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Hysteretic Characteristics of Braced Frames in Modular Steel Buildings

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Abstract: The hysteretic characteristics of a structural system of a building must be well understood in order to effectively mitigate the damage caused by seismic activities. Modular steel construction is fast evolving as an effective alternative to traditional on-site construction, especially in low to medium-rise buildings where repetitive units are required. In this building form, modular units are built and finished under controlled environment and are combined on-site to form larger building structures. The lateral resistance of this steel building type is often achieved by adding diagonal braces but the detailing requirements differ significantly from the traditional counterpart. This may be critical to its performance under earthquake ground motions. In this study, key response and performance characteristics such as the available strength, stiffness, ductility, and energy dissipation in braced frames of modular steel buildings are evaluated under reversed cyclic loading. Comparison is drawn with the hysteretic behaviour of concentrically braced frames in regular steel buildings. The analysis suggests that the unique vertical connection of different units would need to be carefully addressed in design in order to fully develop the lateral seismic resistance of the modular steel braced system.

1. Introduction

The hysteretic behaviour of a building system plays a crucial role in many contemporary approaches to seismic design and analysis. Hysteresis refers to the path-dependence of a structural system's restoring force versus deformation. The hysteresis loops generally offer vital information and must be well understood in order to develop the tools needed to effectively mitigate structural damage under seismic actions. For example, extracting the hysteretic characteristics can lead to an understanding of the structure's degradation and nonlinear response under loading. These characteristics could also serve as a basis for developing numerical models that are used for collapse predictions. Experimental testing has been an effective means for evaluating these characteristics and as such, several experimental studies have been conducted to record hysteretic data for analysing and designing structural building systems.

Many studies have revealed the complexity of the hysteretic behaviour of steel-braced frames (Jain et al. 1978, Ikeda and Mahin 1984, Tremblay 2002, Tremblay et al. 2003). For concentrically-braced frames, the lateral response is dominated by the inelastic behaviour of the bracing members. Brace hysteretic behaviour is unsymmetric in tension and compression, and typically exhibits substantial degradation of strength and stiffness when subjected to monotonic compression or loaded cyclically. Under reversed cyclic loading, bracing members are expected to buckle in compression and yield in tension. After brace buckling, plastic hinges may develop which could lead to permanent plastic deformations and deterioration of the brace resistance. However, the complete hysteretic response and overall performance of any framed structure is dependent, to a large extent, on the global frame configuration including the connection type and construction methodology.

The Modular Steel Building (MSB) is fast evolving as an effective alternative to the traditional on-site steel construction, particularly in low to medium-rise buildings where repetitive units are required. The modular technique involves the design of buildings to be built and finished under controlled environment and be used at another. The finished modular units are transported and combined both horizontally and vertically at the building site to form larger buildings. Lateral stability of the entire MSB is often achieved by adding diagonal braces. The unique detailing requirements of the MSB braced system have been described extensively in previous publications (Annan et al. 2009a, 2009b, 2009c) and are summarised below.

Figure (1) shows typical plan and sections of an MSB. A typical modular unit consists of a set of columns, a floor framing made up of floor beams (FB) and floor stringers (FS), as well as a ceiling framing made up of ceiling beams (CB) and ceiling stringers (CS). These components are connected together mainly by direct welding. The horizontal connection (HC) between different units in an MSB involves field bolting of clip angles that are shop-welded to the floor beams (section A-A of Fig. 1). The vertical connection (VC) consists of field welding of base plates of upper module columns to cap plates of lower module columns (section B-B of Fig. 1). Diagonal braces are welded to gusset plates as shown in section B-B of Fig. 1. Lateral loading at each floor is transferred through the horizontal connections to the modular braced frame and then through the vertical connections to the foundation.

