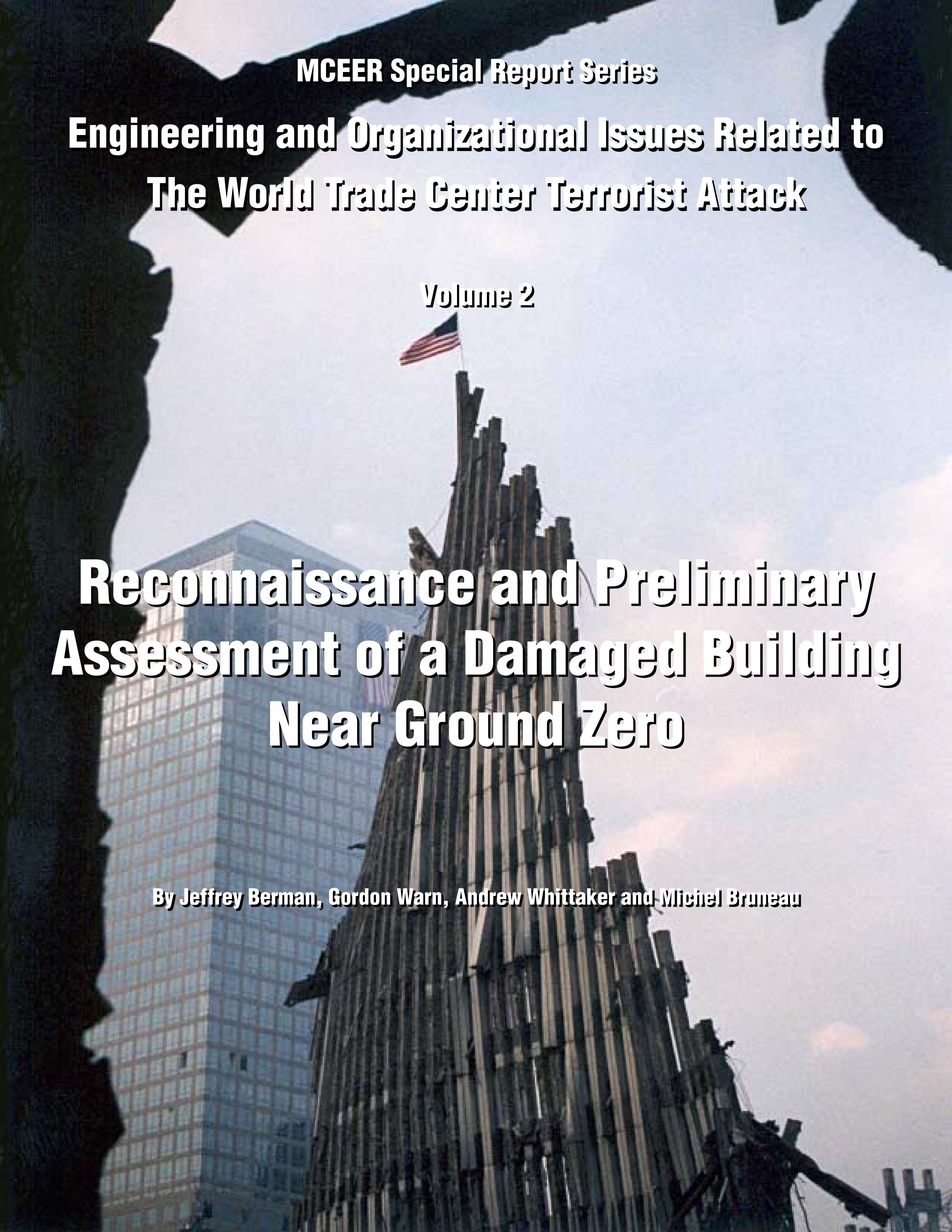
MCEER Special Report Series

Engineering and Organizational Issues Related to The World Trade Center Terrorist Attack

Volume 2



Reconnaissance and Preliminary Assessment of a Damaged Building Near Ground Zero

By Jeffrey Berman, Gordon Warn, Andrew Whittaker and Michel Bruneau

MULTIDISCIPLINARY CENTER FOR EARTHQUAKE ENGINEERING RESEARCH

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Foreword

The terrorist attack that took place on September 11, 2001 in New York City resulted in thousands of lives lost, the collapse of the twin towers of the World Trade Center as well as damage to adjacent buildings, and extensive disruption of transportation and other lifeline systems, economic activity, and other social activities within the city and the surrounding area. When the final accounting takes place, this attack will almost certainly constitute one of the most deadly and costly disasters in U. S. history.

In a very real sense, the September 11 tragedy, the nature of the damage that occurred, the challenges that the city's emergency response community faced, and the actions that were undertaken to meet those demands can be seen as a "proxy" - albeit a geographically concentrated one - for what a major earthquake can do in a complex, densely-populated modern urban environment. Like an earthquake, the terrorist attack occurred with virtually no warning. As would be expected in an earthquake, fires broke out and multiple structural collapses occurred. As has been observed in major urban earthquakes and in other disasters (e.g., Hurricane Andrew), structures housing facilities that perform critical emergency functions were destroyed, heavily damaged, or evacuated for life-safety reasons. Additionally, because the majority of the damage occurred to relatively new and well-engineered structures and because the emergency response system in New York City was considered very well prepared for all types of emergencies, particularly terrorist attacks, the attack and its aftermath provide a useful laboratory for exploring a variety of engineering and emergency management issues.

In this perspective, the Multidisciplinary Center for Earthquake Engineering Research initiated a research project (funded by the National Science Foundation) to collect perishable data in the aftermath of the attack for later study to gain a better understanding of how resilience is achieved in both physical, engineered systems and in organizational systems. The project is divided into two major components, focusing on the impact of the disaster on engineering and organizational systems:

(a) Damage to Buildings in the Vicinity of Ground Zero - The objective of this effort is to collect perishable information on the various types of damage suffered by buildings at Ground Zero, including, most importantly, those that suffered moderate damage from the impact of large debris but that did not collapse, and to investigate whether state-of-practice analytical methods used in earthquake engineering can be used to explain the observed structural behavior.

(b) Organizational and Community Resilience in the World Trade Center Disaster - The objective of this effort is to collect information on the response activities of the City's Emergency Operations Center and on other critical emergency response facilities. Of particular interest is to identify the plans that were in place at the time of the disaster, as well as how decision systems were used and coordinated with engineering decisions. Efforts will also include identifying the technologies and tools that were most useful or failed (or did not meet expectations) during the emergency period, the types of adaptations that had to be made by these organizations, how well intra-organizational communication and coordination functioned, and whether any emerging technologies were used during the emergency period.

The MCEER special report series "Engineering and Organizational Issues Related to The World Trade Center Terrorist Attack" was initiated to present the findings from this work. The decision to publish a number of brief individual reports focusing on different topics was prompted by the desire to provide timely access to this information. As such, each report in the series focuses on a narrow aspect of the disaster as studied by MCEER researchers. A compendium of these short reports is planned at a later time. It is hoped that this work will provide a useful contribution that can lead to a better understanding of how to cost-effectively enhance the resilience of buildings against catastrophic events.

Abstract

An MCEER research team, sponsored by the National Science Foundation, visited Ground Zero twice in the two weeks following the attacks of September 11, 2001, to collect perishable data related to the collapse of the two 110-story towers and collateral damage to buildings and infrastructure surrounding the World Trade Center complex. The visit on September 23 involved a walk-through of one high-rise building that was badly damaged by large pieces of debris that were ejected from World Trade Center Tower 2 as it collapsed. This summary report presents information from the building interior reconnaissance on September 23 and the subsequent analysis of a building frame with properties similar to those of the damaged building. Linear and nonlinear analyses were undertaken. Such analyses have shown that the use of rigid beam-to-column connections in the building frame enabled gravity loads in the frame above the segment of the building that partially collapsed to be transferred to adjacent undamaged vertical components.

Acknowledgements

The authors wish to acknowledge the substantial technical contributions of the expert structural engineering staff at LZA Technology of New York, a division of the Thornton/Tomasetti Group. Special thanks are due to Mr. Edward Sweirtz (Associate Partner, Chicago), Mr. Daniel Cuoco (Managing Principal, New York), and Mr. Emmanuel Velivasakis (Managing Principal, New York).

This work was supported by the Earthquake Engineering Research Centers Program of the National Science Foundation under a supplement to Award Number EEC-9701471 to the Multidisciplinary Center for Earthquake Engineering Research. However, any opinions, conclusions, and/or recommendations presented in this paper are those of the authors and do not necessarily reflect the views of the sponsors.

