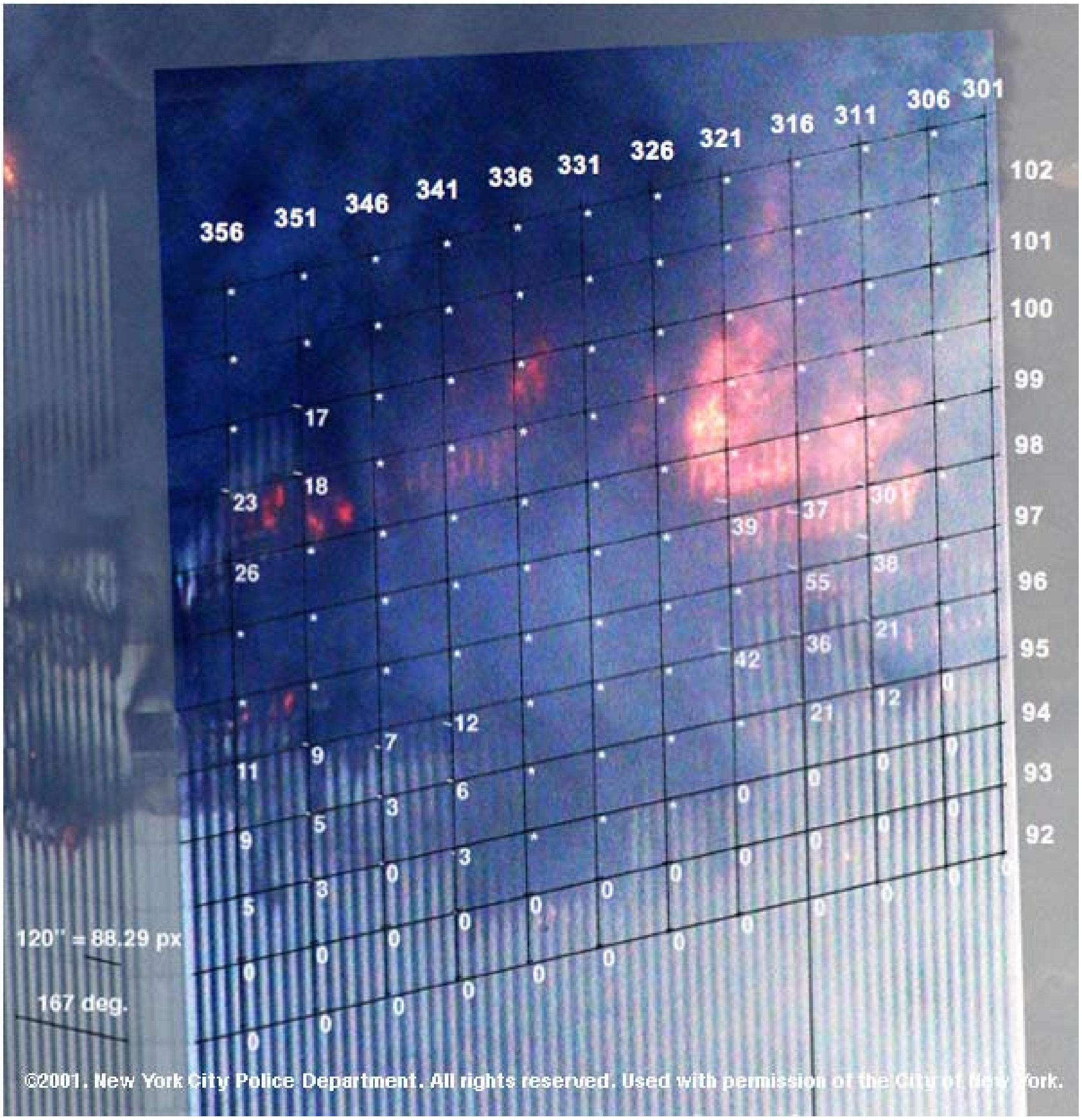
Key observations were used to develop a timeline of structural events for each tower. Structural analyses were used to support development of the collapse hypotheses for each tower and to develop and refine understanding of the probable sequence of events.

WTC 1 key observations were:

- Inward bowing of the south exterior wall was first observed at 10:23 a.m., as shown in Fig. E–5.
- The time to collapse initiation was 102 minutes from the aircraft impact (9:46:30 a.m. until 10:28:22 a.m.).
- From exterior observations, tilting of the building section appeared to take place near Floor 98. Column buckling was then observed to progress rapidly across the east and west faces.
- The WTC 1 building section above the impact and fire area tilted to the south as the structural collapse initiated, as shown in Fig. E–6. A tilt to the south of at least 8 degrees occurred before dust clouds obscured the view and the building section began to fall downwards.

WTC 2 key observations were:

- Following the aircraft impact and fireballs, hanging objects were observed through the windows of the east and north faces. The hanging objects suggest that there was structural damage to WTC 2 Floor 83 along the east face and to Floors 81 to 83 of the north face near the northeast corner.
- Inward bowing of the east wall was first observed at 9:21 a.m. The inward bowing was approximately 10 in. at Floor 80.
- An increase of the inward bowing of the east wall was observed at 9:53 a.m. The greatest bowing was approximately 20 in.±1.0 in. at Floor 80 on the east face of WTC 1.
- Collapse initiated 56 minutes after the aircraft impact (9:02:59 a.m. to 9:58:59 a.m.).
- From a northeast viewpoint, initial downward motion was observed as columns moved inward on the north side of the east face, as shown in Fig. E–7. Tilt of the building section above the impact and fire area appeared to take place near Floor 82. Column buckling was then seen to progress across the north face.
- The building section above the impact and fire area tilted to the east and south as the structural collapse initiated as shown in Fig. E–8. There was approximately a 3 to 4 degree tilt to the south and a 7 to 8 degree tilt to the east prior to significant downward movement of the upper building section.



1. Measurements were based on calibration measures shown on the west face
2. Foreshortening into depth of field across the south face of 17% was included in the measurements
3. Measurement error was at least ± 6 inches

Figure E–5. WTC 1 exterior columns bowing inward across most of the south face between floors 95 to 98 at 10:23 a.m.

Figure E–6. WTC 1 building section above impact damage zone tilts to the south.

Figure E–7. View of WTC 2 buckling of east wall near northeast corner as collapse initiates from southeast.

Figure E–8. View of upper building section of WTC 2 tilting to the east.

Structural Response of Major Tower Subsystems

Prior to conducting the analysis of the global structural response of each tower, major structural subsystems were analyzed to provide insight into their behavior within the WTC global system. The three major structural subsystems, the core framing, a single exterior wall, and full tenant floors, were analyzed separately for their response to impact damage and fire. The hat truss was not analyzed separately as its structural behavior did not require significant reduction in the global analysis. The component analyses provided a foundation for these large, nonlinear analyses with highly redundant load paths by determining component behavior and failure modes and enabling a significant reduction in finite element model complexity and size. The major subsystem models used final estimates of impact damage and elevated temperatures determined from the aircraft impact analysis and the fire dynamics and thermal analyses.

The capacity of each subsystem to sustain loads for the imposed damage and elevated temperatures was evaluated. The isolated subsystem models lacked the restraint and load paths to other subsystems found in the global analysis. Even so, the isolated subsystem response was useful for refining the global models and interpreting subsystem behavior in the global system. For instance, when the column connections to the hat truss in WTC 2 failed at the southeast corner of the core, the only load path available to carry those column loads was the floor system within the core structure. However, in the global structure, the hat truss at the top of the core would transfer loads to other core columns or the exterior walls, assuming the connections between the core columns and hat truss remained intact.

