



Assessment of Overstrength and Ductility of a Four-story Modular Steel Building Braced frame

C.D. Annan¹, M.A. Youssef² and M.H. El Naggar³

¹Ph.D. Candidate (cannan@uwo.ca); ²Assistant professor (myoussef@uwo.ca); ³Professor (helnaggar@uwo.ca)
Dept of Civil and Environmental Engineering, The University of Western Ontario, London, ON, N6A 5B9, Canada

Abstract

Structural overstrength and ductility are two key characteristics that affect a reasonable assessment of the vulnerability of a building to seismic events. Overstrength results from sources inherent in the structural system and its response mechanism under loading, as well as design assumptions and simplifications. Ductility on the other hand is tied to the inelastic characteristics of the structural system, such as energy dissipation and strength degradation. In this paper, these parameters are assessed for a braced frame of a four-storey modular steel building using non-linear static analyses. The design of the frame, particularly of columns, considers two widely used assumptions/simplifications; the Square Root of the Sum of the Squares (SRSS) approach and the Direct Summation approach. Analytical modeling of the modular braced frame takes into account the unique detailing requirements of this structural system. The implication of the results obtained from the analyses to the design of modular steel building braced frames is presented.

Keywords: modular steel building, overstrength, ductility, braced frames.

Introduction

The seismic behaviour factor, R , is a critical parameter in contemporary seismic design. It has been used to reduce the code-specified forces resulting from idealized elastic response spectra, which are representative of site seismicity. In the 2005 edition of the National Building Code of Canada (NBCC 2005), the R factor consists of ductility related force modification factor, R_d , and overstrength related force modification factor, R_o . The choice of these factors for design primarily depends on the structural system type. R_d is tied to the inelastic characteristics of the structural system, such as energy dissipation and strength degradation. Structural systems with large energy dissipation capacity have large R_d values, resulting in design for lower forces than systems with relatively limited energy dissipation capacity. It is also observed that building structures usually possess a considerable amount of reserve strength due to factors such as material effects and properties, structural system configuration and design assumptions and

simplifications. This extra strength is known to be one of the key characteristics, which influence seismic response of these building structures.

Several sources of overstrength have been identified and reviewed by Uang (1991). Many studies, both experimental and analytical, have been conducted to assess overstrength of different structural systems ([Uang and Bertero 1986](#); [Rahgozar and Humar 1998](#); [Balendra and Huang 2003](#); [Kim and Choi 2005](#)). Nonlinear static analysis has been a reliable analytical tool employed to evaluate this parameter ([Rahgozar and Humar 1998](#); [Balendra and Huang 2003](#); [Kim and Choi 2005](#)). [Rahgozar and Humar \(1998\)](#) conducted studies of 2 to 30 storey concentrically steel braced frames designed for the same lateral load and observed that, the main parameter that controls the reserve strength is the slenderness ratio of bracing members. They found the height of the building and the effect of building sway to have almost no effect on overstrength. For the frames considered, the average reserve strength accounting for internal force redistribution was about 2.1. [Balendra and Huang \(2003\)](#) observed from an investigation of two different configurations of concentrically braced frame (CBF) that overstrength and ductility factors are almost the same for inverted V-braced and split X-braced frames having the same height. Comparative studies have also shown a generally good match between results of nonlinear static pushover analysis and nonlinear incremental dynamic analysis ([Kim and Choi 2005](#)). The NBCC (2005) recommends an overstrength factor of 1.3 for both moderately and limited ductile concentrically steel braced frame, regardless of the height of the building and the magnitude of design earthquake. It is important to note that overstrength factors provided by different codes can only be achieved by applying the design and detailing provisions of the appropriate standard.

Many new building codes and standards (CISC 2001; SEAOC 2005; ASCE 2005) require design for ductile response. Provision of ductile structures is essential in ensuring seismic survival. Such seismic-resistant building structures must possess the ability to dissipate energy while undergoing large inelastic deformations. This response is achieved by adopting appropriate design strategies that allow for stable and reliable hysteretic energy-dissipation mechanism and avoid conditions that may lead to brittle failures. In this regard, the concept of capacity design emerges as an important design tool. Capacity design allows the designer to take advantage of zones of considerable plastic deformation capacity by controlling the failure mechanism of the frame and dictating where inelastic deformation should and should not occur.

