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Appendix D
INTERIM REPORT ON PRELIMINARY STABILITY ANALYSIS OF THE WTC TOWERS

D.1 INTRODUCTION

The objective of this appendix is to present preliminary system stability analyses of the World Trade Center (WTC) towers to: (1) examine the overall stability of the towers when floors are removed; (2) study possible redistribution mechanisms when core columns are destroyed by aircraft impact; and (3) study the response of the tower when columns and spandrels in the exterior walls and columns in the core are destroyed by aircraft impact, and columns in the exterior are damaged due to the subsequent fires, as observed in photographs and videos of WTC 1. The analyses use a reduced and modified version of the global reference structural model of WTC 1 and the model of a typical truss-framed floor (floor 96 of WTC 1), both developed within the framework of Project 2 of the investigation using SAP2000, version 8 (see Appendix B). Although analyses are conducted using models of WTC 1, some of the results and findings apply to WTC 2 as well.

The analyses use a staged construction technique to account for the sequential construction of the towers, especially in the zone of the hat trusses. Linear buckling analysis and nonlinear analysis with plastic hinges are used in the reduced global model of WTC 1 to study the effects of removal of, respectively, floors and damaged exterior and core columns, representing the effects of aircraft impact and subsequent fire effects. In addition, a linear analysis of the typical floor model is used to study the load redistribution mechanisms after losing columns in the core of the tower.

Section D.2 presents a description of the reduced global model of WTC 1 used in this study, including the modifications that were made to the reference model. Section D.3 outlines the analysis procedures, including the staged construction methodology, the eigenvalue buckling analysis, the nonlinear analysis with plastic hinges, and the linear analysis of the typical floor model. The results of these analyses are presented in Section D.4, and Section D.5 presents a summary of the analysis and results.

D.2 MODEL DESCRIPTION AND ASSUMPTIONS

The models considered for the preliminary system stability analyses of the WTC towers were based on the reference structural analysis global model of WTC 1 and the typical truss-framed floor model developed by the firm of Leslie E. Robertson Associates, R.L.L.P. (LERA) under contract from NIST within the framework of Project 2. These reference models, developed using Computers and Structures, Inc.'s SAP2000 Software, Version 8, were reviewed and approved by National Institute of Standards and Technology (NIST) (see Appendix B). The reference global models are linear elastic, three-dimensional structural analysis models and include the 110-story above grade structure and 6-story below grade structure for each of the two towers. The original models use frame elements to represent the exterior columns and spandrels, the core columns, and the hat trusses. Each element in the models is assigned cross-sectional properties and steel strength according to the original design documents, as well as later modifications made to the towers.

A reduced version of the original WTC 1 global model was used in this project to assess typical behavior of the intact structure, as well as the performance of the damaged structure, due to aircraft impact and fire effects. The intent of the reduced model was to minimize the computational effort without a major sacrifice in model performance. The reduced model included the global model of the structure above floor 84 of WTC 1. The structure below was removed and replaced with equivalent springs as summarized in Section D.2.2. The modifications and loads applied to the model are summarized below.

The model of a typical truss-framed floor (floor 96 of WTC 1) was used to study the load redistribution mechanisms inside the core upon losing core columns due to aircraft impact. The floor model contains all primary structural members of the floor system, including primary and bridging trusses, beams in the core, strap anchors, viscoelastic dampers, exterior and core columns above and below floor level, spandrel beams, and concrete slabs. The gravity loads applied to the model, including dead loads, superimposed dead loads, and service live loads are presented in Section D.2.4.

D.2.1 Steel Properties

The values of the yield and ultimate strengths of the structural steel used in the WTC 1 reduced tower model were set to match the room-temperature properties, which were determined by Project 3 of the NIST investigation, by replacing the nominal strength included in the reference models with actual strength values. Project 3, "Mechanical and Metallurgical Analysis of Structural Steel," provided estimates for typical steel properties based on test results from a limited number of steel specimens from the towers, construction documents that indicate the occasional substitution of higher strength steels for lower strength steels, and historical data from steels of that era. These estimates differentiate among steels with the same designation from different manufacturers and from different areas of the buildings; in particular, steel in the exterior columns, core columns, and floor trusses each had slightly different properties for steels with the same designation. In cases where steel with a particular designation had different properties in the exterior and core columns, the properties associated with the steel in the exterior columns were used in the reduced global model. The steel properties used for the work reported herein are listed in Table D–1.

D.2.2 Boundary Conditions: Spring Supports

The reduced model of WTC 1 was supported by vertical springs assigned to each joint (core and exterior) at floor 84. The spring stiffness coefficients were obtained from a separate model of the tower below floor 84. For that purpose, a concentrated gravity load was applied to each column node at floor 84 of the tower model below floor 84, and the spring stiffnesses were estimated by dividing the applied load by the measured vertical displacement of each column at floor 84. At the bottom of the reduced model, each joint with an assigned vertical spring was restrained from horizontal translation and rotation about all three axes. These boundary conditions provided results that most closely matched those obtained from analyzing the whole tower (i.e., all 116 floors).

Table D–1. Steel strength used in the reduced tower model.

Design Yield Strength ksi (MPa)	Model Yield Strength F_y ksi (MPa)	Model Ultimate Strength F_u ksi (MPa)
36 (248.2)	35.6 (245.5)	61.2 (422.0)
42 (289.6)	53.1 (366.1)	74.9 (516.4)
45 (310.3)	53.1 (366.1)	74.9 (516.4)
46 (317.2)	53.1 (366.1)	74.9 (516.4)
50 (344.7)	54.0 (372.3)	75.6 (521.2)
55 (379.2)	60.8 (419.2)	82.6 (569.5)
60 (413.7)	62.0 (427.5)	87.3 (601.9)
65 (448.2)	69.6 (479.9)	90.4 (623.3)
70 (482.6)	76.7 (528.8)	92.0 (634.3)
75 (517.1)	82.5 (568.8)	96.8 (667.4)
80 (551.6)	91.5 (630.9)	99.4 (685.3)
85 (586.1)	104.8 (722.6)[a]	116.0 (799.8)[a]
90 (620.5)	104.8 (722.6)[a]	116.0 (799.8)[a]
100 (689.5)	104.8 (722.6)	116.0 (799.8)

a. The steel fabricator used steel with a nominal strength of 100 ksi in place of steels with specified strengths of 85 ksi and 90 ksi.

D.2.3 Floor Systems

Floor systems distribute gravity loads to the core and exterior columns. Actual member properties of the floor elements have relatively little effect on the towers' stability, but would significantly increase model complexity and decrease its efficiency. To capture their effect, each floor of the tower in the reduced global model was modeled with a rigid diaphragm, except for floor 107 to the roof, which were modeled using flexible diaphragms as described in Appendix B. Rigid diaphragms constrain all nodes at a particular floor to move as a single unit. Flexible diaphragms differ only in the level of constraint. Both types of diaphragms do not affect the relative vertical displacements of the nodes. Modeling floors with diaphragms (rigid or flexible) ignores the floor's capability of redistributing loads from column to column in a damaged case (particularly within the core columns). More detailed models developed within the framework of Project 2 for the aircraft impact analysis and Project 6 for the thermal-structural and collapse initiation analysis will address this issue.

D.2.4 Applied Loads

Gravity loads from the floor systems were applied as joint loads on columns in the model. They are comprised of: dead load (DL), which includes the self-weight of all structural members and the floor systems; superimposed dead load (SDL), which includes additional static dead loads such as partitions, fireproofing, ceiling systems, and floor coverings; and service live load (LL) which includes occupants and furnishings. Each load was applied to the model as a separate load case. Identical loads were used in the reduced global model of WTC 1 on floors 85 through 106. These floors were considered to be typical floors as described below. Loads varied on floors 107 through 110, and on the roof. The antenna load

(724 kips) was distributed among eight points near the center of the roof. These loads should be considered preliminary; NIST is working with LERA to further refine them.

The loads were developed based on a realistic assessment of service loads and their distribution throughout the towers. The service live loads were assumed to be 25 percent of the design live loads. This parametric value can easily be varied in a sensitivity analysis. Actual self-weights were used for the dead loads, and additional loads on the mechanical floors were accounted for explicitly.