Contents

1.0 Introduction

Shortly after the attack on the World Trade Center Towers, MCEER dispatched a research team to New York City. The team's mission was to collect perishable data related to the collapse of the two 110-story towers and collateral damage to buildings and infrastructure surrounding the complex. Two visits to Ground Zero were undertaken, the first on September 21 and the second on September 23. The visit on September 23 involved a walk-through of one high-rise building that was badly damaged by large pieces of debris that were ejected from World Trade Center Tower 2 (WTC 2) as it collapsed.

The objectives of the work presented in this summary report were two-fold: (1) to collect information about the structural and nonstructural damage suffered by the building at 130 Liberty Plaza due to the collapse of Tower 2 in the World Trade Center (WTC) complex, and (2) to investigate whether analytical methods used in earthquake engineering can be used to explain the observed structural behavior. The first objective was achieved by the September 23 walk-through of the building. Summary information on the damage suffered by the building is presented in Section 2. The second objective was addressed using linear and nonlinear analysis tools that are used by earthquake engineers. Results of this work are presented in Section 3.

The Banker's Trust Building is a 39-story office building located at 130 Liberty Plaza in lower Manhattan. The building was located to the immediate south of the WTC 2. Figure 1.1 shows the location of the building with respect to the WTC complex.

The 130 Liberty Plaza building was designed in the early 1970s by the structural engineering firm of James Ruderman LLP. Structural drawings were not available to the reconnaissance team but information on the framing system was gathered during the visit and is described in the following section. The typical gravity framing system is composed of a non-composite unreinforced concrete slab on metal decking spanning approximately 8 feet, 8 inches to steel beams that span 26 feet to steel girders that frame into steel wide-flange columns. The concrete slab is 2-1/2 inches thick atop a 20 gauge metal deck. The lateral framing system consists of a three-dimensional steel moment-resisting space frame (that is, all beam-to-column connections are rigid connections) and a steel braced core. Figure 1.2 shows a partial plan of a typical floor. The member sizes indicated on the figure were established by members of the reconnaissance team following independent analysis of the frame as described in Section 3. The grid marks (A through H and 5 through 8) were selected by the reconnaissance team to aid in

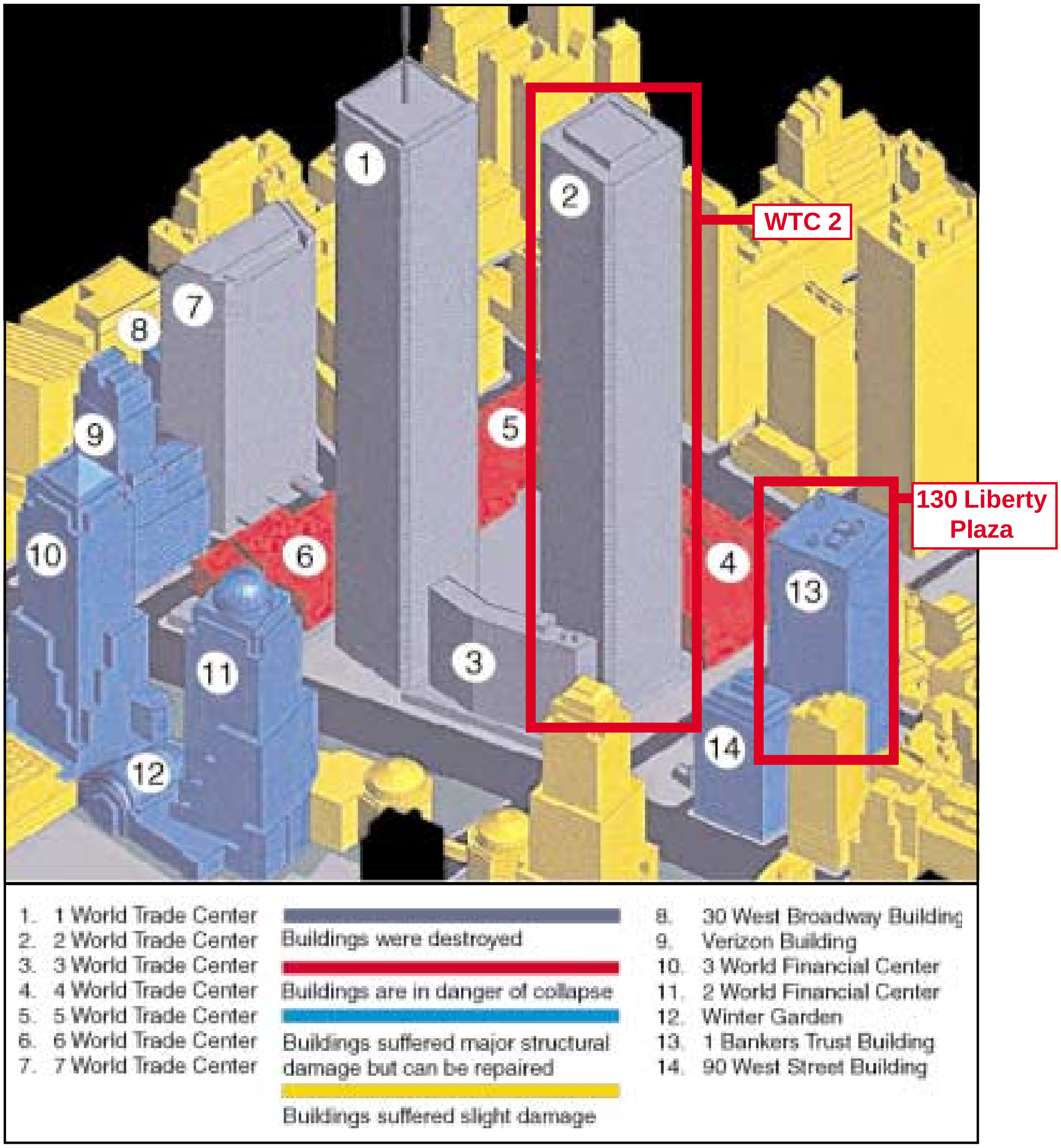


Figure 1.1. Location of subject building with respect to the World Trade Center complex

the identification and interpretation of damage. The next section refers extensively to this figure in describing the damage within the building.

The façade of the building is composed of windows and a lightweight cladding system. An artist's rendering of the northern façade is presented in Figure 1.3. This façade was badly damaged by falling debris in the zone described by the elliptical line in the figure. Tenant Levels are identified in the figure to facilitate the description of damage to the building presented in Section 2.

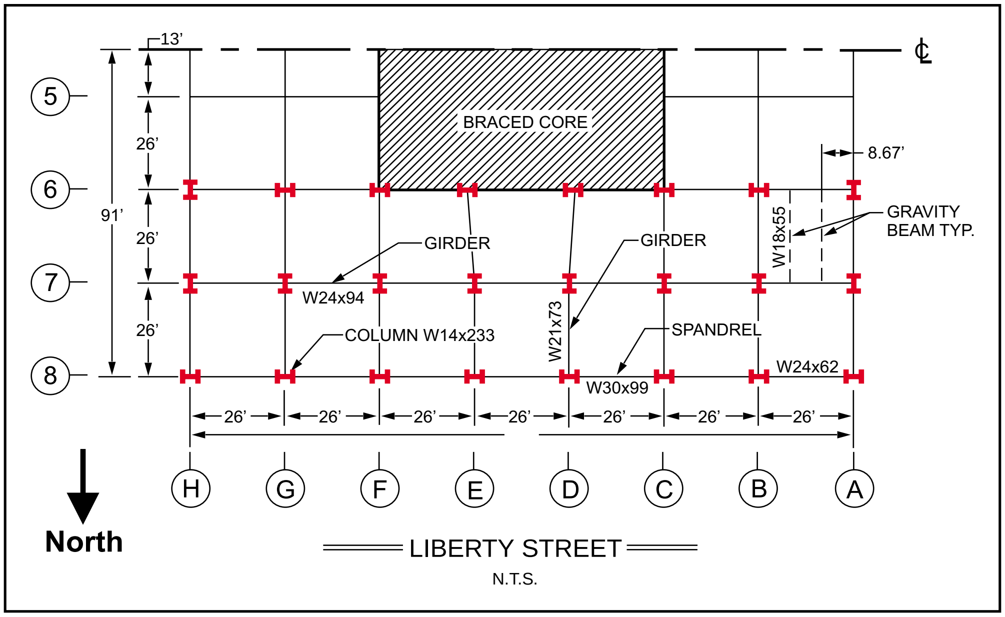


Figure 1.2. Partial plan view of a typical floor with estimated section sizes at Tenant Level 25