The analytical definition of structural overstrength is reasonably established. Considering a typical structural response envelope in Fig. 1, showing the relationship between base shear, V , and roof displacement, Δ , the structural overstrength accounting for all possible sources can be defined by Eq. (1):

$$R_0 = \frac{V_y}{V_d} \quad (1)$$

where V_y is the load that corresponds to the achievement of the specified failure mode and V_d is the design base shear. For the reserve strength that accounts for redistribution of internal forces in the inelastic range, V_d would represent the load corresponding to the first significant yield. Displacement ductility, μ , is defined in terms of maximum structural drift (Δ_u) and the displacement corresponding to the idealized yield strength (Δ_y) as

$$\mu = \frac{\Delta_u}{\Delta_y} \quad (2)$$

The actual force reduction factor R_d is a factor, which reduces the elastic force demand to the level of the maximum yield strength V_y .

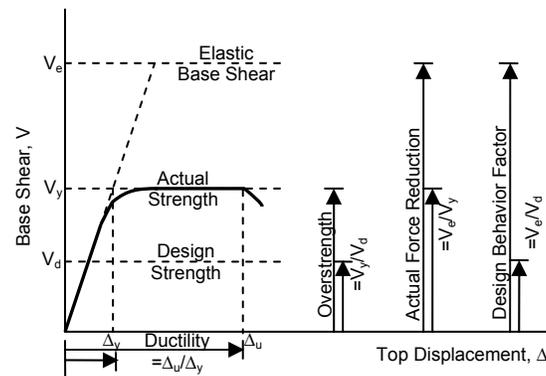


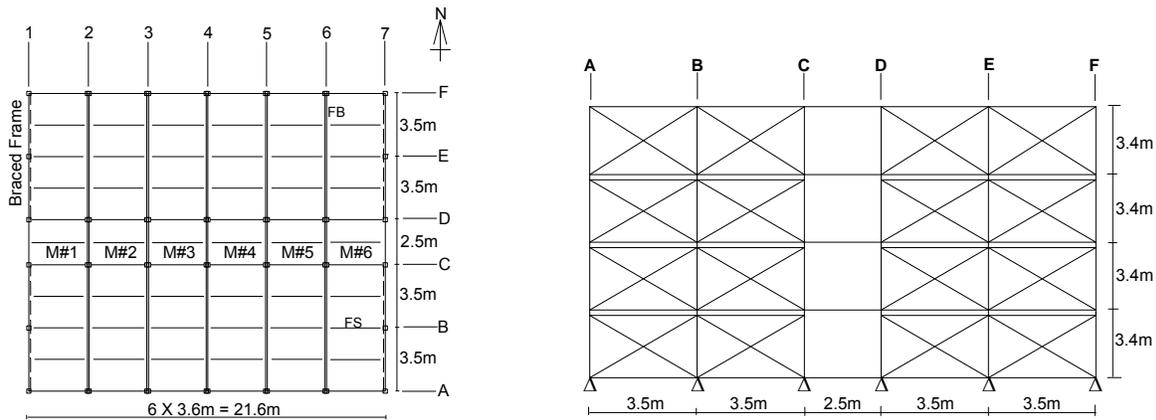
Figure 1. Typical structural response envelope

Modular steel buildings (MSBs) are fast becoming an effective alternative to traditional on-site steel building. They have been typically used for one-to-six storey schools, apartments, hotels, correctional facilities, dormitories and other buildings where repetitive units are required. The modular technique involves the design of buildings, which are built and finished at one location and transported to be used at another. The finished units of a MSB are connected both horizontally and vertically onsite. Lateral stability of the entire MSB is achieved by adding diagonal braces. Typical details of the MSB system has been fully described in Annan et al. (2007). Components of the system, including the floor and ceiling framing, are connected together mainly by direct welding. Results of a finite element study on the response of the MSB floor framing system under gravity loading (Annan et al. 2005) showed that the direct welding between floor beams and floor stringers of the MSB floor framing system significantly affect the design of the stringers but have a negligible effect on the design of the floor beams.

