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Analysis of a Damaged Building near Ground Zero

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Summary

An NSF-funded MCEER research team 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 subsequent analysis of a building frame with properties similar to those of the damaged building. Linear and nonlinear analyses were undertaken, however, only the linear analyses are presented here. Such analyses shows 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.

Introduction

The observation that the subject building did not collapse despite the loss of key structural elements and severe damage (Figure 1) motivated the research team to analyze the building to understand the cause of the observed behavior. Standard tools for 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 an undamaged state as well as for three damage states, one of which corresponds to the observed damage. Linear analysis also 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.

Detailed information on the structural framing system was not available to the research team, although approximate sizes were noted during the building reconnaissance. In order to facilitate 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 prescribed by the 1970 Building Code of the City of New York (BCCNY 1970). 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 2 shows the resulting section sizes and grid lines on floor 25 just above the damaged section of the building.

Figure 1. Building damage to northern façade and column on line D

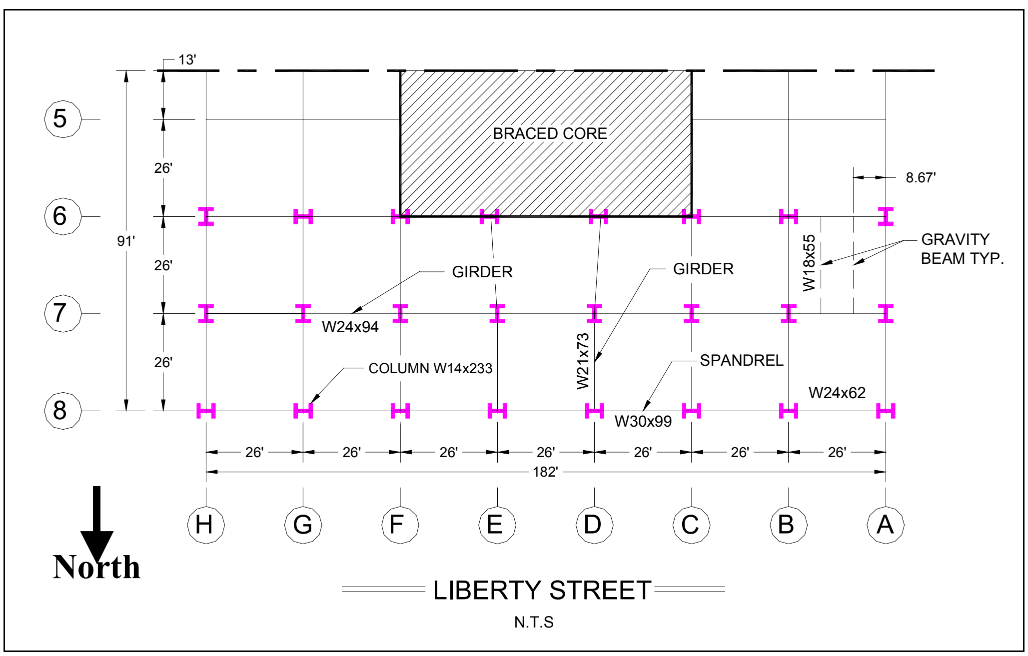


Figure 2. Framing on floor 25 and grid lines

Building Analysis

Two- and three-dimensional finite element models were constructed using SAP2000 (CSI 2000). Firstly, a two-dimensional model (Figure 3a) was prepared considering the 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). Secondly, a three-dimensional model was prepared considering the structural framing over all 39 stories along Lines 6, 7, and 8, including intermediate perpendicular framing. 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.