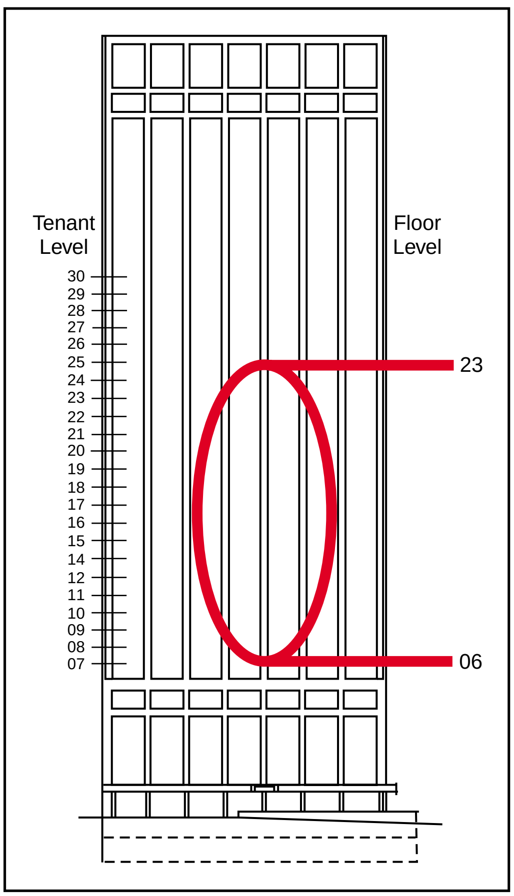


Figure 1.3. Sketch of northern façade showing the zone of damage

2.0 Reconnaissance of 130 Liberty Plaza

2.1 Exterior Reconnaissance of September 21, 2001

The visit of September 21, 2001 involved an exterior inspection of the building from ground level as part of a broader survey reported in Bruneau et al., 2002. The locations of damage observed within the building are referred to by grid lines, shown in Figure 1.2 and/or Tenant Level (TL), shown in Figure 1.3.

Figure 2.1 presents four photographs of the building taken on September 21, 2001. The view in a. is of the northern façade showing the large gash in the exterior wall caused by falling debris that was ejected from WTC 2 as it collapsed. One large piece of the façade of WTC 2 can be seen in this figure at approximately the 9th floor level. It is highly likely that this three-story high piece of debris caused much of the damage that is evident above TL 7. Clearly seen in this figure is the loss of a column on Line D (see Figure 1.2) between TL 7 and TL 25 (see Figure 1.3).

Part b. of Figure 2.1 shows damage to the northern and western faces of the building. Windows were broken on both of these faces of the building but the damage on the western face was confined to a one-bay width between Lines 7 and 8. The cladding to corner column A8 was lost between the ground and TL 5 and between TL 9 and 12.

Parts c. and d. of Figure 2.1 show damage to the lower levels of the building on its north face. Burning debris ignited small fires in the lower levels of the building but these fires appeared to have been confined to the perimeter of the building. Most of the windows in the lower levels, especially below the mechanical plant room at the 5th floor level, were broken but it is not known how many of these windows were broken on purpose by the rescue teams to eliminate falling glazing hazards to the rescue workers operating to the immediate south of WTC 2. Part d. of the figure shows the debris pile to the immediate north of the building, 10 days after the terrorist attacks. Much of the debris was from WTC 2.

Figure 2.2 presents information on the moment-resisting connections used in the building. Part a. of the figure shows fire damage to the lower framing, a cover-plated moment connection of the beam to the column, and the connections used to join the steel floor beams to the perimeter girders. No spray-on fireproofing is evident on the side or bottom of the perimeter beam although it may have been knocked off by the impact of falling debris. Façade construction

a. View of northern facade

b. View of northern and western facades

c. View of lower 10 stories of northern facade

d. Damage to the northern façade at the entry level

Figure 2.1. Damage to the 130 Liberty Plaza building from the exterior

a. First story of northern face   b. First story of the western face

Figure 2.2. Exposed steel moment framing in the first story of the building

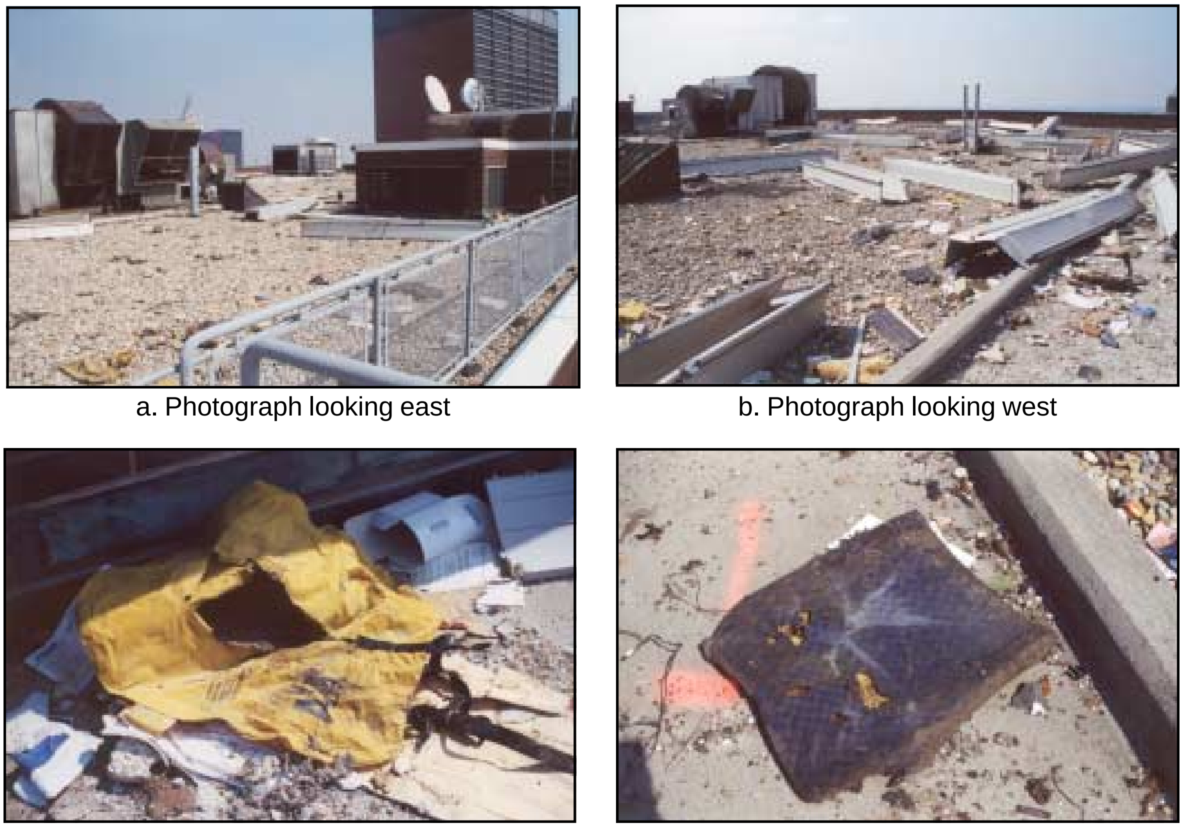
details can be clearly seen in this figure. Part b. shows a typical beam-column moment connection on the western face of the building. Cover-plates are clearly visible in this connection and damaged (or incomplete) fireproofing can be seen on the exterior face (web) of the perimeter beam.

2.2 Interior Reconnaissance of September 23, 2001

One member of the MCEER reconnaissance team returned to Ground Zero on September 23, 2001, to accompany an expert structural engineer from LZA/Thornton Tomasetti on a detailed inspection of the interior of the building. That inspection involved climbing directly from the entry level to the roof via the two stairwells located in the core of the building and returning to the entry level following a floor-by-floor inspection of the building. All quadrants of the building were inspected at each floor with attention being focused on the portion to the north of Line 6 (see Figure 1.2).

Both structural and nonstructural components in the building suffered significant damage, with all of the non-dust and non-mildew related damage above the sixth floor confined to a 2-bay by 2-bay zone contained by Lines 6 and 8 and C and E (see Figure 1.2 for details). Nonstructural damage extended a short distance to the west of Line C and the east of Line E.

Figure 2.3 presents photographs taken at the roof level of the building. Part a. of the figure is the view looking east across the roof. Silver U-shaped aluminum panels from WTC 2 can be seen in the figure together with life vests from American Airlines (AA) Flight 11 that struck the north face of WTC 1 (approximately 120 meters from 130 Liberty Plaza). Part b. of the figure is a view looking west across the roof. Aluminum panels and AA life vests are also



a. Photograph looking east

b. Photograph looking west

c. American Airlines life vest

d. Seat from American Airlines plane

Figure 2.3. Debris at the roof level of 130 Liberty Plaza

visible in this figure. Parts c. and d. are close-up photographs of an AA life vest and a seat from AA Flight 11.

No structural damage was observed at the roof level of the building and there was no evidence that the roof floor slab had been punctured by falling debris from WTC 2. This observation was confirmed by the walk-though of the plant-room spaces immediately below the roof. Apart from broken glass in the stairwells from the roof-level skylights, the structure and mechanical plant in the 38th story were undamaged by the collapse of WTC 2.

