RECONNAISSANCE AND PRELIMINARY ASSESSMENT OF A DAMAGED HIGH-RISE BUILDING NEAR GROUND ZERO

GORDON WARN, JEFFREY BERMAN, ANDREW WHITTAKER* AND MICHEL BRUNEAU
State University of New York at Buffalo, Buffalo, New York, USA

SUMMARY

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 paper 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 non-linear analyses were undertaken. Such analyses showed 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. Copyright © 2003 John Wiley & Sons, Ltd.

1. INTRODUCTION

Within two weeks of the attacks on the World Trade Center Towers, on September 11, 2001, MCEER dispatched a research team to New York City 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 (WTC) 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 (WTC2) as it collapsed.

The objectives of the work described in this paper were twofold: (1) to collect information about the structural and non-structural damage suffered by the building at 130 Liberty Plaza due to the collapse of Tower 2 in the 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 non-linear analysis tools that are used by earthquake engineers. Results of this work are presented in Section 3.

The building studied is a 39-story office building located at 130 Liberty Plaza in lower Manhattan. The building was located to the immediate south of the Tower 2. Figure 1 shows the location of the building with respect to the WTC complex.

* Correspondence to: Dr. Andrew Whittaker, Department of Civil Engineering, State University of New York, Buffalo, NY 14260, USA. E-mail: awhittak@acsu.buffalo.edu

1. 1 World Trade Center	8. 30 West Broadway Building
2. 2 World Trade Center	9. Verizon Building
3. 3 World Trade Center	10. 3 World Financial Center
4. 4 World Trade Center	11. 2 World Financial Center
5. 5 World Trade Center	12. Winter Garden
6. 6 World Trade Center	13. 1 Bankers Trust Building
7. 7 World Trade Center	14. 90 West Street Building

Buildings were destroyed

Buildings are in danger of collapse

Buildings suffered major structural damage but can be repaired

Buildings suffered slight damage

Figure 1. Location of subject building with respect to the World Trade Center 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 reconnaissance that 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 $2\frac{1}{2}$ 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 2 shows a part plan of a typical floor; the member sizes indicated on that figure were established by members of the reconnaissance team following independent analysis of the frame as described in Section 3. The grid marks (A–H and 1–8) were selected by the reconnaissance team to aid in the identification and interpretation of damage.

The façade of the building is composed of windows and a lightweight cladding system. A sketch of the northern façade that was badly damaged by falling debris is presented in Figure 3. The elliptical line in the figure identifies the zone of damage to the façade of the building.
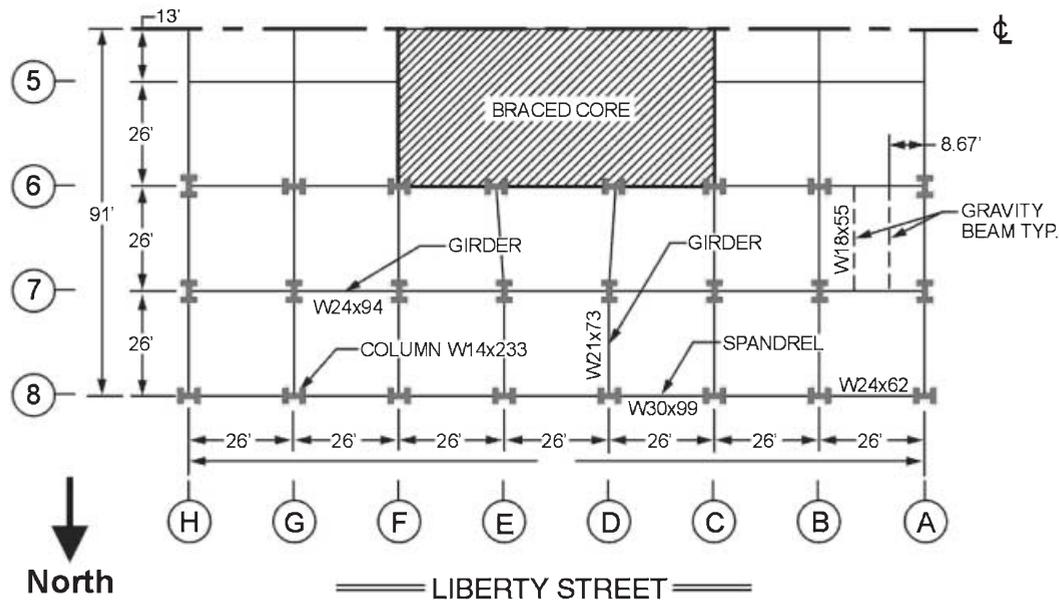
Figure 2. Part plan view of a typical floor

2. RECONNAISSANCE

2.1 Exterior reconnaissance of September 21, 2001

The visit of September 21 involved an exterior inspection of the building from ground level as part of a broader survey reported elsewhere (Bruneau *et al.*, 2002). Figure 4 presents four photographs of the building taken on September 21, 2001. Part (a) is a view of the northern façade showing the large gash in the exterior wall caused by falling debris that was ejected from Tower 2 as it collapsed. One large piece of the façade of Tower 2 can be seen in this figure at approximately the ninth floor level. It is highly likely that this three-storey high piece of debris caused much of the damage that is evident above the ninth floor level. Clearly seen in this figure is the loss of a column on Line D (see Figure 3) between Tenant Level 7 and 25.

