PERFORMANCE OF SEISMIC MOMENT RESISTING CONNECTIONS UNDER COLUMN REMOVAL SCENARIO

Fahim Sadek, H. S. Lew, Joseph A. Main, John L. Gross
Building and Fire Research Laboratory, National Institute of Standards and Technology, Gaithersburg, MD 20899-8611, U.S.A.
fahim.sadek@nist.gov, hsl@nist.gov, joseph.main@nist.gov, john.gross@nist.gov

ABSTRACT

Beam-to-column moment connections in steel frame construction have been studied extensively for seismic applications. The behavior of such connections, however, has not been studied under the monotonic loading conditions expected in progressive collapse scenarios, in which connections are subjected to combined bending and tension. This paper presents an experimental and analytical assessment of the performance of beam-column assemblies with two types of moment resisting connections under vertical column displacement. The connections considered include (1) a welded unreinforced flange – bolted web connection and (2) a reduced beam section connection. The study provides insight into the behavior and failure modes of the connections, including their ability to carry tensile forces that develop in the beams. The results indicate that these connections can sustain larger rotations under monotonic loading conditions than under the cyclic loading conditions developed for seismic applications. Validated models of the connections are developed that capture the primary response characteristics and failure modes.

INTRODUCTION

While structural safety in buildings is implicitly assured through reliability-based load and resistance factors, such provisions in current building codes and standards do not include load combinations to account for abnormal loading events that may lead to progressive collapse. Progressive collapse is the collapse of a disproportionately large portion of a structure that results from localized initial damage (e.g., failure of a column). An accurate characterization of the nonlinear, large-deformation behavior associated with the transfer of forces through the connections in this scenario is critical in assessing the potential for progressive collapse.

The National Institute of Standards and Technology (NIST) has initiated a research program to study the behavior of structures that when exposed to abnormal loads, might lead to progressive collapse. At present, design and evaluation of structures for progressive collapse potential are typically based on acceptance criteria obtained from seismic research (e.g., FEMA 350, 2000). As will be shown in this paper, using this approach to predict the response to monotonic loading similar to that expected during progressive collapse underestimates the rotational capacities of the connections.

To understand the behavior of structural systems near their ultimate strength limit states and to develop reliable tools to quantify the reserve capacity and robustness of structural systems, the NIST study involves analysis of three-dimensional models of structures with various materials and systems to assess the vulnerability of different types of structural systems to progressive collapse. The three-dimensional analyses use experimentally validated subsystem models of the various components and connections of the structure.

The study reported herein covers the development of finite element models of steel moment resisting connections with experimental validation. This paper describes two tests of steel beam-column assemblies with selected moment resisting connections under vertical displacement of a center column, representing a column removal scenario. These tests help fill the gap in defining the response characteristics of these connections under monotonic loading, and also contribute to establishing a database of connection behavior that can be used to assess the robustness of structural systems. Finite element models of the tested assemblies are developed and validated with the purpose of understanding the response characteristics and providing input to three-dimensional system-level models of complete structural systems to be analyzed in future studies.

DESCRIPTION OF BUILDING DESIGNS

Prototype steel framed buildings were designed in the NIST study for the purpose of examining their vulnerability to progressive collapse. The buildings are 10-story office buildings with plan dimensions of 100 ft x 150 ft (30.5 m x 45.7 m). The buildings were designed and detailed for two Seismic Design Categories (SDC) to examine the effectiveness of seismic design and detailing in resisting progressive collapse. One building was designed for SDC C, which resulted in a design using intermediate moment frames (IMFs) for the lateral load resisting system and the other for SDC D, which resulted in a design using special moment frames (SMFs) as defined in the American Institute of Steel Construction (AISC) Seismic Provisions (2002).

Moment frames, located around the perimeter of both buildings, provided the lateral load resistance. Connections used in the moment frames were selected from the prequalified steel connections specified in FEMA 350 (2000): (1) Welded Unreinforced Flange-Bolted Web (WUF-B) connections for the IMFs in the SDC C building, and (2) Reduced Beam Section (RBS) connections for the SMFs in the SDC D building.

