Earthquake Simulator Testing of Metal Building Systems

C. M. Uang, M. ASCE¹, M. D. Smith, M. ASCE², and W. L. Shoemaker, M. ASCE³

¹Professor, Department of Structural Engineering, University of California San Diego, La Jolla, CA 92093-0085. E-mail: cmu@ucsd.edu
²Graduate Student Researcher, Department of Structural Engineering, University of California San Diego, La Jolla, CA 92093-0085. E-mail: mds002@ucsd.edu
³Director of Research & Engineering, Metal Building Manufacturers Association, 1300 Sumner Ave., Cleveland, OH 44115. E-mail: LShoemaker@thomasamc.com

ABSTRACT

Metal Building Systems (MBS) make up a substantial portion of non-residential, low-rise construction in the U.S. Given their widespread use, it is desirable to understand their performance during a seismic event. Previous pseudo-static, cyclic testing indicates an absence of ductility in these systems. This lack of ductility arises from the inherent design concepts behind using web-tapered frames with no flange or web compactness limits, causing buckling limit states to govern the strength of the frames. In light of the lack of ductility, it is obvious that the current practice of designing MBS moment frames as Ordinary Moment Frame (OMF) systems, with \( R = 3.5 \), is inappropriate. Also, simple analytical studies suggest that MBS supporting large masses, such as concrete or masonry exterior cladding or interior mezzanines, may be particularly vulnerable to the lack of ductility. For these reasons, it is desirable to develop a safe, economic, and reliable design procedure for MBS. Earthquake simulation testing of MBS has been performed to provide test data to start this development.

Three full-scale MBS specimens have been tested on the NEES/UCSD Large High Performance Outdoor Shake Table. Specimen 1 consisted of a single story MBS frame with metal side wall panels to represent the majority of MBS buildings. Specimen 2 was similar to the first, except with heavy concrete sidewall panels. Specimen 3 consisted of a MBS frame built with compact flanges, a mezzanine attached to one sidewall, and a concrete wall attached to the other. Results from each battery of tests will be presented in this paper.

INTRODUCTION

A very common form of construction in low-rise, non-residential structures is the metal building system (MBS). MBS are typically built from steel moment frames in their transverse direction and braced frames or shear walls in their longitudinal direction. The transverse frames are usually made with built-up I-sections, many times with tapered webs, which have been optimized to minimize material weight. These built-up sections are usually made without consideration of slenderness limits and frequently have slender or noncompact webs and flanges. These plates often
cause the governing limit state of the frames to be some type of buckling, a non-ductile behavior. These frames can be clad with an assortment of materials ranging from steel decking to concrete tilt-up panels.

MBS are built in a variety of locations, including high seismic zones. Currently, the building codes treat these frames as any other structural steel moment resisting system. MBS frames are usually designed as ordinary moment frames (OMF) or intermediate moment frames (IMF) as defined in ASCE/SEI 7 (ASCE, 2005) and AISC 341 (AISC, 2005b), effectively assuming an ability of the system to dissipate energy through some ductile, inelastic behavior.

While, historically, MBS perform well in large seismic events, previous pseudo-static cyclic testing at UCSD, has revealed a concerning lack of ductility in the cyclic behavior of the transverse frames used in these systems (Hong and Uang, 2006). This suggests that the assumptions made by the current building codes concerning the ductility of these systems are incorrect and the design provisions used for more traditional steel systems with rolled shapes are likely inapplicable to MBS. The research also suggested that the good past performance of these systems is due to a high level of system overstrength. Since MBS are typically very light structures, the transverse frames are usually controlled by non-seismic load combinations. This is the source of the overstrength. Thus, there is a potential lack of safety in systems with large masses, such as MBS with hard wall attachments or mezzanines, when designed according to the current code provisions (Hong and Uang, 2007). It is clear that additional seismic testing was needed to develop applicable, economic, and safe design provisions for MBS frames.

