Experimental and computational assessment of robustness of steel and reinforced concrete framed buildings

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ABSTRACT: This paper presents an experimental and computational assessment of the performance of full-scale steel and reinforced concrete beam-column assemblies under vertical displacement of a center column, simulating a column removal scenario. The assemblies represent portions of structural framing systems designed as Intermediate Moment Frames (IMFs) for Seismic Design Category C. The steel assembly was designed and detailed in accordance with ANSI/AISC 341-02, using prequalified moment connections specified in FEMA 350, and the concrete assembly was designed and detailed in accordance with ACI 318-02 requirements. Each assembly consists of two beam spans and three columns, and a downward displacement of the center column is imposed until a failure mechanism is formed. The study provides insight into the behavior and failure modes of the assemblies, including the development of catenary action. Both detailed and reduced finite element models are developed to capture the primary response characteristics and failure modes. The reduced models can be analyzed rapidly without loss of accuracy and are implemented in analyses of entire structural systems to assess their vulnerability to disproportionate collapse.

1 INTRODUCTION

Since the destruction of the Alfred P. Murrah Federal Building in 1995, caused by a truck bomb attack (FEMA 1996), and the collapse of the World Trade Center towers in 2001, caused by the impact of large passenger jetliners (NIST 2005), the engineering community, including codes and standards development organizations and public regulatory agencies, has paid greater attention to the performance of buildings subjected to damage from abnormal events. In the U.S., the American Society of Civil Engineers Standard 7 (ASCE 2010, Section C1.4), and the guidelines of the U.S. General Services Administration (GSA 2003) and the Department of Defense (DOD 2009) provide guidance to prevent disproportionate collapse (also known as progressive collapse). Disproportionate collapse occurs when an initial local failure spreads progressively, resulting in total collapse or collapse of a disproportionately large part of a structure. Resistance to disproportionate collapse is achieved either implicitly, by providing minimum levels of strength, continuity, and ductility; or explicitly, by (1) providing alternate load paths so that local damage is absorbed and major collapse is averted or (2) providing sufficient strength to structural members that are critical to global stability.

In the alternate path method, structural integrity is assessed through analysis, to ascertain whether the structural system can bridge over failed structural members. For example, if a column is damaged, continuity of the beams adjacent to the top of the damaged column is required to redistribute the loads formerly carried by the damaged column. The analysis must demonstrate the adequacy of the beams and their connections to redistribute these loads, potentially through catenary action. An accurate characterization of the nonlinear, large-deformation behavior associated with the transfer of forces through the connections in such scenarios is critical in assessing the potential for disproportionate collapse. Physical tests are indispensible in validating the analytical models used to represent nonlinear connection behavior in such scenarios.

This paper describes both full-scale testing and finite element-based modeling of steel and reinforced concrete beam-column assemblies. Each assembly comprises three columns and two beams, representing a portion of the second floor framing of a prototype ten-story building. Both assemblies represent portions of intermediate moment frames (IMFs) designed for Seismic Design Category C (SDC C), typical of the Atlanta, Georgia area.

The beam-column assemblies are subjected to monotonically increasing vertical displacement of the unsupported center column to observe their behavior under a simulated column removal scenario, including the development of catenary action in the beams. Each test is continued until a collapse mechanism of the assembly is reached. Both detailed and reduced finite element models of the test specimens are developed, and the model predictions show good agreement with the experimental results, providing validation of the modeling approaches. The tests and associated computational models help fill the gap in defining the response characteristics of the moment-resisting connections under collapse scenarios and contribute to establishing a library of validated connection models for use in collapse analysis.

The reduced models are used to analyze complete structural systems under column removal scenarios, and results are presented from both dynamic column removal and pushdown analyses. Such modeling capabilities are valuable for assessing the vulnerability of structural systems to disproportionate collapse, and evaluating the risk of collapse for structures exposed to abnormal loads.

2 DESCRIPTION OF BUILDING DESIGNS

A rectangular plan of 30.5 m by 45.7 m was chosen for all prototype buildings. The steel prototype building was designed and detailed in accordance with the American Institute of Steel Construction Seismic Provisions (AISC 2002). The building used IMFs at the perimeter for the lateral-load resisting system and gravity frames on the interior. Welded unreinforced flange-bolted web (WUF-B) connections were used in the moment frames, selected from prequalified steel connections specified in FEMA 350 (FEMA 2000a). ASTM A992 structural steel (F_y = 345 MPa) was used in all beams and columns. ASTM A36 steel (F_y = 248 MPa) was used for the shear tabs and continuity plates at the beam-to-column joints. ASTM A490 high strength bolts were used for the bolted connections, and welding requirements followed the recommendations in FEMA 353 (FEMA 2000b).