Damage to the building above TL 29 was modest (relative to the damage below TL 25) and was limited to broken glass (caused by debris ejected from WTC 2 as it collapsed). Figure 2.4 shows typical nonstructural damage above TL 29. Part a. shows damage caused by a section of fascia that was ejected from WTC 2 and which penetrated through a window on the northern face of the building. Part b. shows debris in a corner office at Grid A8. The angle section lying atop the

a. Penetration of WTC 2 fascia piece

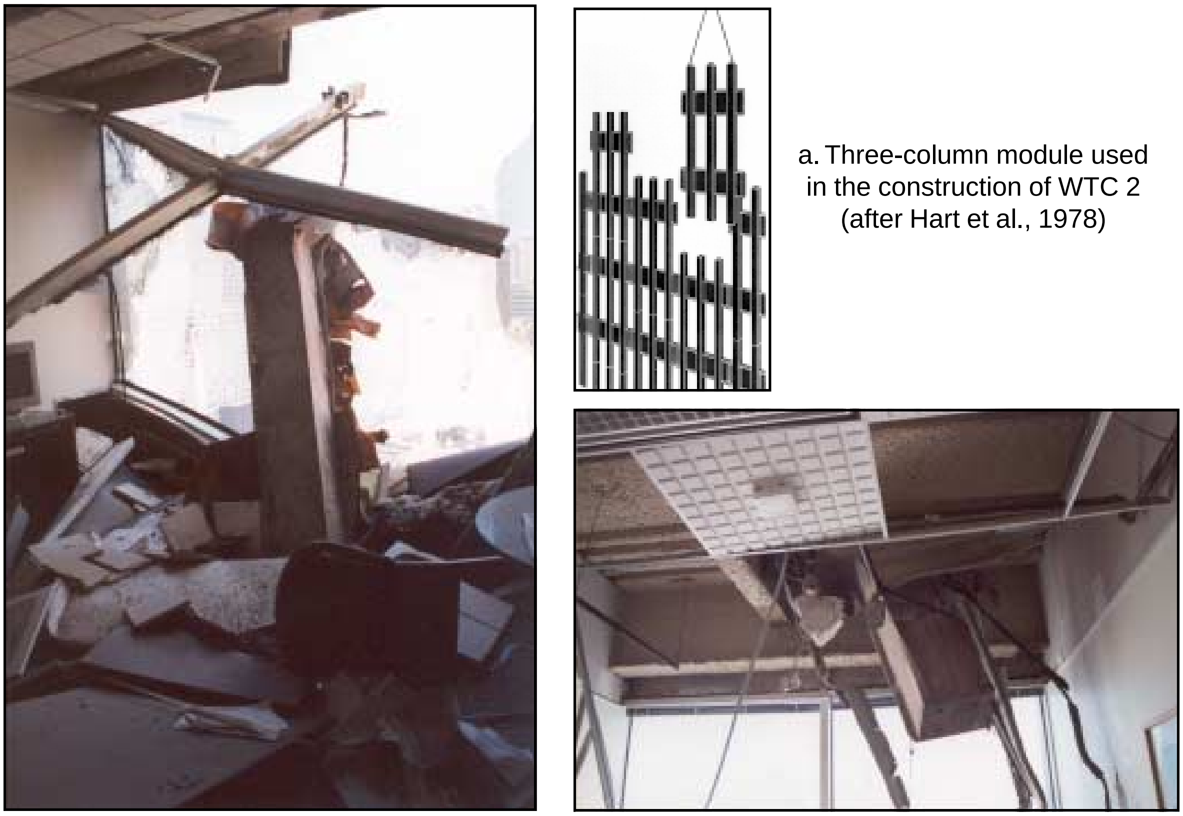
b. Typical damage in office space

Figure 2.4. Office space damage above Tenant Level 29

overturned chair in this figure penetrated through the north-facing window of the building. (The hard hat and flashlight in the photograph belonged to the photographer.)

The first major structural damage was observed at TL 29 and was due to the impact of one single-story tall structural steel column from WTC 2. (Note that, as described in Bruneau et al., 2002, this column was one of three in a typical spandrel module, so the two steel beams linking the three columns were destroyed before the single column hit the 130 Liberty Plaza building.) Figure 2.5a is a sketch of a typical three-column module. Figure 2.5b shows the upper end of the column above TL 29. Figure 2.5c shows the underside of the column and the damage it caused to the ceiling system. It is noteworthy that one unreinforced slab on the metal deck arrested the fall of this substantial steel member.

There was little evidence of damage to or distress in the building frame in the stories immediately above the zone of structural damage. There was no sign of excessive deflection in the framing above TL 25 as evinced by the total lack of deflection-induced damage in brittle components such as glazing and plasterboard partitions.



a. Three-column module used in the construction of WTC 2 (after Hart et al., 1978)

b. Upper end of column-missile above TL 29

c. Lower end of column-missile below TL 29

Figure 2.5. Damage due to column-missile ejected from WTC 2

The degree of damage to the structural framing increased substantially below TL 26. Figure 2.6a shows a north-south spanning floor beam at the underside of TL 23 that has lost its support on Line 8 due to the failure (loss) of the spandrel beam between Lines D and E. Note the clean separation of the metal deck from the beam (made possible by the lack of studs joining the beam to the slab). Part b. shows the underside of the floor slab at TL 22. At this level, both north-south spanning floor beams were lost; the locations of these beams are marked by dark stripes on the underside of the metal decking. Of importance to the integrity of the floor system is the fact that the TL 22 slab at this location was able to span three times the distance assumed in design, namely, 26 feet, albeit with large deflections in the slab system. Part c. shows a view of the slab of part b. looking north-east. Part d. is a view looking north-east of the underside of the slab at TL 21: a photograph taken from approximately the same location as Figure 2.6c but one floor lower in the building. Fracture of the metal deck floor system at the line of the girder spanning north-south on Line E can be seen in part d. of the figure together with gross distortion of two steel floor beams spanning north-south to the (lost) spandrel.

a. Underside of TL 23 slab looking north-west

b. Underside of TL 22 slab looking north-west

c. Underside of TL 22 slab looking north-east

d. Underside of TL 21 slab looking north-east

Figure 2.6. Interior damage at TLs 22 and 21

Figure 2.7 presents photographs taken from TL 19 in the building. Figure 2.7a shows the remnants of a moment-resisting connection at grid D8; the fractured flange connections and the welded web tab are clearly visible. Part b. is a photograph taken from the same location as the photograph of part a. but looking towards the north-south spanning girder on Line C. The World Financial Center and Winter Garden can be seen in the background of this photograph. Figure 2-7c is a view of two stories of framing along Line D. Note the distortion in the column flanges at the level of the beam-to-column connection and that the metal decking fractured cleanly along a butt (noncontinuous) joint atop the girder on Line D. Large deflections in the floor framing are not seen in this figure. Part d. shows the fractured column on Line D at approximately three feet above TL 18: immediately below the bottom of the framing shown in part c. of the figure. The debris pile at the base of WTC 2 can be seen in the lower portion of part d.