OBJECTIVE AND SCOPE

Three full-scale MBS building specimens were tested at the NEES/UCSD Large High Performance Outdoor Shake Table (LHPOST). The objective of these experiments was to test MBS frames to investigate their performance when subjected to high level seismic motions. It was desired to determine if MBS frames with heavy wall attachments or mezzanines exhibited different margins of safety against collapse than typical MBS with metal sidewalls only. Finally, the data gathered from the tests will be used to calibrate analytical models for use with a FEMA P695 (ATC, 2009) study for new design provisions.

TEST PROGRAM

NEES Shake Table

Test specimens were constructed and tested on the LHPOST. The shake table has a surface area of 7.6 m x 12.2 m and has a maximum payload of 20 MN. It is capable of motions with accelerations of up to 4 g, velocities of up to 1.8 m/s, and has a stroke of ±0.75 m.
Test Buildings

Each test specimen was to represent two interior bays of a MBS building. The specimens consisted of two 18.29 m clear span MBS frames with 6.10 m eave height and spaced at 5.49 m. Since the size of the specimens exceeded the dimensions of the table, a frame was built using W36x302 cantilevered beams and mounted to the table with adequately prestressed threaded anchoring rods. The test setup is shown in Figure 1. The frames were composed of web-tapered, built-up sections using steel plates with nominal yield stress of 379 MPa. All connections within the frames were of the bolted end-plate type. Where steel decking was used as cladding, light gauge metal girts or purlins were incorporated to mount the decking onto the frames. Each specimen had diagonal bracing in each sidewall as well as in the plane of the roof. Each frame was bolted to the testing apparatus at the column base plates. Additional mass, in the form of three 25.4 mm x 203 mm x 9.14 m steel plates, was added at the roof level to represent the remainder of the tributary mass of two full interior bays. These masses were located directly above and along each frame, above the roof decking, and were bolted to pieces connecting them to the frame rafters. The rafter flanges were laterally braced with purlins and flange braces.

Specimen 1 had only steel deck cladding at the side walls. The column flanges were laterally braced with girts and flange braces along their height. Specimen 2 had two 254 mm x 3.05 m x 6.71 m precast concrete panels at each side wall. These concrete panels represented a hard wall cladding of approximately 2.873 kN/m² spread over two interior bays. The walls were connected to the frame using spandrel beams bolted at the panel zones. These connections, as well as the connections of the wall base to the test apparatus, were designed to be of pin type. The column flanges were unbraced for their full height. Specimen 3 had one sidewall which resembled Specimen 2, using the same concrete panels and unbraced column flanges. The other side wall was clad with metal decking similar to Specimen 1. This sidewall also had a mezzanine attachment. The mezzanine was composed of built-up, I-beam sections with bolted connections designed as pin type. The floor mass of the mezzanine was represented with one of the 3.05 m x 6.71 m concrete

Figure 1. Test Setup
panels. Diagonal bracing was used to brace the mezzanine in the out-of-plane direction.

Each specimen was designed according to the International Building Code 2006 (ICC, 2006). Gravity loads consisted of self weight, 144 N/m\(^2\) collateral load, 958 N/m\(^2\) reducible roof live load, and flat roof snow load of 1,005 N/m\(^2\). Wind loads were calculated using a wind speed of 38 m/s. The seismic load resisting system was designed and detailed as OMF with one exception. The third specimen used seismically compact flanges (AISC, 2005b) in the columns and in the rafters near the knee joint in addition to other OMF requirements. The first two specimens were designed using \(S_{DS}\) of 1.031 and \(S_{D1}\) of 0.592, while the third specimen used \(S_{DS}\) of 1.000 and \(S_{D1}\) of 0.400. Gravity load combinations controlled the rafter sizes for the first and second specimens. While wind load combinations controlled the column sizes for Specimen 1, seismic drift limitations per ASCE/SEI 7 (ASCE, 2005) controlled the column sizes for Specimen 2. Both rafters and columns for Specimen 3 were primarily sized to meet the compactness criteria set forth by the designer.