The detailed model of a typical truss-framed floor, floor 96 of WTC 1 (see Appendix B), was used to determine the actual loads on floors 85 through 106 of the reduced global model. The detailed floor model contains four distinct areas, each with its own load, as shown in Fig. D–1. The superimposed dead and live loads applied to each of these areas were determined from the design documents and are listed in Table D–2. In the core, the design live loads varied from 40 psf (1.92 kN/m^2) to 100 psf (4.79 kN/m^2) and were scaled to service live loads from 10 psf (0.479 kN/m^2) to 25 psf (1.20 kN/m^2), while the superimposed dead load varied from 29 psf (1.39 kN/m^2) to 49 psf (2.35 kN/m^2). The loads due to the slab, trusses, beams, columns, and spandrels were all calculated based on the actual weights of the members. The reactions at each column due to the loads and self-weight of this typical floor were calculated separately for the dead load and superimposed dead load cases. The weights of the columns and spandrels were then subtracted from the DL results to determine the load on each column from the floor system. These loads were then applied as point loads to the reduced global tower model. The loads due to the self-weight of the columns and spandrels were calculated by the analysis software (SAP2000) for the reduced global tower model.

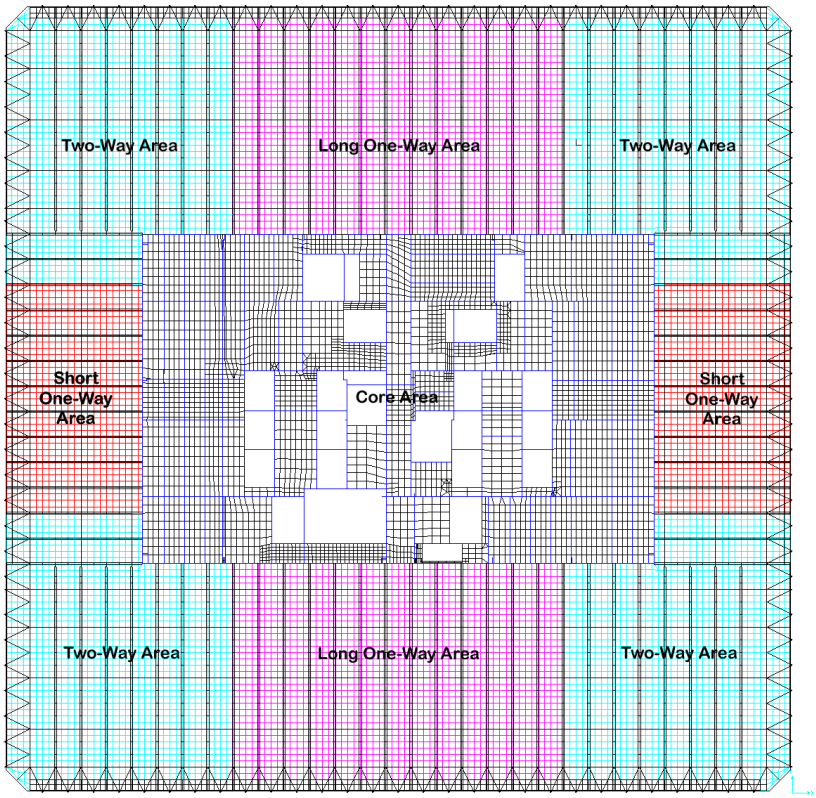


Figure D–1. Plan of typical truss-framed floor with loading areas indicated.

Table D–2. Superimposed dead load and service live load on typical floor.

	Core	Long One-way Slabs	Short One-way Slabs	Two-way Slabs
Area [ft^2 (m^2)]	9274 (861.6)	13186 (1,225.1)	5228 (485.7)	12233 (1,136.5)
Superimposed Dead Loads [psf (kN/m^2)]				
Mechanical & electrical		2.0 (0.096)	2.0 (0.096)	2.0 (0.096)
Ceiling		2.0 (0.096)	2.0 (0.096)	2.0 (0.096)
Floor covering		2.0 (0.096)	2.0 (0.096)	2.0 (0.096)
Fireproofing		2.0 (0.096)	2.0 (0.096)	4.0 (0.192)
Total SDL	**Varies**	**8.0 (0.383)**	**8.0 (0.383)**	**10.0 (0.479)**
Live Loads [psf (kN/m^2)]				
Service LL	Varies	17.5 (0.838)	21.3 (1.020)	13.8 (0.661)

For the mechanical floors (floors 107 through 110) and roof, the design DL, design SDL, and service LL values were determined from the design documents and information provided by LERA. These loads, which are listed in Table D–3, were applied as uniform loads to the typical floor model to estimate the corresponding column reactions. The column reactions were then applied as point loads on the reduced tower model.

Table D–3. Dead, superimposed dead, and live loads on mechanical floors.

Floor	**107**	**108**	**109**	**110**	**Roof**
Dead Loads [psf (kN/m^2)]					
Concrete slab	100.0 (4.788)	69.0 (3.304)	69.0 (3.304)	104.0 (4.980)	48.1 (2.304)
Reinforcing steel	2.0 (0.096)	3.0 (0.144)	3.0 (0.144)	3.0 (0.144)	2.0 (0.096)
Steel deck	2.0 (0.096)	2.0 (0.096)	2.0 (0.096)	2.0 (0.096)	2.0 (0.096)
Structural steel	13.0 (0.622)	20.0 (0.958)	20.0 (0.958)	20.0 (0.958)	(a)
Total DL	117.0 (5.602)	94.0 (4.501)	94.0 (4.501)	129.0 (6.177)	52.1 (2.496)
Superimposed Dead Loads [psf (kN/m2)]					
Partitions	12.0 (0.575)	-	-	-	-
Ceiling	2.0 (0.096)	10.0 (0.479)	10.0 (0.479)	-	-
Mech. & elec.	2.0 (0.096)	3.0 (0.144)	3.0 (0.144)	50.0 (2.394)	50.0 (2.394)
Fireproofing	2.0 (0.096)	5.0 (0.239)	5.0 (0.239)	5.0 (0.239)	5.0 (0.239)
Flooring	57.0 (2.729)	31.0 (1.484)	31.0 (1.484)	-	5.0 (0.239)
Total SDL	75.0 (3.591)	49.0 (2.346)	49.0 (2.346)	55.0 (2.633)	60.0 (2.873)
Live Loads [psf (kN/m2)]					
Service LL	25.0 (1.197)	18.8 (0.898)	37.5 (1.796)	18.8 (0.898)	37.5 (1.796)

a. The roof structural steel is explicitly included in the tower model.

D.3 ANALYSIS PROCEDURES

Both linear and nonlinear analyses were performed on the reduced global model of WTC 1 to examine the tower stability and assess how the tower responded to the representative impact and fire damage. Due to

the difference in stiffness between the core and the exterior columns, and the presence of the hat trusses, it was necessary to use nonlinear, staged construction to analyze the intact structure. Subsequently, two different series of analyses were performed independently.

An eigenvalue-based buckling analysis was performed using the reduced global model of WTC 1 to determine the reserve capacity of the columns to buckling, and to determine how much the unsupported column length would need to increase, through floor-constraint removal, before the columns lacked any reserve capacity.

A nonlinear analysis of the tower with damage to exterior walls and core columns was performed on the reduced global model of WTC 1 to determine if the tower could withstand that level of structural damage, and to assess the response of the tower when columns are lost due to aircraft impact and fire effects.

In addition, an analysis was conducted of the typical truss-framed floor model to study the mechanism by which the floor loads were redistributed when the core columns were severed by aircraft impact. In this analysis, the core columns that were assumed to be missing were replaced by equivalent vertical springs, representing the stiffness of the hat trusses and columns between the affected floors and hat trusses. The following describes the details of the various analyses.

D.3.1 Staged Construction

From a linear analysis of the response of the intact WTC towers to gravity loads, it was determined that a simple linear analysis does not produce realistic stress distributions in the core and exterior columns. All loads in a linear model are applied instantaneously, which is not unreasonable for most structural models. Tall buildings sustain loads gradually, as the structure is built from the ground up, and any differential deformation is accounted for during construction. In addition, the hat trusses atop the tower were applied stress-free to the existing structure subjected to dead loads, but prior to the application of live loads. The linear analysis (without staged construction) of the tower models resulted in unrealistic, large forces and stresses in some hat truss members, connecting spandrels, and core columns within the hat trusses, due to differential settlement between the core and exterior columns in the model.