The subsystem models used modeling reductions from the component analyses, which kept the analysis tractable while maintaining required nonlinear features. As previously noted, such reductions were necessary to maintain a careful balance between model size and complexity as the model size increased. Each of the major subsystem models used temperature histories for the towers. Elevated temperatures were applied to the models in 10 min intervals, where a temperature state was given for all structural components at a given time and linearly ramped to the next temperature state. Examination of structural temperature histories indicated that no significant fluctuations between temperature states occurred for the 10 min intervals selected for analysis.

Core Subsystem

The core subsystem models included temperature-dependent plasticity, creep and plastic buckling behavior in the core column elements. Core models extended from Floor 89 to Floor 106 for WTC 1 and from Floor 73 to Floor 106 for WTC 2, and did not include the hat truss. The models included core columns and floor beams and slabs. Floor slabs were modeled as membrane elements with a relatively coarse mesh which resulted in approximate slab openings for elevators and mechanical shafts. The meshing did not affect the floor's ability to provide a load path between columns. For the purposes of the isolated core model, only the floor beams with partial moment connections were included, as simple shear connections were not capable of transferring significant loads between columns. Impact damage was modeled by removing severed core columns and damaged floor areas. The core subsystem was analyzed for stability under gravity loads. Temperature histories were then applied to the core structure.

By not including the hat truss, the primary load path for core column load redistribution was removed, leaving the core floors which typically provide a secondary load path. The WTC 1 isolated core subsystem was stable with Case A aircraft impact damage and gravity loads. To reach a stable solution for Case C structural damage and gravity loads, the WTC 2 isolated core model required horizontal restraints to be added in the east and south directions at each floor representing the lateral restraint provided by the office area floors. Without the horizontal restraints, the WTC 2 core model tilted significantly due to the severed columns in the southeast corner of the core. The isolated core models did not converge for WTC 1 Case B and WTC 2 Case D structural impact damage, which had more severed columns than Cases A and C. The core needed to redistribute loads to other areas in the global system for a stable solution with Cases B and D structural damage.

Full Floor Subsystem

The full floor subsystem models included large deflection and temperature-dependent material properties with plasticity for all steel components. Creep was not included in the full floor models, as this analysis feature did not work with beam elements in version 8.0 of ANSYS (the detailed truss model had 3D finite strain elements that were changed to beam elements in the full floor model). Creep was included for beam elements in ANSYS 8.1 and subsequent analyses of the core and exterior wall subsystems included creep deformation. The floor slab was modeled as lightweight concrete across the entire floor (tenant and core floor areas) with a bilinear stress-strain constitutive model that did not account for cracking, crushing, or spalling. The concrete material model used the compressive strength as the yield point, with the same yield strength in both tension and compression (the reinforcing steel was assumed to provide the tensile capacity in the composite floor). Separate floor models were created from the Floor 96 structural model by imposing the different damage and temperature conditions for WTC 1 Floors 93 to 99 and WTC 2 Floors 79 to 83. Structural components that were severed due to the aircraft impact were removed from each floor model, based upon the four initial damage cases, WTC 1 Case A_i and B_i and WTC 2 Case C_i and D_i. Each full floor model was analyzed for stability under floor gravity loads. No column loads were applied. Temperature histories were then applied to the floor structure.

The floor analysis results for Cases A_i to D_i were used for Cases A to D in the exterior wall subsystem and global analyses. Final damage Cases A, B, C, and D were completed after the initial set of floor analyses were conducted with Cases A_i, B_i, C_i, and D_i. The full floor models were not rerun for Cases A through D as comparisons showed that the structural temperature histories of the floors were nearly identical for most floors and only slightly different for a few floors.

Exterior Wall Subsystem

The exterior wall subsystem models included temperature-dependent plasticity, creep strains and plastic buckling behavior in the exterior wall components. The exterior wall analysis extended over approximately 20 floors and were centered around the areas of impact and fire zone. The south face of WTC 1 extended from floor 89 to floor 106 and the east face of WTC 2 extended from floor 73 to floor 90. The exterior panel that was severed during the aircraft impact and found south of the tower was removed from the south face of WTC 1. No structural damage to the panels was observed on the east wall of WTC 2. The analysis of a single exterior face provided insight into the conditions that would result in the inward bowing of the south wall of WTC 1 and the east wall of WTC 2 observed in photographs. Conditions examined included pull-in forces resulting from sagging floors, disconnected floors resulting from truss seat failure, additional vertical loads simulating load transfer to the exterior wall, and elevated temperatures.

The exterior wall models were used to estimate the pull-in force magnitude and locations for each tower that would produce the observed bowing of the exterior wall. The inward pull was caused by sagging of the floors. Heating of the inside face of the exterior columns also contributed to inward bowing. Thermal expansion occurred as soon as steel temperatures began to rise; column shortening occurred when creep and plastic strains overcame thermal expansion strains, typically at temperatures greater than 500 °C to 600 °C with accompanying high stresses and duration of temperatures and stress levels.

WTC 1 exterior wall analysis found that an inward pull force of 6 kips at each column at floors 95 to 99, starting 80 min after the aircraft impact, caused a maximum inward bowing of 31 in, shown in Fig. E–9. This inward deflection was smaller than the observed maximum bowing of 55 in. ±6 in., and the wall was stable at 100 min. The magnitude of pull-in forces was expected to be less than 6 kip with the addition of gravity loads from the core subsystem as it also weakened; therefore, pull-in forces of 4 to 5 kips were used in the global model analyses.

WTC 2 exterior wall analysis found that an inward pull force of 1.0 to 1.5 kip and 4.0 to 5.0 kip on the south and north portions of the east wall, respectively, over floors 79 to 83, caused a maximum inward bowing of 9.5 in. at 20 min and 37 in. at 50 min, as shown in Fig. E–10. The observed deflections were 10 in. and 20 in., respectively, at corresponding times. Considering the possible increase in column loads after impact for Case D conditions, a pull-in force of 1.0 kip on the south half and 4.0 kip on the north half of the east wall was selected for the initial estimate for the WTC 2 global model analysis.