**Instrumentation**

Each test building was thoroughly instrumented. Strain gauges and rosettes were located at several sections along each column and rafter. Displacement transducers were used to measure slip, rotation, and uplift of the column bases and slip of the mounting frame. String potentiometers were used to measure the displacement of each frame at the roof and base as well as the mezzanine of the third specimen. Accelerometers were located at several sections along each rafter, on the mounting frame, on the roof mass plates, and in the panel zones to measure accelerations in three perpendicular directions. Over 300 separate data channels were used on each specimen.

**Test Sequence**

Each specimen was subjected to a variety of ground motions with the shake table. Five ground motion records were chosen from motions used in FEMA P695 (ATC, 2009). These ground motions were from 1979 Imperial Valley (H-DLT 352), 1989 Loma Prieta (CAP 000), 1992 Landers (YER 270), 1994 Northridge (LOS 000), and 1999 Chi-Chi (TCU084_001). A white noise motion and a short duration impulse type motion were used to assess the dynamic characteristics (natural periods and equivalent viscous damping) of each specimen at various times in the test sequence, intermittently between seismic record tests to detect and track accumulating damage. Prior to testing, preliminary finite element models were used to estimate the fundamental periods of each specimen. Each ground motion record was scaled such that its 5% damped pseudo-acceleration response spectrum value, at the estimated fundamental period, was some percentage of the value of the Design Basis Earthquake (DBE) response spectrum defined in ASCE/SEI 7 (ASCE, 2005), using \(R=1\), at that same period. An example of this scaling is shown for the Imperial Valley (IV) motion in Figure 2. Tests were named using initials based on the particular ground motion record and a number representing the percentage of DBE to
Figure 2. Imperial Valley ground motion scaled to match 100% DBE at estimated period of Specimen 1

which it was scaled, such as IV150 for the IV motion scaled to 150% DBE. Since the periods at which the records were scaled were only estimates, the actual percentage of DBE acceleration exciting the specimens varied from the number in the test name. The general test sequence for each specimen started with white noise and impulse motions, followed by each ground motion record scaled to a low percentage of DBE. For all specimens, the IV motion was used for high level tests for consistency and ease of result comparison. After the low level tests, IV tests were performed at increasing scale factors until limiting circumstances prevented further testing.

TEST RESULTS

Specimen 1

After the initial white noise and impulse tests, the fundamental period was determined to be 0.39 s. This differed from the value of 0.52 s determined from the initial analytical model. The damping ratio was determined to be about 1.5%. This measured fundamental period was used to set the scale factors for the remaining tests. After some low level ground motion records, the Imperial Valley motion was scaled up progressively to investigate the capacity of the structure. Hysteretic behavior can be seen in Figure 3 for IV100 through IV300. Testing went without problems until the IV150 test.

During the IV150 test, the anchor rods at the southeast columns sheared off. Because of this failure, the remaining three columns had to transfer the seismic loads. Despite the changed structure, no limit states were reached anywhere in the frame. After this test, a fix was made at all four column base plates using welds. These welds were placed such that minimal stiffening of the base plate connection would occur.

After the repair, testing continued with the IV200 and IV250 tests. The response of the structure until the IV250 test was essentially elastic with no limit states reached and no significant energy dissipation. During the IV250 test, the north rafter experienced a lateral-torsional buckle (LTB) at the east pinch point. This was
consistent with the predicted failure mode and location. Strain gauge data from a time just prior to buckling implies a moment diagram which should cause LTB according to member strength predicted by AISC 360 (AISC, 2005a). The buckling caused large forces in the flange braces on either side of the affected unbraced length. The forces were large enough to damage the purlin at the other end of the flange brace (See Figure 4). The buckle in the rafter can be seen in Figure 4. The load-displacement curve showed a sharp drop in strength at the point of buckling, as expected. This sharp drop agrees with the nonductile behavior observed in the previous cyclic testing (Hong and Uang, 2006). After the buckle, the stiffness had degraded but the structure continued to behave elastically with a lower lateral stiffness.

