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PROGRESSIVE COLLAPSE OF BUILDINGS AND MULTI HAZARD MITIGATION

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ABSTRACT

Recent earthquakes have shown that building collapse frequently occurs vertically due to loss of critical vertical load carrying components such as columns or connections under cyclic lateral loads. The subsequent progressive collapse can be prevented if the gravity loads can be redistributed and seismic loads can be carried by the structural members neighboring the failed elements. Progressive collapse performance of two buildings was investigated through experimental testing and computational analysis. The five- and three-story steel frame buildings were located in Ohio and Illinois, and were scheduled for demolition. Four first-story perimeter columns were physically removed from each building. Progressive collapse resistance of buildings was investigated using a computer program following the General Services Administration (GSA) guidelines. The calculated response showed that the demand on several columns were larger than the limit specified in the GSA guidelines. The elastic static and nonlinear dynamic analysis results are presented and their implications are discussed. The research findings can be used to better understand the load redistribution within buildings after one or more axial load carrying elements are lost possibly during an earthquake.

Introduction

Progressive collapse of buildings is typically initiated by loss or failure of one or more vertical load carrying members like columns or connections. When one or a few columns fail, an alternative load path is needed to transfer the gravity load to other structural elements. If the neighboring elements cannot resist and redistribute the additional loading, failure will happen with further load redistribution until equilibrium is reached, sometimes after a substantial part of the structure collapses. Failure of one or more connections or columns in a building and the resulting progressive collapse may be a result of a variety of events with different loading rates, pressures or magnitudes. The magnitude and probability of man-made and natural hazards, including earthquakes, are usually difficult to predict. Therefore, most design standards adopt a non-threat specific approach to increase the overall structural integrity. Some of the requirements of the current progressive collapse design guidelines (e.g., GSA 2003) are similar to those of seismic codes and standards. Although many buildings experience progressive collapse during earthquakes, some buildings survived after the loss of one or more columns (Sezen et al. 2000). The ASCE-41 standard (2006) and FEMA 356 guidelines (2000) imply that an existing building would collapse if one of the primary load carrying components, e.g., a column, fails during an

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earthquake. This study involves simulation of sudden failure of columns in two actual buildings. The objective is to provide insight into understanding of progressive collapse mechanism and load redistribution within the buildings. Linear elastic and nonlinear dynamic analyses of the buildings were performed and demand-to-capacity ratios were calculated following the General Services Administration (GSA 2003) guidelines.

Description of Buildings

Ohio Student Union Building

The Ohio Student Union building was designed in 1949 and constructed in 1951. The building was scheduled for demolition in June, 2007 when field experiment was conducted. It was a four-story steel moment frame with eight and two bays in the longitudinal and transverse directions, respectively. Fig. 1 shows the building and four exposed columns before and after their removal. Experiment details can be found in Song and Sezen (2009) and Song et al. (2010).

(a) (b) (c)

Figure 1. Before and After Removal of Four First-Story Columns of the Ohio Union Building

Bankers Life and Casualty Company insurance building

The second test building was the Bankers Life and Casualty Company (BLCC) building located in Northbrook, Illinois. This building was constructed in 1968, following the sixth edition of AISC Steel Construction Manual (1967) design code, and was scheduled for demolition in August, 2008. The building had nine bays in the longitudinal direction, and eight bays in the transverse direction. Only north side of the building, a two-story steel frame structure, was considered in this study (Fig. 2).

Figure 2. Before and After Removal of Four First-Story Columns of the BLCC Building

Column Removal Process

For progressive collapse analysis, GSA (2003) requires several column loss scenarios. The GSA guidelines imply that immediate removal of an exterior column causes serious damage to the structural bays directly linked to the removed column or to an area of 1800 ft^2 at the floor level immediately above the removed column. Consistent with the GSA recommendations, we considered sequential removal of four first-story columns of each building in the following order: (1) two columns near the middle of the longitudinal perimeter frame, (2) column in the building corner, and (3) column next to the corner column (Figs. 1 and 2). Each test column was first torched or cut through its cross section near the top and bottom as shown in Fig. 3c. The column segment between the torched sections was then pulled out by a bulldozer (Fig. 3d). The column was removed within a very short time period to simulate more realistic instantaneous removal of a column, as recommended in the design guidelines.