Beam-column assemblies consisting of two-span beams connected to three columns (see Figure 2) were selected from the second floor of the moment resisting frames of each of the two buildings for the experimental and computational studies presented herein. The beams had a span length (center to center of columns) of 20 ft (6.10 m). The beams selected from the building in the SDC C zone were W21x73 sections, and were connected to W18x119 columns using WUF-B connections. The beams selected from the building in the SDC D zone were W24x94 sections, and were connected to W24x131 columns using RBS connections. ASTM A992 structural steel (F_y = 50 ksi, 345 MPa) was used in all beams, columns, and doubler plates in the panel zone.

ASTM A36 steel (F_y = 36 ksi, 248 MPa) was used for the shear tabs and continuity plates at connections. ASTM A490 high strength bolts were used for the bolted connections, and welding requirements followed the recommendations in FEMA 353 (2000).

BEAM-COLUMN ASSEMBLY WITH WUF-B CONNECTIONS

Description of WUF-B Connection

The WUF-B connection is similar to the connection commonly used prior to the 1994 Northridge earthquake. After significant research, it was determined that, with several improvements and appropriate quality assurance, this connection can perform reliably. FEMA 355D (2000) provides extensive information on the testing and performance of the WUF-B connections under seismic loading. The acceptable values for inter-story drift angle or rotation capacity of the WUF-B connection was specified in FEMA 350 (2000) based on a statistical analysis of the results from cyclic tests of full-scale connections. The rotation capacity, in radians, corresponding to collapse prevention, characterized by the inability of the connection to maintain its integrity under gravity loading, was estimated to be θ_U = 0.060 - 0.0006d_b, where d_b is the beam depth in inches. For the W21x73 section used with the WUF-B connection, θ_U = 0.047 rad.

The WUF-B connection used in this study is shown in Figure 1. As shown, the beam web is connected to the column flange using a shear plate (shear tab), which is fillet welded to the column using 5/16 in (8 mm) weld and bolted to the beam web using three 1 in (25 mm) diameter, high strength bolts. The bolt holes are standard holes with an edge distance of 2.75 in (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 flanges per the recommendations of FEMA 350 (2000). Continuity plates are provided for both interior and exterior columns as shown in Figure 1. No doubler plates were required for either column.

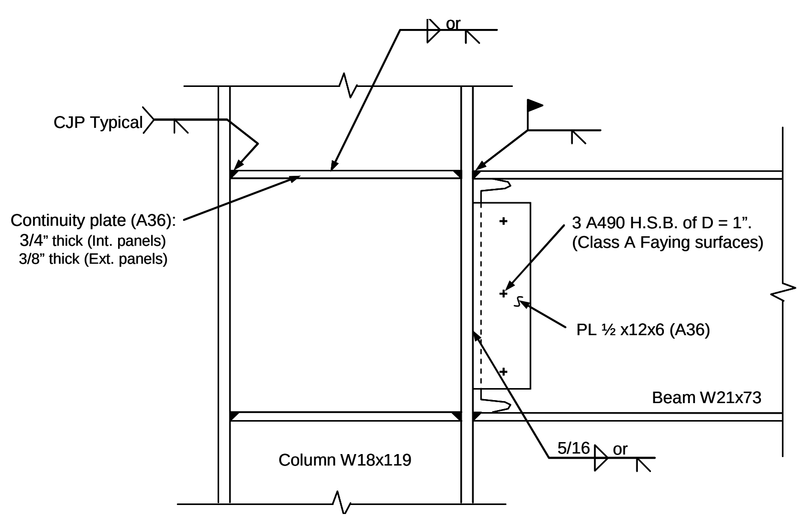

Figure 1. WUF-B Connection Details – Second Floor of Building in SDC C zone

Experimental Setup and Test Results

A schematic of the test specimen is shown in Figure 2 along with details of the instrumentation. Figure 3 shows a photograph of the test specimen along with a close-up of the connections to the center column. As shown in the figures, the double-span beam was supported on two exterior columns, which were anchored to the strong floor of the testing laboratory. Two diagonal braces were rigidly attached to the top of each exterior column to simulate the bracing effect provided by the upper floor. The center column was free at its bottom to simulate a column removal scenario, but its out-of-plane motion was restrained. In addition, the beams were restrained from out-of-plane motion at mid-span by lateral bracings. A hydraulic ram with a capacity of 500 kips (2224 kN) and a 20 in (508 mm) stroke was attached to the top of the center column to apply a vertical load to the specimen. Load was applied under displacement control at a rate of 1 in/min (25 mm/min). The uncertainty in the measured data from the load cells, deflection (D) and strain (S) gages, and inclinometers (I) was within ± 1 %. For more details, see Sadek et al. (2008).