Part (b) of Figure 4 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 Tenant Level 5 and between Tenant Level 9 and 12.

Parts (c) and (d) of Figure 4 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 fifth 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 Tower 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 Tower 2.

Figure 5 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
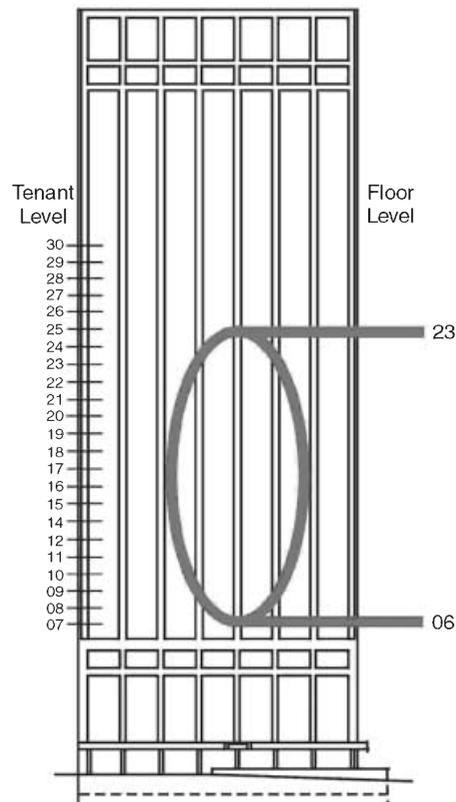
Figure 3. Rendering of northern façade showing the zone of damage

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 knocked off upon impact. Façade construction details can be clearly seen in this figure. Part (b) of the figure 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, 2002, 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, in which all quadrants of the building were inspected at each floor but with attention being focused on that portion of every floor to the north of Grid 6.

   Both structural and non-structural components in the building suffered significant damage but all of the non-dust and non-mildew-related damage above the sixth floor appeared to be substantially con-fined to a two-bay by two-bay zone contained by Lines 6 and 8 and C and E. See Figure 2 for details. Non-structural damage did extend a short distance to the west of Line C and the east of Line E.

a. view of northern façade

b. view of northern and western façades

c. view of lower 10 stories of northern façade

d. damage to northern façade at the entry level
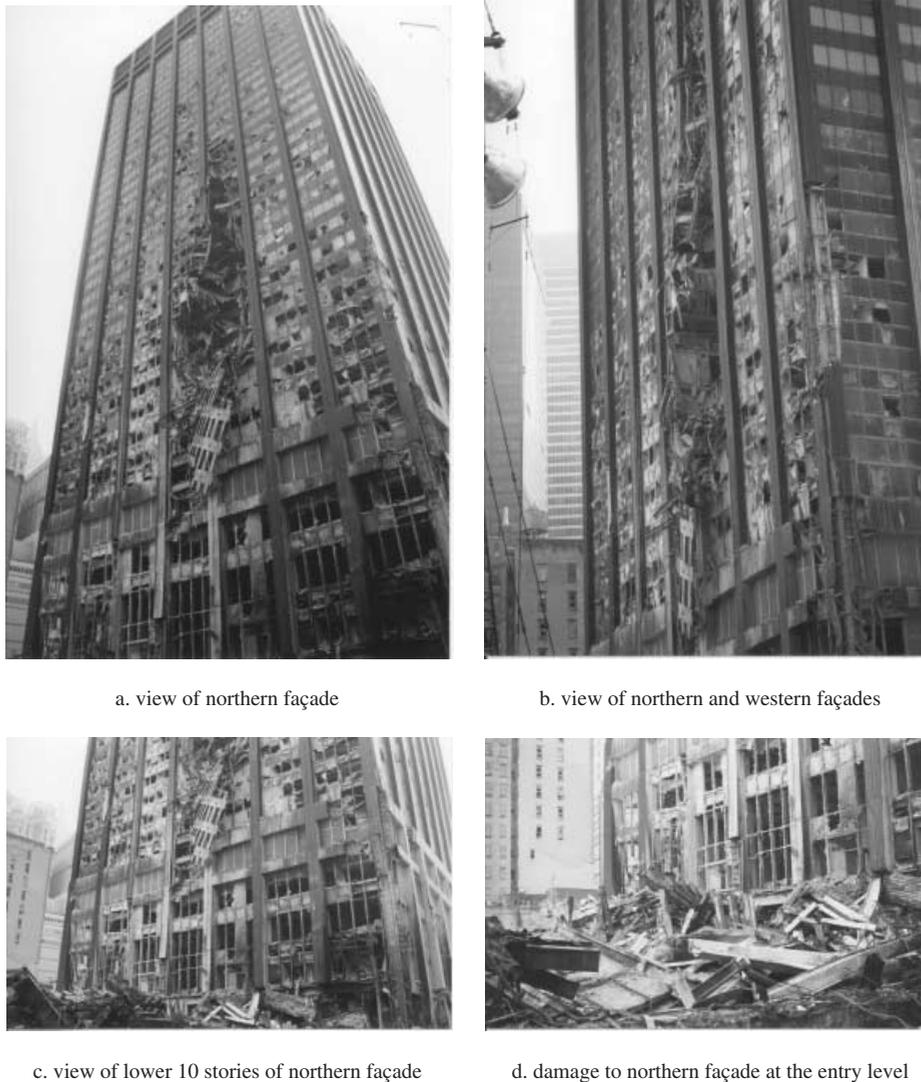
Figure 4. Damage to the 130 Liberty Plaza building from the exterior