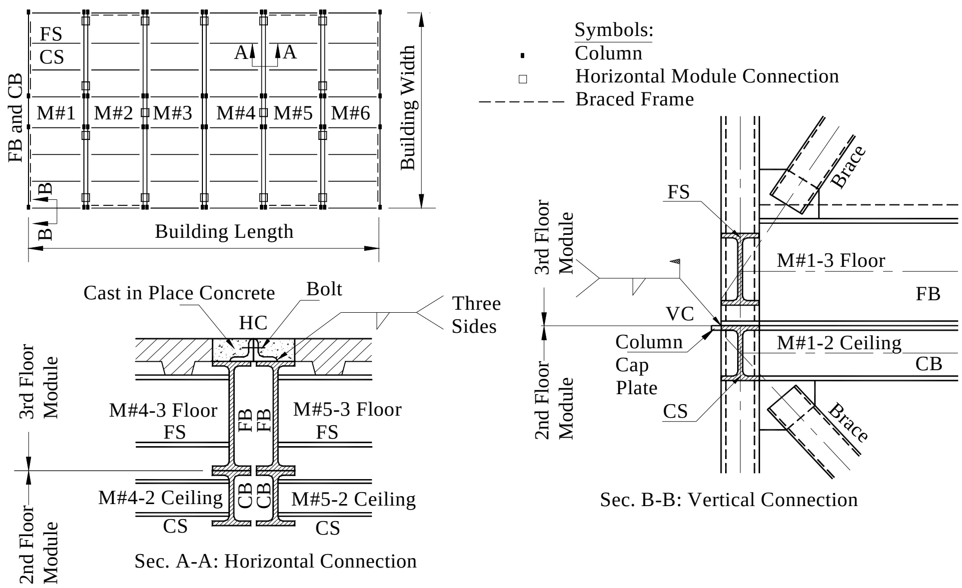


Figure 1. Typical details for a multi-storey MSB

In terms of structural configuration, the following features distinguish the MSB braced frame from braced frames of a regular steel building: (1) the existence of ceiling beams (CB) and ceiling stringers (CS) in the MSB frame system may result in natural periods and mode shapes different from those of conventional systems; (2) the floor beams (FB) are often set directly above the ceiling beams (CB) without mechanical connections except at column locations, and this may result in structural contact interaction between these beam members; (3) the brace members typically do not intersect at a single working point which may lead to high seismic demands on the vertical connection (VC) of different units; (4) the vertical connection (VC) typically involves only partial welding of the columns of a lower and an upper modules which may lead to independent upper and lower rotations at the same joint.

In this study, the hysteretic inelastic behaviour of braced frames of modular steel buildings is investigated under reversed cyclic loading, and key response and performance characteristics such as the strength, stiffness, inelastic deformation and energy dissipation are evaluated. The behaviour is compared to regular concentrically braced frames. A detailed overview of the complete experimental work and findings has been published by Annan et al. (2009c).

2. Specimen Details and Test Setup and Procedure

The test specimens consist of one MSB-braced frame and one regular-braced frame. Both specimens are a one-storey, one-bay crossed braced panel and represent a 3/8 scale models of one bay of four-storey modular steel and regular steel building frames designed by Canadian standards (CSA 2001, NBCC 2005). Schematic diagrams of the frame specimens showing their physical characteristics (sections and dimensions) are given in Figure (2). The specimens were constructed according to current practices in industry. The MSB braced frame specimen consists of two columns (HSS 51x51x5), two cross bracing members (HSS 32x32x3), a floor beam (W 100x19) and a ceiling beam (W 100x19) of a lower modular unit, and a floor beam (W 100x19) of a fictitious upper modular unit. A clearance of 40 mm was allowed between the bottom flange of the floor beam and the top flange of the ceiling beam, which is often provided in practice to allow for a fire protective layer to be installed. Beam-to-column connections in this specimen were achieved by direct welding of the members. The upper brace ends were connected through a 8 mm thick single gusset plates to the ceiling beam and columns, which form part of the lower modular unit. The lower brace ends were connected through a 8 mm thick gusset plates to the floor beam and columns of the same unit. The tube brace members were slotted at each end and fillet welded to the gusset plates. The braces intersection was achieved by cutting a slot through one brace member and welding a 10 mm thick connecting plate to this member and to the other crossing brace member. In the regular braced specimen, beam-to-column connections were achieved by welding a plate to both the beam and column allowing for partial rotation between adjoining members. The boundary conditions for the specimens were selected to simulate those of the full-scale panels.