The braced frame system of MSBs is clearly different from regular steel braced frames and may respond differently to seismic excitations. In terms of structural configuration, the following specific features distinguish MSBs from conventional steel building construction: 1) the existence of ceiling beams in MSBs is expected to result in unique natural periods and mode shapes, 2) in a typical modular steel frame, brace members do not intersect at a single working point leading to high seismic demands on the vertical connections, 3) vertical connections typically involve welding one face of the columns of a lower and an upper modules leading to independent upper and lower rotations at the same joint. Currently, conventional procedure is followed in the design of MSBs. With their unique features identified, it is the aim of this investigation to assess structural overstrength and displacement ductility of a braced frame of a four-storey MSB. The MSB braced frame is designed for ductility using Canadian standards (CISC 2001) and capacity design procedures. Two widely-used assumptions in capacity design of braced frame columns are considered in the design. 2-D frame systems are modeled using the non-linear finite element computer program, SeismoStruct (SeismoSoft 2003). Special attention is given to the unique detailing requirements of a MSB. Non-linear pushover analyses are conducted to determine the ultimate lateral load resistance and ultimate structural drift.

Design and Analysis of Modular Steel Braced Frame

A four-storey modular steel dormitory is considered in the study. Fig. 2 shows a floor plan and elevation of the building. Each storey is made up of six modular units, labelled M#1 to M#6, comprising twelve individual rooms and a corridor. A floor framing of a modular unit is composed of two floor beams, a number of floor stringers and a metal deck with concrete composite floor. The composite floor within a modular unit is rigid in-plane. The horizontal connections between modular units are designed for the elastic earthquake force. The combined behaviour of the composite floor of the modules and the horizontal connections between these modules is found to be sufficiently rigid to transfer lateral loads between the modular floor units and to the braced frames. The ceiling framing includes two ceiling beams and a number of ceiling stringers. The corridor on each floor runs through the middle portion of all the modular units, between the two interior columns. The corridors are without ceiling beams to allow mechanical and electrical ducts to run along it. Only the lateral response of the MSBs in the N-S direction is considered in this study. The lateral force resisting system in this direction is composed of two external X-braced frames (centrelines 1 and 7) as shown by the dashed lines within units M#1 and M#6 in Fig. 2a. These two frames are identical and only one (centreline 7) is considered in the study for each building height. In these frames, the braces are connected to the floor beam-to-column and ceiling beam-to-column joints in each storey. Brace connections to the modular framing system are composed of gusset plates welded to the braces. For the vertical connection of units of these frames, welding is provided only at the outer faces of all of the columns (i.e. on centerlines A, B, C, D, E, and F).



(a) Floor Plan

(b) Elevation (centerline 1 or 7)

Figure 2. 4-storey modular steel braced frame

When designing the MSB braced frame, frame members were initially sized on the basis of traditional strength and stiffness design criteria for the specified imposed gravity and earthquake actions. Then, the braces, columns, floor beams, and ceiling beams sizes obtained from the strength design were evaluated and modified, as necessary, according to ductility design requirements and capacity design procedures. The strength and ductility designs were based on the Canadian standard (CISC 2001). The dead load from a typical floor is composed of the weights of the concrete floor, an all round metal curtain wall system and insulation, a steel deck and the self-weight of the frame members. Superimposed dead load of 0.75, 0.32, and 0.7 kN/m² were applied to account for additional loads on floor, roof, and ceiling respectively. The live loads used for the design were based on the National Building Code of Canada (NBCC 2005)

and are 1.9 kN/m² for the individual rooms and 4.8 kN/m² for the corridors. A snow load of 1.0 kN/m² was assumed for the roof. The seismic loading on each frame was based on the NBCC Equivalent Static Approach (NBCC 2005). The location of the MSBs was selected as Vancouver, British Columbia, Canada. The buildings were assumed to be founded on a very dense soil with a shear wave average velocity range between 360 m/s and 760 m/s. The design base shear values of the frames were calculated assuming moderate ductility with an overstrength factor of 1.3 and a ductility factor of 3.0 as per the NBCC (2005). The design base shears were distributed over the height of the building according to this code.