Figure 6 presents photographs taken at the roof level of the building. Part (a) of the figure is a photograph looking east across the roof. Silver U-shaped aluminum panels from Tower 2 can be seen in the figure together with life vests from American Airlines (AA) Flight 11 that struck the north face of Tower 1 (approximately 120 m from 130 Liberty Plaza). Part (b) of the figure is a photograph looking west across the roof. Aluminum panels and AA life vests are also visible in this figure. Parts (c) and (d) of the figure are close-up photographs of an AA life vest and a seat from the 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 Tower 2. This observation was confirmed by the walk-though of the plant-room spaces immediately below the roof. Apart from broken glass in

a. first story of northern face   b. first story of western face

Figure 5. Exposed steel moment framing in the first story of the building

a. photograph looking east   b. photograph looking west

c. American Airlines life vest   d. seat from American Airlines plane
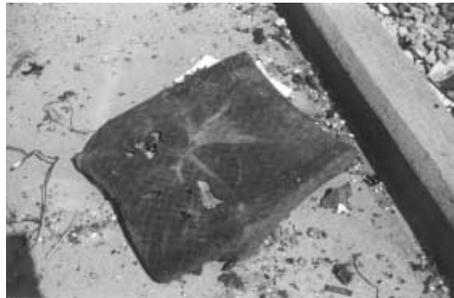
Figure 6. Debris at the roof level of 130 Liberty Plaza

the stairwells from the roof-level skylights, the structure and mechanical plant in the 38th story were undamaged by the collapse of Tower 2.

Damage to the building above Tenant Level 29 was modest (relative to the damage below Tenant Level 25) and was limited to broken glass (caused by debris ejected from Tower 2 as it collapsed). Figure 7 shows sample non-structural damage above Tenant Level 29. Part (a) of the figure shows damage caused by a section of fascia that was ejected from Tower 2 and which penetrated through a

a. penetration of Tower 2 fascia piece   b. typical damage in office space

Figure 7. Office space damage above Tenant Level 29

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 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 Tenant Level 29 and was due to the impact of one single-story tall structural steel column from Tower 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 8(a) is a sketch of a typical three-column module. Figure 8(b) shows the upper end of the column above Level 29. Figure 8(c) is a photograph of the underside of the column and the damage it caused to the ceiling system. It is noteworthy that the unreinforced slab on 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 Tenant Level 25 as evinced by the total lack of deflection-induced damage in brittle components such as glazing and plasterboard partitions.

The degree of damage to the structural framing increased substantially below Tenant Level 26. Figure 9(a) shows a north–south spanning floor beam at the underside of Tenant Level (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) of the figure shows the underside of the floor slab at Tenant Level 22. At this level both north–south spanning floor beams were lost; the locations of these beams are marked

a. three-column module used in the
construction of Tower 2 (adapted from
Hart et al., 1978)

b. upper end of column-missile above Level 29

c. lower end of column-missile below Level 29

Figure 8. Damage due to column-missile ejected from Tower 2

by dark stripes on the underside of the metal decking. Of importance to the integrity of the floor system is the fact that the Tenant Level 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) of the figure shows a view of the slab of part (b) looking northeast. Part (d) of the figure is a view looking north-east of the underside of the slab at Tenant Level 21: a photograph taken from approximately the same location as Figure 9(c) 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. Figure 10 presents photographs taken from Tenant Level 19 in the building.

Figure 10 shows the remnants of a moment-resisting connection; the fractured flange connections and the welded web tab are clearly visible. Part (b) of the figure 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 10(c) is a view of two stories of framing along Line D. Note the distortion in the column flanges

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
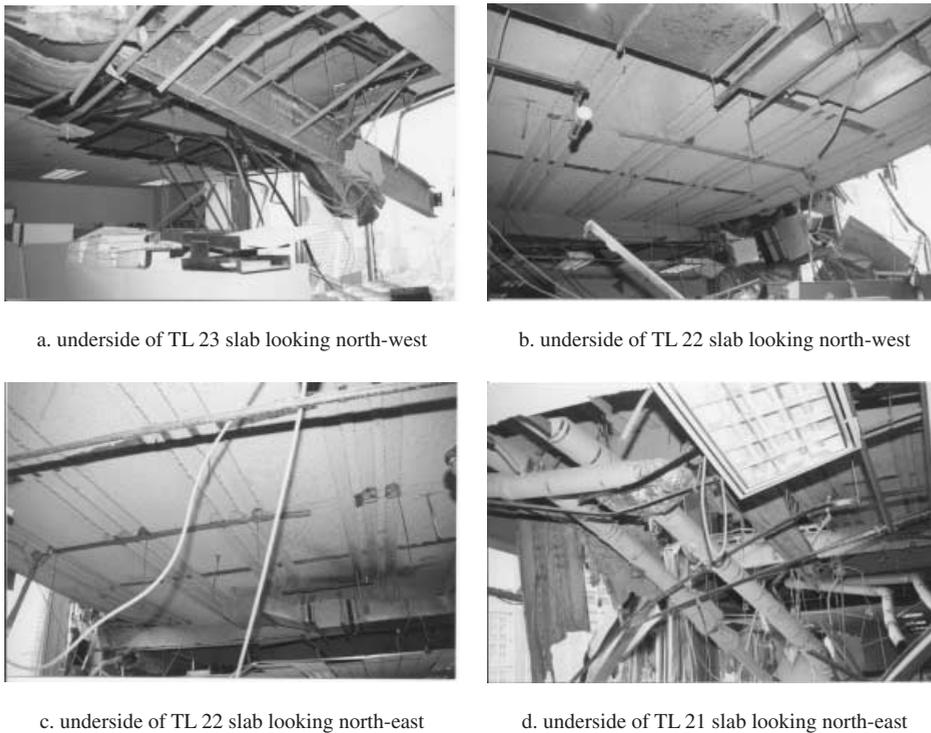
Figure 9. Interior damage at Tenant Levels 22 and 21