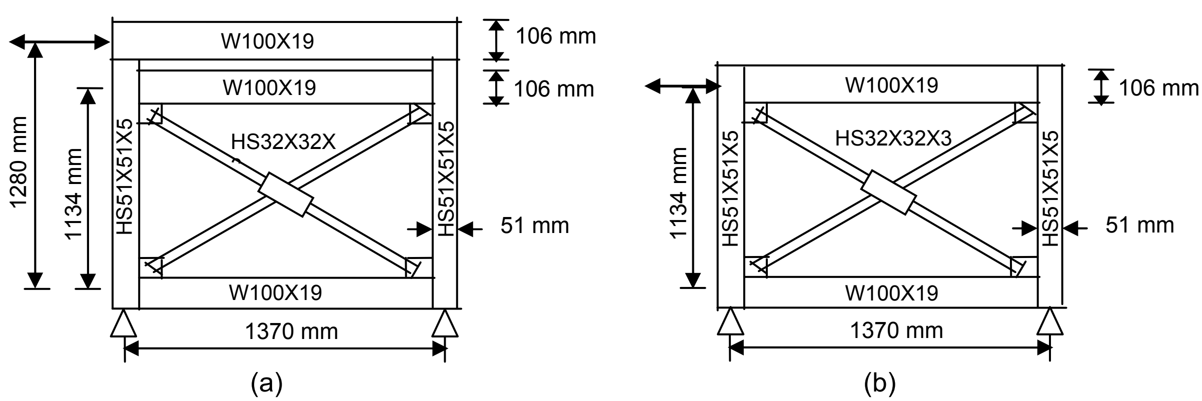
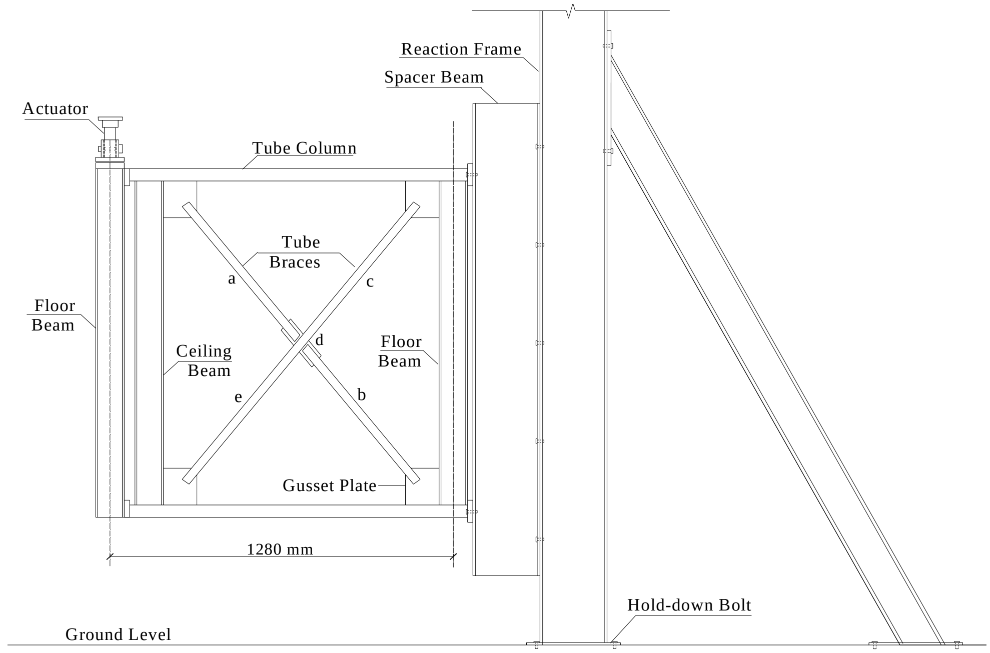


Figure 2. Test Specimens (a) MSB Frame (b) Regular Braced Frame

Figure (3) shows a schematic representation and photos of the setup. The specimens were connected to a well-braced reaction frame by bolting of base plates of the two tube columns. They were connected to the actuator (having a maximum load capacity of 250 kN) by means of a 25.4 mm thick plate welded to one end of the top floor beam and bolted to a plate-tube welded assembly which is attached to the load actuator by means of a pin. The applied load and specimen drift were monitored at the top floor beam levels by the actuator. A number of strain gauges and LVDTs were installed at several locations on members of each specimen to measure strains and deformations. These readings were used to study the distribution of forces in the various members and at different sections of the specimens. Out-of-plane deformations of brace members were measured with LVDTs that were installed at ¼, ½, and ¾ times the distance between the free length regions of the gusset plates (shown as a, b, c, d, and e in Figure 3). All the instrumentations were plugged into a computerized data logger.

Prior to testing of the specimens, the basic monotonic steel stress-strain properties of the tube brace material were obtained from standard coupon tests following the ASTM standard test methods (ASTM, 2002). These data were used in a nonlinear static analysis to predict the base shear-top displacement behaviour of the specimens in order to develop suitable loading history and evaluate need for instrumentation.