The concrete prototype building was designed and detailed in accordance with the American Concrete Institute's Building Code Requirements for Structural Concrete (ACI 2002). The building used IMFs for the lateral-load resisting system. The building was designed using normal weight concrete having a specified compressive strength of 27.6 MPa and reinforcing steel of ASTM A615 having a minimum specified yield strength of 414 MPa.

3 STEEL BEAM-COLUMN ASSEMBLY

3.1 *Test specimen and setup*

The steel test assembly comprised two W21x73 beams connected to three W18x119 columns by WUF-B connections. The span length (center to center of columns) of the beams was 6.10 m. The WUF-B connection is similar to the connection commonly used prior to the 1994 Northridge earthquake. FEMA 355D (FEMA 2000c) provides extensive information on testing and performance of WUF-B connections under cyclic loading. Details of the WUF-B connection used in the test are shown in Figure 1. The beam web is connected to the column flange using a shear plate (shear tab), which is welded to the column using an 8 mm fillet weld and bolted to the beam web using three 25 mm diameter ASTM A490 bolts. The bolt holes are standard holes with an edge distance of 70 mm. The beam flanges are joined to the column flange using complete joint penetration (CJP) groove welds. Weld access holes are cut from the beam webs according to the recommendations of FEMA 350 (FEMA 2000a). Continuity plates are provided for both interior and exterior columns as shown in Figure 1. No doubler plates were required.

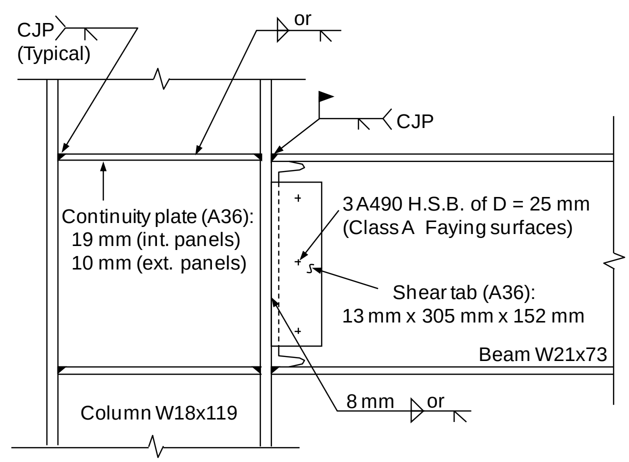

Figure 1. WUF-B connection details for the steel assembly.

A schematic of the test setup for the steel beam-column assembly is shown in Figure 2. Figure 3 shows photographs of the steel assembly, including a close-up view of the connections to the center column.

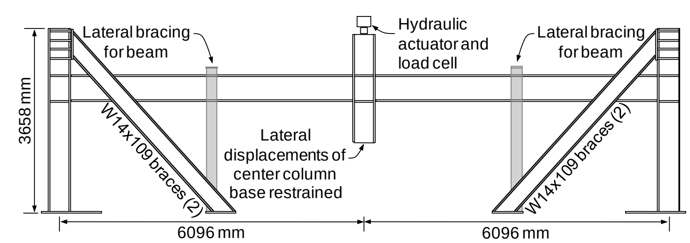

Figure 2. Test setup for the steel assembly.

Figure 3. Photographs of the steel assembly test.

The bases of the exterior columns were anchored to the strong floor of the test facility, and the tops were rigidly attached to two diagonal braces to restrain horizontal movement. The base of the center stub column was unrestrained vertically, but out-of-plane movement was restrained. In addition, the beams were restrained from out-of-plane movement at mid-span by lateral bracing. A hydraulic actuator with a capacity of 2670 kN and stroke of 508 mm was attached to the top of the center column to apply a vertical load to the assembly. Load was applied under displacement control at a rate of 25 mm/min. The estimated uncertainty in the measured data from the load cells, deflection gages, strain gages, and inclinometers was ±1 %. More details on the testing setup and instrumentation are provided in Sadek et al. (2010).