CISC Grade 350W steel with a specified yield stress, F_y , of 350MPa was used to design the beam, column and brace members in accordance to the Canadian standard, CAN/CSA-S16.1-01 (CISC 2001). The least weight section required for strength for each frame element was selected. For all brace members and columns, specified sections were limited to a square hollow structural section (HSS), which is widely used in the MSB industry. W shape sections were specified for the floor, ceiling and roof beams as per common practice. Column 3 of Table 1 gives a summary of the resulting sections from the strength design for each member of the modular braced frame.

Table 1. Member sections from strength and ductility designs of a 4-storey MSB braced frame

Frame Member	Story / Floor #	Strength Design	Ductility Design (column design by SRSS approach)	Ductility Design (column design by DS approach)
Braces	4	HS 76X76X5	HS 76X76X6	HS 76X76X6
	3	HS 76X76X5	HS 76X76X6	HS 76X76X6
	2	HS 89X89X6	HS 89X89X6	HS 89X89X6
	1	HS 89X89X6	HS 89X89X6	HS 89X89X6
Columns	4	HS 76X76X5	HS 102X102X6	HS 102X102X6
	3	HS 178X178X5	HS 178X178X6	HS 178X178X6
	2	HS 178X178X5	HS 203X203X6	HS 203X203X10
	1	HS 178X178X6	HS 203X203X8	HS 254X254X10
Beams	Roof	W100X19	W100X19	W100X19
	Floor 4	W100X19	W100X19	W100X19
	Floor 3	W100X19	W100X19	W100X19
	Floor 2	W100X19	W100X19	W100X19
	Floor 1	W100X19	W100X19	W100X19
	Ceiling	W100X19	W100X19	W100X19

The bracing members are assumed to belong to class H (hot-formed or stress relieved) of the CAN/CSA-S16.1-01 standard. Brace member capacities were calculated based on the same standard. The buckling strength, C_r' , of compression brace members is given in this code in terms of the compressive yield strength, C_r , and the slenderness coefficient, λ , by Eq. (3):

$$C_r' = \frac{C_r}{1 + 0.35\lambda} \quad (3)$$

The ductility provision by the Canadian code (CISC 2001) for the design of steel braced structures is based on the assumption that the braces reach their ultimate strength, and the columns, beams and brace connections within the structure must be able to resist the resulting induced forces. Specific requirements for brace members are given in clause 27 of the CAN/CSA-S16.1-01 standard (CISC 2001). The effect of the reduction in compressive strength of the brace members due to repeated buckling (Jain and Goel 1978) was accounted for by checking the forces in the bracing members against the reduced brace compressive strength,

given by Eq. 3. In the common case where the tension brace in the same bent and at the same level had excess capacity to compensate for this reduction in compressive strength, the reduction factor, $[1/(1+0.35\lambda)]$, was not applied. Columns 4 and 5 of Table 1 contain a summary of the brace member sections for ductile response of the MSB braced frame.

In the ductility design of the ceiling, roof and floor beam members, the effect of redistribution of loads due to brace buckling or yielding are considered in for the beams in braced bays. Beams are thus designed as beam-columns, with the design moment resulting from tributary gravity loads and the axial compression coming from unequal capacity of braces in tension and compression, considering a horizontal equilibrium of brace induced forces at each beam end. The configuration of the braced frame would clearly play a significant role in determining these axial loads in the beams. Redistribution of loads due to brace buckling or yielding can only be determined for beams located within braced bays. The resulting section at any level is applied to beams located in non-braced bays at the same level. A summary of the beam member sections resulting from the ductility design is also shown in Table 1.