Several strain gauges were installed on the columns and beams closely linked to the removed columns of the Ohio Union building and the BLCC building, respectively (Fig. 2a). During the column removal process, a portable data acquisition system recorded strain measurements to monitor the change in the axial forces and deflections (Fig. 2b).

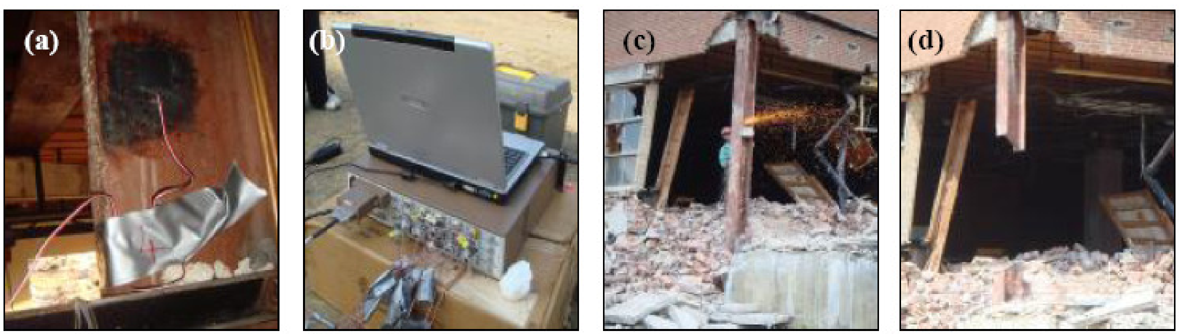

Figure 3 - Building Tests: (a) Strain Gauge Attached on a Steel Beam, (b) Portable Data Acquisition System, (c) Torching of the Corner Column, and (d) Corner Column Removed

Building Models and Analysis

Progressive collapse performance of the Ohio Student Union building and the BLCC building was investigated using the computer program SAP2000. The longitudinal edge frame of each building was modeled as a 2-D frame. Linear static analysis of both buildings and nonlinear dynamic analyses of the Ohio Union building were carried out. The recommendations of GSA guidelines were followed to evaluate the progressive collapse potential of the buildings.

At the time of testing, the building carried dead loads from walls, slabs, beams, and columns. In the linear static analysis, the dead loads were multiplied by 2.0 as recommended in the GSA guidelines (2003), which provides justification for this load amplification. Live load was assumed to be zero because test buildings were not occupied, and most partitions, furniture and other non-structural loads were removed from the buildings. To calculate the dead load of the walls, the assumed densities of glass and brick were 160 lb/ft^3 and 120 lb/ft^3, respectively. The weight and properties of slab and frame members were obtained from the original structural drawings and design notes of each building. Fig. 4 shows SAP2000 models of the Ohio Union and BLCC buildings. Column removal order for both buildings is also shown in Fig. 4.