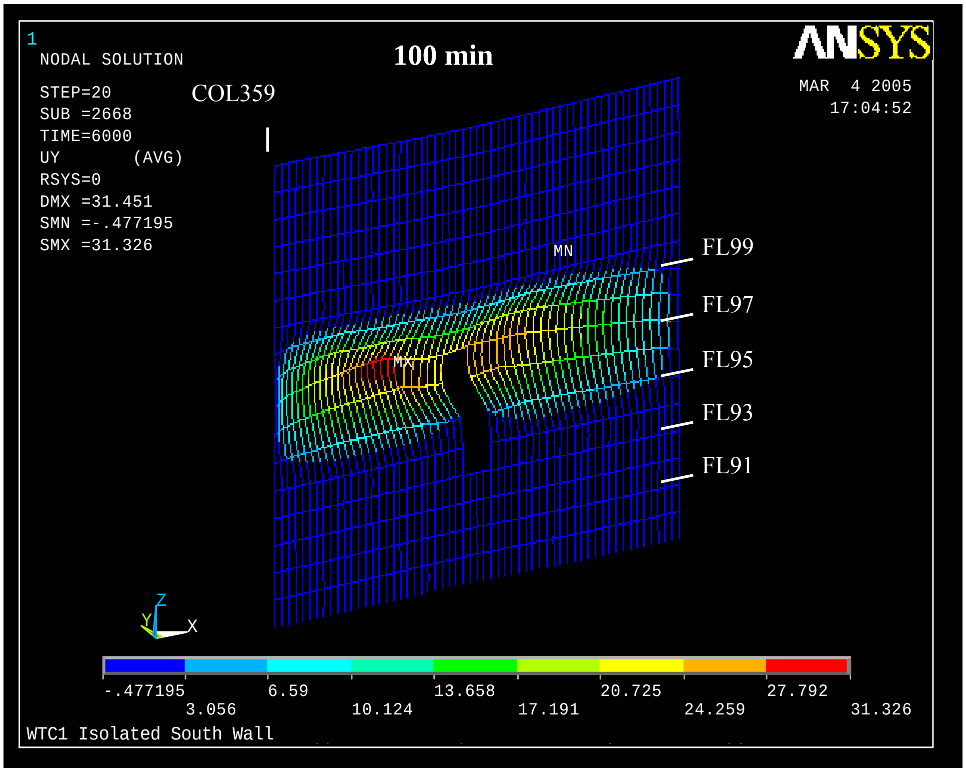
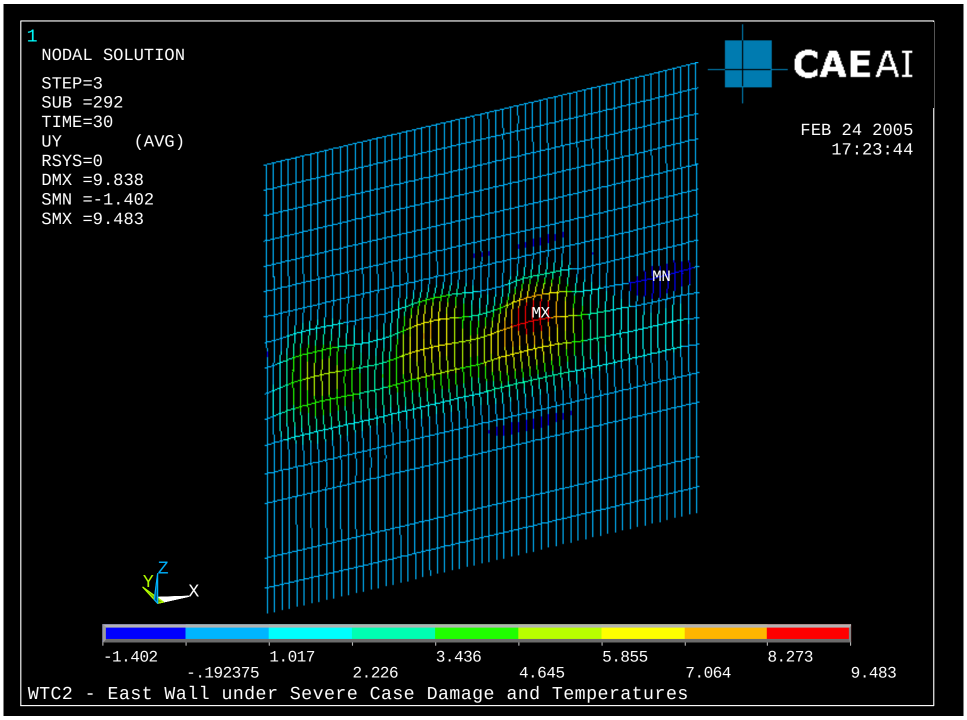
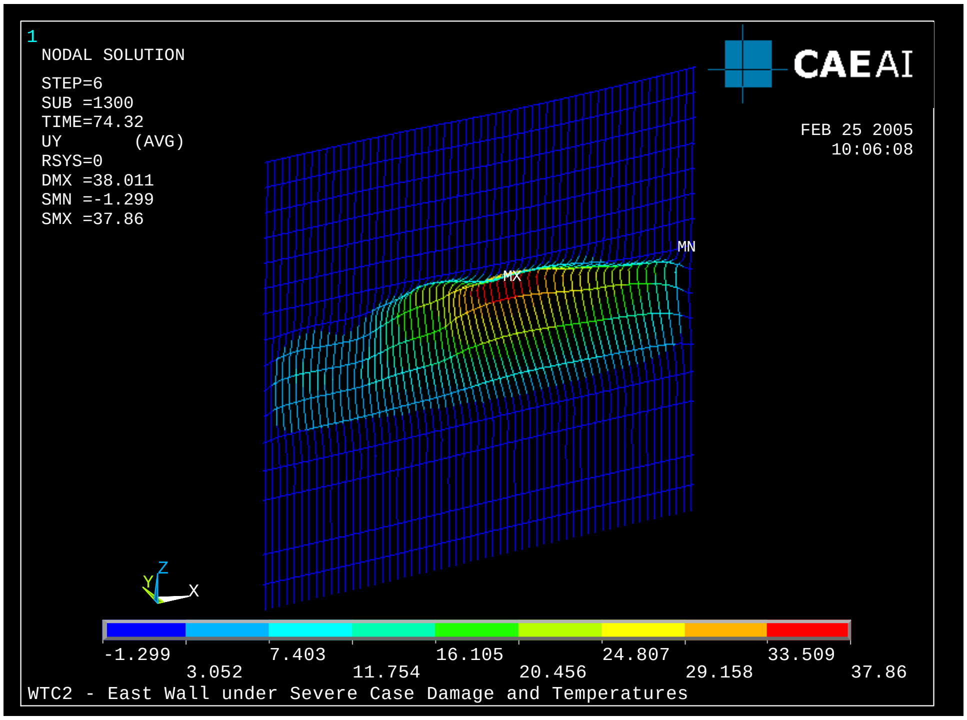


Figure E–9. Inward displacement of the WTC 1 south wall at 100 min of the Case B temperatures with floor disconnections and 6 kip pull-in forces over five floors.



At 20 min



At 50 min

Figure E–10. Out-of-plane displacements of east wall of WTC 2 calculated with pull-in forces of 1.5 kip on the south half and 5.0 kip on the north half.

Structural Response of the WTC Towers

A separate global analysis of each tower helped determine the relative roles of impact damage and fires with respect to structural stability and sequential failures of components and subsystems and was used to determine the probable collapse initiation sequence.

Results of the major subsystem analyses were incorporated into the global models, reducing the complexity of the modeling approach and/or level of detail where possible, while retaining sufficient detail for nonlinear structural responses. The global models of the towers extended from several stories below the impact area to the top of the structure. WTC 1 was truncated at floor 91 and WTC 2 was truncated from floor 77. The global models included the core subsystem, the exterior wall subsystem, the hat truss, and an equivalent plate representation of the floor system. The core columns and exterior columns and spandrels were modeled with elements and features similar to those used in the isolated core and exterior wall analyses. Column analysis features included the effects of thermal expansion, plastic, and creep strains on column behavior within the global structural system. The full floor model was not included in the global models, as it would have made the models computationally too large. Instead, office area and core floors were modeled with an equivalent floor slab thickness and modulus calculated to match the in-plane stiffness of the composite floor system, including the concrete slab, floor trusses, and the floor seats. Floor loads applied as concentrated loads at the column connections. These modeling simplifications of the floor system were able to capture the floor behaviors observed in the full floor subsystem analyses while keeping the analysis tractable.

Each global model was first evaluated for stability under gravity loads with structural impact damage modeled by removing severed and heavily damaged columns and floor areas. Temperature histories were applied in 10 min intervals and linearly ramped to the next temperature state. Pull-in forces from sagging floors were also applied during the appropriate 10 min intervals. The global analysis results provided a sequence of component and subsystem failures that led to the onset of global instability and collapse initiation.

WTC 1 Global Analysis Results

After the aircraft impact, gravity loads that were previously carried by severed columns were redistributed to other columns. The north wall lost about 7 percent of its loads after impact. Most of the load was transferred by the hat truss, and the rest was redistributed to the adjacent exterior walls by spandrels. Due to the impact damage and the tilting of the building to the north after impact, the south wall also lost gravity loads, and about 7 percent was transferred by the hat truss. As a result, the east and west walls and the core gained the redistributed loads through the hat truss.