A staged construction analysis of the towers eliminates these nonexistent, large stresses. This method more closely approximates the way in which the towers were constructed and the loads applied. The staged construction analysis had three stages. First, the floors below the hat truss (up to floor 106) constituted the initial model. The dead and superimposed dead loads were applied to these elements, and the model was analyzed. Second, the upper, remaining stories including the hat truss were added to the model. These newly added components did not initially have internal forces or stresses, even though the components added in the first stage were loaded and stressed. The remaining portions of the dead and superimposed dead loads were then added to these top floors, and the model was again analyzed; this analysis continued from the stress and strain state at the end of the first stage. In the third construction stage, the live and antenna loads were added to the entire model, and the analysis continued from the end of the second stage. This analysis method produced reasonable stresses in the hat truss region of the undamaged towers.

D.3.2 Eigenvalue Buckling Analysis

The objective of this analysis was to assess the overall stability of one of the towers, namely WTC 1, under service loads, without any aircraft impact damage and subject to a progression of floor removal. Stability was measured through column buckling strength, which was reduced in each column as floors were removed in the model and column unbraced lengths were increased. Floor removal was modeled by removing the rigid diaphragm constraint at all columns, discussed in Section D.2.3, for that particular floor. Each node within that floor was then free to translate (e.g., buckle) in either lateral direction. The four columns above and below each removed floor were subdivided into sixteen segments per floor to achieve sufficient resolution for estimation of buckling loads. If instability was identified using a linear stability (eigenvalue buckling) analysis, the analysis was rerun after buckled columns were removed and their loads were redistributed to neighboring columns. This process continued until either the structure was stable or the progression of local instabilities indicated overall system instability.

The buckling analysis began with the "removal" of floor 96. The analysis calculated the load factor (eigenvalue), λ, for the first buckled column. If λ was greater than one, all columns were stable under the given loading condition, which signifies system stability. If λ was less than one, a column had buckled under the applied loads. This column was identified by visually examining the buckled mode shape of the structure at the end of the analysis. Only the first buckling mode was considered in the analysis.

Buckling of a single column might not result in a collapse of the tower due to the load-redistribution capability of the structure. To investigate overall stability, the buckled column was removed from the model above and below the removed floor(s). Any joint loads applied to a removed column were distributed to neighboring nodes. This eliminated any load carrying capacity of the failed column without eliminating its applied load, but rather redistributing it. The analysis was rerun, and the next buckled column was identified until λ was greater than one or until the progression indicated that a global instability had likely been attained. If the structure attained stability, floor "removal" progressed sequentially to floors 95, 97, 94, 98, etc.

The linear bucking analysis in SAP2000 only provided the load factor, λ_L, for the linear combination of DL, SDL, service LL, and antenna loads, but without staged construction. Since staged construction was employed to best represent the application of these loads, λ must be obtained from a relationship with λ_L. The buckling load in the linear case (staged construction not used) is equal to the axial force in the critical (i.e., buckled) column, F_L, times λ_L. Since the buckling load is the same in either the linear case or the staged construction case, the load factor is defined as:

$$\lambda = \frac{\lambda_L F_L}{F_{SC}} \tag{1}$$

where F_{SC} is the axial load in the same column from the staged construction analysis.

The procedure described above was also performed on the undamaged WTC 1 reduced global model with a reduced modulus of elasticity (E') applied to all core and exterior columns directly above and below removed floors. A value for E' equal to 21,460 ksi (E' = 0.74E), corresponding to a uniform column temperature of 600 ºC, was used in the analysis.

D.3.3 Redistribution of Forces within the Core Areas

Analyses of the global models of the towers indicated that when columns are severed in the exterior walls, the walls can redistribute their load through the vierendeel action of the wall above the severed columns. However, when columns are severed in the core, the possible load redistribution mechanisms include: (1) load redistribution to neighboring core columns through the nonlinear, large deflection, tensile membrane action of the floor, (2) load redistribution to the hat truss through tensile loads on the columns between the affected floors and the hat truss, or (3) a combination of both. The objective of this analysis is to determine the actual mechanism that occurs for a given damage pattern in the core columns of WTC 1.

The reduced global model of the tower lacks a complete floor system. As described in Section D.2.3, the floor systems were modeled as rigid or flexible diaphragms, which do not provide a path for vertical loads to be redistributed within the floors. Instead, when a core column is assumed to be damaged, all loads on that column from floors above the damage zone are redistributed through the hat truss in the model. This causes large tension forces in the damaged core columns.

A two-step approach was used to examine how the loads might redistribute. First, the typical floor model was analyzed with assumed damage to core columns. The severed columns were replaced by equivalent vertical springs, representing the combined stiffness of the hat truss and the axial stiffness of the columns between the floor and hat truss. In the analysis of the floor system, damage to the exterior walls of the tower was ignored, since it is assumed that the walls are capable of redistributing their loads. This analysis estimated what portion of the load would be redistributed as forces in the springs that will be transmitted to the hat truss, and what portion would be redistributed to neighboring columns through the floor system. Second, the tensile capacities of the core column splices between the affected floors and the hat trusses were estimated to determine if the columns could carry the calculated tensile loads.

To determine the equivalent stiffness of the hat truss, a separate model of the hat truss was first analyzed. For that purpose, a concentrated gravity load was applied at the node corresponding to the severed column, and the spring stiffness was estimated by dividing the applied load by the measured vertical displacement at that node. Then the axial stiffness of each of the columns above the damaged area was calculated. Finally, the model of the typical floor (Floor 96 of WTC 1) was modified to simulate the case of a floor above the damaged zone of the tower. The vertical support was removed from the base of the severed core columns, and spring restraints equivalent to the combined stiffness of the columns above and the hat truss were added to the tops of these columns. The floor model was then analyzed to determine how the loads would redistribute.

Two damage patterns in the core of WTC 1 were considered for this analysis: the first assumes destruction of fifteen columns (Core Damage Case 1), and the second assumes that only eight columns were severed (Core Damage Case 2). Table D–4 contains a list of core columns that were assumed to be destroyed for both damage cases (see Fig. D–2 for column locations).

Table D–4. Core columns removed from WTC 1, assumed destroyed by aircraft impact.

Core Column	Core Damage Case 1		Core Damage Case 2	
	Lowest Floor	Highest Floor	Lowest Floor	Highest Floor
503	96			
504	95	96	94	97
505	95	96	94	96
506	95			
603	96			
604	95	96	94	97
605	95	96	94	95
606	94			
703	96		94	95
704	96		94	97
705	96			
706	96		94	
803	96			
804	96			
805	96			
903			95	96