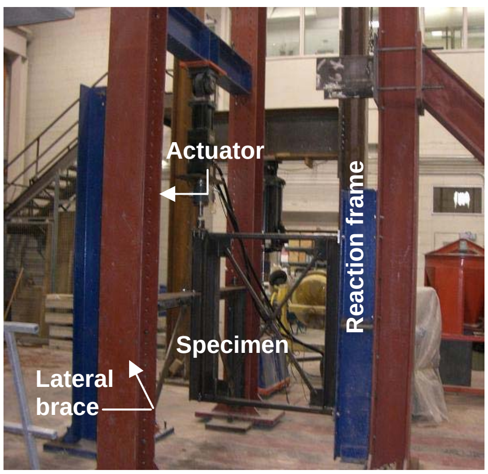


Figure 3. Schematic Representation and Overall View of Test Setup

The test specimens were subjected to symmetric reversed-cyclic loading histories to characterize their performance. The Applied Technology Council's ATC-24 (ATC, 1992) single specimen testing program was followed during testing. The yield values of specimen forces from the preliminary analytical prediction were used to initially control the test up to the elastic limit, after which the experimentally obtained values of the yield displacements were used as test control parameters. Figures (4) summarises the loading histories for both elastic (force-controlled) and inelastic (displacement-controlled) cycles applied to the specimens. Table (1) shows the ductility levels reached by the test specimens at different stages of loading. Δ_y represents the actual yield displacement.