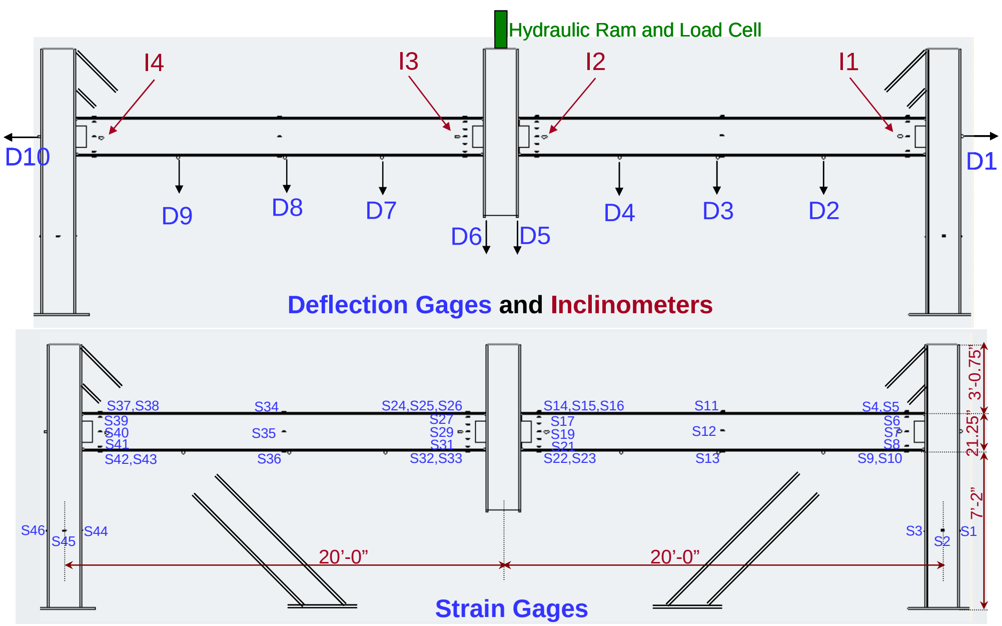

Figure 2: WUF-B Test Specimen Schematic and Instrumentation Layout

Figure 3: Photographs of the WUF-B Test Specimen

The specimen experienced large deflections and rotations prior to failure. The connection failed at a vertical displacement of the center column of about 19.5 in (495 mm), with a corresponding beam end rotation of about 0.088 rad. At that displacement, the applied vertical load was about 200 kips (890 kN). The failure was characterized by 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 lowest and middle bolts connecting the beam web to a shear tab at the center column, and (3) fracture of the bottom flange near the weld access hole immediately thereafter.

Figure 4: Failure mode of the WUF-B Test Specimen

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 7. The beam axial forces are estimated based on the measured strains on the beams. Also presented are the results of the computational models. As the plots indicate, the specimen was unloaded at a vertical displacement of about 18 in (457 mm) to adjust the stroke of the hydraulic ram and then was reloaded again to failure. Figure 7 indicates that the assembly remained in the elastic range up to a vertical displacement of the center column of about 2 in (50 mm). At the early stages of the response, the behavior was dominated by flexure indicated by the compressive axial forces in the beams. With increased vertical displacement, tensile axial forces developed in the beams and the behavior was dominated by catenary action. At the time of failure, the axial tension in the beams was about 150 kips (667 kN).

Finite Element Models and Results

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 that measured during the test. 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 5.