a. Failed moment connection at grid D8

b. Floor framing on Line 8 between Lines C and D

c. Damaged framing at TLs 19 and 20

d. Failed column immediately above TL 18

Figure 2.7. Interior damage photographs from TL 19

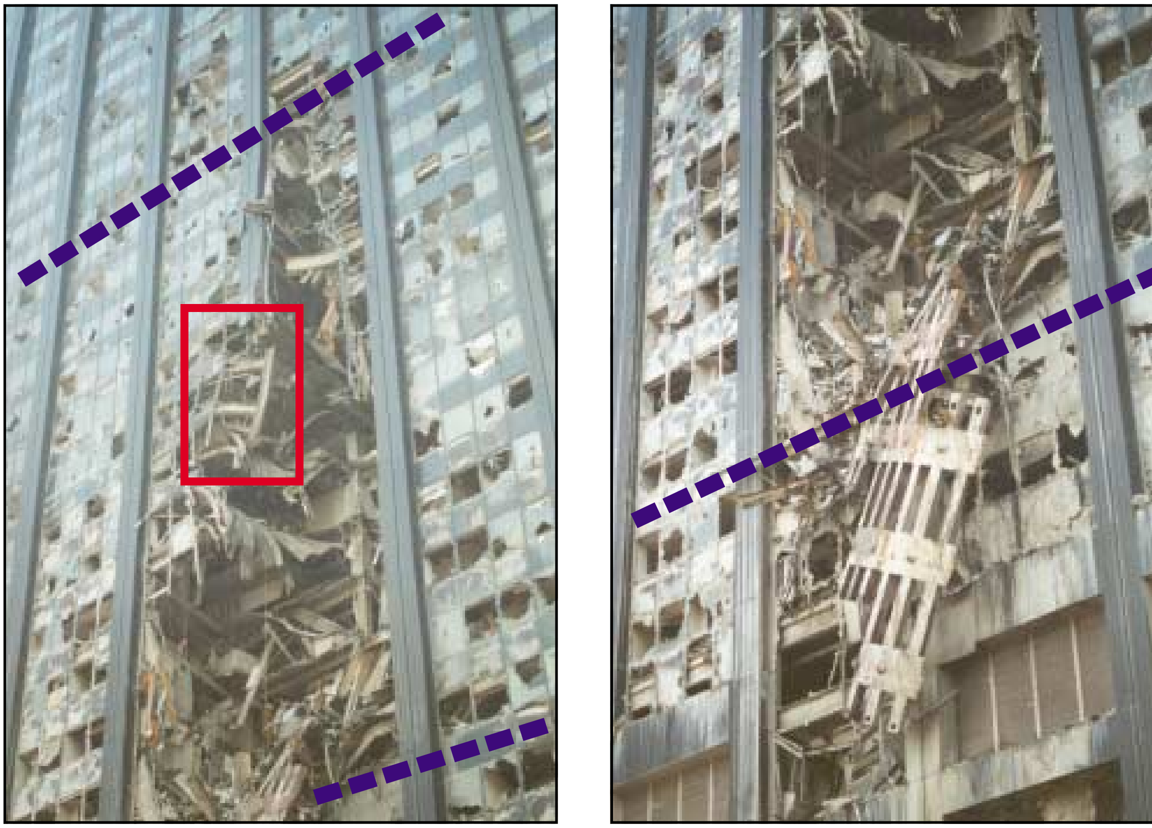
Figures 2.8a and 2.8b show the fractured column on Line D taken from TLs 18 and 17, respectively. The façade of WTC 2 can be seen in the background of part a. and World Trade Center 4 can be seen in the background of part b.

Figure 2.9 presents two photographs taken from outside the building looking toward the damage zone. The box superimposed on Figure 2.9a shows the column of Figure 2.8. The fracture evident in Figure 2.8a is located at the top of the box. The distorted but intact 2-story section of column seen in Figure 2.8b is located in the center of the box. In Figure 2.9a, the upper dashed line is at TL 25 and the lower dashed line is at TL 7. The dashed line in Figure 2.9b corresponds to the lower dashed line of Figure 2.9a. The large piece of debris that likely caused most of the damage above TL 10 is seen in Figure 2.9b precariously attached to the façade of the building.

The damage to the structural and nonstructural components between TLs 16 and 10 was most severe. The near-total destruction of the ceiling, mechanical, and plumbing systems in the areas surrounding the collapsed structural framing prevented the reconnaissance team from gaining access to the perimeter of the

a. View at TL 18 b. View at TL 17

Figure 2.8. Fractured column on Line D from TLs 18 and 17



a. View of damage zone between TLs 23 and 10

b. View of damage zone adjacent to TL 10

Figure 2.9. Damage to the northern face of the building between TLs 23 and 10

damage zone. As seen in Figures 2.9a and 2.9b, the zone of extreme damage expanded below TL 16 to the 2-bay by 2-bay zone bounded by Lines C and E and 6 and 8.

Figure 2.10 shows the damage at TL 9. Figure 2.10a shows the complete destruction of one zone of the floor immediately adjacent to the northern face of the building. Part b. shows the interior face of the section of WTC 2 façade, seen in Figures 2.1a and 2.9b, that caused much of the damage to the building.

Structural damage below the mechanical plant rooms that were located below TL 9 was modest and no photographs were taken at the levels between TL 9 and the entryway to the building. Figure 2.11 is a photograph taken inside the building looking north towards a standing section of the WTC 2 façade.

a. Complete destruction of one zone of the building

b. View of inside face of WTC 2 façade element

Figure 2.10. Building damage at TL 9

Figure 2.11. View of the WTC 2 façade looking north-west from the foyer of the building

3.0 Building Analysis

The observation that the building at 130 Liberty Plaza did not collapse, despite the loss of key structural elements and severe damage, motivated the research team to analyze the building to understand the cause of the observed behavior. Standard tools for the linear and nonlinear analysis of buildings subjected to earthquake shaking were employed for these studies. Linear analysis was performed to determine demand-to-capacity ratios for the undamaged state as well as three damage states, one of which corresponds to the observed damage. Linear analysis further provided an estimate of the elastic limit of the framing system for each of the damage states considered. Two-dimensional and three-dimensional linear analyses were performed. Small displacement theory was employed for these analyses. Simple plastic analysis was then performed to determine an upper bound to the capacity of the framing system for each of the damage states considered. Both two- and three-dimensional framing systems were considered for simple plastic analysis.

Detailed information on the structural framing system was not available to the research team, although approximate sizes were noted during the building reconnaissance. To facilitate the linear and nonlinear analysis of the building, sizes of the beams and columns in the moment-resisting frame were estimated by analysis of the building frame for gravity and winds loads as described in the following section. All beam-to-column connections were assumed to be moment resisting. The estimated sizes of the wide-flange beams and columns were checked against the approximate sizes noted during the building reconnaissance. Because no information was available on the steel braced core, sizes were not estimated for the steel braces.

3.1 Preliminary Analysis and Design

A preliminary design was undertaken per the 1970 Building Code of the City of New York (BCCNY, 1970) to determine beam and column section sizes. Resulting sizes of the beams and columns were used in the analytical studies presented herein. Because actual sizes were unavailable, the results of the analysis should be interpreted with care with regard to the performance of the 130 Liberty Plaza building.

Both gravity and wind loads were considered for the preliminary design. Gravity loads were taken to be 50 psf (dead load) and 100 psf (live load), based on measured structural properties and the occupancy of the building. Live loads were reduced for the design of columns. Wind pressures on the face of the building were taken to be 30 psf for elevations above 301 feet, 25 psf for elevations between 101 feet and 300 feet, and 20 psf for elevations below 100 feet. A one-third increase in allowable stress was used for the gravity and wind load combinations per the 1970 BCCNY. Limits on maximum lateral drift under wind loads were not considered.

Only the moment-resisting frame along Line 8 (Figure 1.2) was considered for preliminary analysis and design. The tributary building width for calculating wind loads for the Line 8 frame was taken to be 39 feet: one and a half bay widths. Design actions were first estimated using simple analysis tools such as the portal method. Steel section sizes were then established using the AISC *Manual of Steel Construction, Allowable Stress Design* (AISC, 1989).

To facilitate three-dimensional finite element analysis of the building, section sizes along frame Lines 6 and 7, including intermediate perpendicular framing, were determined based on: (1) information from the September 23, 2001, reconnaissance visit; and (2) gravity load considerations. The resulting column and beam sections at TL 25 are shown in Figure 1.2 for one half of a typical building floor plan.

3.2 Linear Elastic Analysis

Two- and three-dimensional finite element models were constructed using SAP2000 (CSI, 2000). First, a two-dimensional model (Figure 3.1a) was prepared that considered structural framing over all 39 stories along Line 8 (Figure 1.2). This model was constructed to study the response of a single frame with varying degrees of damage (or damage states). Second, a three-dimensional model was prepared that considered structural framing over all 39 stories along Lines 6, 7, and 8, including intermediate perpendicular framing (Figure 3.2). This model was prepared to better understand the response of the building for the observed damage state and to compare the results of two- and three-dimensional analysis. The red dashed line in Figure 3.2 identifies the zone of observed damage per Figure 3.1b.

The analyses presented below considered only gravity loads with a dead load and reduced live load of 50 psf each. A uniform distributed load of 260 lb/ft per story was assumed to account for the curtain wall loading. Mathematical models were analyzed for the undamaged state, denoted ND, and three damage states, one of which corresponds to the existing damage shown in Figure 2.1a. Each damage state involved the removal of columns on Line 8 from TL 7 to TL 25. The three damage states involved the removal of (1) the column on Line D (the observed damage per Figure 3.1b; (2) the columns on Lines D and E; and (3) the columns in Lines C, D, and E, denoted DS1, DS2, and DS3 in Figure 3.1,