The column members obtained from the strength design were also reviewed to meet ductility requirements. According to the Canadian code (CISC 2001), columns are to be proportioned to resist the gravity loads together with the forces induced by the brace loads. In order to meet this requirement, many structural engineers design the columns to withstand accumulation of the vertical components of yielding and buckling brace forces in addition to gravity loads. For a multi-storey frame, however, a widely used approach for capacity design of column at lower levels is based on the assumption that all the bracing members would not reach their capacities simultaneously. Thus, a statistical accumulation of earthquake-induced brace forces using the Square Root of the Sum of the Squares (SRSS) approach (Khatib et al. 1988) is sometimes used instead of a direct summation (DS) of the vertical components of yielding and buckling brace loads. The SRSS approach has been found to be reasonably conservative for regular braced frames. The effect of any of these approaches adopted in design on the performance of MSB braced frames was presented by Annan et al. (2007). In the current study, both the SRSS and the DS assumptions/simplifications were considered to study their effect on structural overstrength and ductility. In the SRSS approach, the induced force in a column under consideration is taken as equal to the vertical components (nominal capacity) of braces connected to the top of the column, plus the SRSS of load components of braces above the column under consideration. The resulting loads are combined with specified dead and live loads. The Direct Summation (DS) simplification, where column actions are derived from a direct sum of vertical components of yielding and buckling brace forces, results in much higher forces for columns located at lower levels of the braced frame. Columns 4 and 5 of Table 1 also contain a summary of the revised column sections obtained from the use of the SRSS and DS accumulation approaches. There is significant difference in sizes of columns located at lower levels of the frame (i.e. first and second stories) resulting from the two load accumulation approaches. It is noted that column sections at all levels of the MSB frame obtained from strength design are found to be inadequate for the required ductility for both the SRSS and DS accumulation approaches.

The brace end connections are expected to remain elastic at all times. They are therefore designed to support the full yielding brace resistance, given by the brace nominal tensile strength, $A_g F_y$. The design of the vertical welded connections of units of the MSB is based on traditional elastic method and it accounts for the eccentric loading which results from welding one side (i.e. outside faces) of the connected columns. The Canadian standard (CISC 2001) is used in the design of these welded connections.

The SeismoStruct nonlinear computer program (SeismoSoft 2003) is employed in the modeling and analysis of the MSB braced frame. Two-dimensional models are developed based on centerline dimensions. This is deemed sufficient for the objectives of the study. A bilinear material model for steel is employed, with a kinematic strain hardening parameter of 1%, a yield stress of 350 N/mm^2 , and an elastic modulus of $200 \times 10^3 \text{ N/mm}^2$. Inelastic beam-column frame element, which employs a cubic shape function, is used to represent all structural frame members. This element type accounts for geometric and material nonlinearities. The element formulation is based on the fibre modeling approach that models the spread of material inelasticity along the member length and across the section area so as to allow for an accurate estimation of structural damage distribution. A joint element with uncoupled axial, shear and moment actions is utilised to simulate the assumed pin-jointed behaviour at the ends of bracing members. All beam-column joints are assumed rigid to represent the fully welded direct connection between these members in MSB framing.

The model of the vertical connection of different modular units is likely to influence the lateral response of the entire frame. These vertical connections typically involve welding one face (i.e. the outer face) of the columns of a lower and upper frame units, leading to independent upper and lower rotations at the same joint. A model that utilises a number of rigid elastic beam-column elements and a joint element is developed and used. The joint element is defined between the two vertical units to capture the independent rotation expected between column members present at this vertical connection.

Under a strong earthquake, a brace member in a CBF will be subjected to large inelastic deformations in cyclic tension beyond yield and compression into the post-buckling range. The post-elastic compression behaviour will play an important role in seismic performance evaluation as significant degradation in compressive resistance results after a few cycles of loading. In nonlinear static procedure, the reduction in strength of a brace after buckling is included in the model by assuming an elastoplastic brace behaviour for the compression brace with a yield force taken as the residual strength after buckling (Rahgozar and Humar 1998; FEMA 2000). The modeled MSB frames were subjected to static non-linear pushover analyses. The gravity loads, lumped at nodal points, were held constant while the magnitude of lateral forces with an assumed triangular distribution pattern along the building height is gradually increased until the formation of structural mechanism.

Results and Discussions

Primarily, overstrength is a direct consequence of redundancies resulting from member effects, system/structural configurations and design assumptions/simplifications. The main simplification in the design procedure for CBFs is related to the treatment of buckling and post-buckling behaviour of compression brace members. Many of the sources of overstrength can be easily identified but not all can be readily quantified. Redistribution of internal forces in the inelastic range due to redundancy in the structural system is probably the most dependable estimate of overstrength. For tension-compression braced frames, overstrength arises mainly from the difference between the load causing buckling of compression braces and the load required to develop yielding in the tension braces. For ductile CBFs, overstrength is generally identified as the difference between the strength corresponding to the first buckling of a compression brace and the ultimate lateral strength of the structure. The first brace buckling strength would coincide with the design strength of the structure if internal force redistribution in