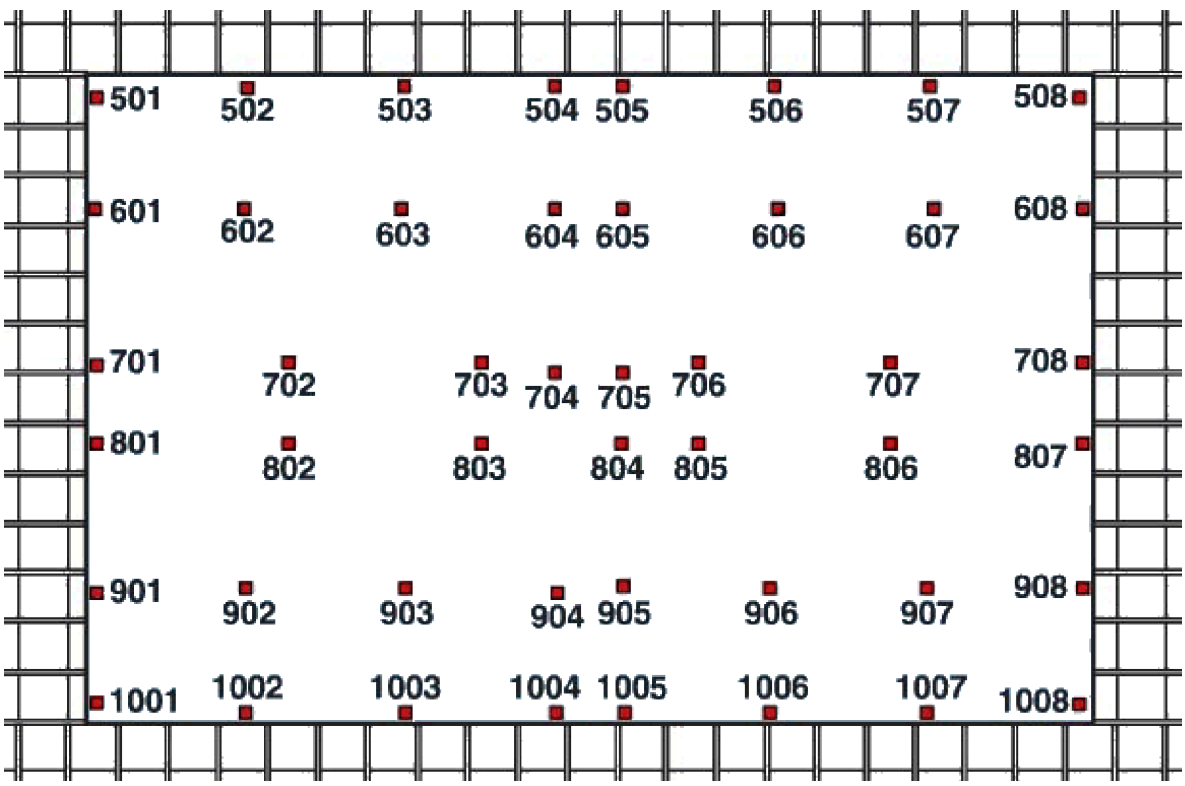


Figure D–2. Plan of typical core column layout (courtesy FEMA).

D.3.4 Nonlinear Analysis with Plastic Hinges

This analysis considered the nonlinear response of WTC 1 when an estimated pattern of damage had occurred. The damage scenario considered for this analysis included the following:

- Representative aircraft impact damage: based on photographic evidence, members in the north exterior wall of WTC 1 that were visibly severed or missing were assumed to be incapable of carrying load and were removed from the model, while members that appeared to be mostly intact were assumed to be capable of still carrying full load. This damage case also includes an exterior panel in the south face of the tower (columns 329 through 331 between floors 94 and 96) that was destroyed by the aircraft impact. In addition, eight columns in the core were assumed severed (see Core Damage Case 2 in Table D–4).

- Representative fire damage: 24 columns on the south face of WTC 1 between floors 96 and 98 were assumed to have buckled and lost all load carrying capacity. This assumption is based on video evidence that indicates that columns in this area were visibly deformed inward a few minutes before the tower collapsed.

The exterior members that were removed in this damage scenario are indicated in Fig. D–3.

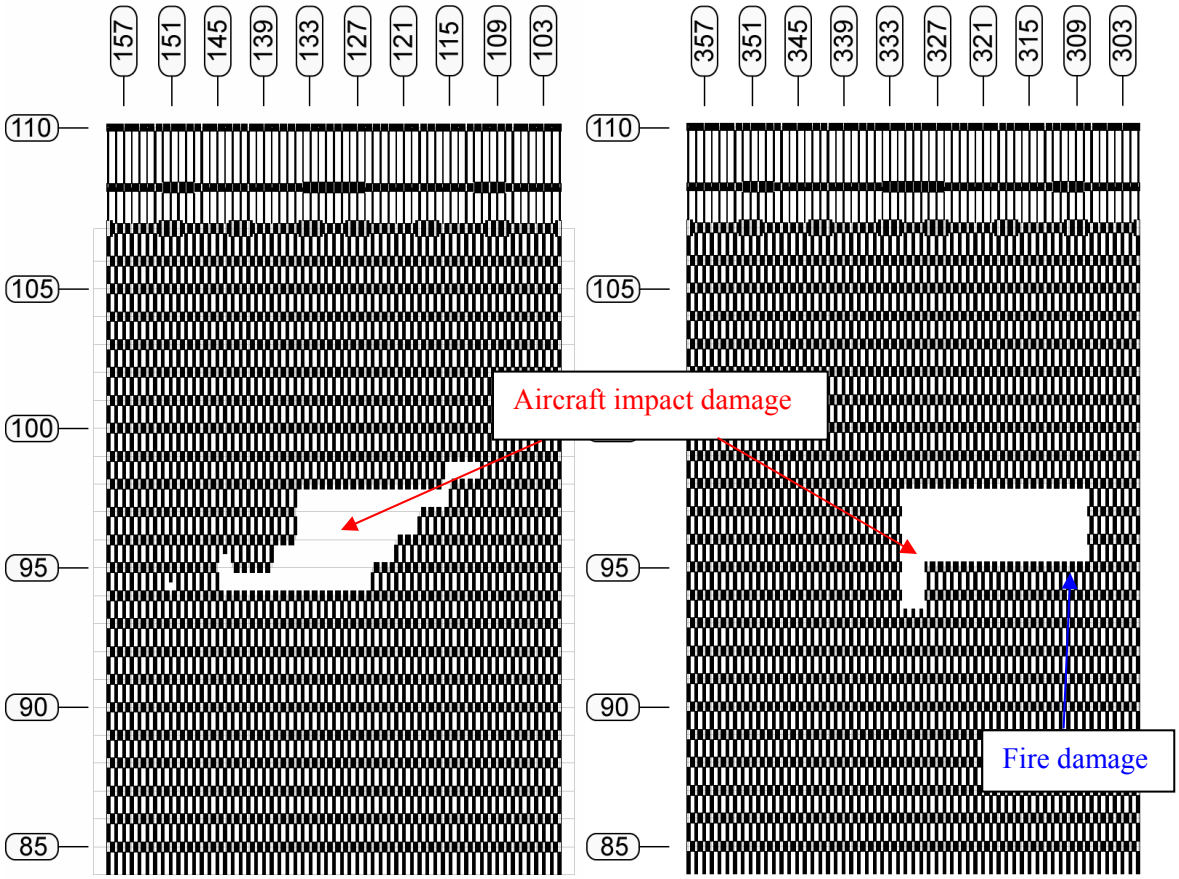


Figure D–3. North and south elevations of WTC 1 indicating columns and spandrels removed due to aircraft impact and fire effects.

To estimate how the damaged structure responded, the analysis considered geometric nonlinearities (large deflections and P-Δ effects) and material nonlinearities through a series of nonlinear, plastic hinges that were added to capture the post-yield behavior of structural members. The plastic hinges were placed in the reduced global model of WTC 1 based on a linear analysis of the damaged structures to determine the most stressed zones using a demand/capacity analysis. These hinges allow the members to act as nonlinear components, yielding once the stress on the member exceeds the material yield stress, continuing to accept some load at a reduced stiffness, and finally failing once an ultimate strain has been reached at an assumed ductility of 6. Hinges that considered both axial and bending forces (PMM hinges) were used in columns and hat truss members. Hinges that considered bending about the primary axis of the member, and shears in both the primary and secondary directions (MVV hinges) were used for most of the spandrels. Hinges that considered only bending about the primary axis (M3 hinges) were used for a small number of spandrels at the tower corners.

This analysis does not account for local bucking of columns; neither does it consider the failure or the role of the floor system in redistributing the loads. More detailed models, currently being developed within the framework of Projects 2 and 6, will account for these factors.

The damage due to aircraft impact analysis started from the end of the staged construction, described in Section D.3.1. At this stage, the set of damaged structural members that represent members destroyed by aircraft impact were removed from the reduced global model of WTC 1. This was followed by another stage, where the set of damaged structural members that represent members severely weakened by fire were removed from the model. For all analysis stages, room-temperature mechanical properties were used for all steels.

D.4 RESULTS

D.4.1 Results of Linear Stability Analysis

An initial analysis of the reduced undamaged model of WTC 1 under service loads with the 96th floor removed (i.e., the diaphragm constraint removed for all nodes at floor 96) produced a load factor for staged construction, λ, of 1.91. This indicated that no columns buckled under the application of DL, SDL, service LL, and antenna loads, and that the structure was stable. The structure was still stable with the additional removal of floor 95 ($\lambda = 1.03$). The analysis with floors 95, 96, and 97 removed yielded $\lambda = 0.65$ and the buckled column (core column 705, see Fig. D–2) was identified through visual observation of the first buckled mode shape.