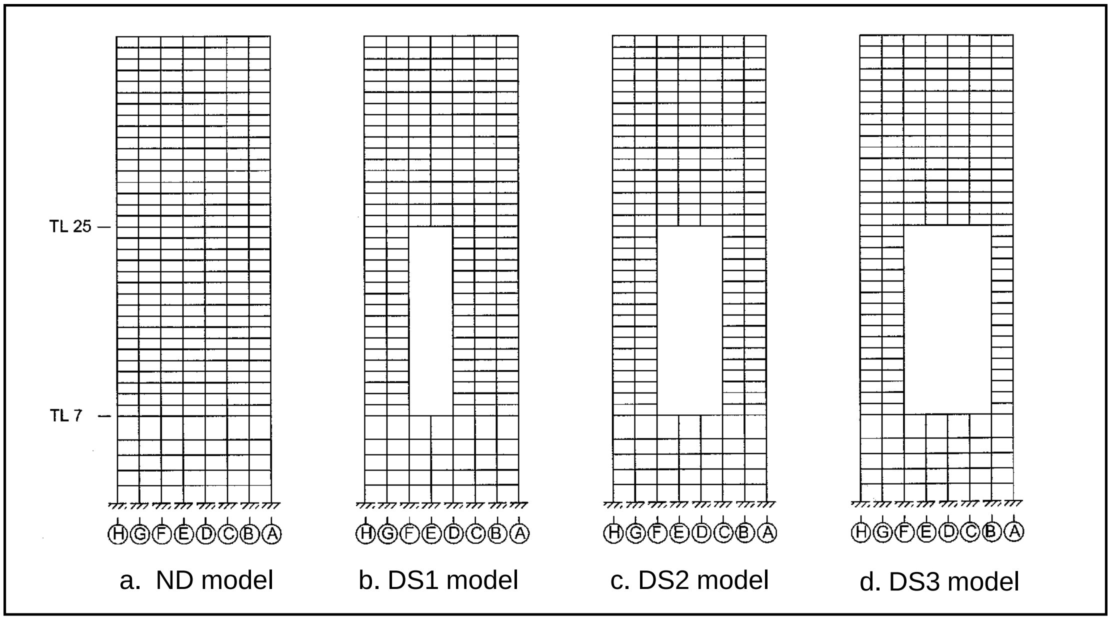


Figure 3.1. Two-dimensional mathematical models of framing on Line 8

respectively. Maximum member actions under gravity loads were calculated for the undamaged state and the three damage states.

Figures 3.3 and 3.4 present some of the results of the two- and three-dimensional analyses, respectively. Shown in these figures are moments normalized by the yield moment for the assumed beam section sizes at TL 25; the floor level immediately above the observed damage. The spandrel beam designation (e.g., DE) refers to the grid lines between which the beam spans (e.g., Lines D and E). The girder designation (e.g., E87) refers to the grid line along which the girder is aligned (e.g., Line E) and the grid lines between which the girder spans (e.g., Lines 8 and 7). Such normalized moments represent demand-to-capacity (D/C) ratios for these elements, albeit not exactly, because M_{yield} is used in lieu of $\phi \cdot M_{nx}$ and each beam is assumed to be fully braced.

Damage state DS1 (the observed damage) and the two-dimensional analysis is considered first. All D/C ratios are substantially less than one. The three-dimensional analysis shows similar results. These results provide an explanation for the observed behavior of the framing along Line 8 following the impact of debris from WTC 2 and the loss of a column on Line D, namely, that the moment-resisting framing above TL 25 provided an alternate (redundant) path for gravity loads around Line D and to the foundation without distress to the structural framing. As observed from DS1, models with increasing levels of damage (i.e., DS2 and DS3) showed that the moment-resisting framing above the damage provided an alternate path for gravity loads. For DS2 and DS3, Vierendeel truss action becomes more apparent.

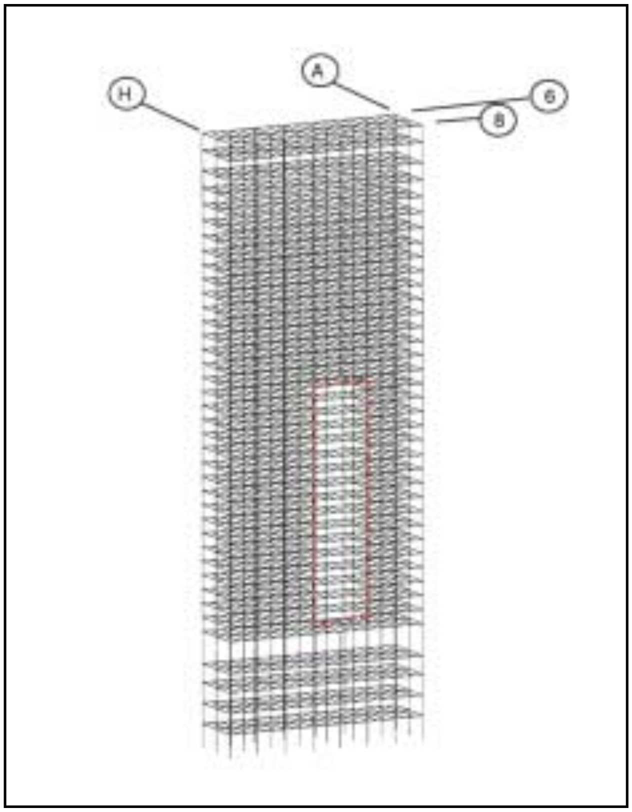


Figure 3.2. Three-dimensional mathematical model of framing on Lines 6, 7 and 8 for DS1

The mathematical models for damage states DS2 and DS3 were prepared to evaluate the robustness of a building frame with characteristics similar to those of 130 Liberty Plaza, where robustness herein is judged by the ability of the framing system to support gravity loads following the loss of multiple perimeter columns. The results of the two-dimensional analysis of the DS2 model (see Figure 3.3) show that the frame on Line 8 would have been compromised by the loss of columns on Lines D and E unless the moment-resisting connections were ductile (i.e., possessed some degree of inelastic rotation capacity). Review of the three-dimensional analysis results of Figure 3.4, however, shows that using the results of the two-dimensional analysis leads to conservative conclusions and that the moment-resisting framing perpendicular to Line 8 also participated in the redistribution of load around the lost columns on Lines D and E. Analyses for both the two- and three-dimensional models of DS3 (the loss of columns on Lines C, D, and E) show modest overloads in both instances. For the three-dimensional analysis, the D/C ratios are greatest in the framing perpendicular to Line 8, namely, Girders C87, D87, and E87.

Demand-to-capacity ratios were calculated for columns on Line 8 between TLs 24 and 25 (Figure 3.1) for both two- and three-dimensional analyses. The column

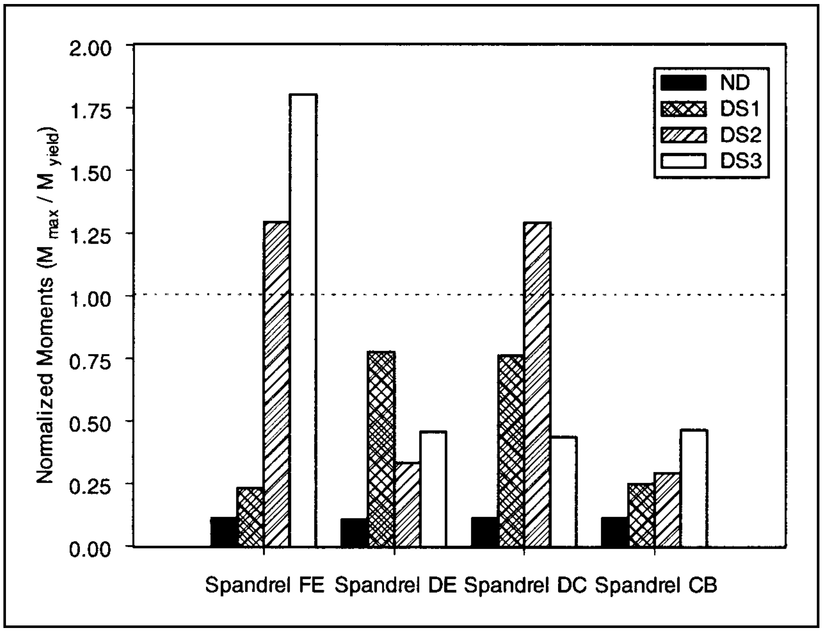


Figure 3.3. Demand-to-capacity ratios for two-dimensional linear elastic analysis