In the early stages of the fire, structural temperatures in the core rose and the thermal expansion of the core was greater than the thermal expansion of the exterior walls. The difference in the thermal expansion increased the loads in the core columns at about 20 min. Thereafter, the core lost gravity loads due to its thermal weakening and shortening until the south wall started to bow inward. At about 100 min, approximately 20 percent of the core loads were transferred by the hat truss to the exterior walls due to thermal weakening of the core; the north and south walls each gained about 10 percent more loads and the east and west walls each gained about 25 percent more loads. Since the hat truss outriggers to the east and west walls were stiffer then the outriggers to the north and south walls, they transferred more loads to the east and west exterior walls.

The inward bowing of the south wall caused failure of exterior column splices and spandrels, and induced column instability. The instability progressed horizontally across the entire south face. The south wall unloaded and redistributed its gravity loads to the thermally weakened core through the hat truss and to the east and west walls through the spandrels. The building section above the impact zone began tilting to the south as column instability progressed rapidly from the south wall along the adjacent east and west walls, and increased the gravity load on the core columns. The change in potential energy due to downward movement of building mass above the buckled columns exceeded the strain energy that could have been absorbed by the structure. Global collapse then ensued.

WTC 2 Global Analysis Results

Before aircraft impact, the load distribution across the exterior walls and core was symmetric with respect to the centerline of each exterior wall. After aircraft impact, the exterior column loads on the south side of the east and west walls and on the east side of south wall increased. This was due to the leaning of the building towards the southeast. After aircraft impact, the core carried 6 percent less loads. The north wall loads reduced by 6 percent and the east face loads increased by 24 percent. The south and west walls carried 2 percent to 3 percent more load.

In contrast to the fires in WTC 1, which generally progressed from the north side to the south side over approximately an hour, the fires in WTC 2 were located on the east side of the core and floors the entire time, with the fires spreading from south to north. With fireproofing dislodged over much of the same area, the structural temperatures became elevated in the core, floors, and exterior walls at similar times. During early stages of the fires, columns with dislodged fireproofing elongated due to thermal expansion. As the structural temperatures continued to rise, the thermal expansion was overcome by plastic and creep deformations under compressive loads.

Vertical displacements of the south and east exterior columns were essentially constant after impact and remained around 7.5 in. (over the severed columns) on the south face and about 3.5 in. on the east face until the east wall became unstable at 43 min. The east wall, which had bowed inward to a total of approximately 62 in., suddenly unloaded. The west wall also unloaded. Loads increased on the core and on the north and south walls. The core had weakened on the east side and shortened by 3.0 in. at the southeast corner. At the same time, the northwest corner of the exterior wall displaced upwards about 2.0 in., as the tower was tilting to the southeast around an axis passing through the southwest and northeast corners.

The inward bowing of the east wall caused failure of exterior column splices and spandrels, and induced column instability. The instability progressed horizontally across the entire east face. The east wall unloaded and redistributed its gravity loads to the thermally weakened core through the hat truss and to the east and west walls through the spandrels. The building section above the impact zone began tilting to the east as column instability progressed rapidly from the east wall along the adjacent north and south walls, and increased the gravity load on the weakened east core columns. The change in potential energy due to downward movement of building mass above the buckled columns exceeded the strain energy that could have been absorbed by the structure. Global collapse then ensued.

Structural Response of the WTC Towers to Fire Without Impact Damage

Whether the towers would have collapsed if subjected to the same fires with no aircraft impact damage was considered as part of understanding the relative roles of the impact damage and fires. It was found that both WTC 1 and WTC 2 were stable after the aircraft impact and that they had considerable reserve capacity from the global analyses with structural impact damage. The global analyses also found that the combined effect of structural and fireproofing impact damage with the ensuing fires caused both towers to collapse. The effect of the fires on the towers without structural or fireproofing damage was considered by examining the subsystem and global analysis results for portions of the structures with intact fireproofing that were subject to the fires.

The towers would likely not have collapsed under the combined effects of aircraft impact and the subsequent multi-floor fires, if the fireproofing had not been dislodged or had been only minimally dislodged by aircraft impact. The existing condition of the fireproofing prior to aircraft impact and the fireproofing thickness on the WTC floor system did not play a significant role in initiating collapse of the towers.

Probable Collapse Sequences

To determine the probable collapse sequence for each tower, NIST adopted an approach that combined mathematical modeling, statistical and probability based analysis methods, laboratory experiments, and analysis of photographs and videos. The approach accounted for variations in models, input parameters, analyses, and observed events. It included the evaluation and comparison of possible collapse hypotheses based on different damage states, fire paths, and structural responses to determine the following:

- The probable sequence of events from the moment of aircraft impact until the initiation of global building collapse;
- How and why WTC 1 stood nearly twice as long as WTC 2 before collapsing (102 min for WTC 1 versus 56 min for WTC 2), though they were hit by virtually identical aircraft (Boeing 767-200ER);
- What factors, if any, could have delayed or prevented the collapse of the WTC towers.

Collapse hypotheses were developed over the course of the NIST Investigation. The first hypotheses were published in the May 2003 NIST Progress Report, and were updated in the June 2004 NIST Progress Report and October 2004 Public Meeting at NIST. The Probable Collapse Sequence for each tower was presented at the April 2005 Public Meeting in New York City. The stages of hypothesis development are summarized as follows:

- **Possible Collapse Hypotheses** (May 2003) – not building specific; key events not identified
- **Working Collapse Hypothesis** (June 2004) – single hypothesis for both WTC towers; identified chronological sequence of major events

- **Leading Collapse Hypotheses** (October 2004) – separate hypothesis for each WTC tower; identified building-specific load redistribution paths and damage scenarios in addition to chronological sequence of major events

- **Probable Collapse Sequences** (April 2005) – refined building specific collapse sequences with chronological sequence of major events, load redistribution paths, and damage scenarios.

To determine the probable collapse sequence for each tower, the following steps were required:

- identification of key observables, primarily from photographs and videos

- development of collapse hypotheses, which were updated periodically through the course of the investigation with the acquisition of new data and analysis results

- sensitivity studies to identify influential parameters, through the application of a formal statistical approach, orthogonal factorial design (OFD)

- development and refinement of mathematical modeling –fire dynamics simulation with computational fluid dynamics and structural response to aircraft impact and fire with finite element analyses

- evaluation of analysis results against observed and expected structural behavior, with adoption of the event tree concept, and pruning and updating branches based upon comparisons with observed data

These steps were applied to the degree needed for the sequence of analyses, from aircraft impact to structural response.

E.3 Probable Collapse Sequence of WTC 1 and WTC 2

The specific factors in the collapse sequences relevant to both towers (the sequences vary in detail for WTC 1 and WTC 2) are:

- Each aircraft severed exterior columns, damaged interior core columns and knocked off fireproofing from steel as the planes penetrated the buildings. The weight carried by the severed columns was distributed to other columns.

- Subsequently, fires began that were initiated by the aircraft's jet fuel but were fed for the most part by the building contents and the air supply resulting from breached walls and fire-induced window breakage.

- These fires, in combination with the dislodged fireproofing, were responsible for a chain of events in which the building core weakened and began losing its ability to carry loads.
- The floors weakened and sagged from the fires, pulling inward on the exterior columns.
- Floor sagging and exposure to high temperatures caused the exterior columns to bow inward and buckle—a process that spread across the faces of the buildings.
- Collapse then ensued.

The sequences are supported by extensive computer modeling and the evidence held by NIST. The probable collapse sequence for WTC 1 and WTC 2 are presented in Figs. E–11 and E–12.