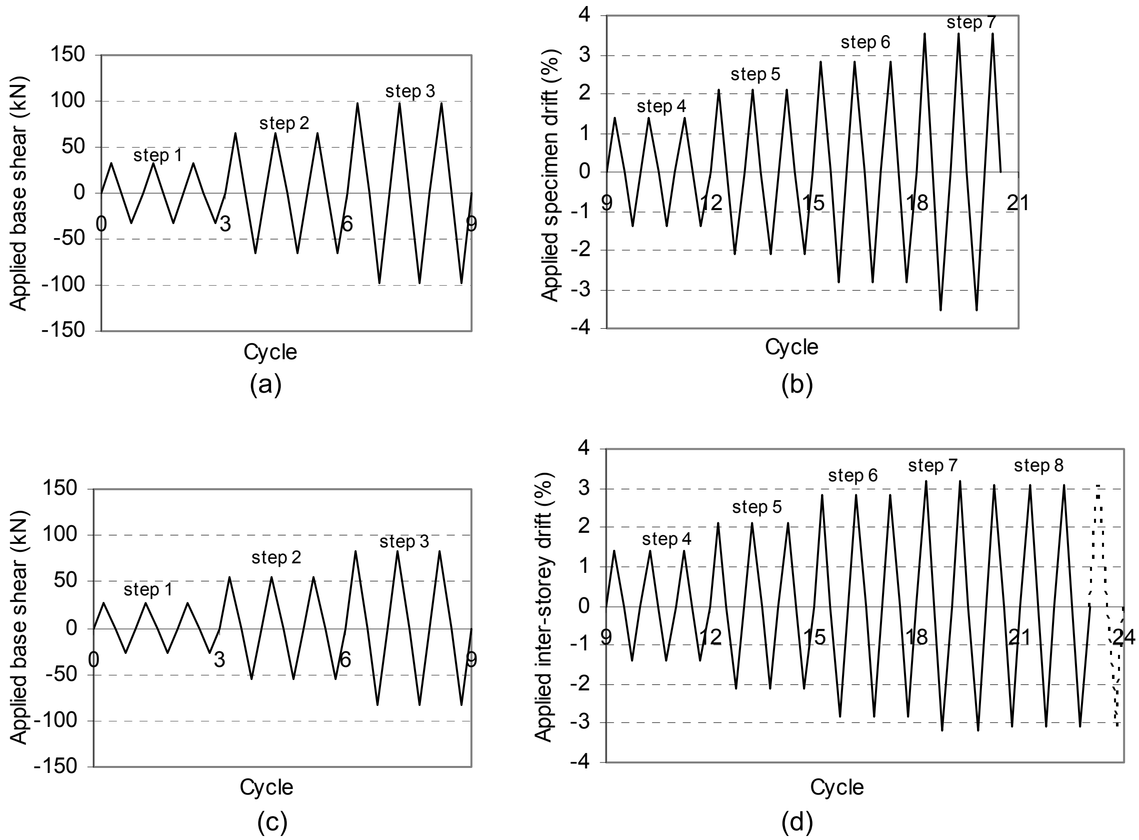


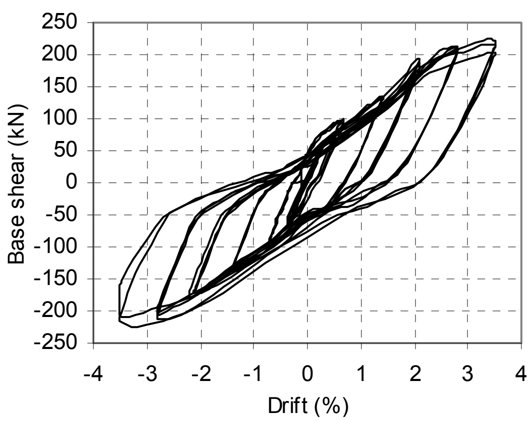
Figure 4. Loading Scheme (a) Elastic Cycles for Regular Specimen (b) Inelastic Cycles for Regular Specimen (a) Elastic Cycles for MSB Specimen (b) Inelastic Cycles for MSB Specimen

3.1 Hysteretic Behaviour

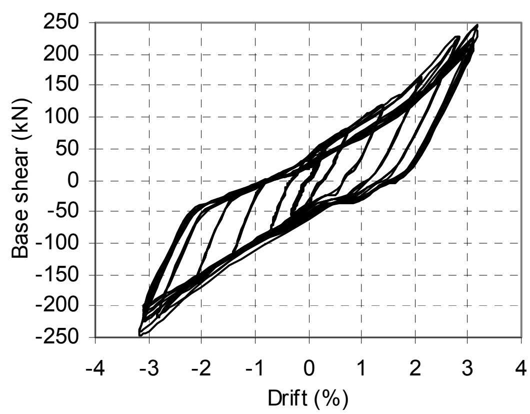
The hysteresis curves, shown in Figure (5), display the base shear versus peak drift behavior of the test specimens. Both specimens showed ductile behaviour and are stable up to large drift levels, although some degree of pinching is apparent in the hysteretic loops, especially for the regular specimen. Clearly, both specimens exhibited almost linear elastic response upon applying the first two load steps (i.e. six cycles of loading) in spite of possible existence of initial imperfections. The initial stiffnesses were evaluated as 14.3 kN/mm and 15.5 kN/mm for the MSB and regular braced specimens, respectively. Within the third load step, significant nonlinearity was observed mid-way in the load-displacement curve for each specimen. The movements in the actuator corresponding to this point were found to be about 4.5 mm and 3.8 mm for the MSB and regular braced specimens, respectively. These displacements correspond to a 0.35% and 0.34% drift, respectively. Strain guage data obtained from the various members indicated that, in the regular braced specimen, buckling of compression brace resulted in the first significant sign of nonlinear behaviour, while in the MSB specimen, flexural response due to column yielding cause initial sign of nonlinearity.

Table 1. Ductility Levels reached by Specimens at Different Cycles

Step	MSB braced specimen		Regular braced specimen	
	Cycle number	Normalised peak deformation ($\Delta/\Delta y$)	Cycle number	Normalised peak deformation ($\Delta/\Delta y$)
1	1 - 3	0.49	1 - 3	0.48
2	4 - 6	0.97	4 - 6	0.95
3	7 - 9	2.00	7 - 9	2.00
4	10 - 12	4.00	10 - 12	4.00
5	13 - 15	6.00	13 - 15	6.00
6	16 - 18	8.00	16 - 18	8.00
7	19 - 20.5	10.00	19 - 20	9.00
8			21 - 40	8.75



(a)

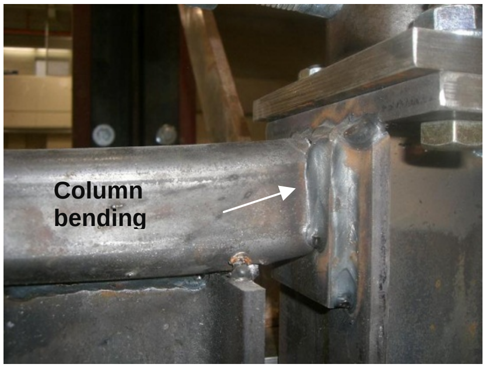


(b)

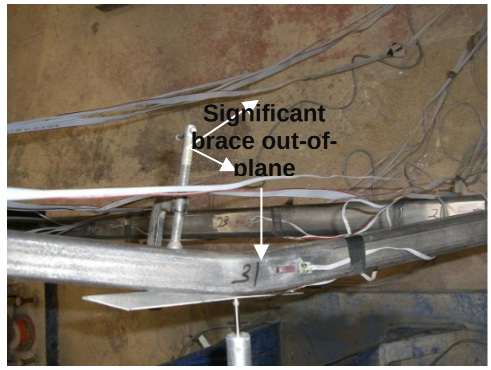
Figure 5. Base Shear versus Drift Hysteretic Response (a) MSB (b) Regular Braced Specimen