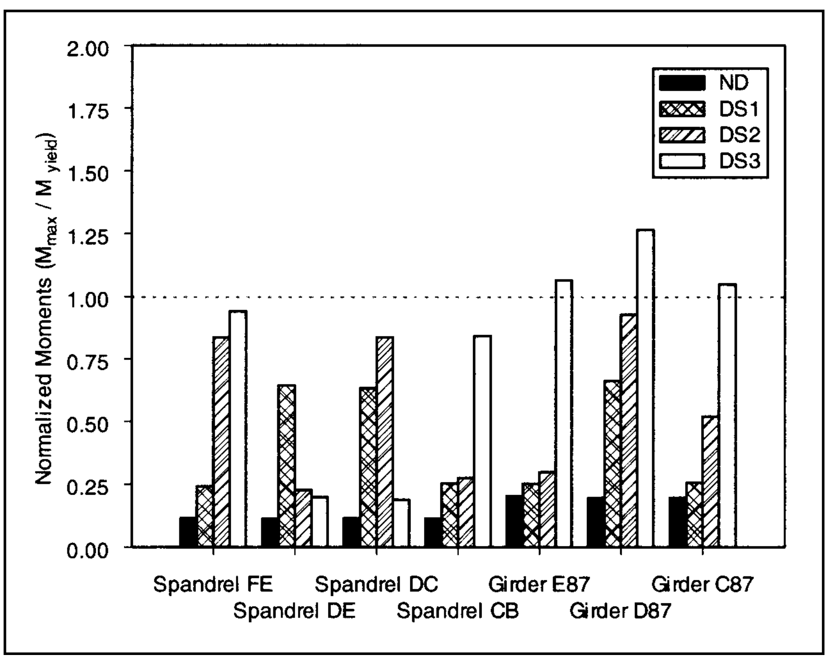


Figure 3.4. Demand-to-capacity ratios for three-dimensional linear elastic analysis

designation (e.g. F8) in Figures 3.5 and 3.6 refers to the column between TLs 24 and 25 at the intersection of Lines F and 8. Shown in Figures 3.5 and 3.6 are column D/C ratios. These ratios were calculated using the 1998 *Edition of the AISC Manual of Steel Construction, Load and Resistance Factor Design* (AISC 1998) nominal strength equation for members under combined forces (LRFD Eqn. H1-1a), namely,

$$\frac{P_u}{\phi \cdot P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi_{b \cdot M_{nx}}} + \frac{M_{uy}}{\phi_{b \cdot M_{ny}}}\right) \leq 1.0 \tag{3.1}$$

To facilitate calculation of the D/C ratios and comparison of analysis results, the value of the effective length factor was assumed to be 1.0 for all columns. This value is the largest assuming that side-sway is prevented, which is a reasonable assumption for the moment frame considering the lateral stiffness of the braced core (significantly greater than that of the moment frame) and the presence of rigid floor diaphragms.

The results of the two-dimensional analyses shown in Figure 3.5 indicate D/C ratios less than unity for the undamaged state, ND, and damage states, DS1 and DS2. For damage state DS3, the D/C ratios for columns F8 and B8 exceed unity. Ratios greater than unity can be attributed to an increase in both axial forces and moments due to the removal of columns C8, D8 and E8. This observed increase in bending moment for the two-dimensional analyses ranged from essentially zero for ND to approximately 0.25 M_p (where M_p is the plastic moment of the section) for DS3. Noting that the span between adjacent columns for DS3 is four times the span in the undamaged state ND, an increase in moment demand is expected. The D/C ratios for the three-dimensional model (Figure 3.6) are less than unity for ND and DS1, DS2, and DS3. Again, the results of the three-dimensional analyses indicate that the use of two-dimensional analysis leads to conservative conclusions and that the presence of perpendicular framing provides additional redundancy and capability for gravity load redistribution to adjacent framing.

3.3 Simple Plastic Analysis

Simple two- and three-dimensional plastic analyses were performed to determine an upper bound on the load carrying capacity of the framing system for each of the three damage states. In both the two and three-dimensional analyses, beam plastic moments were calculated assuming a yield stress of 36 ksi and the section sizes determined in the preliminary design (see Figure 1.2). All sections were assumed to be compact. Further, all beam-to-column connections were assumed to have unlimited rotation capacity. For the two-dimensional analyses, vertical "panel" mechanisms were assumed to form for each damage state. Figure 3.7

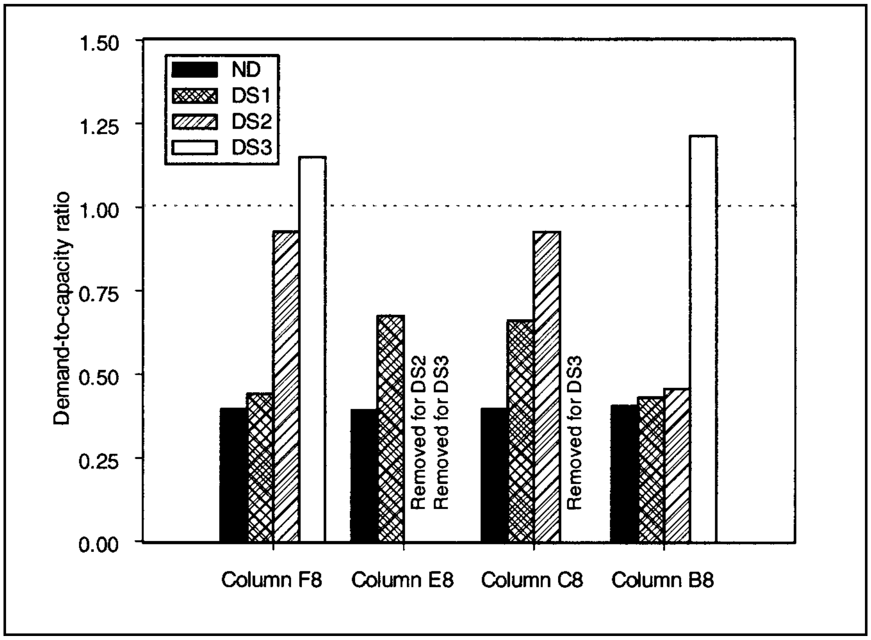


Figure 3.5. Demand-to-capacity ratios for two-dimensional linear elastic analysis

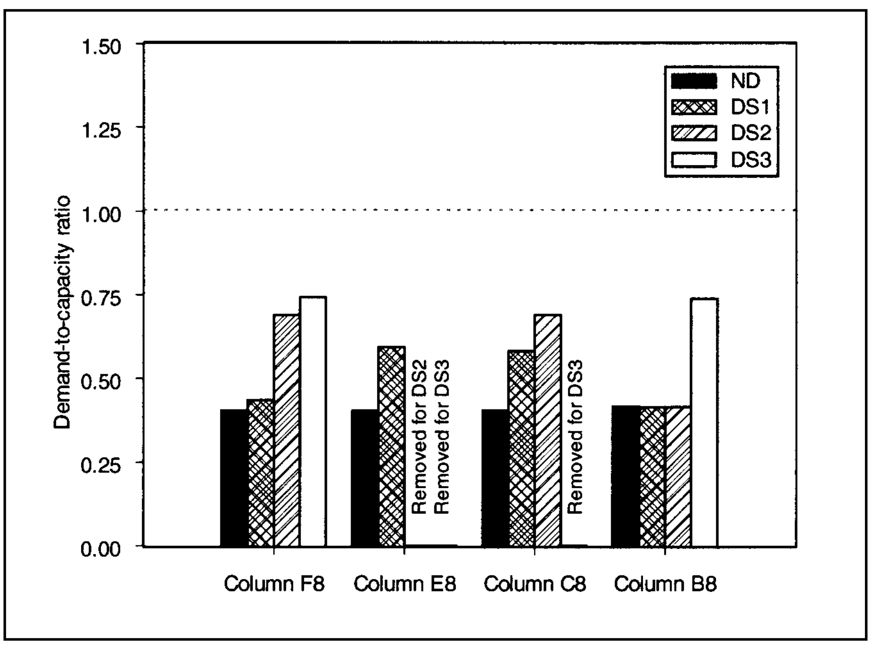


Figure 3.6. Demand-to-capacity ratios for three-dimensional linear elastic analysis

shows the assumed mechanism for DS1. The assumed mechanisms for the three-dimensional analyses were similar, with the addition of hinges forming in the girders of the perpendicular framing where appropriate. Curtain wall loads and beam plastic moments were taken as known quantities, and the maximum corresponding floor load that could be sustained by the framing was determined for each case.

The results of these analyses are presented in Table 3.1 in terms of both the *maximum* floor loading (measured in psf) and the maximum floor loading normalized by the likely maximum loading at the time of the attacks of September 11, 2001, judged by the reconnaissance team to be approximately 100 psf.

These analyses support the results of the elastic analysis, namely, that the framing system could have tolerated the loss of two columns without collapse. Note the additional load-carrying capacity that results from considering the framing perpendicular to Line 8.

However, it must be noted that the cover-plated moment-resisting connections employed in the construction of the building likely have limited plastic rotation capacity as demonstrated by a series of tests conducted following the 1995 Northridge earthquake (Kim et al., 2000). Therefore, the results of the simple plastic analyses must be interpreted with care.