at the level of the beam-to-column connection and that the metal decking fractured cleanly along a butt (non-continuous) joint atop the girder on Line D. Large deflections in the floor framing are not seen in this figure. Part (d) of the figure shows the fractured column on Line D at approximately 3 feet above Tenant Level 18, immediately below the bottom of the framing shown in part (c) of the figure. The rubble pile at the base of Tower 2 can be seen at the bottom of Figure 10(d).

Figures 11(a) and 11(b) present photographs of the fractured column on Line D taken from Tenant Levels 18 and 17, respectively. The façade of Tower 2 can be seen in the background of part (a) of the figure and WTC 4 can be seen in the background of part (b) of the figure.

Figure 12 presents two photographs taken from outside the building looking toward the damage zone described in the previous pages. The box superimposed on Figure 12(a) shows the column of Figure 11. The fracture evident in Figure 11(a) is located at the top of the box. The distorted but intact, two-story section of column seen in Figure 11(b) is located in the center of the box. In Figure 12(a), the upper dashed line is at Tenant Level 25 and the lower dashed line is at Tenant Level 7. The dashed line in Figure 12(b) corresponds to the lower dashed line of Figure 12(a). The large piece of debris that likely caused most of the damage above Tenant Level 10 is seen in Figure 12(b) precariously attached to the façade of the building.

The damage to the structural and non-structural components between Tenant Levels 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 damage zone. As seen in Figures 12(a) and 12(b), the zone of extreme damage

a. floor framing on Line D between Lines 7 and 8

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

c. damaged framing at Levels 19 and 20

d. failed column immediately above Level 18
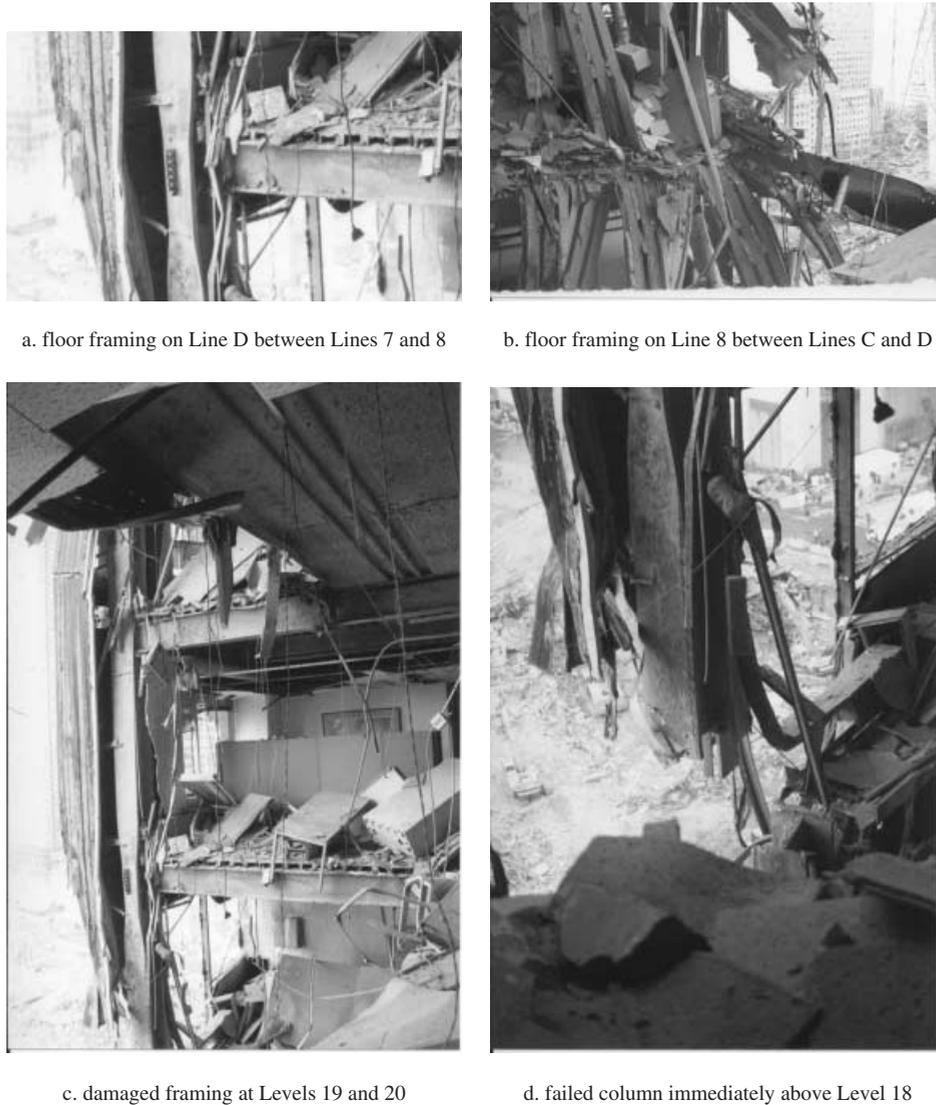
Figure 10. Interior damage photographs from Tenant Level 19

expanded below Tenant Level 16 to the two-bay by two-bay zone bounded by Lines C and E and 6 and 8.