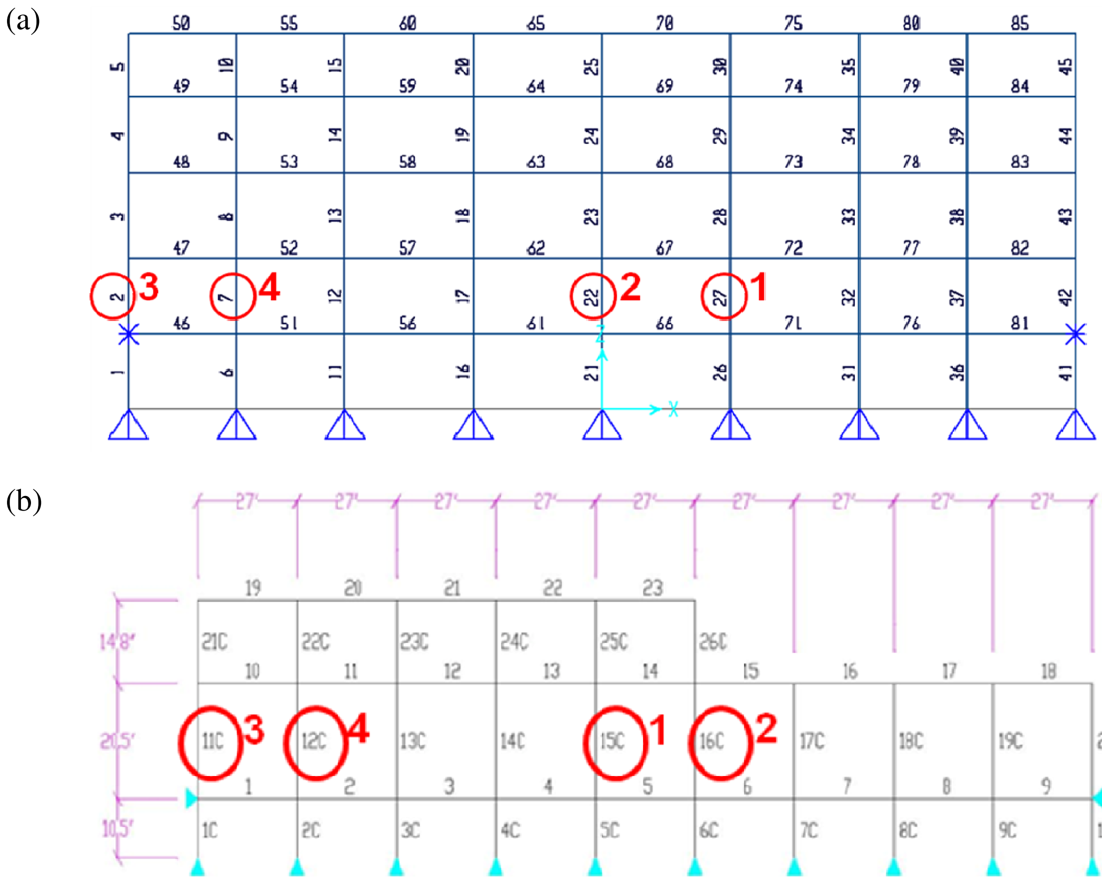

Figure 4 - Two-Dimensional Model for the Longitudinal Perimeter Frames of (a) Ohio Union Building and (b) BLCC Building (Circled Columns were Removed in the Order Shown)

Ohio Union Building

Linear Static Procedure

Linear static analysis was performed because it is a simpler and commonly used method to investigate progressive collapse potential of a building (ASCE-7, 2005). The results were evaluated by comparing the demand-to-capacity ratios (DCR) based on the GSA guidelines. DCR for moment is defined as the ratio of the maximum moment, M_{max} of the beam or column calculated from linear elastic analysis to its ultimate moment capacity, M_p. M_p is calculated as the product of plastic section modulus and yield strength. In M_p calculations for columns, the effect of the axial load is neglected in this study because the column axial loads were relatively small and did not affect the moment capacity of the cross section significantly.

$$DCR = \frac{M_{\max}}{M_p} \tag{1}$$

DCR values were calculated for frame members for each column removal scenario. In general, the DCR values increase with increasing number of removed columns. DCR values for most members were less than 2.0, which is the specified upper limit for regular structural configurations (Section 4.1.2.3.1 of GSA 2003). It is, therefore, concluded that the Ohio Student Union building does satisfy the GSA 2003 progressive collapse criteria for most frame members.

Columns were impacted more than beams when four columns were removed from the frame. After all four columns were removed, no beams and only five columns (columns 8, 9, 10, 20 and 25) exceeded the DCR limit of 2.0.

Fig. 5 shows the elastic moment diagrams after the removal of each column. When the first two columns were removed, the largest bending moments were localized and occurred in the members above or immediately next to the removed columns. Maximum moments significantly increased and spread within the frame when three and four columns were removed. After four columns were removed, the structure was much more susceptible to progressive collapse, which was also reflected in the maximum displacements calculated by linear static analysis. As columns were sequentially removed, the maximum vertical displacements were calculated as 1.41, 3.70, 3.70 and 10.0 inches at the joints above columns 27, 22, 2 and 7, respectively.