3.2 *Test results*

Under prescribed vertical displacement of the center column, the assembly experienced large deflections and rotations prior to failure. The connection failed at a vertical displacement of the center column of about 495 mm. At that displacement, the applied vertical load was about 890 kN. The connection failed in the following sequence (see Figure 4): (1) local buckling of the top flanges of the beams at the center column, (2) successive shear fractures of the lowermost and middle bolts connecting the beam web to the shear tab, and (3) fracture of the bottom flange near the weld access hole.

Figure 4. Failure mode of the steel assembly.

Plots of the applied vertical load versus vertical displacement of the center column and the beam axial force versus the vertical displacement of the center column are shown in Figure 5. Experimental measurements are presented along with finite element model predictions that will be discussed subsequently. The experimental beam axial force is calculated based on measured strains in the beams. As the plots indicate, the assembly was unloaded at a vertical displacement of about 460 mm, to adjust the stroke of the hydraulic ram, and was then reloaded to failure. Figure 5 indicates that the assembly remained in the elastic range up to a vertical displacement of the center column of about 50 mm. In the early stages of loading, the behavior of the assembly was primarily flexural. As the loading progressed with increased vertical displacement of the center column, the response of the assembly was dominated by catenary action, as indicated by the development of axial tension in the beams. At the time of failure, the axial tension in the beams was about 667 kN.

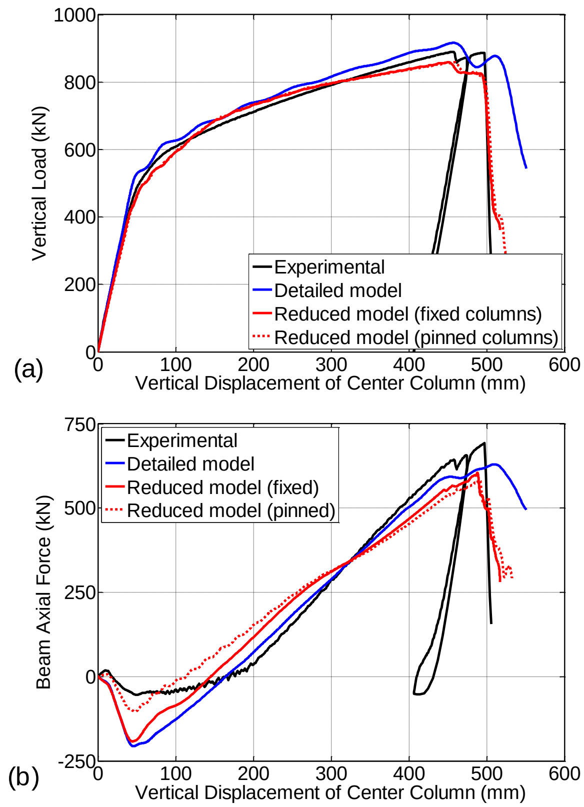

Figure 5. (a) Vertical load and (b) beam axial force versus vertical displacement of center column for the steel assembly (measurement uncertainty: ±1 %).

3.3 *Finite element models*

Two finite element models of the beam-column assembly with WUF-B connections were developed to study the behavior of the connections and to compare the calculated response with experimental values. The first was a detailed model of the assembly with approximately 300 000 elements, while the second was a reduced model with about 150 elements. The analyses were conducted using LS-DYNA, an explicit formulation, finite element software package (Hallquist 2007). Overviews of both models are shown in Figure 6.

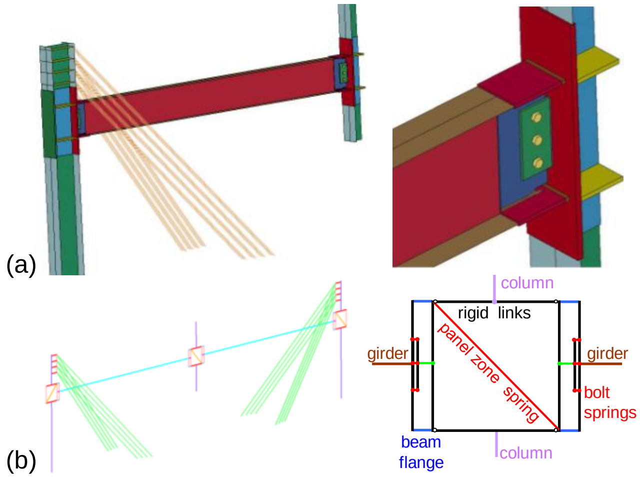

Figure 6. (a) Detailed and (b) reduced models of the steel assembly.