1. Aircraft Impact Damage

- Aircraft impact severed a number of exterior columns on the north wall from floors 93 to 98, and the wall section above the impact zone moved downward.
- After breaching the building's exterior, the aircraft continued to penetrate into the building, severing floor framing and core columns at the north side of the core. Core columns were also damaged toward the center of the core and, to a limited extent on the south side of the core. Fireproofing was damaged from the impact area to the south exterior wall, primarily through the center of WTC 1 and at least over a third to a half of the core width.
- Aircraft impact severed a single exterior panel at the center of the south wall between floors 94 and 96.
- The impact damage to the exterior walls and to the core resulted in redistribution of severed column loads, mostly to the columns adjacent to the impact zones. The hat truss resisted the downward movement of the north wall, and rotated about the east-west axis.
- As a result of the aircraft impact damage, the north and south walls each carried about 7 percent less gravity loads after impact, and the east and west walls each carried about 7 percent more loads. The core carried about 1 percent more gravity loads after impact.

2. Effects of Subsequent Fires and Impact Damaged Fireproofing

A. Thermal Weakening of the Core:

- The undamaged core columns developed high plastic and creep strains over the duration the building stood, since both temperatures and stresses were high in the core area. The plastic and creep strains exceeded thermal expansion in the core columns.
- The shortening of the core columns (due to plasticity and creep) was resisted by the hat truss which unloaded the core over time and redistributed loads to exterior walls.
- As a result of the thermal weakening (and subsequent to impact and prior to inward bowing of the south wall), the north and south walls each carried about 10 percent more gravity loads, and the east and west walls each carried about 25 percent more loads. The core carried about 20 percent less gravity loads after thermal weakening.

B. Thermal Weakening of the Floors:

- Floors 95 to 99 weakened with increasing temperatures over time on the long-span floors and sagged. The floors sagged first and then contracted due to cooling on the north side; fires reached the south side later, the floors sagged, and the seat connections weakened.
- Floor sagging induced inward pull forces on the south wall columns.
- About 20 percent of the connections to the south exterior wall on floors 97 and 98 failed due to thermal weakening of the vertical supports.

C. Thermal Weakening of the South Wall:

- South wall columns bowed inward as they were subjected to high temperatures and inward pull forces in addition to axial loads.
- Inward bowing of the south wall columns increased with time.

Figure E–11. WTC 1 probable collapse sequence.

3. Collapse Initiation

- The inward bowing of the south wall induced column instability, which progressed rapidly horizontally across the entire south face.
- The south wall unloaded and tried to redistribute the loads via the hat truss to the thermally weakened core and via the spandrels to the adjacent east and west walls.
- The entire section of the building above the impact zone began tilting as a rigid block (all four faces; not only the bowed and buckled south face) to the south (at least about 8º) as column instability progressed rapidly from the south wall along the adjacent east and west walls.
- The change in potential energy due to downward movement of building mass above the buckled columns exceeded the strain energy that could be absorbed by the structure. Global collapse then ensued.

Figure E–11. WTC 1 probable collapse sequence (cont).

1. Aircraft Impact Damage

- Aircraft impact severed a number of exterior columns on the south wall from floors 78 to 84, and the wall section above the impact zone moved downward.
- After breaching the building's exterior, the aircraft continued to penetrate into the building, severing floor framing and core columns at the southeast corner of the core. Fireproofing was damaged from the impact area through the east half of the core up to the north and east exterior walls. The floor truss seat connections over about 1/4 to 1/2 of the east side of the core were severed on floors 80 and 81 and over about 1/3 of the east exterior wall on floor 83.
- Aircraft impact severed a few columns near the east corner of the north wall between floors 80 and 82.
- The impact damage to the exterior walls resulted in redistribution of severed column loads, mostly to the columns adjacent to the impact zones. The impact damage to the core columns resulted in redistribution of severed column loads mostly to other intact core columns and the east exterior wall. The hat truss resisted the downward movement of the south wall, and rotated about the east-west axis.
- As a result of the aircraft impact damage, the core carried 6 percent less gravity loads after impact and the north face carried 10 percent less loads. The east face carried 24 percent more gravity load, while the west face and the south face carried 3 percent and 2 percent more gravity load, respectively.
- After impact, the core was leaning toward the east and south exterior walls. The exterior walls acted to restrain the core structure.

Figure E–12. WTC 2 probable collapse sequence.

2. Effects of Subsequent Fires and Impact Damaged Fireproofing

A. Thermal Weakening of the Core:

- Several of the undamaged core columns near the damaged and severed core columns developed high plastic and creep strains over the duration the building stood, since both temperatures and stresses were high in the core area. The plastic and creep strains exceeded thermal expansion in the core columns.
- The core continued to tilt toward the east and south due to the combination of column shortening (due to plasticity, creep, and buckling) and the failure of column splices at the hat truss in the southeast corner.
- As a result of thermal weakening (and subsequent to impact), the east wall carried about 5 percent more gravity loads and the core carried about 2 percent less loads. The other three walls carried between 0 and 3 percent less loads.

B. Thermal Weakening of the Floors:

- Floors 79 to 83 weakened with increasing temperatures over time on the long-span floors on the east side and sagged.
- Floor sagging induced inward pull forces on the east wall columns.
- About an additional 1/3 of the connections to the east exterior wall on floor 83 failed due to thermal weakening of the vertical supports.

C. Thermal weakening of the east wall:

- East wall columns bowed inward as they were subjected to high temperatures and inward pull forces in addition to axial loads.
- Inward bowing of the east wall columns increased with time.

3. Collapse Initiation

- The inward bowing of the east wall induced column instability, which progressed rapidly horizontally across the entire east face.
- The east wall unloaded and tried to redistribute the loads via the hat truss to the weakened core and via the spandrels to the adjacent north and south walls.
- The entire section of the building above the impact zone began tilting as a rigid block (all four faces; not only the bowed and buckled east face) to the east (about 7º to 8º) and south (about 3º to 4º) as column instability progressed rapidly from the east wall along the adjacent north and south walls. The building section above impact continued to rotate to the east as it began to fall downward, and rotated to at least 20 to 25 degrees.
- The change in potential energy due to downward movement of building mass above the buckled columns exceeded the strain energy that could be absorbed by the structure. Global collapse then ensued.

Figure E–12. WTC 2 probable collapse sequence (cont).

E.4 FACTORS THAT AFFECTED PERFORMANCE

- From the collective knowledge and insights gained through the Investigation of the collapse of the WTC towers, the following factors were identified that enhanced performance of both towers on September 11, 2001: The closely spaced columns, along with deep short spandrels, allowed a redistribution of loads as a result of aircraft impact damage to the exterior wall.
- Because there was effectively no wind on the morning of September 11, 2001, the capacity of the exterior wall provided to accommodate design wind loads was available to carry redistributed gravity loads.
- The large dimensional size of the WTC towers helped the buildings withstand the aircraft impact.
- The composite floor system with primary and bridging trusses forming a 2-way grid, and the two layers of welded wire fabric in the slab, acted to bridge over damaged areas without propagation of collapse from areas of aircraft impact damage to other locations, thereby avoiding larger scale floor collapse upon impact.
- The hat truss played a major role in the post-impact performance of the building. This was accomplished through redistribution of the loads from the significant weakening of the core, due to aircraft impact damage and subsequent thermal effects, by redistributing loads from the damaged core columns to adjacent intact columns and, ultimately, by redistributing loads to the exterior walls from the thermally weakened core columns that lost their ability to support the buildings' weight.
- The buildings would likely not have collapsed under the combined effects of aircraft impact and the subsequent jet-fuel ignited multi-floor fires, if the fireproofing had not been dislodged or had been only minimally dislodged by aircraft impact. The existing condition of the fireproofing prior to aircraft impact and the fireproofing thickness on the WTC floor system did not play a significant role in initiating collapse on September 11, 2001.