The MSB specimen was tested successfully to a 3.5% drift, corresponding to ten times the yield displacement, Δ_y, and a base shear of 225 kN. This level of loading resulted in a significant bending deformation in the column segment between the bottom flange of the upper floor beam and the top flange of the ceiling beam, as shown in Figure (6a). The test was terminated at this high ductility level. Prior to the end of the test, a maximum out-of-plane deformation of 2.2% of the brace length was measured at the mid-section of a lower half side of a brace member (labeled “b” in Figure 3). The ratio of the maximum attained base shear to the yield base shear was evaluated as 3.5.

The regular braced specimen was successfully tested to 3.1% drift at displacement ductility of 9. The maximum base shear at this drift level was found to be 245 kN, which was about the maximum load capacity of the actuator. At this load step, significant out-of-plane deformation was observed at mid-section of the lower half of the brace member labeled “b” in Figure (3). The specimen was further subjected to 20 cycles at 3.05% drift during which the brace member lower half suffered severe out-of-plane buckling, as shown in Figure (6b). Maximum out-of-plane deformation of 4.0% of the total length of brace member was measured at this point. The level of ductility reached was deemed sufficient to terminate the test. The ratio of the maximum attained base shear to the yield base shear was evaluated as 4.2.



(a)



(b)

Figure 6. (a) Column Bending Deformation in MSB Braced Specimen (b) Brace Out-of-Plane Deformation in Regular Braced Specimen

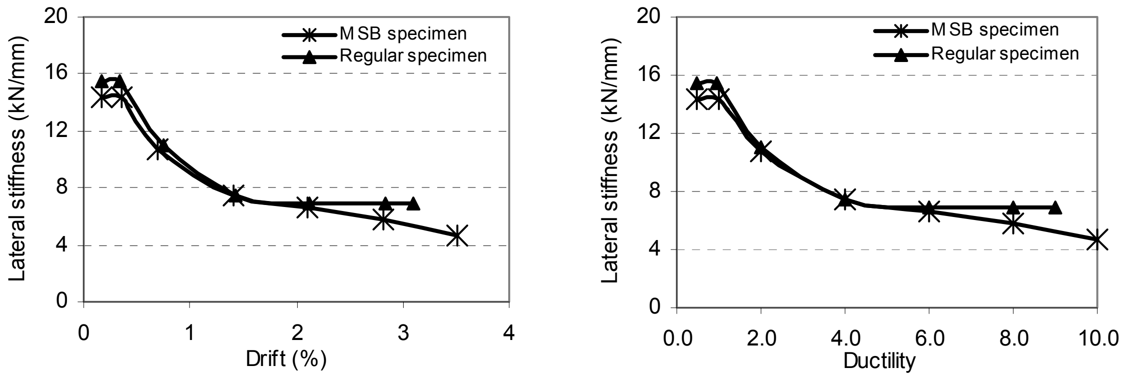
Strain gauges installed on the floor and ceiling beams showed that these members remained elastic throughout the different loading phases of the specimens as anticipated from the design. A maximum strain of 0.84% (8400 micro strain) was reached at the mid-section of the lower half of a tube brace member of the MSB specimen at 3.5% drift level. At the upper end section of the same brace member, a maximum strain of 0.23% (2300 micro strain) was recorded at this stage of loading. At 2.8% drift, maximum strain of 0.33% was recorded at the mid-section of the lower half side of the brace member. For the regular braced specimen, strain of 0.35% was reached at the mid-section of the lower half side of a brace member at 2.8% drift. At 3.1% drift, maximum strain of 0.73% was recorded at the same section. Upon a number of repeated cycling at load step 8, brace members in this specimen reached maximum strain of 2.9% before the test was terminated.

3.2 Strength and Stiffness Characteristics

As explained in the previous section, the regular braced specimen showed greater initial lateral stiffness than the MSB braced specimen (i.e. initial lateral stiffness of MSB specimen was about 93% that of the regular specimen). This is because the regular braced specimen has greater lateral resistance provided by the truss action typical of concentrically braced frames. The MSB specimen in this range derived its lateral resistance from a combination of the brace action and some moment resisting action due to direct welding of members and moment connection between the column and the bottom flange of the upper floor beam.