the inelastic range was the only source of overstrength. The analyses results for the two design approaches showed that the base shear force corresponding to the first buckling of a compression brace member is higher than the design base shear. The design base shear was however used to evaluate overstrength resulting from redistribution of internal forces in the inelastic range, design assumptions and strain hardening behaviour of steel. In the Canadian code (NBCC 2005), the overstrength factor accounting for the braced system's ability to mobilize full capacity before collapse (i.e. due to redistribution of internal forces) is conservatively set to unity in view of the strength degradation of compression braces under reversed cyclic loading. Results of several studies of regular braced frames (Uang and Bertero 1986; Whitaker et al. 1989; Rahgozar and Humar 1998) suggest the code's provision to be rather conservative.

The capacity envelopes obtained from nonlinear pushover analyses were used to estimate the reserve strength ratio. The base shear force versus lateral roof drift for the MSB frames under the two design assumptions (SRSS and DS) are depicted in Figs. 3a and 3b. The roof drift is defined as the ratio of the top displacement to the height of the MSB frame. In both design cases, failure of the MSB frame is caused by formation of a collapse mechanism when the frame is no longer able to carry additional loads. Results of the analyses revealed that overstrength of the four-storey MSB resulting from the use of both the DS accumulation approach and the SRSS approach is the same. This suggests that for this building height, overstrength is only sensitive to the properties of brace section and less so to columns sectional properties as it arises mainly from the difference between the load causing buckling of compression braces and the load required to develop yielding in the tension braces. The reserve strength of a critical story is also the global reserve strength of the frame. The overstrength factor was calculated as 2.2 for the MSB braced frame resulting from both design cases, indicating that lateral forces 120% greater than those considered during design are necessary to trigger failure of the frames. It is noted that the overstrength ratio estimated for the MSB braced frame above was obtained from the use of nominal material properties and the actual overstrength accounting for material effects may be higher. Thus, the use of R_0 given by the Canadian code (NBCC 2005) is shown to be conservative for the design of MSB frames.

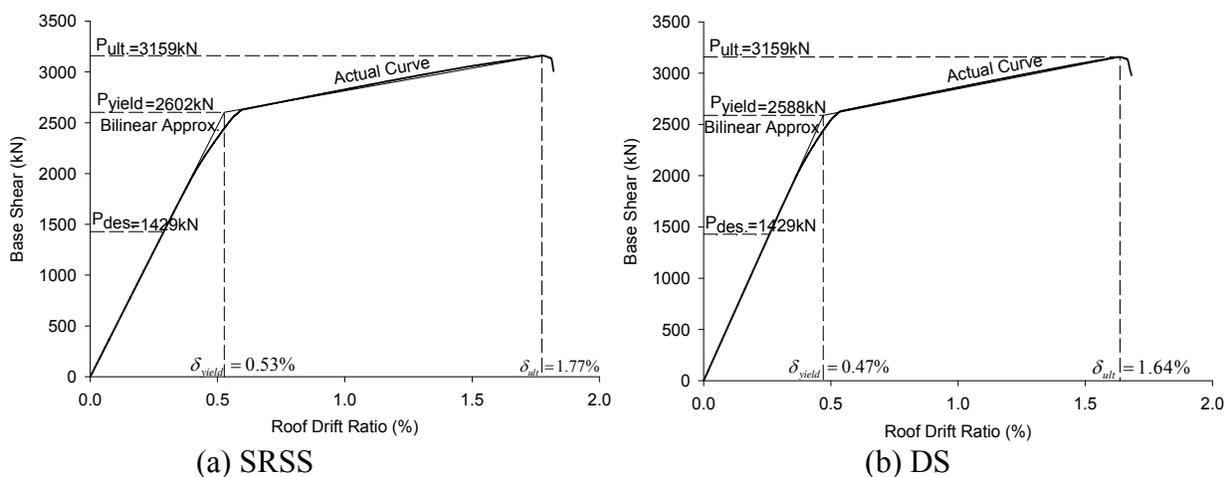


Figure 3. Horizontal capacity curve of a four-storey MSB braced frame

Figs. 3a and 3b also provide some evidence that the four-storey MSB frame shows some ductile behaviour. Structural ductility is defined as the ratio of the ultimate structural drift to the