E.5 FINDINGS

E.5.1 PASSIVE FIRE PROTECTION

The passive fire protection applied to the steel structural components in the WTC towers was investigated to provide information on the in-place condition of the fireproofing before and after aircraft impact. The specified and "as applied" thicknesses, the variability in thickness, the condition of the fireproofing over a 30-year service life, and the effects that the variability and condition have on the structural behavior of insulated steel members were studied. The rationale behind the selection of the effective thickness of thermal insulation for use in thermal analyses was presented. Additionally, the procedures and practices used to provide the passive fire protection for the floor system of the WTC tower structures was documented.

Building Code Requirements for Structural Fire Resistance

Finding 1: The WTC towers were classified as Class 1B, as defined by the 1968 New York City Building Code. This classification required a 3 h fire rating for columns and 2 h for floors. The towers could have been classified as Class 1A since both Class 1A and 1B permitted buildings of unlimited height. Class 1A required a 4 h fire resistance rating for columns and a 3 h rating for floors. In 1969, the Port Authority specified the 0.5 in. fireproofing for all beams, spandrels and trusses, to maintain the Class 1-A Fire Rating of the New York City Building Code. A condition assessment conducted in 2000 reported that the WTC towers were classified as Class-1B—noncombustible, fire-protected, and retrofitted with sprinklers in accordance with Local Law 5/1973.

Selection of Fire Resistive Materials

Finding 2: The passive fire protection for the floor trusses was specified to be 0.5 in. of CAFCO BLAZE-SHIELD Type D, although the technical basis for the selection of this product and required thickness value is not known. After applying the Type D sprayed fire resistive materials to the lower 40 floors of WTC 1, the CAFCO insulating material was switched to Type D/CF (reported to meet or exceed the insulating properties of Type D) which did not contain asbestos. In 1995, the Port Authority conducted a study to establish the fireproofing requirements for the floor trusses in areas undergoing major tenant renovation. The thickness required to achieve a 2 h fire rating was determined to be 1.5 in. using the CAFCO BLAZE-SHIELD II product. At the time of the WTC disaster, fireproofing had been upgraded on a number of floors in the WTC towers: 18 floors in WTC 1, including all of the floors affected by the aircraft impact and fires, and 13 floors in WTC 2, although none that were affected by the aircraft impact and fires.

Equivalent thickness of SFRM

Finding 3: Based on analyses of SFRM thickness measurements and interpretation of photographs showing the condition of the originally applied material, the average thickness of the original thermal insulation on the floor trusses was estimated to be 0.75 in. with a standard deviation of 0.3 in. (coefficient of variation of 0.40). The average thickness of the upgraded thermal insulation was estimated to be 2.5 in. with a standard deviation of 0.6 in. (coefficient of variation of 0.24). Based on finite-element simulations, it was concluded that the original passive fire protection on the floor trusses was thermally equivalent to a uniform thickness of 0.6 in. and the upgraded insulation was thermally equivalent to a uniform thickness of 2.2 in. These values were used in the thermal analyses for determining temperature histories of structural components.

Finding 4: No information was available on in-place conditions of the thermal protection on the exterior columns and spandrel beams, and little information was available on the conditions of fire resistive material on core beams and columns. For thermal analyses of the towers, the SFRM on these elements was taken to have uniform thicknesses equal to the specified thickness. This assumption was supported by the observation that measured average thickness tended to be *greater* than the specified thickness while, due to variability, the effective thickness tended to be *less* than the average uniform thickness. The specified thickness values were 0.5 in. for beams and spandrels, 2.06 in. (2 1/16 in.) for columns lighter than 14WF228, and 1.19 in. (1 3/16 in.) for columns equal to or heavier than 14WF228.

Finding 5: The adhesive strength of CAFCO BLAZE-SHIELD DC/F to primed steel was found to be a third to a half of the adhesive strength to steel that had not been coated with primer paint. The SFRM products used in the WTC towers were applied to steel components with primer paint.

E.5.2 FIRE RESISTANCE TESTS

Four Standard Fire Tests (ASTM E 119) were conducted on floor assemblies constructed to duplicate, as closely as practical, the floor system used in the WTC towers. Full scale tests with a 35 ft span, and having ¾ in. thick SFRM were tested; one in the restrained test condition and the other in the unrestrained test condition. Tests of half-scale specimens, which spanned approximately 17 ft, were conducted using fireproofing conditions simulating the “as specified” condition (0.5 in. thick SFRM) and the “as-applied” condition (0.75 in. thick SFRM). The following findings are based on this series of four tests and a comparison of their results.

Structural Performance

Finding 6: Exposure of the WTC floor assemblies to the Standard Fire Test (ASTM E 119) conditions resulted in extensive spalling on the underside of the flor slab. thermal damage to the bridging trusses, and buckling of compression diagonals and vertical struts of the main trusses.

Finding 7: All four tests demonstrated that the floor assemblies were capable of sagging without failure. The unrestrained test, which had two 0.875 in. bolts fastening the main truss to the truss seats, did not sag sufficiently to bear on the bolts. The three restrained tests welded the main truss ends to the truss seats to provide the required restraint. The magnitude of the sagging observed in the tests was consistent with that computed from finite element structural analyses. No evidence of knuckle failures was seen in the tests.

Finding 8: All four test assemblies supported their full design load under standard fire conditions for two hours without collapse.

Fire Resistance Ratings

Finding 9: The 1968 New York City building code—the code that the WTC towers were intended but not required to meet when they were built—required a 2 h fire rating for the floor system.

Finding 10: The restrained WTC floor system obtained a fire resistance rating of 1.5 h while the unrestrained floor system achieved a 2 h rating. This finding was unexpected since the unrestrained rating is typically less than the restrained rating.

Finding 11: The test of the 17 ft specimen with as-applied fireproofing did not produce the same rating as the 35 ft test specimen, giving 2 h and 1.5 h, respectively. In both cases, the rating was established on the basis of temperatures of the unexposed surface (top of concrete slab) and not on the ability of the specimen to support the load.

Finding 12: The 45 min rating for the standard 17 ft test with the specified 0.5 in. fireproofing did not meet the 2 h requirement of the NYC building code. This test had no fireproofing on the bridging trusses nor on the underside of the metal deck.

Finding 13: The 2 h rating for the standard 17 ft test with the as-applied average 0.75 in. fireproofing met the 2 h requirement of the NYC building code. This test had half the fireproofing thickness on the bridging trusses (0.375 in.) and overspray on the underside of the metal deck.

Finding 14: The difference in test results for the two 17 ft specimens is due primarily to the concrete slab performance (spalling and cracking) and the presence or lack of fireproofing overspray on the metal deck and not due to the fireproofing thickness on the trusses. Differences in the degree of concrete spalling were possibly due to differences in moisture content and the slab cracking.

E.5.3 STRUCTURAL RESPONSE OF COMPONENTS

The response of the structural components and their connections for the tenant floors and exterior walls was examined with detailed structural models. Results of the floor and exterior wall component and connection analyses identified structural behaviors and failure modes that were required for inclusion in the global analyses.

Floor System

Finding 15: The interior truss seats had a greater vertical shear capacity than the exterior truss seats. The controlling failure mode for vertical shear was weld fracture. However, the vertical load at the truss connection of approximately 16 kips had to increase by a factor of 2 to 6 to reach failure (weld fracture) for temperatures near 600 °C to 700 °C.

Finding 16: Detailed structural analysis of a single truss section of the composite floor system subjected to elevated uniform temperatures was found to initially push out on the exterior columns as a result of the concrete slab thermal expansion, and then pull inward as the web diagonals buckled and the truss sag increased. The magnitude of the pull-in force was found to depend highly on the stiffness of the exterior box column which, in turn, depends on expansion of floors above and below.