The detailed model, shown in Figure 6(a), consisted of finely meshed solid elements representing the beams, columns, continuity plates, shear tabs, bolts, and welds in the vicinity of the connection. Contact with friction was defined between the bolts, shear tabs, and beam webs to model the transfer of forces through the bolted connection. Away from the connection zones, the beams and columns were modeled with shell elements. Spring elements were used to model the braces at the top of the exterior columns. All nodes were fixed at the bases of the exterior columns. In order to reduce the time required for computation, only half of the assembly was modeled, with appropriate boundary conditions enforced along the plane of symmetry. The steel for the various elements was modeled using a piecewise-linear plasticity model based on coupon test data obtained for all steel sections and plates. Fracture was modeled using element erosion.

The reduced model used Hughes-Liu beam elements (Hallquist 2007) to model the beams and columns, as well as the shear tabs and beam flanges in the connection regions. The steel was modeled using a piecewise-linear plasticity model based on coupon test data, and fracture was modeled using element erosion. An arrangement of beam and spring elements, connected with rigid links, was used to model the WUF-B connection as shown in Figure 6(b). Nonlinear spring elements were used to model the shear behavior of the bolts, along with bearing-induced deformations of the shear tab and beam web. The shear load-deformation curve for these spring elements was based on the results of a detailed solid-element model of the bolted connection (Sadek et al. 2010). Spring elements were also used to model the diagonal braces and the shear behavior of the panel zone. For the panel zone, the diagonal springs had an elasto-plastic load deformation curve based on the geometry and strength of the panel zone. Further details are provided in Sadek et al. (2010). Two analyses were conducted in which the bases of the exterior columns were modeled as either fixed or pinned.

Based on the analysis of the detailed model, the beam-column assembly responded initially in a purely flexural mode before catenary action developed. The beam remained essentially elastic except for the sections in the vicinity of the connections to the center and exterior columns, where significant yielding was observed. The failure mode of the connection based on this analysis, shown in Figure 7, was very similar to that observed in the experiment (Figure 4). The results from the reduced model were consistent with those from the detailed model, albeit without the same level of detail.

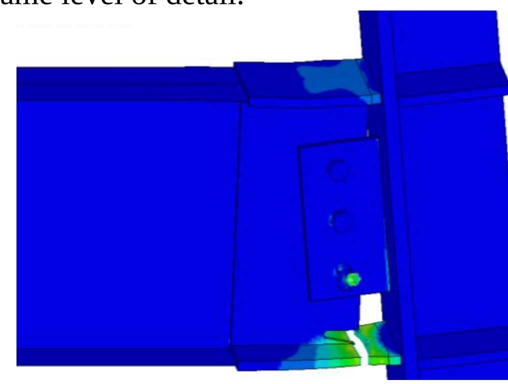
Figure 7. Failure mode from detailed model of the steel assembly.

In Figure 5 comparisons are presented between the finite element model predictions and the experimental measurements of the vertical load and the beam axial force, plotted against the vertical displacement of the center column. The plots indicate a good agreement between the experimental and computational results and provide validation for the detailed and reduced models.

4 REINFORCED CONCRETE BEAM-COLUMN ASSEMBLY

4.1 *Test Specimen and Setup*

The reinforced concrete test assembly for the IMF frames comprised two 711 mm by 508 mm beams supported by three 711 mm by 711 mm columns as shown in Figure 8. The span length of the beams (center-to-center of columns) was 6.10 m.

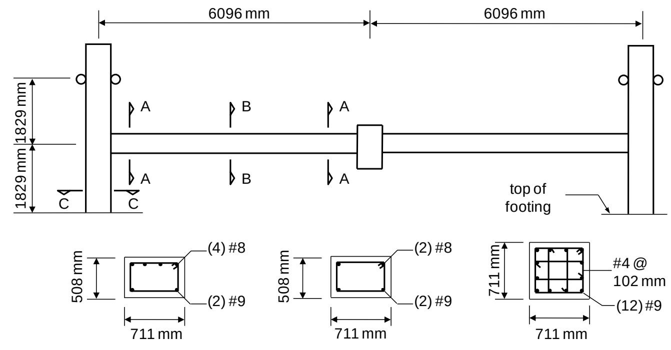

Figure 8. Schematic of the reinforced concrete assembly.

A schematic of the test setup used for the concrete beam-column assembly is shown in Figure 9. Figure 10 shows a photograph of the reinforced concrete assembly in the initial position.