Column 705 was removed from the model between floors 94 and 98, and the column's joint loads at floors 95, 96, and 97 were evenly distributed to joints of columns 704, 706, and 804. The analysis produced $\lambda = 0.78$ and indicated that column 704 had buckled. Column 704 was then removed from the model in a similar fashion to column 705. The combined joint loads of columns 704 and 705 were then distributed to neighboring joints at columns 703, 706, 803, 804, and 805. The load redistribution was proportional to the distance of each joint from the point halfway between joints 704 and 705. This analysis produced $\lambda = 1.38$, which indicated a stable tower with three floors removed.

The rigid diaphragm constraint at floor 94 was then removed from this latest model, i.e., floors 94 through 97 were unconstrained, columns 704 and 705 were omitted between floors 93 and 98, and the joint loads

at each removed floor from these two columns were redistributed as above. The eigenvalue buckling analysis produced a load factor of 0.92 for this model and indicated that column 601 buckled. Column 601 was removed between floors 93 and 98, and its joint load was distributed to columns 501, 502, 602, and 701. This analysis produced $\lambda = 0.97$ and indicated column 608, similarly located along the perimeter of the core columns like column 601, buckled. The model with column 608 removed and its load distributed to columns 508, 507, 607, and 708 yielded $\lambda = 1.25$. Thus, the tower was stable with four floors removed and with the redistribution of loads from removed columns 705, 704, 601, and 608. Subsequent models with floor 98 removed suggested that the structure did not attain λ greater than one, and that the structure had likely reached a failed state.

When the modulus of elasticity was reduced from E = 29,000 ksi to E' = 21,460 ksi in columns above and below removed floors, the conclusions changed slightly but the progression of column failures remained the same. The tower maintained its overall stability with floor 96 removed ($\lambda = 1.38$). With floors 96 and 95 removed, the model of the intact structure indicated that column 705 buckled ($\lambda = 0.83$), but that stability was achieved through the removal of column 705 ($\lambda = 1.02$). With the removal of a third floor (97th), column 704 was also removed and its load redistributed in the model to maintain overall stability ($\lambda = 1.11$). The additional removal of a fourth floor (94th) produced a series of buckled columns (601, 608, 904, and 604) that indicated the structure would likely not achieve overall stability.

To summarize, the eigenvalue analysis examined the stability of the undamaged tower under service loads through increased unbraced column lengths in the absence of material nonlinearities. For the case with columns at room temperature, the tower was stable when two floors were removed. Two core columns buckled when three floors were removed, but the tower maintained its overall stability. Similarly, the tower maintained its stability when four columns buckled with four floors removed. This analysis suggested that global instability of the tower occurred when five floors were removed from the model. The case with columns at the region of removed floors at temperature of 600 ºC showed the tower maintained overall stability with one floor removed, with two floors removed and one buckled column, and with three floors removed and two buckled columns. This case produced tower instability with four floors removed from the model.

D.4.2 Results of Redistribution of Forces within the Core Analysis

The typical floor model was analyzed with 15 severed core columns (Core Damage Case 1) replaced with springs representative of the combined stiffness of the columns and hat truss. The analysis indicated that most of the load redistribution would take place initially through the hat truss, with the columns above the damaged zone carrying large tensile forces. A small portion of the load would be redistributed within the floor system. This is due to the greater stiffness of the hat truss-column assembly relative to the flexural stiffness of the floor system with fifteen severed columns.

When eight columns were assumed severed in the core (Core Damage Case 2), the floor system had a larger stiffness than that with fifteen columns severed (Core Damage Case 1). Consequently, the contribution of the floor in redistributing the gravity loads was larger, and the tensile forces in the columns above the damaged zone were reduced relative to the tensile forces for the case of fifteen severed core columns.

Above the impact levels, the core columns were typically two or three stories high, wide flange segments that were connected together with bolted splices. In the upper floors of WTC 1, where the column tensile capacities were analyzed, the columns were spliced at floors 98, 101, 104, and 106. Only the splices for columns that were assumed to be damaged were analyzed. The splice connections on these columns typically consisted of two splice plates, one bolted to each flange of the column, connected to the columns by eight or twelve, 3/4 in. (19 mm) A325 bolts. At floor 106, where the columns connected to the hat truss, 7/8 in. (22.2 mm) bolts were used. The splice plates were made with A36 steel, which was assumed to have an ultimate tensile capacity of 61 ksi (422 MPa) (see Table D–1). Since the connections were bearing connections with the bolts in single shear, the ultimate shear capacity of each 3/4 in. (19 mm) bolt was estimated to be 31.8 kip (219 MPa). When the strengths of both the splice plates and bolts were estimated, the splice plates were found to be consistently stronger than the bolts, so the columns would fail in tension through shearing of the bolts. Table D–5 lists the ultimate capacities of the splices on each of the columns assumed to be damaged in this analysis.

Table D–5. Ultimate tensile capacities of core column splices.

Column Number	Floor 98		Floor 101		Floor 104		Floor 106	
	kip	(kN)	kip	(kN)	kip	(kN)	kip	(kN)
503	519.6	(2,311)	381.6	(1,697)	381.6	(1,697)	346.4	(1,541)
504	381.6	(1,697)	381.6	(1,697)	254.5	(1,132)	346.4	(1,541)
505	381.6	(1,697)	381.6	(1,697)	254.5	(1,132)	346.4	(1,541)
506	519.6	(2,311)	381.6	(1,697)	381.6	(1,697)	346.4	(1,541)
603	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
604	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
605	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
606	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
703	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
704	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
705	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
706	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
803	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
804	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
805	254.5	(1,132)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)
903	381.6	(1,697)	254.5	(1,132)	254.5	(1,132)	346.4	(1,541)

A comparison of the columns' tensile forces with the capacities of column splice connections is presented in Table D–6 for Core Damage Case 1. The table indicates that, for the assumed service loads and damage pattern, the splice connections for interior core columns at the 700 and 800 lines are capable of resisting the tensile forces imposed on them. Splice connections on the 600 line are at the critical stage, but splices at the north perimeter core columns (500 line) are likely to fail. Failure of the column connections at the 500 line will require the floor to redistribute its loads to neighboring intact columns. This will result in overloading the columns at the 600 line, and consequently, the floor has to redistribute its loads through nonlinear, large deflection, tensile membrane action.

Table D–6. Tensile loads on columns above damaged area, and column load to ultimate capacity ratios for Core Damage Case 1.

Column Number	Floor 98		Floor 101		Floor 104		Floor 106	
	Column Load kip (kN)	Load to Capacity Ratio	Column Load kip (kN)	Load to Capacity Ratio	Column Load kip (kN)	Load to Capacity Ratio	Column Load kip (kN)	Load to Capacity Ratio
503	109.3 (486)	0.21	273.2 (1215)	0.72	437.1 (1944)	***1.15***	546.3 (2430)	***1.58***
504	82.8 (368)	0.22	207.0 (921)	0.54	331.2 (1473)	***1.30***	414.0 (1841)	***1.20***
505	92.7 (412)	0.24	231.8 (1031)	0.61	370.9 (1650)	***1.46***	463.6 (2062)	***1.34***
506	214.7 (955)	0.41	375.7 (1671)	0.98	536.7 (2387)	***1.41***	644.0 (2865)	***1.86***
603	64.6 (287)	0.25	161.5 (718)	0.63	258.4 (1149)	***1.02***	323.0 (1437)	0.93
604	57.5 (256)	0.23	143.8 (640)	0.57	230.1 (1024)	0.90	287.7 (1280)	0.83
605	72.3 (321)	0.28	180.7 (804)	0.71	289.1 (1286)	***1.14***	361.3 (1607)	***1.04***
606	138.7 (617)	0.55	242.8 (1080)	0.95	346.9 (1543)	***1.36***	416.2 (1851)	***1.20***
703	39.1 (174)	0.15	97.8 (435)	0.38	156.5 (696)	0.61	195.6 (870)	0.56
704	20.1 (90)	0.08	50.4 (224)	0.20	80.6 (358)	0.32	100.7 (448)	0.29
705	27.3 (121)	0.11	68.2 (303)	0.27	109.1 (485)	0.43	136.3 (606)	0.39
706	27.2 (121)	0.11	68.0 (303)	0.27	108.9 (484)	0.43	136.1 (605)	0.39
803	24.8 (110)	0.10	62.0 (276)	0.24	99.2 (441)	0.39	124.0 (552)	0.36
804	36.2 (161)	0.14	90.4 (402)	0.36	144.7 (644)	0.57	180.9 (805)	0.52
805	18.9 (84)	0.07	47.1 (210)	0.19	75.4 (335)	0.30	94.3 (419)	0.27

For the case where eight columns were assumed severed in the core (Core Damage Case 2), the results are presented in Table D–7. The Table indicates that the columns splice connections are capable of resisting the tensile loads except for column 505, where the load to capacity ratio is approximately 1.25 at floor 104 and 1.15 at floor 106. Successive removal of columns that were assumed to lose their splice connections indicated that the failure of connections would propagate to the 500 and 600 column lines in the core. Splice connections at columns 704, 705, and 903 should remain intact. Table D–7 indicates, however, that the load to capacity ratio at the splices did not exceed 1.25 for all cases considered. These values might not be conclusive to determine connection failure or survival due to the uncertainties in the loads on the floors and the capacities of the splice connections.