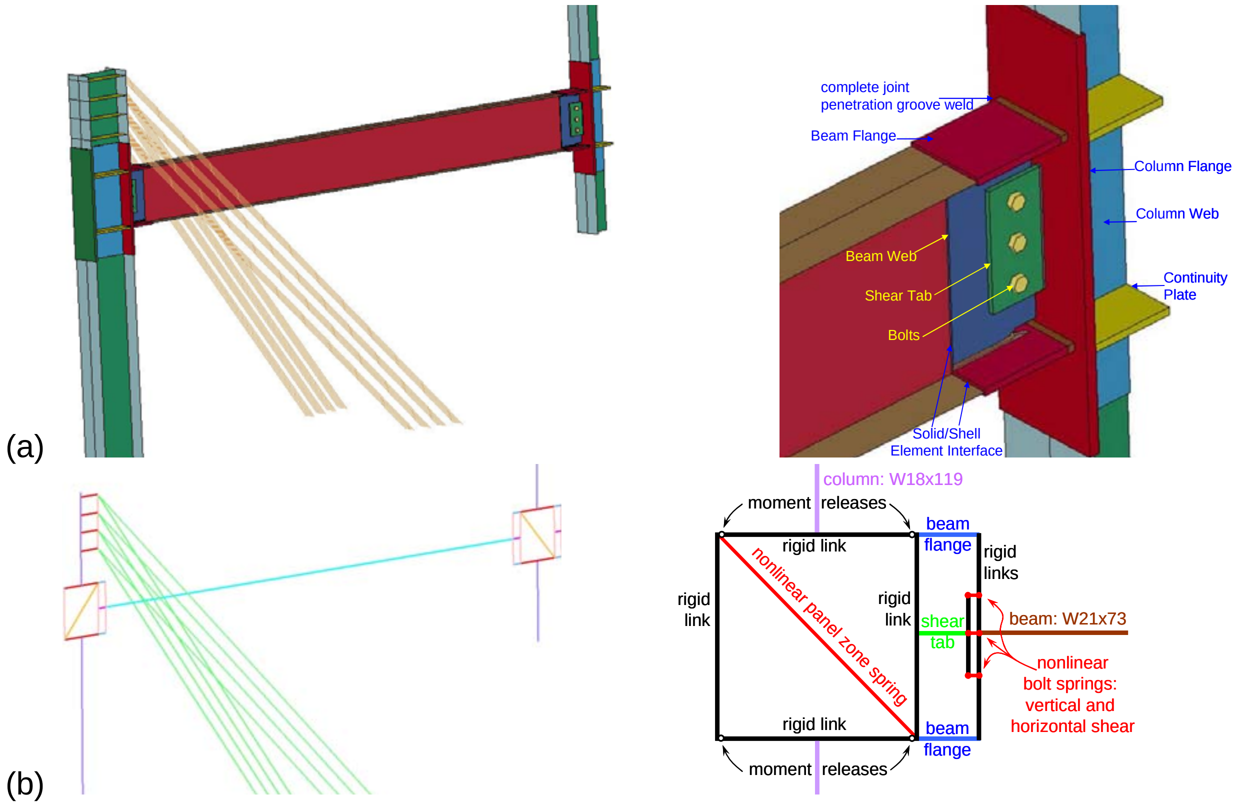

Figure 5: (a) Detailed and (b) Reduced Models of the WUF-B Test Specimen

The detailed model, Figure 5(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. The steel for the various elements was modeled using a piecewise-linear plasticity model based on coupon tensile test data obtained for all steel sections and plates.

The reduced model used beam elements with Hughes-Liu formulation (Hallquist, 2007) to model the beams and columns. An arrangement of beam and spring elements, connected with rigid links, was used to model the WUF-B connection as shown in Figure 5(b). Nonlinear spring elements represented the bolts, while beam elements represented the shear tab and the top and bottom flanges of the beam. 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 (for more details, see Sadek et al., 2008). Two analyses were conducted in which the bases of the end columns were modeled as fixed and 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 next to the center and end columns where significant yielding was observed. The failure mode of the connection based on this analysis was very similar to that observed in the experiment, see Figure 6. The results from the reduced model were consistent with those from the detailed model, albeit without the same level of detail.

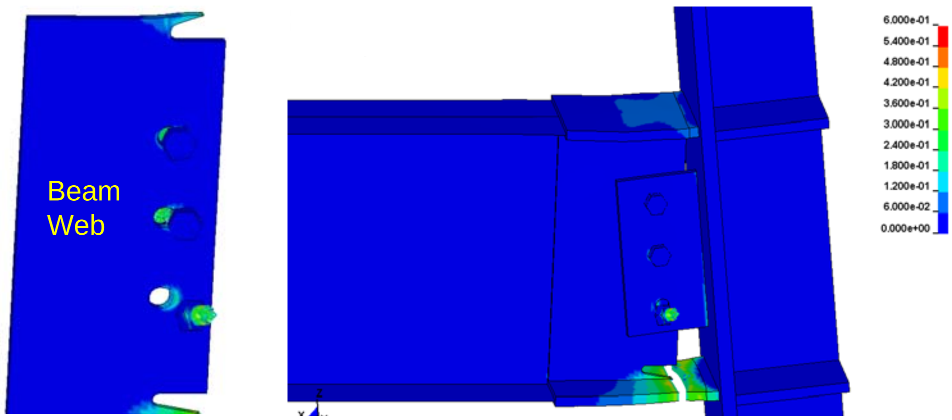

Figure 6: Failure Mode from the WUF-B Detailed Model

Figure 7 shows plots of (a) the applied vertical load and (b) the beam axial force against the vertical displacement of the center column from the experimental results and the two finite element models. The plots indicate a good agreement between the experimental and computational results and provide validation for the detailed and reduced models.