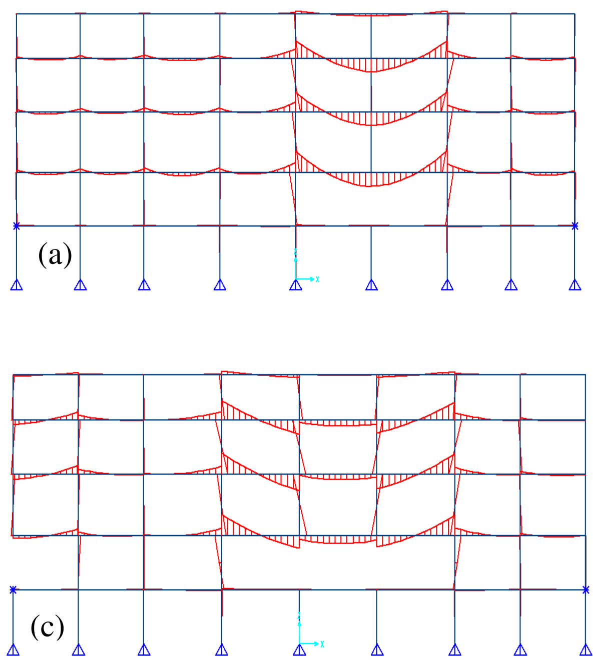

Figure 5 – Elastic Moment Diagrams After the Removal of: (a) One Column, (b) Two Columns, (c) Three Columns, and (d) Four Columns

Fig. 6 shows the moment diagram and corresponding DCR values at the top of each column and beam ends after four columns were removed from the frame. The columns in the top story show higher DCR values, mainly as a result of smaller cross section used in those columns. The maximum DCR value (2.83) was calculated for Column 10 in the top story. The maximum DCR value calculated in beams was 0.94 in Beam 63 in the third floor level.

Fig. 7 shows DCR values for each frame member for all cases. Frame member numbers up to 45 are columns, and beams are numbered from 46 to 85 (Fig. 4). After the first column was removed, DCR values for all columns and beams were below 0.5. The DCR values after the loss of the second column was similar to those of the third column loss, all of which were less than 1.5. The DCR values for columns were remarkably increased after the fourth column was lost. There were five columns with DCR values exceeding the acceptance criteria in this case. However, the change in DCR values for beams was not significant, compared with that of columns. The DCR values of beams were always less than 1.0. This is probably due to potential redistribution of loads to the adjacent beams in the analyzed frame.