Figure 13 shows two photographs of damage at Tenant Level 9. Figure 13(a) shows the complete destruction of one zone of the floor immediately adjacent to the northern face of the building. Part (b) of the figure shows the interior face of the section of Tower 2 façade seen in Figures 4(a) and 12(b) that caused much of the damage to the building.

Structural damage below the mechanical plant rooms that were located below Tenant Level 9 was modest and no photographs were taken at the levels between Tenant Level 9 and the entryway to the

a. view at Level 18   b. view at Level 17

Figure 11. Fractured column on Line D from Tenant Levels 18 and 17

building. Figure 14 is a photograph taken inside the building looking north towards a standing section of the Tower 2 façade.

3.   BUILDING ANALYSIS

3.1   Introduction

The observation that the subject building 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 non-linear analysis of buildings subjected to earthquake shaking were employed for these studies. Linear analysis was performed to determine demand-to-capacity ratios for an 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 the 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 non-linear analysis of the building, sizes of the beams and columns in the moment-resisting frame
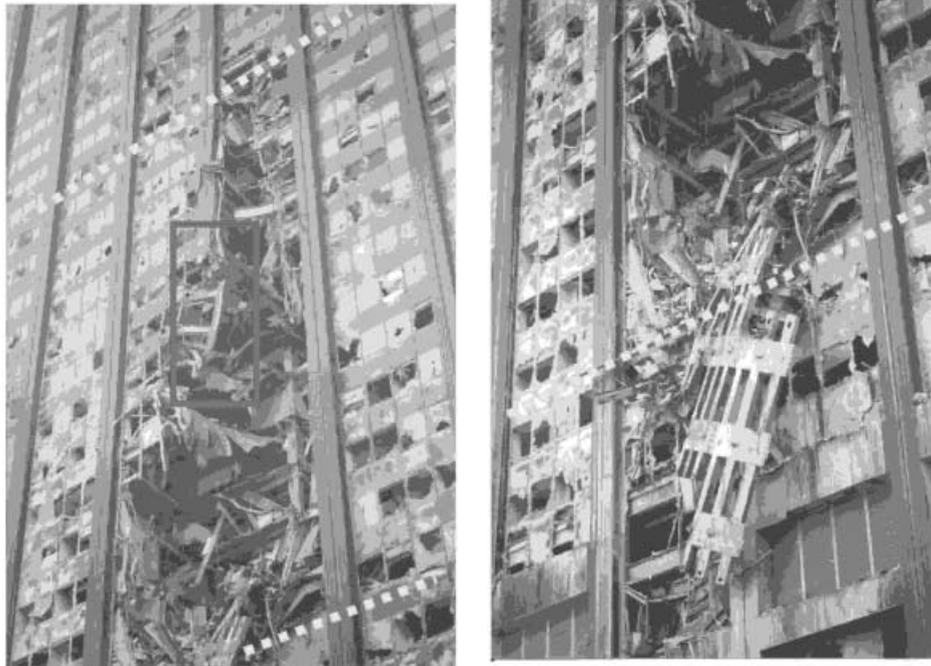
a. view of damage zone between Levels 23 and 10   b. view of damage zone adjacent to Level 10

Figure 12. Damage to the northern face of the building between Tenant Levels 23 and 10

a. complete destruction of one zone of the building   b. view of inside face of Tower 2 façade element

Figure 13. Building damage at Tenant Level 9

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 WF 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.

Figure 14. View of the Tower 2 façade looking north-west from the foyer of the building

3.2 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 reader should interpret the analysis results presented below 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 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 WF sections were then sized using the AISC Manual of Steel Construction, Allowable Stress Design (AISC, 1989).

To facilitate three-dimensional finite element analysis of the building, sections 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 Tenant Level 25 are shown in Figure 2 for one half of a typical floor plan.

3.3 *Linear elastic analysis*

Two- and three-dimensional finite element models were constructed using SAP2000 (CSI, 2000). First, a two-dimensional model (Figure 15a) was prepared that considered structural framing over all 39 stories along Line 8 (Figure 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 16). 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 light dashed line in Figure 16 identifies the zone of observed damage per Figure 15(b).

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 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 4(a). Each damage state involved the removal of columns on Line 8 from Tenant Level (TL) 7 to Tenant Level 25. The three damage states involved the removal of (1) the column on Line D (the observed damage per Figure 15b); (2) the columns on Lines D and E; and (3) the columns in Lines C, D, and E, denoted DS1, DS2, and DS3 respectively. Maximum member actions under gravity loads were calculated for the undamaged state and the three damage states.