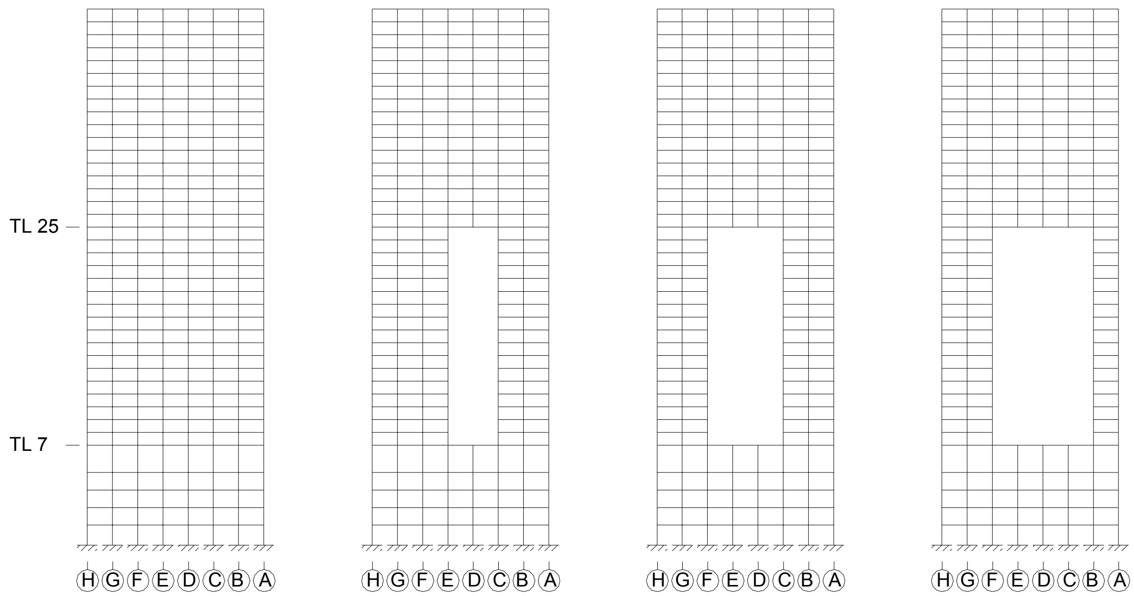
The analyses presented below considered only gravity loads with dead load and reduced live load equal to 50 psf each. A uniform distributed load of 260 lb/ft per story was assumed to account for the curtain wall loading. Mathematical models were analyzed for the undamaged state, denoted ND, and three damage states, one of which corresponds to the existing damage shown in Figure 1. Each damage state involved the removal of columns on Line 8 from Tenant Level 7 to Tenant Level 25. The three damage states involved the removal of: (1) the column on Line D (the observed damage per Figure 3b); (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 4 and 5 present some 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 ϕM_{nx} and each beam is assumed to be fully braced.

Consider first damage state DS1 (the observed damage) and the two-dimensional analysis. All D/C ratios are substantially less than unity. 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) show that the moment resisting framing above the damage provided an alternate path for gravity loads. However, for DS2 and DS3, Vierendeel truss action become 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. 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., they possess some degree of inelastic rotation