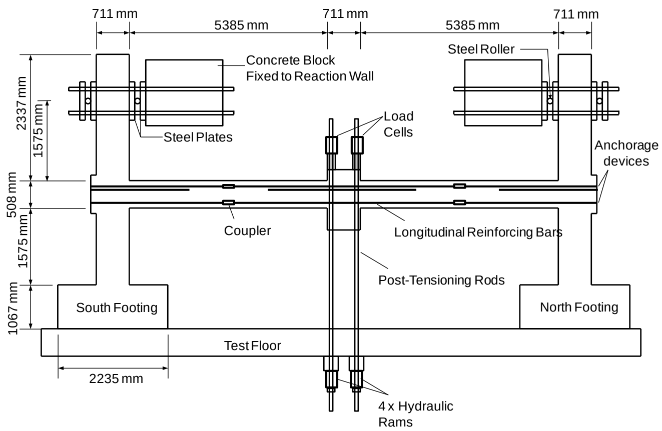

Figure 9. Test setup for the reinforced concrete assembly.


Figure 10. Photograph of the reinforced concrete assembly in the initial position.

As shown in Figure 9, the load was applied to the center stub column by means of four post-tensioning rods that were pulled down by four hydraulic rams, each having a capacity of 534 kN and a stroke of 102 mm. Because this loading scheme is self-centering, lateral bracing of the center stub column was not required. The load was applied under displacement control at a rate of 25 mm/min. The instrumentation used in the concrete assembly tests included displacement transducers, inclinometers, vertical displacement encoders, Optotrak (a surface position measuring device) targets, and strain gages which were cemented to both beam and column reinforcing bars. The estimated uncertainty in the measured data from the load cells was ± 1 %. The tops of the exterior columns were restrained from horizontal movement by steel rollers (see Figure 9), while vertical motion was permitted. The bases of the exterior columns were fixed to large footings which in turn were anchored to the test floor. Both top and bottom longitudinal beam reinforcing bars were spliced with threaded couplers at mid-span of the beams. Mechanical bar couplers were used instead of lap splices in order to evaluate their effectiveness in the development of catenary action. All longitudinal beam reinforcing bars were anchored at the exterior beam-column joints by means of threaded mechanical anchorage devices. These anchorage devices are supplemented with 19 mm thick steel plates as shown in Figure 9.

4.2 *Test results*

Figure 11(a) shows a plot of the vertical load versus the vertical displacement of the center column. Experimental measurements are presented along with model predictions that will be discussed subsequently. As the load was increased, flexural cracks developed in the tension zones, at the top of the beams adjacent to the exterior columns and at the bottom of the beams adjacent to the center column. Yielding of the longitudinal reinforcing bars in the cracked regions was first detected at about 267 kN. The load reached an initial peak of 296 kN at a vertical displacement of 127 mm and started to decrease with additional displacement. This decrease in load was associated with crushing of concrete at the top of the beams adjacent to the center column. The load leveled at 196 kN at a displacement of 406 mm. With further increases in displacement, the load began to increase again due to the development of catenary action, while the cracks at the bottom of the beams near the center column widened. The assembly attained a maximum load of 547 kN at a vertical displacement of 1090 mm, at which point the assembly failed due to rupture of one of the bottom bars. A second bar ruptured at a displacement of 1130 mm (see Figure 12).