Finding 17: Detailed analysis of the knuckles (shear connectors in the floor system for composite action) through test simulation and detailed truss analysis 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 (ASTM E119) of the floor truss assemblies with twice the floor load that was on the WTC floors.

Exterior Wall System

Finding 18: Large inelastic deformations and buckling of the spandrels at elevated temperatures were predicted, but were found not to significantly affect the stability of the exterior columns. Partial separations of the spandrel splices were also predicted at elevated temperatures, but were found not to significantly affect the stability of the exterior columns.

Finding 19: Analyses of bolted splices in the exterior columns found that the splice may slide or open when the exterior columns are bowing and have large lateral deflections. No column splice bolts were predicted to have failed.

Finding 20: An exterior wall section (9 columns wide and 9 floors high) was found to bow inward when the floor connections applied an inward pull force. For the condition where three sequential floors were disconnected, there was no bowing of the columns for five different elevated temperature conditions. When the column section with three disconnected floors was subjected to increased axial column loads, the wall section bowed outward over the unsupported column length.

E.5.4 FIREPROOFING AND PARTITION DAMAGE DUE TO AIRCRAFT IMPACT

The aircraft impact of the WTC towers caused extensive damage to the buildings' exterior, penetrated into the interior causing further damage to the structural system, dislodged fireproofing, and ignited multi-floor fires. The structural damage to each tower resulting from the aircraft impact was estimated using a transient finite element analysis. Results of this analysis were used to predict damage to the structure, fireproofing, and partition walls in the path of the debris field.

Finding 21: For WTC 1, partitions were damaged and fireproofing was dislodged by direct debris impact over five floors (Floors 94, 95, 96, 97, and 98) and included most of the north floor areas in front of the core, the core, and central regions of the south floor areas, and on some floors, extended to the south wall. For WTC 2, partitions were damaged and fireproofing was dislodged by direct debris impact over six floors (Floors 79, 80, 81, 82, and 83) and included the south floor area in front of the core, the central and east regions of the core, and most of the east floor area, and extended to the north wall.

Finding 22: The fireproofing damage estimates were conservative as they ignored possibly damaged and dislodged fireproofing in a much larger region that was not in the direct path of the debris but was subject to strong vibrations during and after the aircraft impact. A robust criteria to generate a coherent pattern of vibration-induced dislodging could not be established to estimate the larger region of damaged fireproofing.

E.5.5 OBSERVATIONS AND TIMELINE

Thousands of photographs and hours of video tape were reviewed for insights into the structural performance of the towers. A timeline of significant events that characterized the weakening and eventual collapse of the WTC towers was developed with the photographs and videos that were time-stamped. Quantitative information, such as the amount of inward bowing observed on the exterior walls of the buildings, was extracted from key photographs through image enhancement and scaled measurements. Key observations and the timelines were used to guide the global collapse analyses.

WTC 1

Finding 23: Inward bowing of the south exterior wall was first observed at 10:23 a.m. The bowing appeared to extend between Floors 94 to 100 and Columns 305 to 359. The maximum bowing was estimated from images to be 55 in.±6 in. at Floor 97 on the east side of the south face of WTC 1. The central area in available images was obscured by smoke. The extent of fires observed on all faces of WTC 1 was similar, although somewhat more extensive on the east and west faces (where short span floors were located) and similar in extent on the north and south faces (where long span floors were located). Inward bowing was observed only on the south face. The north face had extensive aircraft impact damage and the damaged floors were not capable of imposing inward pull forces on the north face.

Finding 24: The time to collapse initiation was 102 minutes from the aircraft impact (9:46:30 a.m. until 10:28:22 a.m.).

Finding 25: From exterior observations, tilting of the building section appeared to take place near Floor 98. Column buckling was then observed to progress rapidly across the east and west faces.

Finding 26: The WTC 1 building section above the impact and fire area tilted to the south as the structural collapse initiated. The tilt was toward the side of the building that had long span floors. Video records taken from east and west viewpoints showed that the upper building section tilted to the south. Video records taken from a north viewpoint showed no discernable east or west component in the tilt. A tilt to the south of at least 8 degrees occurred before dust clouds obscured the view and the building section began to fall downwards.

WTC 2

Finding 27: On the east face and north face of WTC 2, draped objects were observed through the windows of Floor 82 on the east face and Floors 81 to 83 on the north face near the northeast corner. The draped objects appeared to be hanging floors. The drape of these objects was observed to increase with time and extend across approximately half of the east face.

Finding 28: Inward bowing of the east wall was first observed at 9:21 a.m. The inward bowing was approximately 10 in.±1 in. at floor 80 and extended between Floors 78 to 83 and Columns 304 to 344. The remaining portion of the face to the south of Column 344 was not included in the image. The bowing appeared to extend over a large fraction of the east face, and to be greatest near the center of the face. Fires were more extensive along the east face (where long span floors were located) and at the east side of the north and south faces (where short span floors were located). Fires were not observed on the west face (where long span floors were located). Inward bowing was observed only on the east face. The south face had extensive aircraft impact damage and the damaged floors were not capable of imposing inward pull forces on the south face. There was no impact damage or fires on the west floors to cause pull-in forces on the west face.

Finding 29: An increase of the inward bowing of the east wall was observed at 9:53 a.m. The inward bowing appeared to extend between Floors 78 to 84 and Columns 305 to 341. The remaining portion of the face to the south of Column 344 was not included in the image. The maximum bowing was estimated from images to be 20 in.±1 in. at floor 80 on the east face of WTC 1.

Finding 30: The time to collapse initiation was 56 minutes after aircraft impact (9:02:59 a.m. to 9:58:59 a.m.).

Finding 31: From exterior observations, tilt of the building section above the impact and fire area appeared to take place near floor 82. Column buckling was then seen to progress across the north face.

Finding 32: The building section above the impact and fire area tilted to the east and south at the onset of structural collapse. The tilt occurred toward the east side with the long span floors. Estimates made from photographs indicate that there was approximately a 3 degree to 4 degree tilt to the south, and a 7 to 8 degree tilt to the east, prior to significant downward movement of the upper portion of the building.

The tilt to the south did not increase any further as the upper building section began to fall, but the tilt to the east continued reaching 20 degrees to 25 degrees before dust clouds obscured the view.

E.5.6 STRUCTURAL RESPONSE OF MAJOR TOWER SUBSYSTEMS

Prior to conducting the analysis of the global structural response of each tower, major structural subsystems were analyzed to provide insight into their behavior within the WTC global system. The three major structural subsystems, the core framing, a single exterior wall, and full tenant floors, were analyzed separately for their response to impact damage and fire. The hat truss was not analyzed separately as its structural behavior did not require significant simplification in the global analysis. The component analyses provided a foundation for these large, nonlinear analyses with highly redundant load paths and they enabled a significant reduction in finite element model complexity and size. The major subsystem models used final estimates of impact damage and elevated temperatures determined from the aircraft impact analysis and the fire dynamics and thermal analyses.

Isolated Core Subsystem

Finding 33: The WTC 1 isolated core subsystem analysis found that the core structure was most weakened from impact and thermal effects at the center of the south side of the core. Smaller displacements occurred in the global model due to the constraints of the hat truss and floors.

Finding 34: The WTC 2 isolated core subsystem analysis found that the core structure was unstable for the estimated structural damage to core columns. The core was most weakened from impact and thermal effects at the southeast corner and along the east side of the core. Larger displacements occurred in the global model as the isolated core model had lateral restraints imposed that were somewhat stiffer than in the global model.

Full Floor Subsystem

Finding 35: Floor sagging was caused primarily by either buckling of truss web diagonals or disconnection of truss seats at the exterior wall or the core perimeter. Except for the truss seat failures near the southeast corner of the core in WTC 2 following the aircraft impact, web buckling or truss seat failure was caused primarily by elevated temperatures of the structural components.