The hystereses for the two specimens are fairly symmetrical in both the elastic and inelastic cycles. Both specimens showed stable behaviour (only small strength and stiffness degradation) in almost every step of repeated loading. Figure (7) shows the change in lateral stiffness with specimen peak drift and with maximum ductility. Within the elastic response range, the lateral stiffness was evaluated as the slope of the base shear versus displacement relationships. Beyond this range, it was estimated as the slope of the line joining the peaks of positive and negative drifts in each remaining load step.

Up to a ductility level of 2, the MSB braced specimen provided less lateral stiffness than the regular braced specimen. Both specimens, however, showed a sharp drop in lateral stiffness after the first sign of brace buckling in the regular braced specimen and column flexural yielding in the MSB braced specimen. Between ductilities of 2 and 6, there was no significant difference in lateral stiffness between the two specimens. Beyond ductility of 6, the regular braced specimen again showed superior lateral stiffness, almost remaining constant up to a ductility of 9. The overall reductions in lateral stiffness from start of loading up to ductility of 6 were 46% and 45% for the MSB specimen and regular braced specimen, respectively.



(a) (b)

Figure 7. Variations of Lateral Stiffness with (a) Peak Drift (b) Maximum Ductility

Within a specified load step, there was no significant degradation in stiffness with cycling in both specimens (Figure 5). However, the strength of the MSB braced specimen deteriorated slightly with cycling in load steps 5, 6, and 7 as drift exceeded 2.1%. At 3.5% drift (in load step 7), the strength of this specimen dropped by about 9% at the end of that load step. For the regular braced specimen, there was no significant strength degradation with cycling up to the maximum actuator load capacity (i.e. corresponding to 3.1% specimen drift). After completing 20 load cycles of this specimen at 3.05% drift (in load step 8), its strength dropped by about 18% before the test was terminated. Figure (8) shows the base shear-drift response for only this final load step.

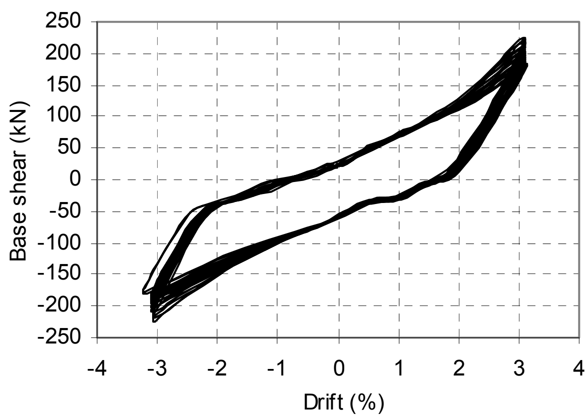


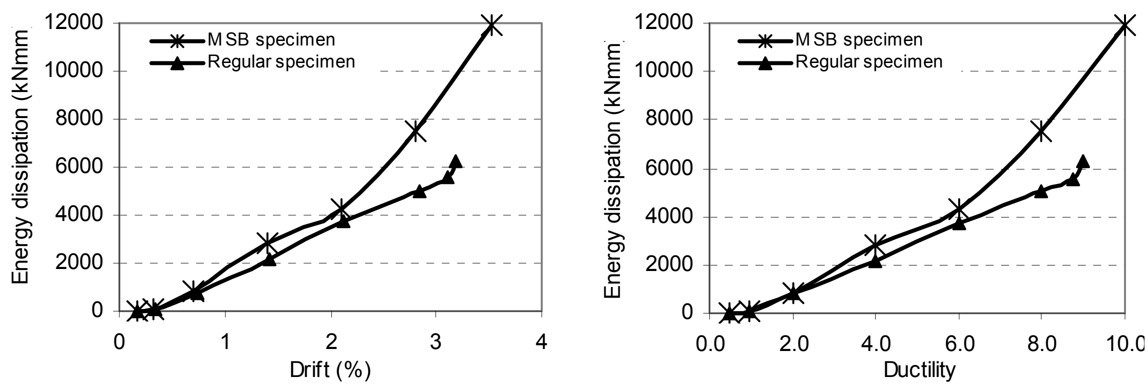
Figure 8. Base Shear versus Drift Relationship for Regular Braced Specimen at 3.05% Drift

3.3 Energy Dissipation Characteristics

Seismic performance of a framed structure can be measured by its energy dissipation characteristics. Energy dissipation is represented by the experimentally obtained hysteretic area, which is evaluated as the area enclosed by the base shear-deformation diagram. The cumulative hysteretic energy dissipation is a useful measure of the seismic efficiency of a structural system. In this study, both the energy dissipation per cycle, using the first complete cycle in each load step, and the cumulative energy dissipation were presented and used in assessing individual performances and comparing the relative effectiveness of the test specimens. The cumulative dissipated hysteretic energy was normalized at each cycle by the product of specimen yield base shear and yield displacement, $V_y\Delta_y$, to eliminate the effect of varying yield loads and displacements. For both specimens, there was no significant amount of energy dissipation within the elastic range of loading. Appreciable energy was dissipated after the elastic cycles, and increased in the following cycles.