Figures 17 and 18 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 Tenant Level 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_y is used in lieu of $\phi \cdot M_{nx}$ and each beam is assumed to be fully braced.
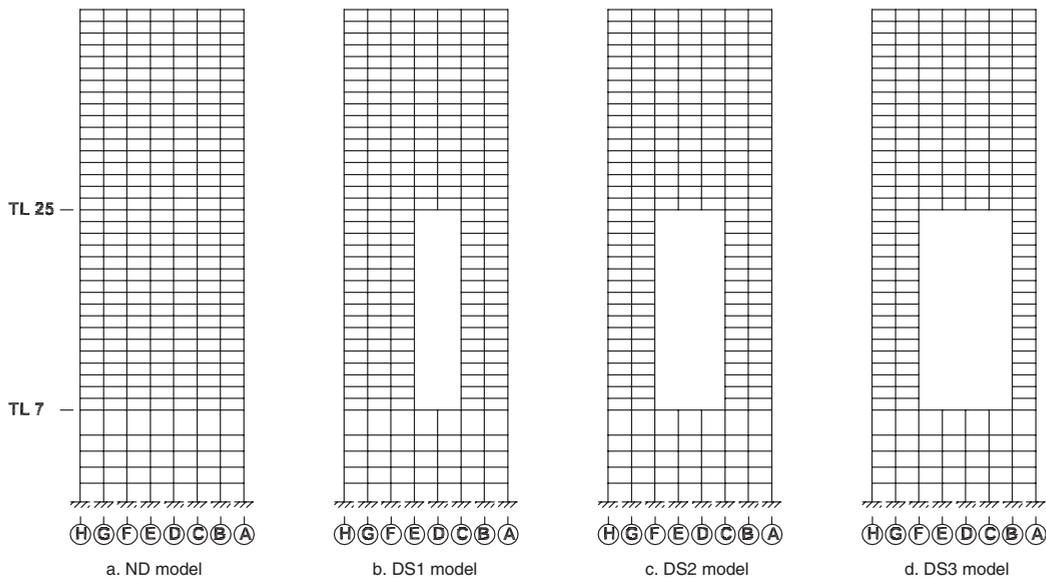
Figure 15. Two-dimensional mathematical models of framing on Line 8
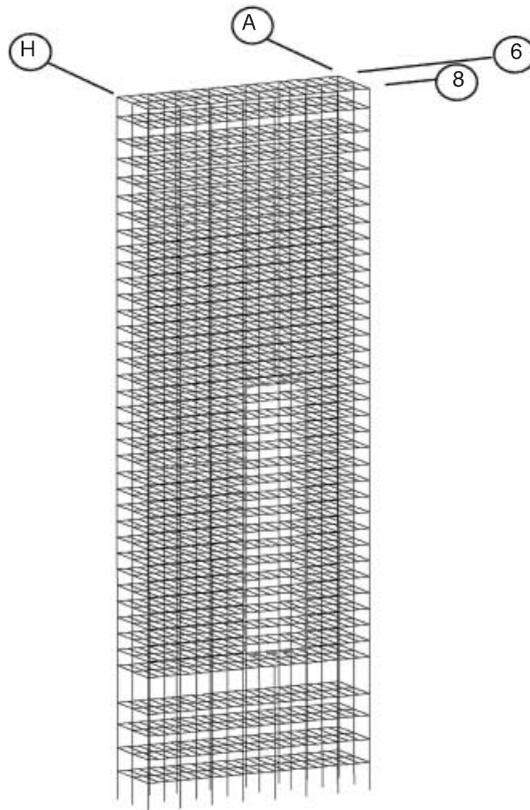
Figure 16. Three-dimensional mathematical model of framing on Lines 6, 7 and 8 for DS1
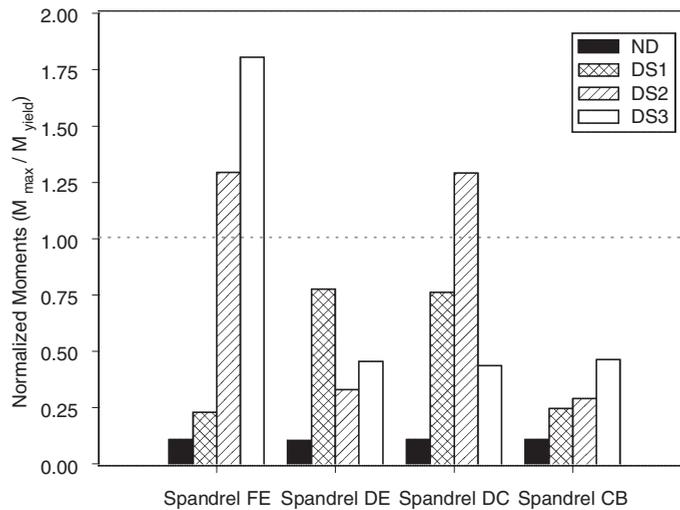
Figure 17. Demand-to-capacity ratios for two-dimensional linear elastic analysis
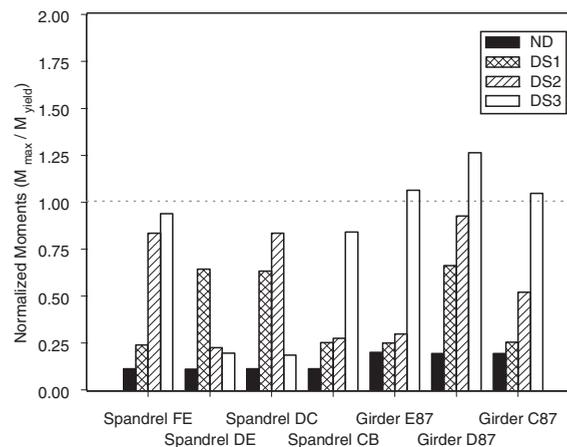
Figure 18. Demand-to-capacity ratios for three-dimensional linear elastic analysis