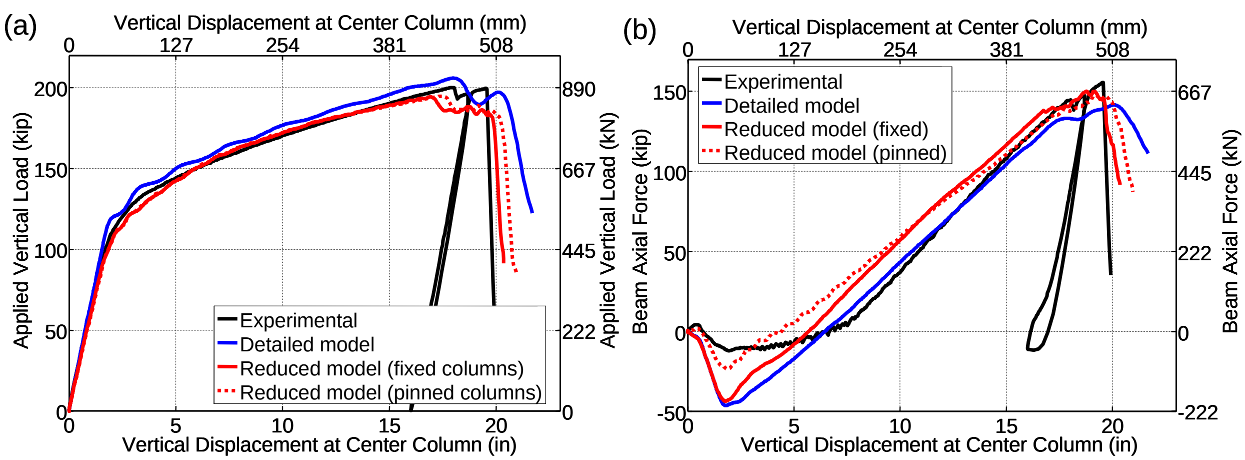

Figure 7: (a) Applied Vertical Load and (b) Beam Axial Force versus Vertical Displacement at Center Column of the WUF-B Specimen

BEAM-COLUMN ASSEMBLY WITH RBS CONNECTIONS

Due to the similarities between the test layout, boundary conditions, and loading system of the WUF-B and RBS specimens, only a brief overview of the RBS test is presented herein. The reader is referred to Sadek et al., 2008 for further details.

Description of RBS Connection

The RBS connection is created by cutting away a portion of the top and bottom flanges of the beam at a distance from the beam-column interface so that yielding would be concentrated in this reduced area. The RBS connection was developed as a result of extensive research following the 1994 Northridge earthquake and has been used for seismic design since then. FEMA 355D (2000) provides extensive information on the testing and performance of the RBS connections under seismic loading. The rotation capacity of the RBS connection, in radians, corresponding to collapse prevention, was specified in FEMA 350 (2000) based on full-scale cyclic tests as θ_U = 0.080 - 0.0003d_b. For the W24x94 section used with the RBS connection, θ_U = 0.073 rad.

The RBS connection used in this study is shown in Figure 8. As shown in the figure, the beam flanges and web are connected to the column flange using CJP groove welds. The connection is created by circular radius cuts in both top and bottom flanges of the beam. Continuity plates are provided for both center and end columns, while doubler plates were required only for the center column.