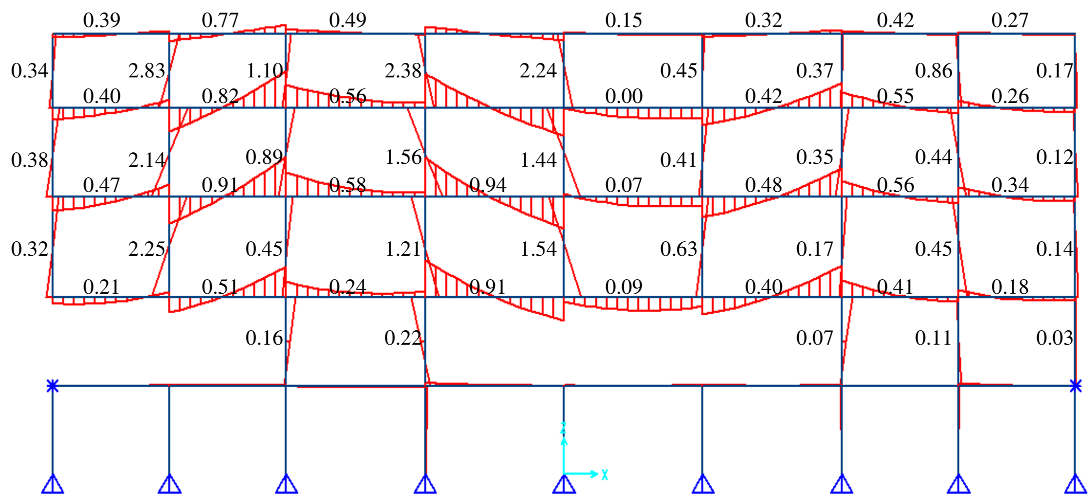

Figure 6 - DCR Values and Moment Diagram after Loss of Four Columns

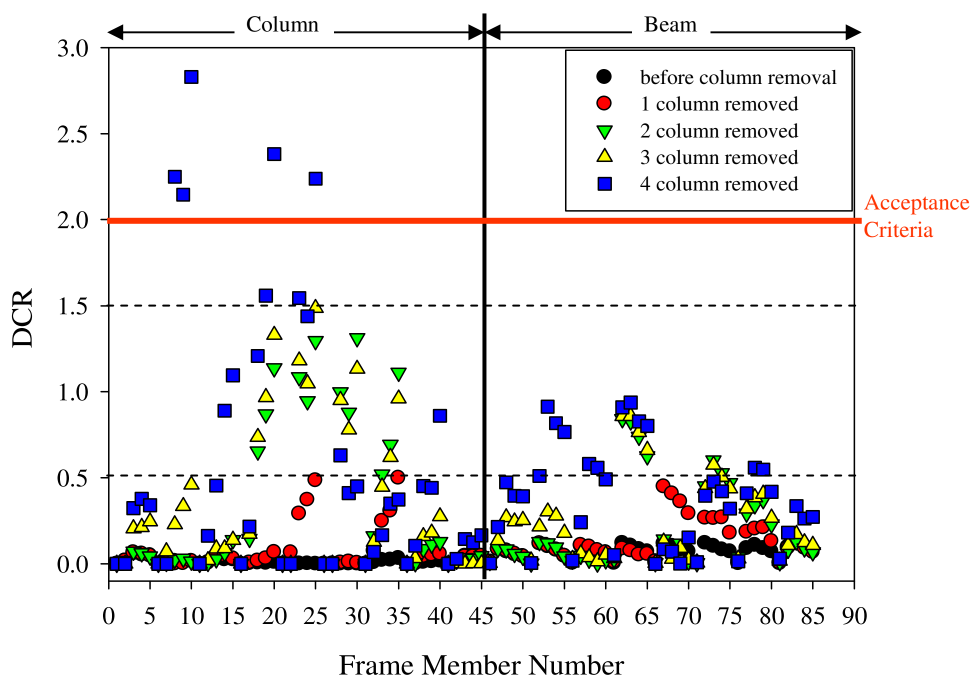

Figure 7 - Changes in DCR Values of Each Frame Member for All Cases

Nonlinear Dynamic Procedure

Nonlinear dynamic analysis provides more detailed information for the progressive collapse potential. Fig. 8 illustrates the column removal application in the nonlinear dynamic analysis procedure. First, the building is modeled with its dead load assigned. After the equivalent forces for the column are determined, the removed column is replaced with its equivalent forces to simulate the instantaneous removal of that column. As shown in Fig. 9, the equivalent load was first assigned with a uniform time history function, and then removed over time using a time history function. The sum of a uniform time history function and the column loss function represents the sudden column loss.

After the equivalent column loads were removed, the building was allowed to deform until it settled. The dynamic displacement history of the joint above each removed column was calculated. Fig. 10 shows the displacement of the joint above the first removed column (Joint 1). This joint settled down in 0.5 seconds at a displacement of 0.09 in. After all columns were removed (Fig. 11), all four joints settled in 5 seconds. It took more time for the structure to settle as it became more susceptible to progressive collapse. Joints 1 through 4 (joints above the first through fourth removed columns, i.e., above columns 27, 22, 2 and 7, respectively) settled at 0.85, 0.76, 1.04 and 0.52 inches, respectively. These joint displacements are smaller than those calculated from linear static analysis (2.16, 2.18, 6.19, and 3.66 in., respectively).