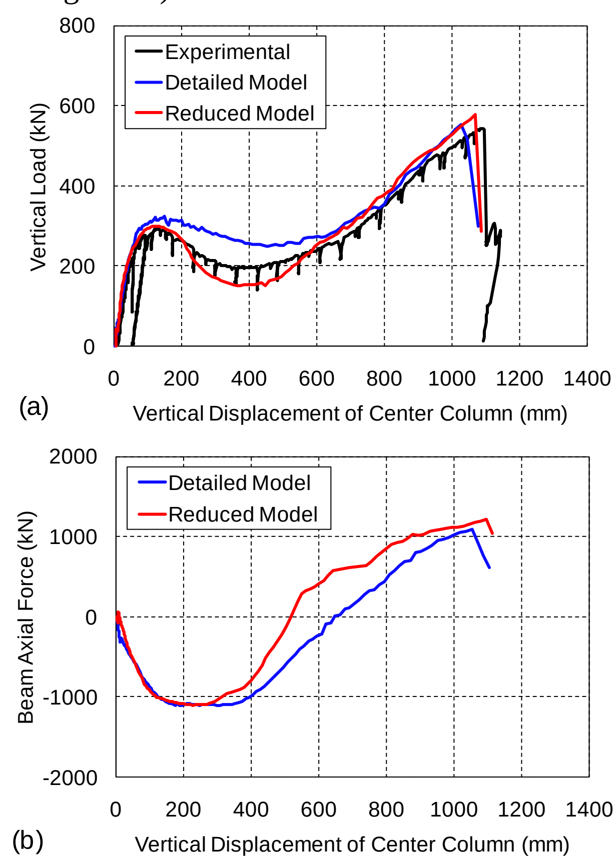

Figure 11. Vertical load and beam axial force versus vertical displacement of center column for the reinforced concrete assembly (measurement uncertainty ±1 %).

Figure 12. . Failure mode of the reinforced concrete assembly.

4.3 *Finite element models*

The reinforced concrete assembly was modeled using two different approaches: a detailed finite element model with approximately 70 000 elements and a reduced component-based model with about 150 elements. Calculated structural responses were compared to those measured from the experiment. The analyses were conducted using LS-DYNA (Hallquist 2007). Overviews of both models are shown in Figure 13.

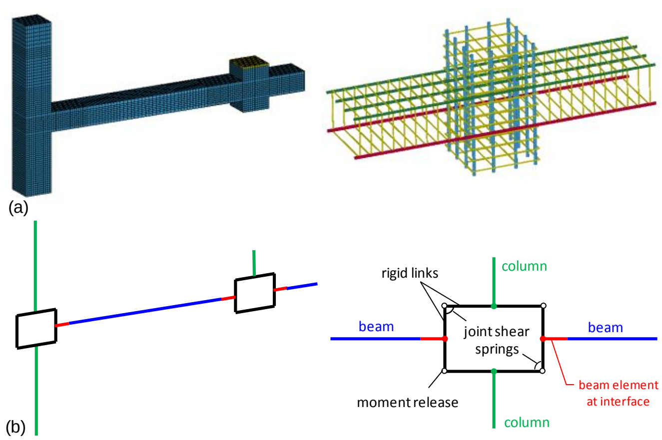

Figure 13. (a) Detailed and (b) reduced models of the reinforced concrete assembly.

In the detailed model, concrete was represented by finely meshed solid elements and reinforcing bars were modeled as beam elements. A contact interface between beam elements and solid elements was defined to describe the bond-slip behavior of reinforcing bars in the beams. The bottom of the exterior columns was assumed fixed. Since the tops of the exterior columns were restrained horizontally by steel rollers (see Figure 9), contact was defined between the columns and rigid cylinders representing the rollers. Steel properties of reinforcing bars were modeled using a piecewise-linear plasticity model based on test data, and fracture was modeling using element erosion. The concrete material was modeled by a continuous surface cap model (material 159 in LS-DYNA).

In the reduced model, the beams and columns were modeled using beam elements with fiber-discretized sections. An arrangement of beams, spring elements, and rigid links was used to simulate the behavior of beam-column joints. Joint shear was represented by rotational springs. Critical sections at the beam-to-column interface were modeled using a beam element with bond-slip effect incorporated into the material property of the reinforcing bars (Bao 2008).

The failure mode predicted by both the detailed and reduced models was fracture of the longitudinal bottom bars of the beams near the center column, as shown in Figure 14 for the detailed model. These predictions were consistent with the failure mode observed in the test (see Figure 12).

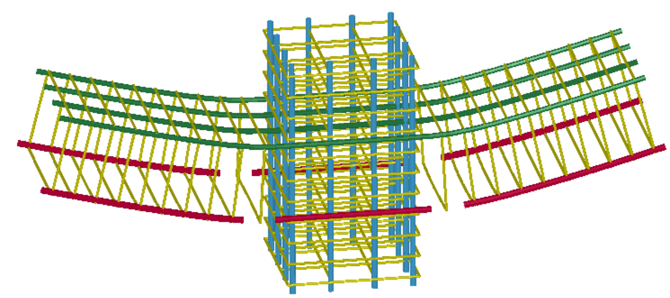

Figure 14. Failure mode from detailed model of the reinforced concrete assembly.