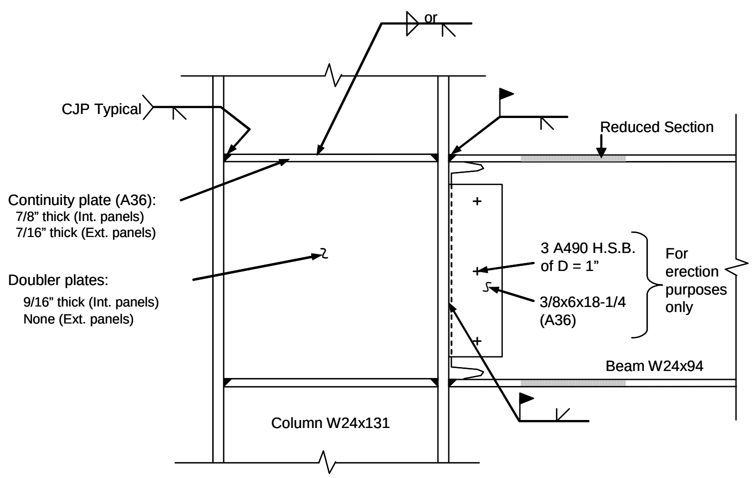

Figure 8. RBS Connection Details – Second Floor of Building in SDC D zone

Test Results

The connection failed at a vertical displacement of the center column of about 33.5 in (851 mm), corresponding to a beam end rotation of about 0.155 rad. At that displacement, the applied vertical load was about 400 kips (1780 kN). The failure was characterized by the fracture of the bottom flange in the middle of the reduced section of one of the connections near the center column. As shown in Figure 9, the fracture propagated through the web until the specimen could no longer carry the applied load.

Plots of the 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 12. Also shown are the results of the computational models. Similar to the WUF-B specimen, in the early stages of loading, the response of the beam was primarily in flexure. As the loading progressed with increased vertical displacement of the center column, the beam response was dominated by tensile axial forces. At the time of failure, the beam axial tensile forces were about 550 kips (2447 kN).

Figure 9: Failure mode of the RBS Test Specimen

Finite Element Models and Results

Similar to the WUF-B specimen, two finite element models were used to estimate the response of the RBS specimen. The detailed model consisted of shell elements representing the columns, beams, continuity and doubler plates, and welds. Finer meshes were used in the vicinity of the reduced section. The reduced model consisted of beam and spring elements. Each reduced beam section was modeled using five beam elements with varying section properties. Both fixed and pinned bases were considered for the end columns.

The detailed model showed that the beam-column assembly responded initially in a flexural mode before catenary action developed. The failure mode of the connection was very similar to that observed in the experiment, see Figure 10. The results from the reduced model were consistent with those from the detailed model.

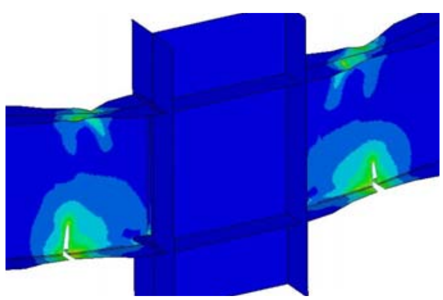

Figure 10: Failure mode from the RBS Detailed Model

Figure 11 shows plots of (a) the applied vertical load and (b) the beam axial force against the vertical displacement of the center column from the experimental results and the two models. The agreement between the experimental and computational results is good and validates the detailed and reduced models. The plots also indicate that the results using the reduced models with pinned and fixed boundary conditions at end column bases generally bracketed the experimental results.

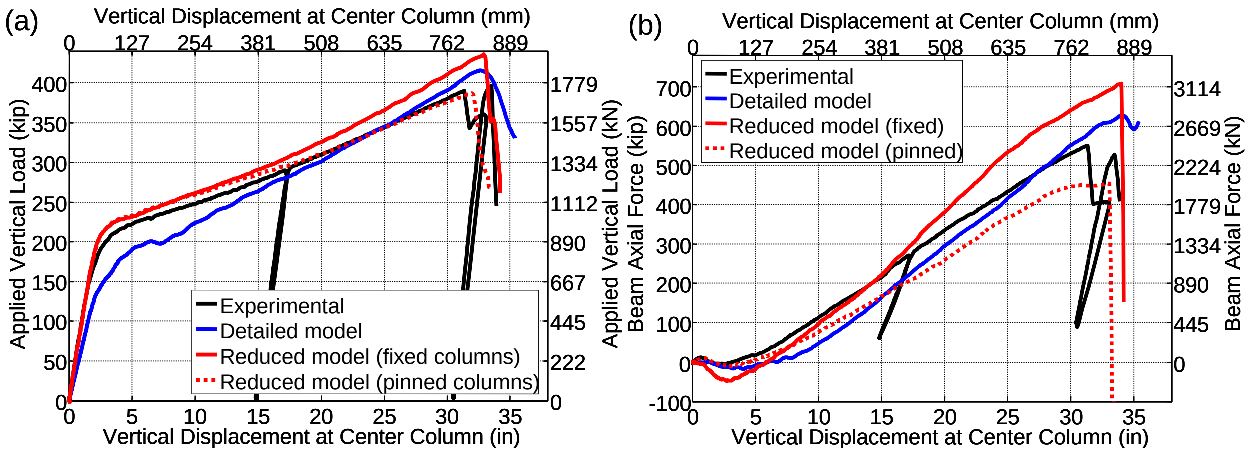

Figure 11: (a) Applied Vertical Load and (b) Beam Axial Force versus Vertical Displacement at Center Column of the RBS Specimen