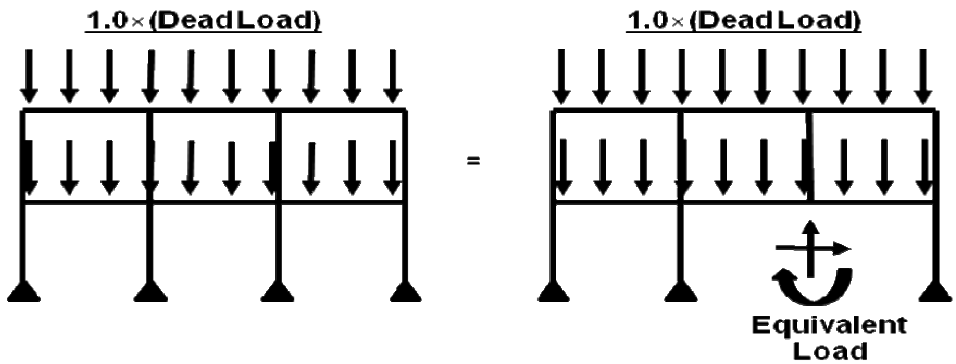

Figure 8 – Column Removal Load Representation for Nonlinear Dynamic Analysis

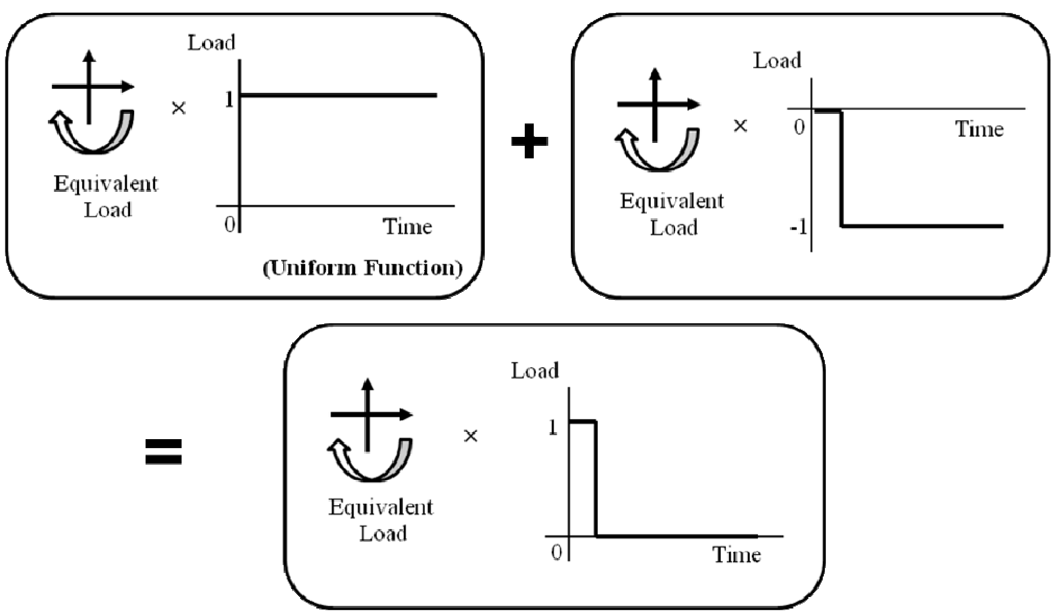

Figure 9 – Time History Function for Column Loss Simulation

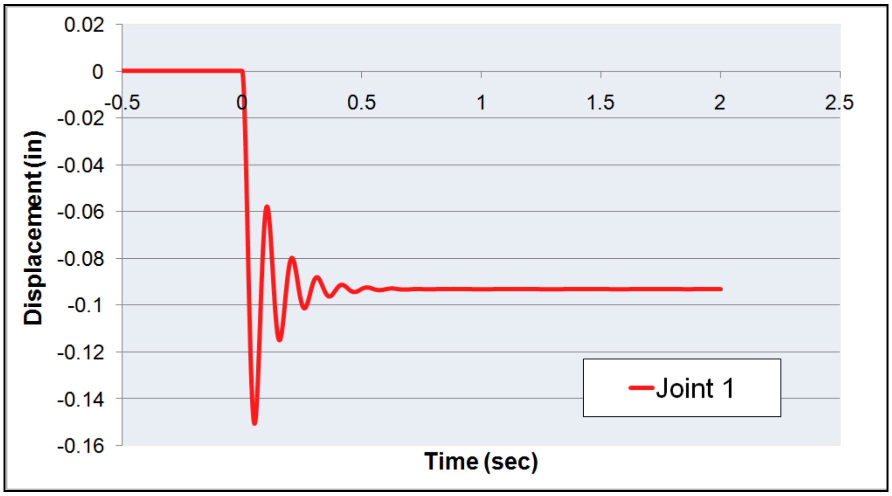

Figure 10 –Displacement above the First Removed Column (Joint 1) after First Column Removal