D.4.3 Results of Nonlinear Analysis

Two cases were considered for this analysis based on the results presented in Section D.4.2. The first assumed that core column splices above the eight severed columns (Core Damage Case 2) had failed, and the load was being distributed through the floor system to neighboring columns (Case A). The second case assumed that splices were intact, and the load was being transmitted to the hat truss via tensile forces in the columns (Case B). For both cases, the results of the nonlinear analysis show that WTC 1 had significant reserve structural capacity after aircraft impact. Moreover, the loads and deformations in critical members varied little between the two cases. The results are described in detail for Case A only.

Table D–7. Tensile loads on columns above damaged area, and L/C ratios indicating likely progression of splice failures for Core Damage Case 2.

Column Number	Floor 98		Floor 101		Floor 104		Floor 106	
	Column Load kip (kN)	L/C Ratio	Column Load kip (kN)	L/C Ratio	Column Load kip (kN)	L/C Ratio	Column Load kip (kN)	L/C Ratio
Loads with all column splices intact								
504	34.7 (154)	0.09	138.9 (618)	0.36	243.1 (1081)	0.96	312.5 (1390)	0.90
505	79.5 (354)	0.21	198.8 (884)	0.52	***318.1 (1415)***	***1.25***	***397.6 (1769)***	***1.15***
604	21.3 (95)	0.08	85.2 (379)	0.33	149.1 (663)	0.59	191.7 (853)	0.55
605	77.0 (343)	0.30	154.0 (685)	0.61	231.0 (1028)	0.91	282.4 (1256)	0.82
703	45.0 (200)	0.18	90.0 (400)	0.35	134.9 (600)	0.53	164.9 (734)	0.48
704	3.6 (16)	0.01	14.3 (64)	0.06	25.1 (112)	0.10	32.2 (143)	0.09
706	21.0 (93)	0.08	36.7 (163)	0.14	52.4 (233)	0.21	62.9 (280)	0.18
903	36.2 (161)	0.09	90.6 (403)	0.36	145.0 (645)	0.57	181.2 (806)	0.52
Loads with splices at columns 504 and 505 failed								
504	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00
505	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00
604	29.0 (129)	0.11	115.9 (515)	0.46	202.8 (902)	0.80	260.7 (1160)	0.75
605	103.6 (461)	0.41	207.3 (922)	0.81	***310.9 (1383)***	***1.22***	***380.0 (1690)***	***1.10***
703	43.1 (192)	0.17	86.2 (383)	0.34	129.3 (575)	0.51	158.0 (703)	0.46
704	3.0 (13)	0.01	12.0 (53)	0.05	20.9 (93)	0.08	26.9 (120)	0.08
706	20.1 (89)	0.08	35.1 (156)	0.14	50.2 (223)	0.20	60.2 (268)	0.17
903	36.2 (161)	0.09	90.5 (403)	0.36	144.8 (644)	0.57	181.0 (805)	0.52
Loads with splices at columns 504, 505, and 605 failed								
504	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00
505	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00
604	42.8 (191)	0.17	171.4 (762)	0.67	***299.9 (1334)***	***1.18***	***385.6 (1715)***	***1.11***
605	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00
703	45.8 (204)	0.18	91.5 (407)	0.36	137.3 (611)	0.54	167.8 (746)	0.48
704	3.5 (16)	0.01	14.0 (62)	0.05	24.5 (109)	0.10	31.5 (140)	0.09
706	22.6 (101)	0.09	39.6 (176)	0.16	56.5 (251)	0.22	67.8 (302)	0.20
903	36.2 (161)	0.09	90.5 (403)	0.36	144.8 (644)	0.57	181.0 (805)	0.52
Loads with splices at columns 504, 505, 604, and 605 failed								
504	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00
505	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00
604	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00
605	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00	0.0 (0)	0.00
703	57.6 (256)	0.23	115.2 (513)	0.45	172.9 (769)	0.68	211.3 (940)	0.61
704	5.1 (23)	0.02	20.6 (91)	0.08	36.0 (160)	0.14	46.3 (206)	0.13
706	25.5 (113)	0.10	44.6 (198)	0.18	63.7 (284)	0.25	76.5 (340)	0.22
903	36.1 (161)	0.09	90.3 (402)	0.35	144.4 (642)	0.57	180.5 (803)	0.52

The analyses show that the most stressed members were the columns next to the damaged area on the north wall of the tower. The analyses show that the tower also remained standing after losing columns in the south wall due to fire effects with some reserve capacity left. This indicates that additional loss or weakening of columns in the core, weakening of additional columns in the exterior, or additional loss of floors is needed to collapse the tower.

Figures D–4 and D–5 show the load deformation curves for columns 111 and 145 (see Fig. D–3) on either side of the damage on the north face of the tower. Both of these columns yielded during the impact damage analysis, but had sufficient strength and ductility to resist the peak loads. Note that the loads on these two columns were slightly reduced during the fire damage analysis. This occurred because the south face of the tower lost stiffness when members were lost to the fire, which caused the upper portion of the tower to rotate slightly toward the south. This redistributed a small portion of the load on the north face through the hat trusses to the core. The columns at the edge of the damage on the south face experienced the opposite effect. As can be seen in Fig. D–6, column 332 on the west side of the damage on the south face (see Fig. D–3) did not have a significant change in its load during the impact damage stage, but received a large additional load during the fire damage analysis. This column remained within the linear response range throughout the analysis.

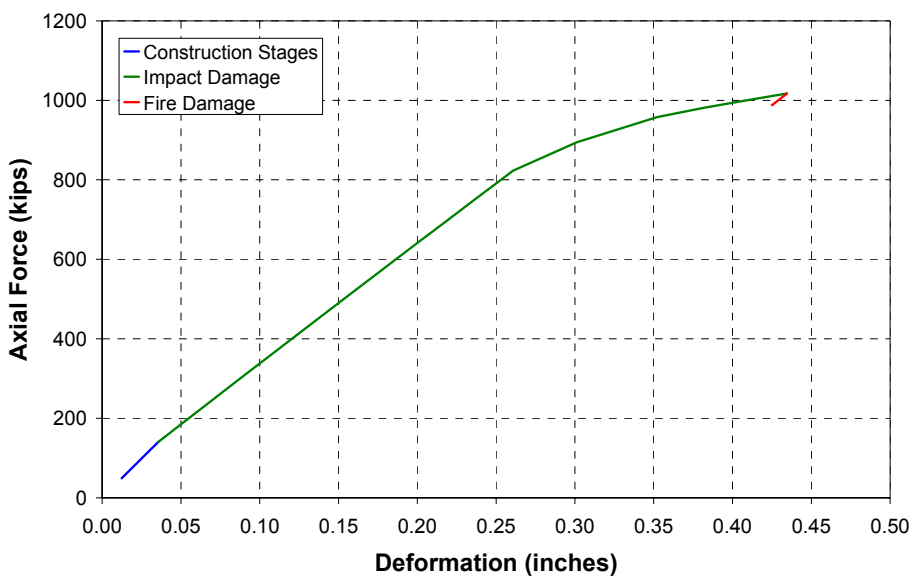


Figure D–4. Load vs. deformation in column 111 at floor 98 (north face, west side of damage).

Table D–8 shows the distribution of axial loads in columns at floor 99, immediately above the damaged zone for the various loading stages. For the floors below the hat trusses (construction stage 1) about 57 percent of the dead load was carried by the core columns, with the rest distributed among the four exterior walls. The mechanical floors and roof tended to have a large percentage of their loads carried by the exterior, so at the end of the final construction stage, the load was nearly evenly distributed between the core and exterior. Significant load redistribution occurred during the damage cases; however most of the redistribution was from the north and south walls to the east and west walls. Only 1.7 percent of the total load was redistributed from the exterior to the core.