Finding 36: Analysis results from both the detailed truss model and the full floor models found that the floors began to exert inward pull forces when floor sagging exceeded approximately 25 in. for the 60 ft floor span.

Finding 37: Sagging at the floor edge was due to loss of vertical support at the truss seats. The loss of vertical support was caused in most cases by the reduction in vertical shear capacity of the truss seats due to elevated steel temperatures.

Isolated Exterior Wall Subsystem

Finding 38: Inward pull forces were required to produce inward bowing that was consistent with displacements measured from photographs. The inward pull was caused by sagging of the floors. Heating of the inside face of the exterior columns also contributed to inward bowing.

Finding 39: The observed inward bowing of the exterior wall indicated that most of the floor connections were intact to cause the observed bowing.

Finding 40: The floors that were affected by the fires and the dislodged fireproofing matched well with the floors that participated in the inward bowing of the exterior walls.

Finding 41: The extent of floor sagging required at each floor was greater than that predicted by the full floor models. The estimates of the extent of sagging at each floor were governed by the combined effects of fireproofing damage and fire; fireproofing damage estimates were limited to areas subject to direct debris impact. Other sources of floor and fireproofing damage from the aircraft impact and fires (e.g., fireproofing damage due to shock and subsequent vibrations as a result of aircraft impact or concrete slab cracking and spalling as a result of thermal effects) were not included in the floor models.

E.5.7 STRUCTURAL RESPONSE TO AIRCRAFT IMPACT DAMAGE AND FIRE

Global analysis of WTC 1 and WTC 2 used final estimates of impact damage and elevated temperatures to determine the structural response and sequence of component and subsystem failures that led to collapse initiation.

General Findings

Finding 42: The structural analyses of WTC 1 and WTC 2 found that the collapse of the towers was due to the combined effects of structural and fireproofing damage from aircraft impact and the subsequent fires on the core, floor systems and exterior walls. The towers collapsed when the weakened core and exterior columns could no longer redistribute or support the building loads with their reduced load carrying capacity.

Finding 43: Impact damage alone did not cause collapse of the towers, as they were stable after the aircraft impact. Global analyses showed that both towers had substantial reserve capacity after the aircraft impact.

Finding 44: The multi-floor fires alone did not cause collapse of the towers. Without impact damage to the fireproofing, the structural steel temperatures would have been generally less than 200 °C to 300 °C, with a few isolated locations of structural steel temperatures less than 400 ˚C in WTC 1 floors and 500 ˚C in WTC 2 floors. The core would not have weakened, the floor sag would have been insufficient to pull inward on the exterior columns, and the exterior walls would not have bowed inward.

Finding 45: The towers would likely not have collapsed under the combined effects of aircraft impact and the subsequent multi-floor fires, if the fireproofing had not been dislodged or had been only minimally dislodged by aircraft impact. Had fireproofing not been dislodged by debris field, temperature rise of structural components would likely have been insufficient to induce global collapse. Structural components that became thermally weakened were generally determined by impact of the debris field. The existing condition of the fireproofing prior to aircraft impact and the fireproofing thickness on the WTC floor system did not play a role in initiating collapse of the towers.

Finding 46: Creep strain was significant in the core and exterior columns over the 56 to 102 min period of fire exposure in columns with temperatures greater than 500 ˚C to 600 ˚C and high stress. Columns with creep strains of sufficient magnitude to cause column shortening played a significant role in the collapse initiation.

Finding 47: The faces of the buildings that exhibited inward bowing were associated with the long span direction of the floor system. The primary direction of tilting at collapse initiation for WTC 1 and WTC 2 was in the direction of the bowed faces.

Performance with Intact fireproofing

Finding 48: A detailed thermal-structural analysis, which did not include slab delamination/spalling effects, showed that a full collapse of the WTC floor system would not occur even with a number of failed trusses or connections.

Finding 49: Most of the horizontal and vertical capacity of the floor connections to the exterior and core columns significantly exceeded the demand under design load conditions.

E.5.8 PROBABLE COLLAPSE SEQUENCES

The results of structural analyses conducted in this study on components, subsystems, isolated exterior walls and cores, and global models of WTC 1 and WTC 2 showed that the collapses of the towers were initiated due to the combined effects of the structural and fireproofing damage from aircraft impact and the subsequent intense fires. The probable collapse sequence for WTC 1 and WTC 2 are based upon the collective consideration of structural analyses, statistical based methods, observations, and laboratory testing.

Role of the Building Core

Finding 50: The core columns were weakened significantly by the aircraft impact damage and thermal effects. Thermal effects dominated the weakening of WTC 1. As the fires moved from the north to the south side of the core, following the debris damage path, the core was weakened over time by significant creep strains on the south side of the core. Aircraft impact damage dominated the weakening of WTC 2. Immediately after impact, the vertical displacement at the southeast corner of the core increased 6 in. (from 4 in. to 10 in.). With the impact damage, the core subsystem leaned to the southeast and was supported by the south and east floors and exterior walls.

Finding 51: As the core was weakened from aircraft impact and thermal effects, it redistributed loads to the exterior walls primarily through the hat truss. Additional axial loads redistributed to the exterior columns from the core were not significant (only about 20-25 percent on average) as the exterior columns were loaded to approximately 20 percent of their capacity before the aircraft impact.

Role of the Building Floors

Finding 52: The primary role of the floors in the collapse of the towers was to provide inward pull forces that induced inward bowing of exterior columns (south face of WTC 1; east face of WTC 2).

Finding 53: Sagging floors continued to support floor loads as they pulled inward on the exterior columns. There would have been no inward pull forces if many of the floors truss seats had failed and disconnected.

Role of Exterior Frame-Tube

Finding 54: Column instability over an extended region of the exterior face ultimately triggered the global system failure as the loads could not be redistributed through the hat truss to the already weakened building core. In the area of exterior column buckling, load transferred through the spandrels to adjacent columns and adjacent exterior walls. As the exterior wall buckled (south face for WTC 1 and east face for WTC 2), the column instability propagated to adjacent faces and caused the initiation of the building collapse.

Finding 55: The exterior wall instability was induced by a combination of thermal weakening of the columns, inward pull forces from sagging floors, and to a much lesser degree, additional axial loads redistributed from the core.

Probable Collapse Sequences

Finding 56: Although the north face of WTC 1 had extensive impact damage, thermal weakening of the core columns on the south side of the core and inward bowing of the south face caused the building to tilt to the south at collapse initiation. The extent of fires observed on all faces of WTC 1 was similar, although somewhat more extensive on the east and west faces (where short span floors were located) and similar in extent on the north and south faces (where long span floors were located). Thermal weakening of exterior columns with floor sagging (which induced inward pull and occurred on south side) caused inward bowing of the south face and tilting in the south direction.

Finding 57: Although the south face of WTC 2 had extensive impact damage, thermal weakening of the core columns on the east side of the core and inward bowing of the east face caused the building to tilt more to the east and less to the south at collapse initiation. Fires were more extensive along the east face and at the east side of the north and south faces. Thermal weakening of exterior columns with floor sagging (which induced inward pull and occurred on the east side) caused inward bowing of the east face and primary tilting in that direction (with additional southward tilting due to the aircraft impact damage).

Finding 58: The time it took for each WTC tower to collapse was due primarily to the differences in structural damage, the time it took the fires to travel from the impact area across the floors and core to critical locations, and the time it took to weaken the core and exterior columns. WTC 2 had asymmetric structural damage to the core including the severing of a corner core column and WTC 1 had more symmetrical damage. The fires in WTC 2 reached the east side of the building more quickly, within 10 to 20 minutes, than the 50 to 60 minutes it took the fires in WTC 1 to reach the south side.