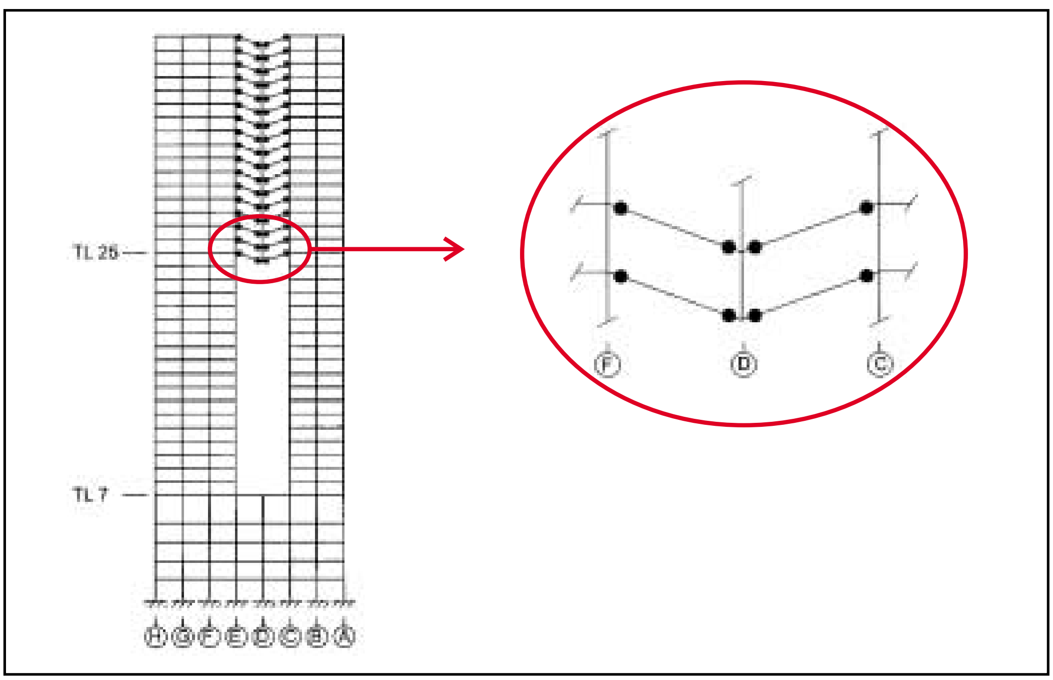


Figure 3.7. Two-dimensional collapse mechanism for DS1

Table 3.1. Plastic analysis results

	Two-dimensional Analysis		Three-dimensional Analysis	
Damage State	**Floor Load (psf)**	**Normalized Load**	**Floor Load (psf)**	**Normalized Load**
1	293	2.9	308	3.1
2	136	1.4	218	2.2
3	84	0.8	182	1.8

3.4 Nonlinear Static Analysis

Two-dimensional nonlinear static or "pushdown" analysis of the framing along Line 8 was performed for the three damage states to obtain insight as to the displacements that could be expected at the maximum floor loads predicted by simple plastic analysis. Elastic-perfectly plastic moment-rotation relationships were assumed for all beams. Connections were assumed to have infinite rotation capacity. The analyses were run using SAP2000 under displacement control and used the node at Frame Line D and TL 25 as the control node. The loading pattern was a series of equal valued point loads at the locations where the floor beams and perpendicular girders frame into the spandrel elements on Line 8. Prior to running the displacement controlled nonlinear static analysis, the curtain wall load was applied as a single force controlled step because it had been accounted for in the plastic analyses. Figure 3.8 shows the observed progression of plastic hinging for DS1. The hinging patterns for DS2 and DS3 were similar to DS1. Hinging started at the beam-to-column connections on Lines E and C near TL 25, progressed to the connections on Frame Line D, and then vertically up through the framing on Line 8.

Figure 3.9 shows the resulting relationships between floor load (psf) and the deflection of Line D at TL 25 for the three damage states, where the floor load is assumed to be present over the entire width of the damaged zone (i.e., between Lines C and E for DS1, Lines C and F for DS2, and Lines B and F for DS3). The initial displacements seen in the figure correspond to the deflection due to the prior application of the curtain wall load. Also shown in this figure is the research team's estimate of the likely maximum floor loading at the time of the attacks of September 11, 2001, of 100 psf. As expected, an increase in damage led to an increase in structural flexibility and reduced the maximum permissible floor load. The results of the nonlinear static analysis and those of the simple plastic analysis are in good agreement for all damage states.

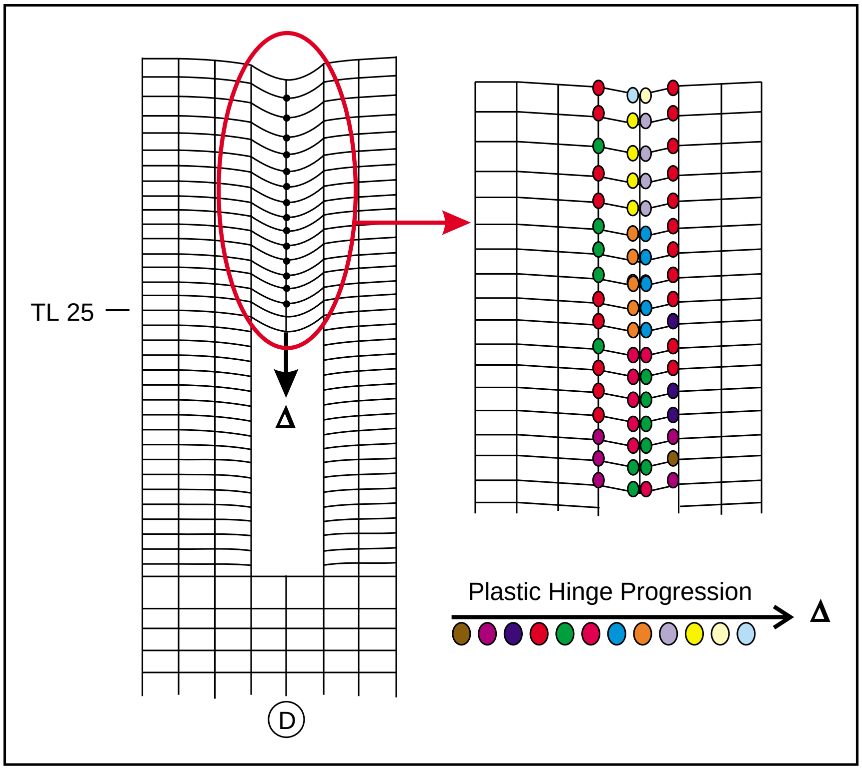


Figure 3.8. Progression of plastic hinge formation in DS1

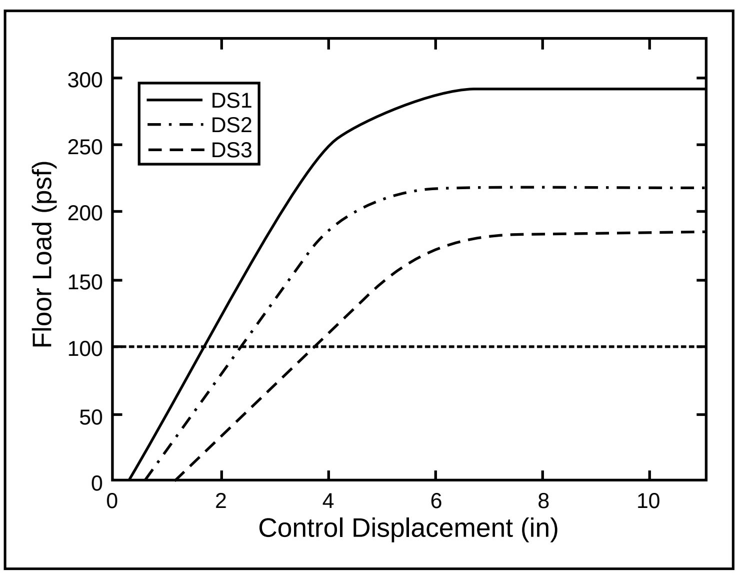


Figure 3.9. Two-dimensional pushdown curves for DS1, DS2, and DS3

4.0 Summary and Conclusions

The 130 Liberty Plaza building sustained severe damage from falling debris during the collapse of World Trade Center Tower 2. Reconnaissance efforts on September 21 and 23, 2001 documented the exterior and interior damage to the building. Despite the loss of a perimeter column over a 17-story height, the building did not collapse because the lateral and gravity load resisting systems were highly redundant. The redundant structural systems permitted gravity loads to be redistributed around the badly damaged region, an observation supported by preliminary elastic and plastic analyses of a building frame with characteristics similar to those of the damaged building. Key observations from the work to date include:

1. Highly redundant gravity and lateral-force-resisting systems are key to the construction of damage tolerant buildings.

2. The use of ductile details (ability to deform well into the inelastic range) will improve the damage tolerance of buildings.

3. Simple framing systems such as unreinforced slabs on metal decking can span substantially further than that assumed in design and such capabilities should be included in the evaluation of buildings for damage tolerance. The addition of inexpensive details (such as continuous slab reinforcement and continuity in the metal decking) could further enhance building performance and prevent partial collapses.

4. Simple two and three-dimensional analysis tools such as those adopted for the work presented in this summary report can be used to judge, in a preliminary sense, the damage tolerance of buildings.

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