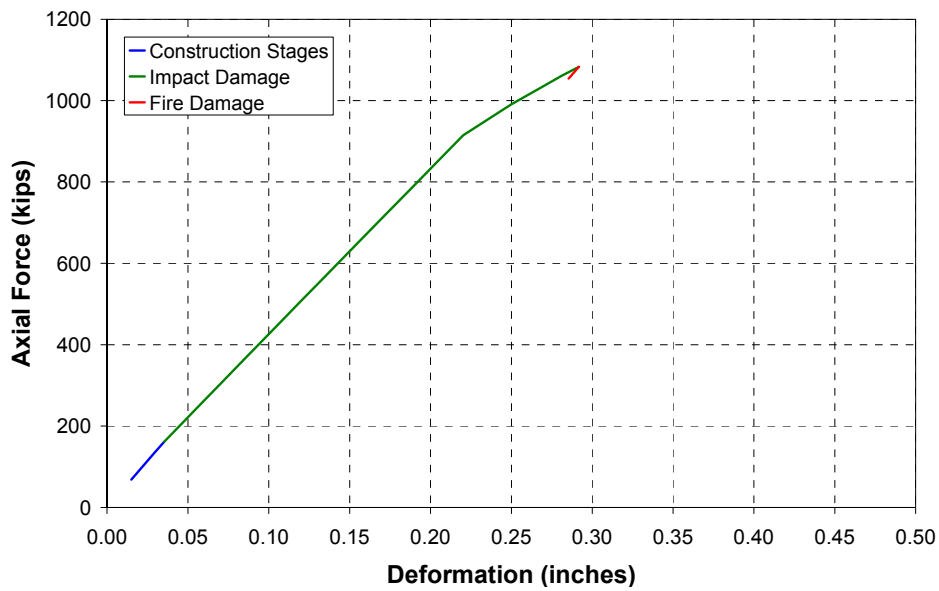


Figure D–5. Load vs. deformation in column 145 at floor 95 (north face, east side of damage).

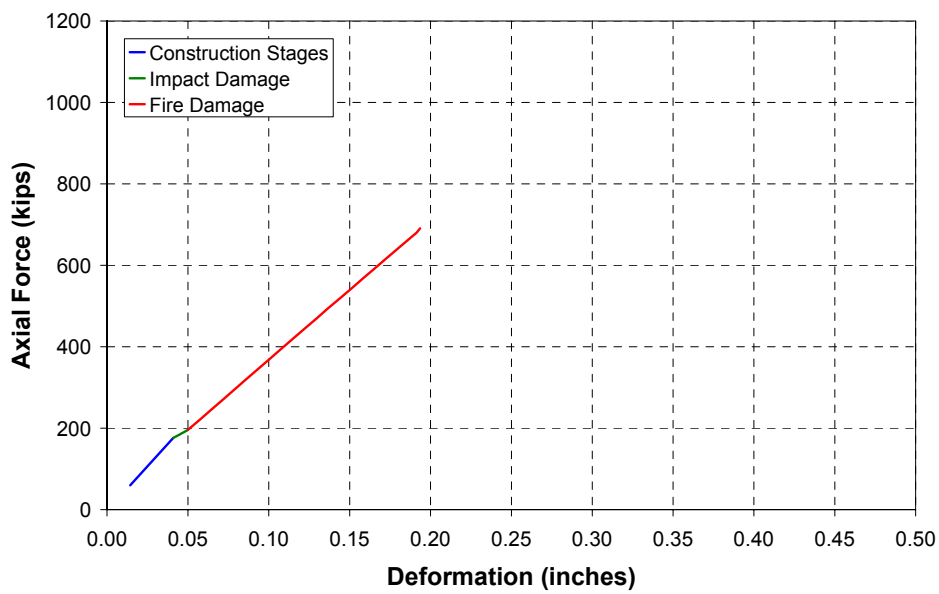


Figure D–6. Load vs. deformation in column 332 at floor 97 (south face, west side of damage).

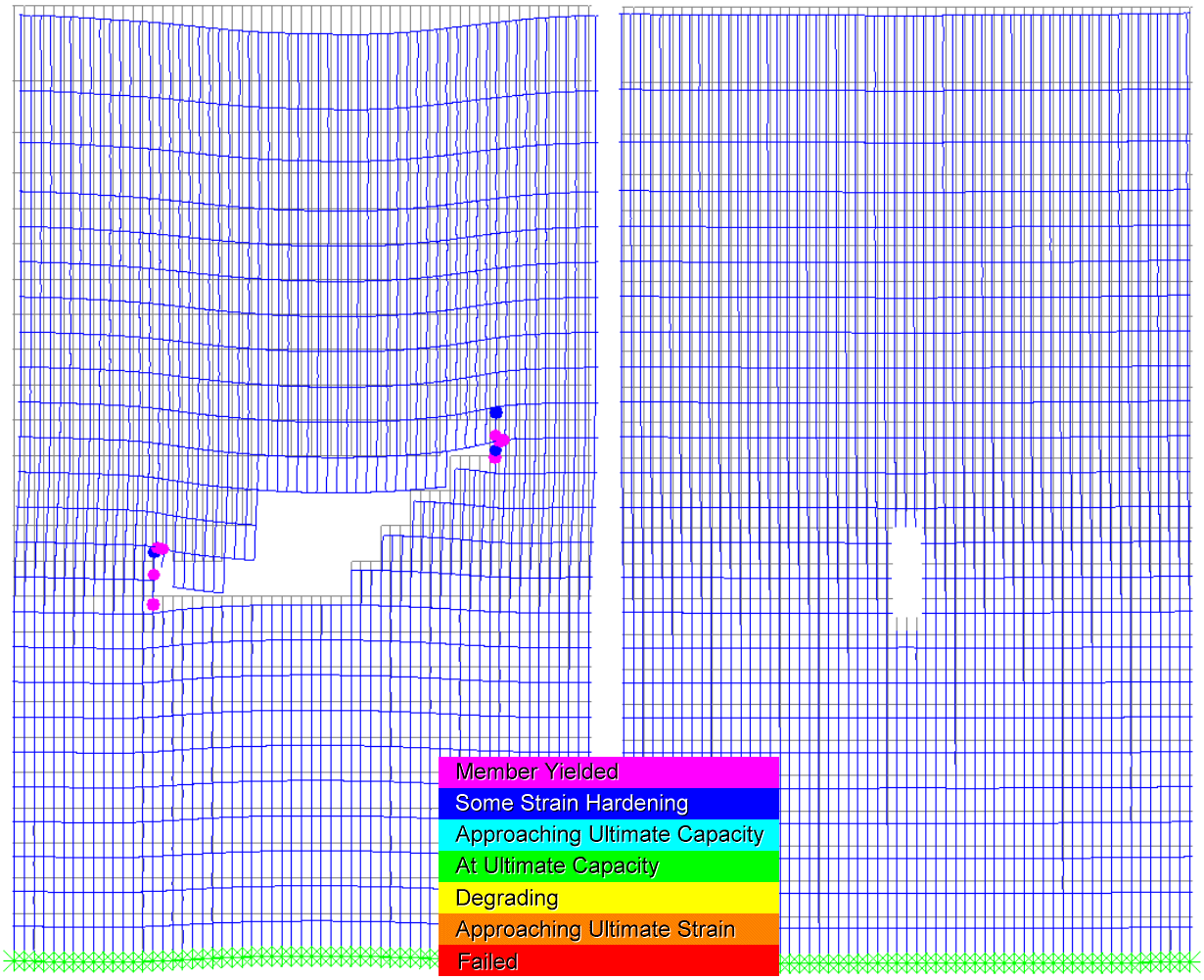
Table D–8. Distribution of loads on exterior walls and core columns at floor 99 for Core Damage Case 2.