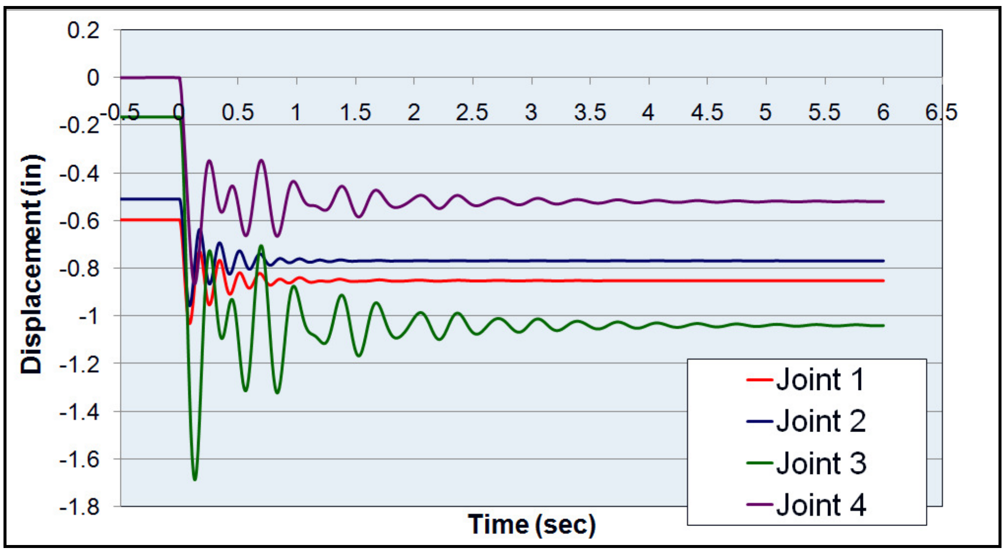

Figure 11 - Displacement of Joints above Each Removed Column after Removal of All Columns

Bankers Life and Casualty Company Insurance Building

Linear Static Procedure

Linear static analysis of 2-D SAP 2000 model was performed after each column removal. Fig. 12 shows the bending moment diagrams and DCR values for the exterior frame of the BLCC building after each column removal. Even after the first column removal, the DCR values have exceeded the acceptance criteria (2.0). After the fourth column removal, the DCR values for columns were remarkably increased. The DCR values for four columns exceeded 10 (Fig. 12d).