In Figure 11(a) comparisons are presented between the model predictions and the experimental measurements of the applied vertical load, plotted against the vertical displacement of the center column. The agreement between the experimental and computational results is good and provides validation for the detailed and reduced models. The plots show that the models are able to correctly predict significant softening after an initial peak load, with subsequent increases in load due to catenary action. Catenary action is indicated by the increasing axial tension evident in Figure 11(b), which shows model predictions of the beam axial force versus the vertical displacement of the center column. In the early stages of loading, the beam is predominantly in compression due to arching action, and subsequently the compression force is reduced due to softening and crushing of concrete. As the loading progresses with increased vertical displacement of the center column, the response of the assembly is dominated by catenary action. Further discussion of the different stages of response is provided in Bao (2008).

5 STRUCTURAL SYSTEM ANALYSES

Because the reduced models described in previous sections can be analyzed much more rapidly than the detailed models, they are used here in the analysis of complete structural systems. To assess the robustness of structural systems, two types of analyses are considered. The first, dynamic column removal, involves sudden removal of selected columns under service loads. Figure 15 shows peak vertical displacements after dynamic removal of two first-story columns for a 10-story steel IMF building. The removed columns were part of a perimeter IMF, as shown in Figure 16. The structural system was able to withstand this column loss without collapse, limiting the peak vertical displacement to about 360 mm.

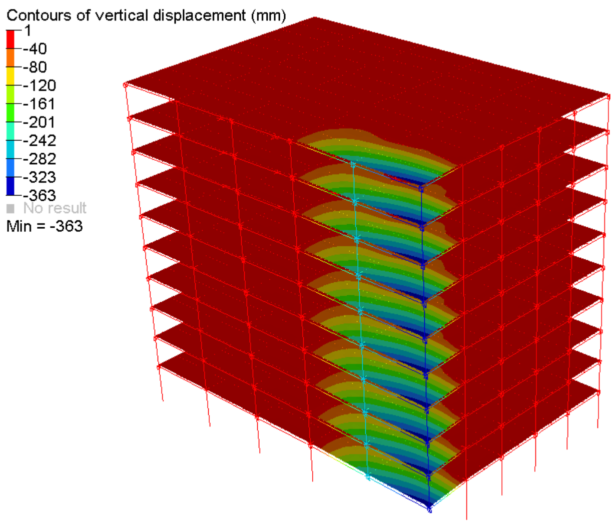

Figure 15. Peak vertical displacements of steel IMF building after dynamic removal of two columns.

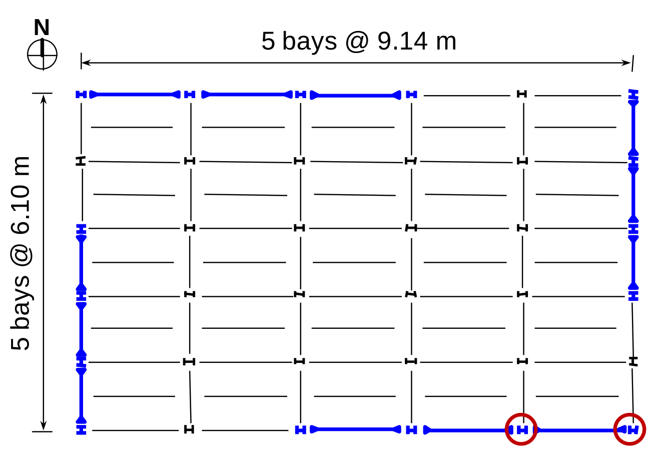

Figure 16. Plan layout for steel IMF building. Perimeter moment frames shown in heavy blue lines; circles indicate columns removed dynamically under service loads.

The second type of analysis considered for assessing robustness is a pushdown analysis, in which vertical loads are gradually increased to assess the ultimate vertical load-carrying capacity of the structural system. Pushdown analyses are performed on the intact structure and also on the structure with missing columns, to assess a structure's capability to redistribute loads. Columns are removed prior to loading in the pushdown analyses. Figure 17 shows contours of vertical displacement from pushdown analysis of a 10-story reinforced concrete frame building with three missing first-story columns, indicated in Figure 18. By performing pushdown analyses with increasing numbers of missing columns, degradation in the vertical load-carrying capacity can be observed, with more robust structures exhibiting less degradation.