Case	Const. Stage 1		Const. Stage 2		Const. Stage 3		Impact Damage		Fire Damage	
	(kip)	(%)	(kip)	(%)	(kip)	(%)	(kip)	(%)	(kip)	(%)
Total axial force	27,719		45,694		54,639		54,641		54,641	
Force Distribution between Core and Exterior										
Core columns	15,828	57.1	24,466	53.5	27,397	50.1	27,791	50.9	28,318	51.8
Exterior columns	11,891	42.9	21,228	46.5	27,242	49.9	26,850	49.1	26,323	48.2
Force Distribution between Exterior Faces										
100 face	3,562	12.9	6,104	13.4	7,610	13.9	6,732	12.3	6,519	11.9
200 face	2,389	8.6	4,551	10.0	6,084	11.1	6,539	12.0	6,765	12.4
300 face	3,562	12.9	6,064	13.3	7,548	13.8	7,000	12.8	6,445	11.8
400 face	2,378	8.6	4,509	9.9	6,000	11.0	6,579	12.0	6,594	12.1

Even with the loss of 34 columns on the north face and 24 columns on the south face, relatively few structural members were overstressed. As listed in Table D–9, plastic hinges had formed in only nine members, and only three members had more than a small amount of plastic deformation. Of the members with plastic hinges, six were exterior columns, and three were exterior spandrels. None of the core columns had hinges form. Three beams in the hat truss at floor 107 experienced some yielding. These members were all light, short connecting members, and the yielding was most likely due to modeling idealization rather than overloading that would have occurred in the real structure. The hinges in the columns and spandrels all formed near the sides of the openings created by the aircraft impact and at the edges of the region that appeared to fail inward due to the fires. Figure D–7 shows the displacements and plastic hinges that occurred in the north and south faces of the tower during the impact damage analysis stage, while Fig. D–8 shows the displacements and plastic hinges from the fire damage analysis stage.

Table D–9. Hinge states of members where plastic hinges formed during nonlinear analysis.

Columns with Plastic Hinges				
Column Line	**Lower Floor**	**Upper Floor**	**Impact Damage**	**Fire Damage**
111	98	99	Some strain hardening	Some strain hardening
111	99	99 mid floor	Yielded	Yielded
111	99 mid floor	100	Some strain hardening	Some strain hardening
145	94	95	Yielded	Yielded
145	95	95 mid floor	Yielded	Yielded
145	95 mid floor	96	Some strain hardening	Some strain hardening
Spandrels with Plastic Hinges				
Floor Level	**Start Column**	**End Column**	**Impact Damage**	**Fire Damage**
99	110	111	Yielded	Yielded
96	144	145	Yielded	Yielded
98	331	332	-	Yielded



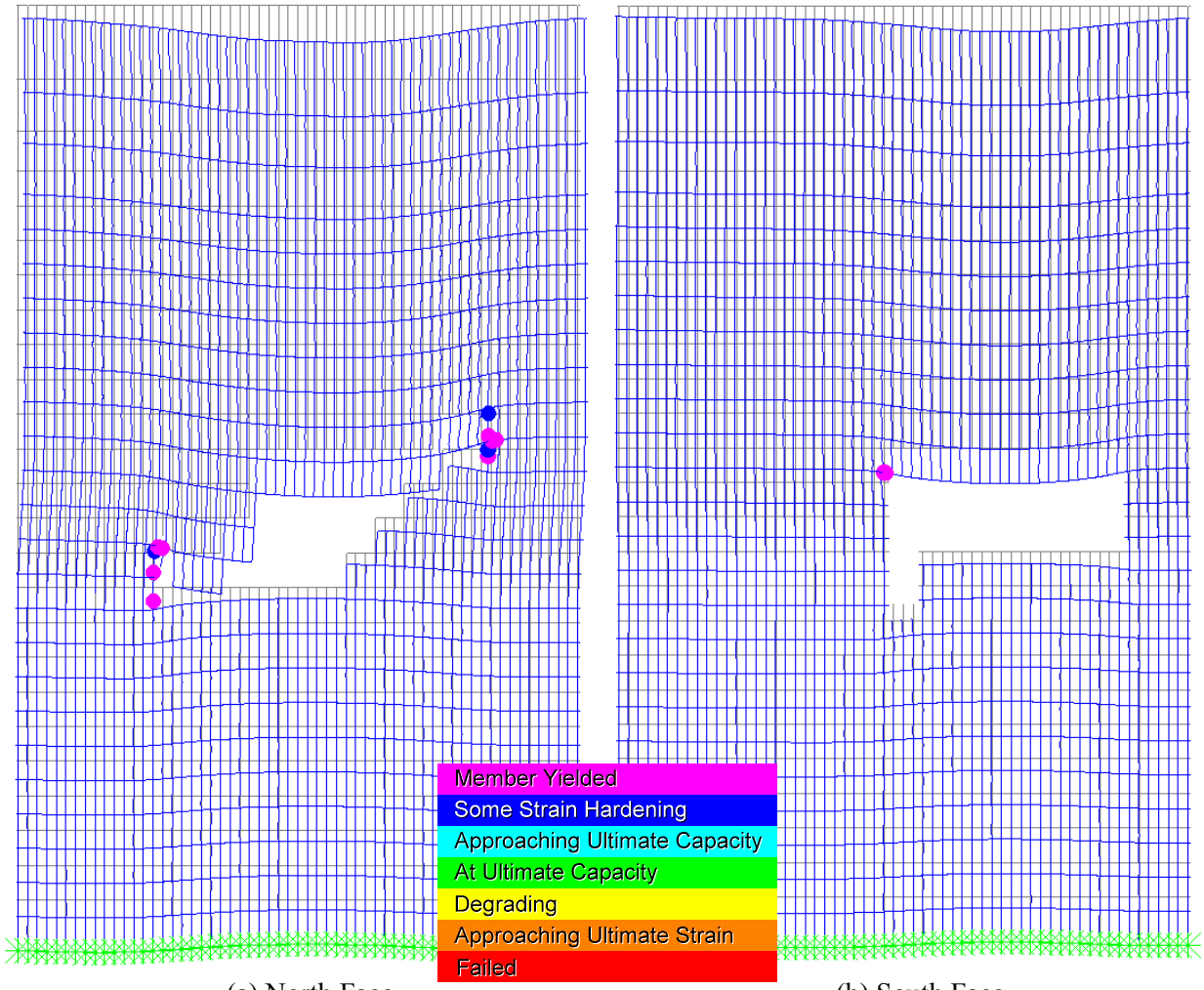
(a) North Face (b) South Face

Figure D–7. Displacements and locations of plastic hinges in the north and south exterior walls of WTC 1 after impact.

D.5 SUMMARY AND PRELIMINARY FINDINGS

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

The analyses used the typical truss-framed floor model and a reduced version of the global reference model of WTC 1 with proper modifications. Modifications included adding vertical springs at the bottom of the reduced models to account for the removed lower portion of the towers, and using actual (vs specified) steel properties and service loads on the towers. The analyses used the staged construction technique to account for the sequential construction of the towers, especially in the zone of the hat trusses.



(a) North Face (b) South Face

Figure D–8. Displacements and locations of plastic hinges in the north and south exterior walls of WTC 1 after impact and fire.

Linear buckling analysis and nonlinear analysis with plastic hinges were used to study the effects of removal of floors and loss of exterior and core columns, respectively. In addition, analysis of the floor system, where severed core columns were replaced by equivalent springs representative of the combined stiffness of the hat trusses and columns between the floors and hat trusses, was conducted to study the mechanism by which the floor loads were redistributed when the core columns were destroyed by aircraft impact.

The following presents some preliminary findings based on the analyses under service loading conditions:

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

region of removed floors reached a temperature of 600 ºC (reduced modulus of elasticity), the analysis indicated that removal of four floors would induce global instability.

- Analysis of the typical truss-framed floor model with fifteen core columns assumed severed indicated that, under service loads, the floors first attempted to redistribute their loads to the hat trusses through tension in the columns above the damage. The load followed this path due to the relatively large stiffness of the hat trusses-column system compared to the flexural stiffness of the floors. This resulted, however, in the ultimate tensile capacity of some column splices below the hat trusses to be exceeded, and ultimately, the floors would have redistributed their loads directly to neighboring core columns. When only eight core columns were assumed severed, the analysis indicated that the tensile forces in the columns were smaller, due to the relatively larger stiffness of the floor. These forces may still have failed the columns at the splices. Since the load to capacity ratio at the splices did not exceed 1.25 when eight columns were severed, and due to the uncertainties in the loads on the floors and the capacities of the splice connections, the results are not conclusive as to whether splice failure would occur or not.

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

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