Consider first damage state DS1 (the observed damage) and the two-dimensional analysis. All D/C ratios are substantially less than 1. 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 Tower 2 and the loss of a column on Line D, namely, that the moment-resisting framing above Tenant Level 25 provided an alternate (redundant) path for gravity loads around Line D and to the foundation without distress of 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. However, for DS2 and DS3, Vierendeel truss action became more apparent.

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 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 18, however, shows that the use of the results of the two-dimensional analysis leads to conservative conclusions and that 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 Tenant Levels 24 and 25 (Figure 15) for both two- and three-dimensional analyses. The column designation (e.g., F8) in Figures 19 and 20 refers to the column between Tenant Levels 24 and 25 at the intersection of Lines F and 8. Shown in Figures 19 and 20 are column D/C ratios. These ratios were calculated using the 1998 edition of the *Manual of Steel Construction: Load and Resistance Factor Design* (AISC, 1998) nominal strength equation for members under combined forces (LRFD Eqn. H1-1a), namely,
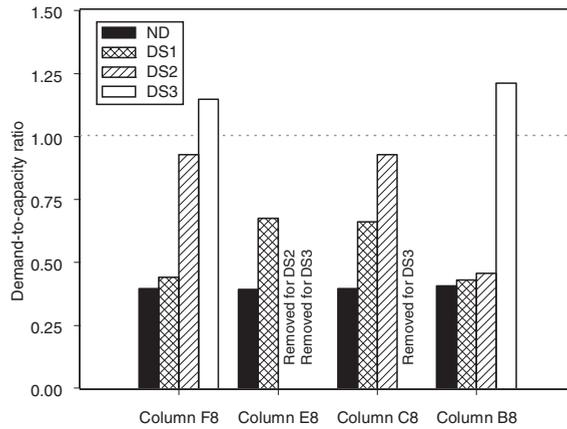
Figure 19. Demand-to-capacity ratios for two-dimensional linear elastic analysis
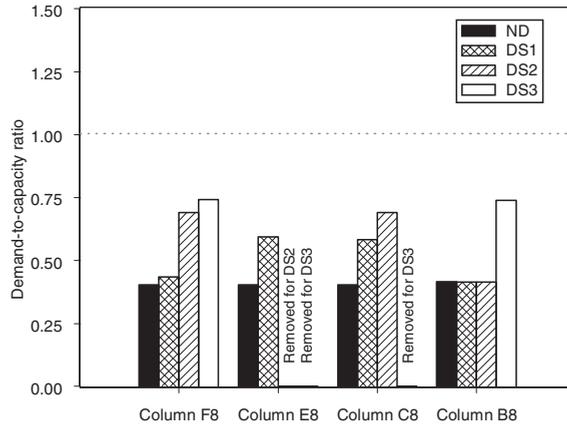
Figure 20. Demand-to-capacity ratios for three-dimensional linear elastic analysis

$$\frac{P_u}{\phi \cdot P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\phi b \cdot M_{nx}} + \frac{M_{uy}}{\phi b \cdot M_{ny}}\right) \leq 1 \cdot 0 \tag{1}$$

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 sidesway 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 19 indicate D/C ratios less than unity for the undamaged state ND and the 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 approxi-

mately $0.25M_p$ (plastic moment of 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 20) 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 loads to be redistributed to adjacent framing.

3.4   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 identified previously. 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 (Figure 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 21 shows the assumed mechanism for DS1. The assumed mechanisms for the three-dimensional analyses were similar, with the addition of hinges 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 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.
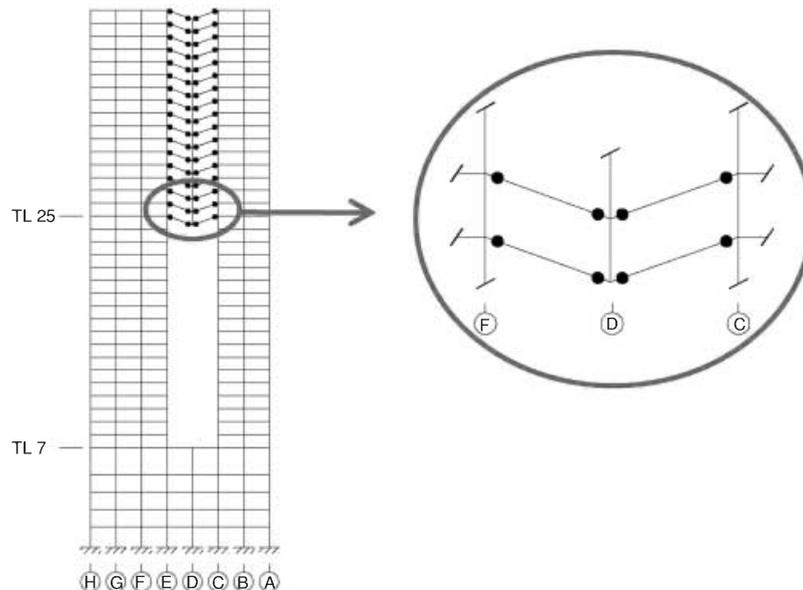
Figure 21. Two-dimensional collapse mechanism for DS1

Table 1. Plastic analysis results

Damage state	Two-dimensional analysis		Three-dimensional analysis	
	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

These analyses support the results of the elastic analysis described above, namely, that the framing system could have tolerated the loss of two columns without collapse. Note the additional load-carrying capacity that results from consideration of 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.