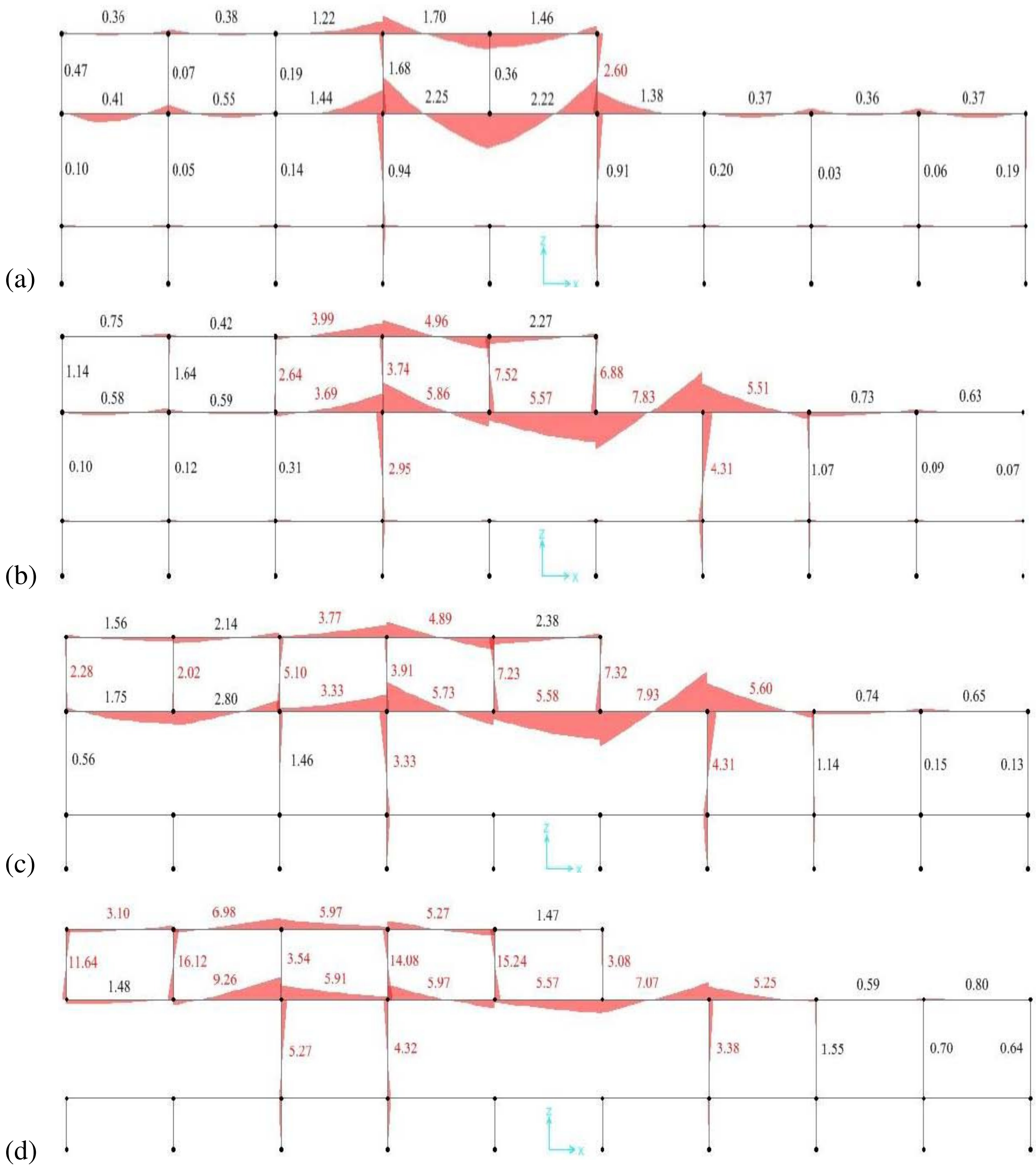

Figure 12 - Calculated Bending Moment Diagrams and DCR Values after the Removal of: (a) First Column, (b) Second Column, (c) Third Column, and (d) Fourth column

Conclusions

An experimental and analytical progressive collapse research was conducted on two actual steel buildings scheduled for demolition. As part of the field experiment, four first-story columns were sequentially removed from each frame building and the buildings did not collapse. At the time of testing, the building did not carry any live load.

The progressive collapse analyses were conducted using the computer program SAP2000. Linear static analysis of both buildings suggests that the columns in the top story are most significantly influenced by the column loss, likely due to smaller cross section and lower moment of inertia. The Ohio Union structure did satisfy the GSA (2003) progressive collapse criteria for all frame members, except for five columns. However, the BLCC building did not satisfy the GSA criteria even after the first column removal. The DCR values were excessively large up to 16.1 (Fig. 12d). The calculated DCR values and maximum displacements showed that the buildings became most susceptible to progressive collapse after the fourth column was removed. The beams were less impacted than columns.

Nonlinear dynamic analyses of the Ohio Union building were performed by replacing the removed columns with equivalent reactions. To simulate the column loss a time history function was used. The nonlinear dynamic analysis resulted in smaller displacements than linear static analyses. Time to stabilization increased as each column is removed, indicating that the structure is becoming more susceptible to progressive collapse.

References

AISC, 1969. Manual of Steel Construction. 6th Edition, American Institute of Steel Construction.

ASCE-7, 2005. Minimum Design Loads for Buildings and Other Structures. American Society of Civil Engineers, Reston, VA.

ASCE-41, 2006. Seismic Rehabilitation of Existing Buildings. American Society of Civil Engineers, Reston, VA.

Byfield, M.P., 2006. Behavior and Design of Commercial Multistory Buildings Subjected to Blast. *ASCE Journal of Performance of Constructed Facilities*, 20 (4), 324-329.

GSA, 2003. Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects. General Services Administration, Washington, D.C.

Marjanishvili, S., and Agnew, E. 2006. Comparison of Various Procedures for Progressive Collapse Analysis. *ASCE Journal of Performance of Constructed Facilities*, 20 (4), 365-374.

NIST, 2005. The Collapse of the World Trade Center Towers. Final Report. National Institute of Standards and Technology (NIST), Gaithersburg, MD.

Sezen H., Elwood K. J., Whittaker A. S., Mosalam K. M., Wallace J. W., and Stanton J. F. 2000. Structural Engineering Reconnaissance of the August 17, 1999 Kocaeli (Izmit), Turkey Earthquake. PEER 2000/09. Technical Report. Pacific Earthquake Engineering Research Center, University of California, Berkeley. pp. 120

Song B., Giriunas K., and Sezen H. 2010. Experimental and Analytical Assessment of Progressive Collapse of Steel Frame Buildings. *ASCE Structures Conference/North American Steel Construction Conference, American Society of Civil Engineers, Orlanda, Florida* May12-15, 2010.

Song B., and Sezen H. 2009. Evaluation of an Existing Steel Frame Building against Progressive Collapse. *ASCE Structures Conference, American Society of Civil Engineers. Austin, Texas, April 30-May 2, 2009.*