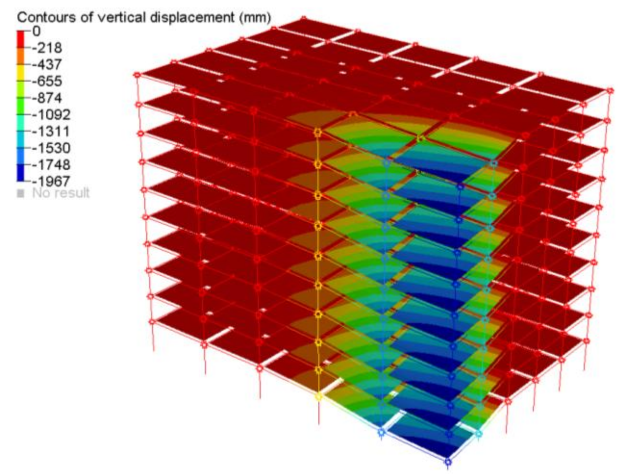

Figure 17. Vertical displacements from pushdown analysis of concrete IMF building with three missing columns.

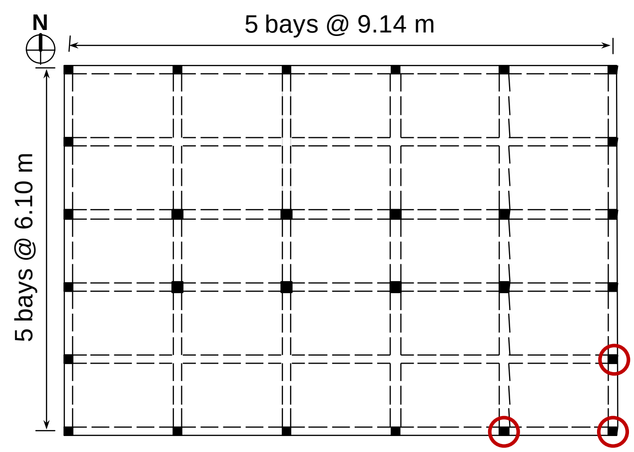

Figure 18. Plan layout of concrete IMF building. Circles indicate columns removed prior to pushdown.

6 DISCUSSION AND CONCLUSIONS

This paper presented both full-scale testing and finite-element based modeling of steel and reinforced concrete beam-column assemblies. Each assembly comprised three columns and two beam spans of 6.10 m, and each was subjected to prescribed vertical displacements of the center column until failure, simulating a column removal scenario. The assemblies represented portions of the structural framing systems of 10-story buildings designed as Intermediate Moment Frames for SDC C.

The steel assembly, which incorporated welded, unreinforced flange, bolted web connections, failed at a vertical column displacement of 495 mm, with a corresponding ultimate load of 890 kN. The vertical load versus displacement curves of the steel assembly exhibited an initial linear portion, a fairly well-defined yield point at a vertical displacement of about 50 mm, and a gradually increasing load beyond the yield point up to failure. The observed hardening behavior beyond the yield point was associated with the development of catenary action, and a peak axial tension value of about 670 kN was measured in the beams of the steel assembly.

The reinforced concrete assembly failed at a vertical column displacement of 1090 mm, with a corresponding ultimate load of 547 kN. The vertical load versus displacement curve of the reinforced concrete assembly exhibited softening behavior, with an initial peak load at a vertical displacement of about 100 mm and reductions in load thereafter, up to a displacement of about 500 mm, at which point the load began to increase again up to the point of failure. The observed softening behavior was associated with softening and crushing of concrete, while the subsequent hardening behavior was associated with the development of catenary action. The initial peak load was about 300 kN, and thus the ultimate load exceeded the initial peak load by a factor of 1.82.

Both detailed and reduced finite element models of the assemblies were developed, and the computational predictions showed good agreement with the experimentally observed response characteristics and failure modes, providing validation of the modeling approaches. The detailed models involved hundreds of thousands of solid and/or shell elements and were capable of representing the behavior and failure of the assemblies in great detail. The reduced models, which involved on the order of a hundred beam and spring elements, also accurately captured the behavior and failure modes of the assemblies, while analyses with the reduced models can be executed much more rapidly.

The validated reduced models developed in this study are being used in the analysis of complete structural systems to assess their reserve capacity and robustness. Results of dynamic column removal and pushdown analyses for steel and reinforced concrete frame structures, respectively, were presented.

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8 ACKNOWLEDGMENTS

The steel assembly tests were carried out at the Engineering Research and Development Center (ERDC) of the U.S. Army Corps of Engineers, Vicksburg, MI. The concrete assembly tests were carried out at the Bowen Laboratory of Purdue University. The steel assembly tests were partially supported by ERDC, the Defense Threat Reduction Agency, the U.S. Air Force, and the American Institute of Steel Construction, and the concrete assembly tests were partially supported by the Department of Homeland Security.

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