3.5 Non-linear static analysis

Two-dimensional non-linear 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 Tenant Level 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 non-linear 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 22 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 Tenant Level 25, progressed to the connections on Frame Line D, and then vertically up through the framing on Line 8.

Figure 23 shows the resulting relationships between floor load (psf) and the deflection of Line D at Tenant Level 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 increased structural flexibility and reduced the maximum permissible floor load. The results of the non-linear static analysis and those of the simple plastic analysis are in good agreement for all damage states.

4.   SUMMARY AND CONCLUSIONS

The 130 Liberty Plaza building sustained severe damage from falling debris during the collapse of the 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
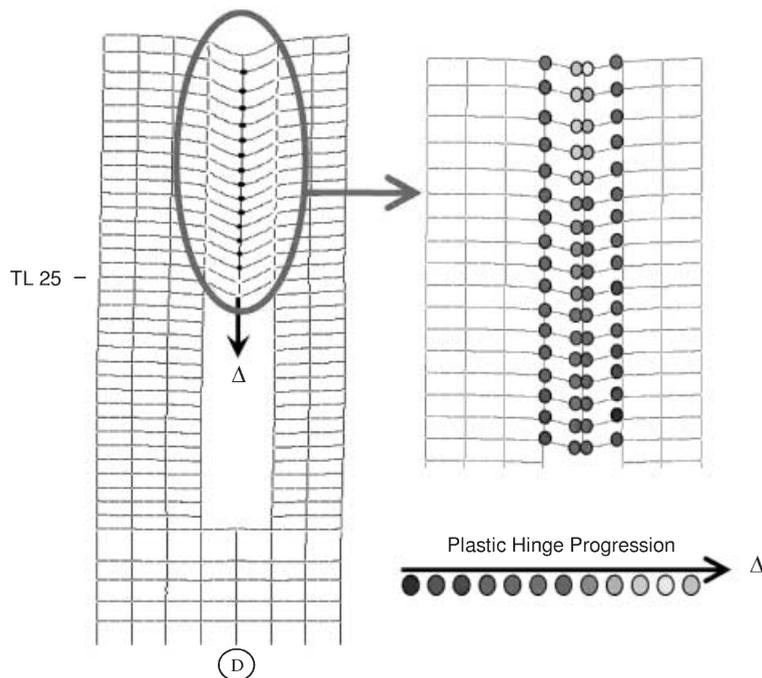
Figure 22. Progression of plastic hinge formation in DS1

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.
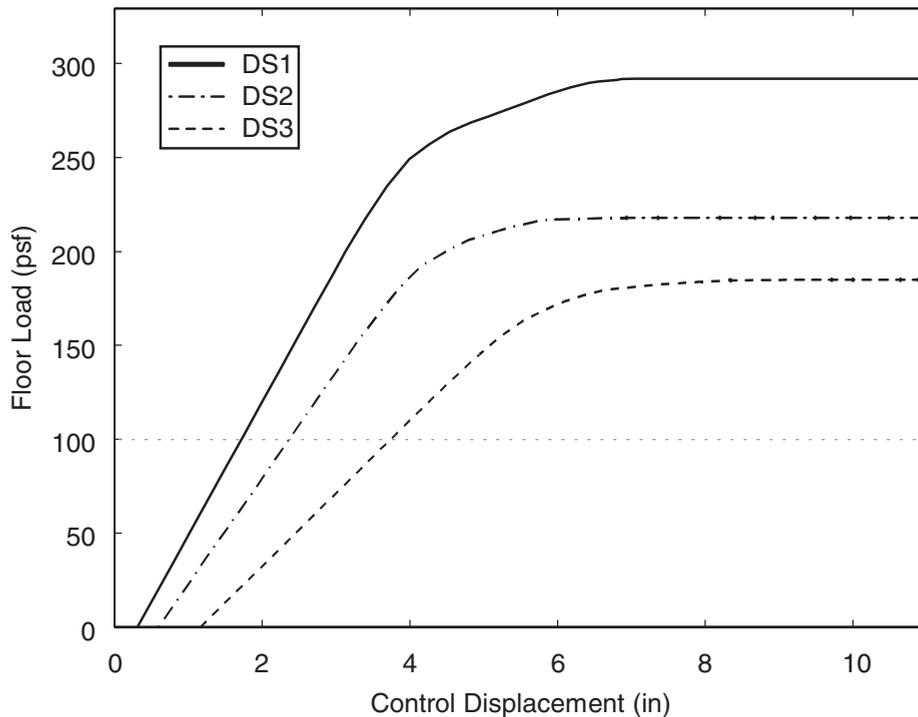
Figure 23. Two-dimensional pushdown curves for DS1, DS2, and DS3

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