a) ND model b) DS1 model c) DS2 model d) DS3 model

Figure 3. Two dimensional model of framing on line 8

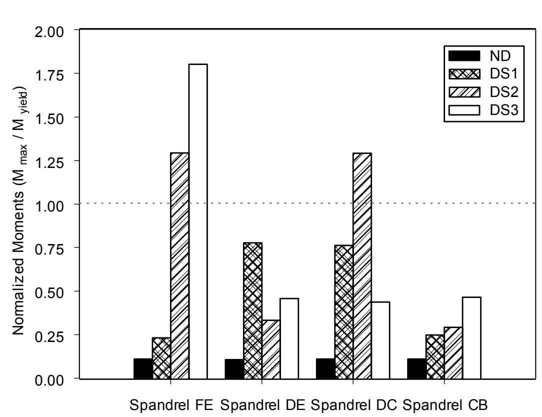


Figure 4. D/C ratios for 2-D analysis

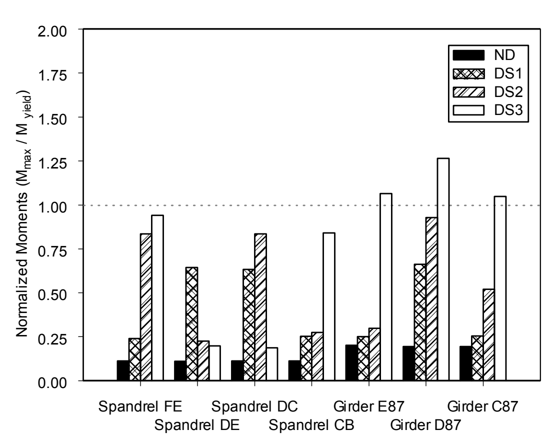


Figure 5. D/C ratios for 3-D analysis

capacity). Review of the three-dimensional analysis results of Figure 5, 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 3) for both two- and three-dimensional analyses. The column designation (e.g., F8) in Figures 6 and 7 refers to the column between Tenant Levels 24 and 25 at the intersection of Lines F and 8. Shown in Figures 6 and 7 are column D/C ratios. These ratios were calculated using the nominal strength equation for members under combined forces included in the 1998 Edition of the AISC Manual of Steel Construction, Load and Resistance Factor Design (AISC 1998). To facilitate calculation of D/C ratios and comparison of analysis results, the value of the effective length factor was assumed equal to 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 when considering the lateral stiffness of the braced core (significantly greater than that of the moment frame) and the presence of rigid floor diaphragms.

Results of the two-dimensional analysis shown in Figure 6 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 approximately 0.25 M_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 7) are less than unity for ND and DS1, DS2, and DS3. Again, results of the three-dimensional analysis 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.

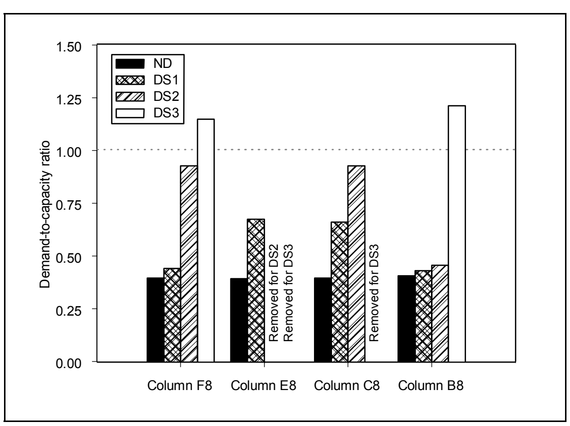


Figure 6. D/C ratios for 2-D analysis

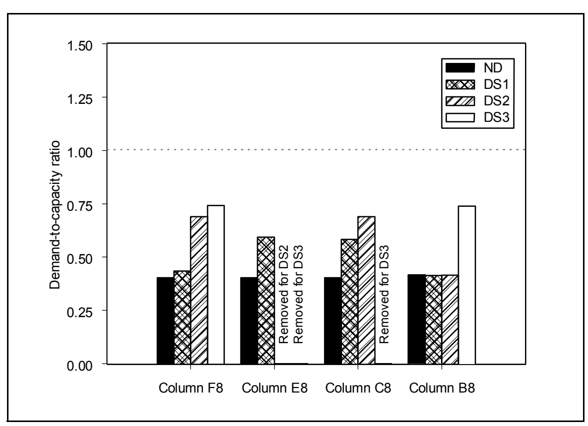


Figure 7. D/C ratios for 3-D analysis

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 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) improves the damage tolerance of buildings.

3. Simple framing systems such as unreinforced slabs on metal decking can span substantially further than what is assumed in design and such capabilities should be included in the evaluation of buildings for damage tolerance. Addition of inexpensive details (such as continuous slab reinforcement and continuity in the metal decking) further enhances building performance and prevents 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.

For a more complete report of the reconnaissance and analysis of this building, the reader is referred to Berman et al., (2002).

Acknowledgements

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