Even though the structure reached a limit state, the structure was still standing and safely stable. Some weld cracking occurred at the column base fixes made previously. These welds were repaired and an additional IV300 test was performed. During this test, LTB occurred at two additional pinch point locations in the rafters. These buckles were associated with more sharp drops in strength and further degraded stiffness. In addition to the new LTBs, the initial LTB location experienced a rupture at the bottom flange (See Figure 4). This was due to low-cycle fatigue and the large lateral displacements at the LTB location. Since the structure was ruptured, it was deemed unsafe to continue testing.

During the testing this structure experienced a base shear of 224 kN when the first LTB limit state was reached. A maximum base shear of 325 kN was observed. When compared to the design base shear of 22.2 kN, a very significant amount of system overstrength is apparent. A maximum drift of 3.6% was reached during the final test.
Specimen 2

The fundamental period for Specimen 2 was determined to be 0.48 s. The damping ratio was determined to be between 1% and 2%. After determining the proper scaling factors for the ground motions from the fundamental period, low level testing was performed. The analytical model suggested a failure at a much lower level of base shear compared with the first specimen, so low level testing was performed at the 10% DBE level instead of 25%. Higher level testing was then performed with the Imperial Valley motion. Hysteretic behavior is shown in Figure 5. Behavior was essentially elastic until the IV100 test.

During the IV100 test, the north rafter experienced LTB at the east pinch point, the same limit state reached as the first specimen. Once again, the strength dropped sharply at the point of buckling and the structure experienced a degradation of stiffness. The LTB at the pinch point was the predicted failure mode and location.

The IV150 test caused LTB at all three remaining pinch point locations in the rafters. The LTBs all caused stiffness degradations and sharp decreases in strength. The initial LTB experienced a flange rupture in the same manor as the first specimen. Also, the large forces in the flange braces caused damage to the connections at the purlin. Photos of the flange brace damage, LTB, and flange rupture can be seen in Figure 7.

When the first limit state was reached, the base shear was 348 kN and a maximum base shear was observed at 427 kN. Compared with a design base shear of 100 kN, the system overstrength was far less in this specimen than in the first. This specimen exhibited no ductile behavior and had no significant energy dissipation. A maximum drift of 3.6% was reached during the IV150 test.
After removing the wall attachments and roof decking, this specimen was subjected to a pseudo-static pull-over test. The top of each frame was attached to a stationary point with a cable and the table was used to pull the base away from the support. This test provided a pull-over curve for the damaged structure. The measured response is shown in Figure 6, corrected for second-order effects which would have existed prior to removal of the large masses. A maximum base shear of 166 kN was observed, showing a greatly reduced strength capacity of the buckled structure. However, this post-buckling strength continued through to a very large displacement of 47 cm (7.8% drift). The extreme displacements further distorted the LTBs in the east rafter pinch points, while new top flange local buckles occurred in the western portions of the rafters. Extreme nonlinearities, eventually causing rupture, were seen in the base plates during the pull-over test. This suggests that column bases may be a source of some energy dissipation. Figure 8 - Figure 9 shows the displaced shape and various damages observed during the pull-over test.

![Figure 5. Specimen 2 hysteretic responses](image)

![Figure 6. Specimen 2 pull-over test load-displacement response](image)
Figure 7. Observed damage during shake table tests of Specimen 2

(a) Rafter LTB
(b) Flange brace damage
(c) Flange rupture

Figure 8. Observed damage during pull-over test of Specimen 2

(a) Global deformed configuration
(b) Top flange buckle
(c) Rafter LTB
Specimen 3

The third specimen had a fundamental period of approximately 0.45 s and a damping ratio of 1% - 2%. Low-level motions up to IV50 showed no inelastic behavior. Hysteretic behavior can be seen in Figure 10. During the IV100 test, elastic shear buckling occurred in the south east panel zone. Comparing moment and axial force demands calculated from the strain gauges to the panel zone strength defined by AISC equations (AISC, 2005a), this buckling was easily predicted.