DISCUSSION AND COMPARISONS

This study indicates a good agreement between the experimental results and the computational predictions. Both detailed and reduced models were capable of capturing the primary response characteristics and failure models. The validated reduced models developed in this study will be valuable in the analysis of complete structural systems for assessing reserve capacity and robustness of building structures. The analyses confirm that the loads under a column removal scenario are primarily resisted by axial tensile forces in the beams. These tensile forces increase until the connection can no longer sustain the axial force.

For the WUF-B and RBS connections, the rotations at peak load were about 0.088 rad and 0.155 rad, respectively based on the experimental results in this study. The rotational capacities of these connections based on seismic testing data are approximately 0.047 rad and 0.073 rad for the WUF-B and RBS connections, respectively. These results show that the rotational capacities of these connections under monotonic column displacement are about twice as large as those based on seismic test data. Contributors to this difference may include: (1) cyclic loading leads to significant degradation in the strength and stiffness of the connection, while no such degradation is expected under monotonic loading, and (2) the applied loads are resisted by different mechanisms in the two cases, with the connection in pure flexure for seismic loading but subjected to both flexure and tension under vertical column displacement, with tension being the dominant load.

SUMMARY AND CONCLUSIONS

This paper presented an experimental and computational assessment of the performance of beam-column assemblies with two types of moment-resisting connections (WUF-B and RBS) under monotonic vertical displacement of a center column. The study provided insight into the behavior and failure modes of the connections. The results indicate that these connections can accommodate substantially larger rotations (prior to significant strength degradation) under monotonic loading conditions than under the cyclic loading conditions considered in seismic tests. Both detailed and reduced models are capable of capturing the primary response characteristics and failure modes of the connections. The reduced models, in particular, would be valuable in the analysis of complete structural systems for assessing reserve capacity and robustness of building structures.

REFERENCES

AISC (2002), *Seismic Provisions for Structural Steel Buildings*, American Institute of Steel Construction, Chicago, IL.

FEMA 350 (2000), *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, SAC Joint Venture and Federal Emergency Management Agency, Washington, D.C.

FEMA 353 (2000), *Recommended Specifications and Quality Assurance Guidelines for Steel Moment-Frame Construction for Seismic Applications*, SAC Joint Venture and Federal Emergency Management Agency, Washington, D.C.

FEMA 355D (2000), *State of the Art Report on Connection Performance*, SAC Joint Venture and Federal Emergency Management Agency, Washington, D.C.

Hallquist, J. (2007), *LS-DYNA Keyword User's Manual*, Livermore Software Technology Corporation, Livermore, CA, Version 971.

Sadek et al. (2008), *An Experimental and Analytical Study of the Robustness of Moment Resisting Steel Connections*, National Institute of Standards and Technology, Gaithersburg, MD, under preparation.

ACKNOWLEDGEMENTS

The experimental program was conducted at the Engineering Research and Development Center (ERDC) of the U.S. Army Corps of Engineers with partial support from ERDC, the Defense Threat Reduction Agency, the U.S. Air Force, and the American Institute of Steel Construction. Mr. Stephen D. Robert and Mr. Vincent Chiarito of ERDC led the experimental work. The authors acknowledge their contributions. The authors acknowledge the discussions with Professor Sherif El-Tawil of the University of Michigan in the development of finite element models. The authors acknowledge the through review by Drs. Long T. Phan and William E. Luecke of NIST.

Certain commercial software or materials are identified to describe a procedure or concept adequately. Such identification is not intended to imply recommendation, endorsement, or implication by NIST that the software or materials are necessarily the best available for the purpose. The policy of NIST is to use metric units in all its published materials. Because the paper is intended for the U.S. construction industry which uses inch-pound units, units are presented in the inch-pound and metric systems.