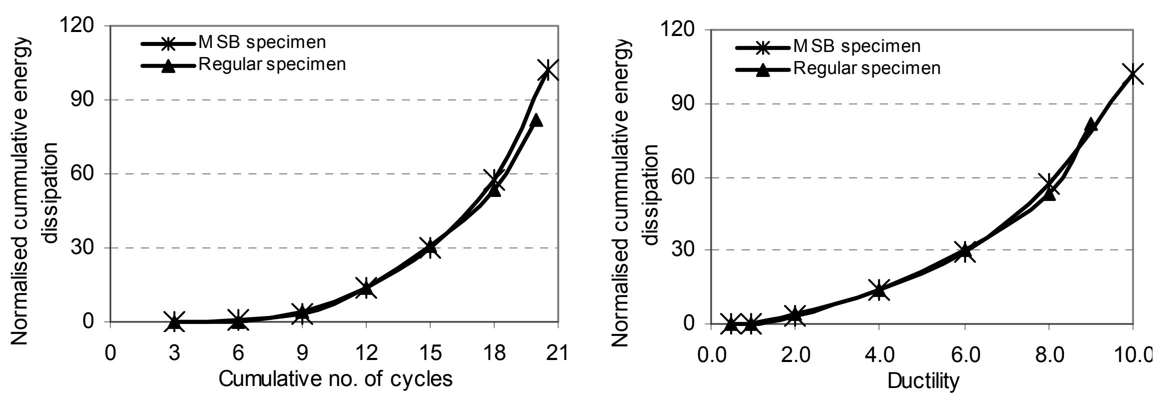
Figure (9) shows the variation in energy dissipation per cycle, using the first complete cycle at each load step, versus peak drift and maximum ductility reached during that same load cycle. It can be observed that up to maximum displacement ductility of 2, both the MSB and the regular braced specimens exhibited similar energy dissipation per cycle. Beyond this ductility level, the MSB specimen showed superior energy dissipation per cycle in each of the load steps until the end of the test. At the maximum ductility of 4, for example, the energy dissipated per cycle by the regular braced specimen was about 77% that by the MSB specimen. For the regular braced specimen, there was no significant deterioration of dissipated energy with cycling in load step 8 (i.e. at 3.05% drift) before the test was terminated. Energy dissipated in the first cycle of this load step reduced by only about 1.5% at the end of the 18th cycle of that same load step.

Figure (10) shows the variation of normalized cumulative energy dissipation with cumulative number of cycles and maximum displacement ductility of the test specimens. Clearly, both specimens dissipated similar amount of normalized cumulative energies. The majority of dissipated energy in the MSB braced specimen was the result of a combination of bending deformation in the tube column segment between the top flange of the ceiling beam and the bottom flange of the floor beam, and tension yielding and inelastic buckling of the bracing members. For the regular braced specimen, energy was mainly dissipated through tension yielding and inelastic buckling of the HSS bracing members.



(a) (b)

Figure 9. Energy Dissipation per Cycle versus (a) Peak Drift (b) Maximum Ductility



(a) (b)

Figure 10. Comparison of Normalized Cumulative Energy Dissipation for Specimens (a) against Cumulative Number of Cycles (b) against Ductility

4. Conclusion

The hysteretic characteristics of modular steel building (MSB)-braced frame under repeated cyclic loading has been evaluated in this paper. The strength, stiffness, ductility, and cumulative hysteretic energy dissipation capacity were assessed. The design and construction of the test specimen accounted for the building's unique detailing requirements. A regular concentrically-braced frame with similar physical characteristics, such as clear height and member proportions and properties, was also tested to compare behaviour. The results of the tests revealed some similarities as well as some differences in hysteretic behaviour and characteristics. The differences particularly suggest that for improved performance of the MSB-braced system, its unique detailing requirements, such as the vertical connection of consecutive stories, need to be incorporated in their design. This would allow the frame system to develop its full capacity as predicted by design.

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