After the IV100 test, additional tests were run through IV250. During these tests, all four panel zones experienced elastic shear buckling and low-cycle fatigue, eventually causing large ruptures in each web. Interestingly, during the test which caused rupture, IV200, and subsequently, the specimen exhibited significant energy dissipation as evidenced in the larger hysteretic loops seen in IV200 and IV250. Another important observation was that the structure remained elastic and undamaged outside of the panel zone areas. Panel zone damage can be seen in Figure 11.

This specimen reached its first panel zone buckling limit state at a base shear of 262 kN. Comparing this with a design base shear of 87.4 kN, it is seen that this specimen had much lower system overstrength than the first specimen. The maximum base shear reached was 405 kN and the maximum drift was 5.3%. This is the only specimen that exhibited any significant energy dissipation.

IMPLICATIONS

The results of these experiments have a few significant implications. Firstly, a low level of structural damping was observed. It would be prudent for designers of MBS to account for the structures having 1% - 2% damping, rather than the 5% assumed by the codes, when determining seismic force demands. Secondly, the differing levels of system overstrength verify that MBS with large masses have a propensity to be less reliable compared with light MBS. All three structures were designed with $R=3.5$. This assumes that if the structure were to remain elastic during a DBE level earthquake, the elastic demand would be about 3.5 times its design base shear. The specimens behaved primarily elastically up to this demand level. Since, for the most part, the test buildings did not exhibit ductile post-buckling behavior, it is reasonable, in design, to call the load causing the first limit state to be reached the capacity. A safety factor can be easily calculated for each specimen by dividing the
observed capacity by the elastic design base shear. Safety factors for each specimen, in order, were 2.88, 0.99, and 0.86. This clearly shows that the two specimens with heavier masses were insufficient to resist DBE level earthquakes without reaching their reliable capacity. It is easily seen that the original assumption of $R=3.5$ is incorrect and an $R=1$ is likely more applicable. Use of this $R$ value would have ensured that the structures were designed to resist the elastic DBE level forces which were demanded. However, since neither of the two specimens collapsed, perhaps a rethinking of the overall concepts of the seismic design should be considered for these elastic systems.
CONCLUSIONS

Overall, the MBS specimens were surprisingly robust. The first specimen, despite lacking ductility, showed very large system overstrength and was able to withstand a base shear approximately 290% of the DBE level without damage. Even though the second and third specimens were damaged at or below the DBE level, they remained standing and did not collapse. These results verify that the safety margin against collapse of MBS with larger masses is much lower than a typical, light MBS. They also show a significant lack of ductility in the buckling limit states. Finally, the panel zone limit states in the third specimen suggest a possible energy dissipation mechanism using shear buckling.

Currently, this research is continuing by the development of numerical models calibrated to the test data gained by these experiments. These numerical models will be used in a FEMA P695 (ATC, 2009) study. This study will test newly developed design criteria, which take into account the unique characteristics of these systems and provide an economic and safe solution for seismic design of MBS.

REFERENCES

ASCE (2005), Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, New York, New York, USA, 2005;
Hong, J.K. and Uang, C.M. (2006), Cyclic Performance Evaluation of a Metal Building System with Web-Tapered Members, Report No. SSRP-06/23, Department of Structural Engineering, University of California, San Diego, La Jolla, CA.
Hong, J.K. and Uang, C.M. (2007), Development of A Seismic Design Procedure for Metal Building Systems, Report No. SSRP-07/17, Department of Structural Engineering, University of California, San Diego, La Jolla, CA.

ACKNOWLEDGEMENT

This research was sponsored by NEES, MBMA, and AISI.