displacement corresponding to the yield strength. Global yield displacement for braced frames is essentially controlled by buckling of bracing members which in turn depends on the brace member sectional properties (i.e. brace slenderness ratio and width-to-thickness ratio, b/t). The yield strength can be obtained by idealizing the actual structural response curve by a bilinearly elasto-plastic curve, as shown in the figures, such that the total energy dissipation up to the point of ultimate deformation before collapse is the same for both curves. It is known that this simplified response idealization is well representative only for systems that can dissipate energy in a stable manner, especially in simple single storey frames. For multi-storey buildings, especially those that exhibit significant strength degradation, the definition of the yield deformation is more complicated and analytical methods may not be very reliable in estimating structural ductility. The behaviour of the MSB has so far not been studied extensively to conclude on its energy dissipation characteristics. Nonetheless, yield displacement of the four-storey MSB braced frame for both design assumptions considered in the study were obtained by this simplified method. As shown in Fig. 3, there is a slight difference in yield displacement resulting from the two design assumptions (0.53% roof drift ratio for SRSS against 0.47% roof drift ratio for DS). The maximum structural drift is controlled by the capacity of tension braces as well as the mechanics of internal force redistribution in the inelastic range from compression braces to tension braces. The latter is largely influenced by the behaviour of the entire braced frame, including the boundary frame behaviour. The configuration of the MSB braced frame, particularly the vertical connections of different modular units, would influence the behaviour of the entire frame. As shown in Fig. 3, there is also a slight difference (similar to the yield displacement) in the maximum structural drift resulting from the two design assumptions (1.77% roof drift ratio for SRSS against 1.64% roof drift ratio for DS). Displacement ductility was estimated as 3.3 and 3.4 for the MSB braced frame under the SRSS and DS design assumptions respectively.

Conclusions

This paper has highlighted some unique features of the MSB and has assessed structural overstrength and displacement ductility of a four-storey MSB braced frame using nonlinear static pushover analyses. The frame was designed for ductility using Canadian standards and following capacity design procedure. The SRSS and DS design assumptions were considered in capacity design of frame columns. Modeling and analyses of the braced frames were done using the nonlinear finite element computer program, SeismoStruct. The unique detailing requirements of the MSB were particularly taken into account during modeling. The analysis results revealed that the four-storey MSB braced frame possess considerable overstrength due to intrinsic redundancies in the frame system. Overstrength ratio was independent of the two design assumptions considered in the study. Overstrength factor for this frame under the two assumptions was evaluated to be 2.2. The use of the overstrength value provided by the Canadian code in designing MSB system may, thus, to be conservative. The results also show significant displacement ductility in the MSB frame system considered. This was assessed to be 3.3 for the frame under the SRSS design assumption and 3.4 under the DS assumption.

Acknowledgements

This research was funded by the National Sciences and Engineering Research Council of Canada (NSERC).

References

1. Annan, C. D., M. A. Youssef, and M. H. El-Naggar, 2005. Analytical investigation of semi-rigid floor beams connection in modular steel structures, *33rd Annual general conference of the Canadian Society for Civil Engineering*, Toronto, Ontario, Canada.
2. Annan, C. D., M. A. Youssef, and M. H. El-Naggar, 2007. Seismic performance of modular steel braced frames, *Ninth Canadian conference on earthquake engineering*, Ottawa, Ontario, Canada.
3. ASCE (2005), "ASCE/SEI 7-05: Minimum design loads for buildings and other structures", *Standard Committee, American Society of Civil Engineers*, Virginia, USA.
4. Balendra, T. and Huang, X. (2003), "Overstrength and Ductility factors for steel frames designed according to BS5950", *Journal of Structural Engineering, ASCE*, 129 (8), 1019-1035.
5. CISC, 2001. *Handbook of Steel Construction, 7th Edition*, Canadian Institute of Steel Construction, Willowdale, Ontario, Canada.
6. FEMA (2000), "*Prestandard and Commentary for the seismic rehabilitation of buildings*", prepared by the American Society of Civil Engineers for the Federal Emergency Management Agency, Washington, D.C, (FEMA Publication No. 356).
7. Jain, A. K., and S. C. Goel, 1978. Hysteresis models for steel members subjected to cyclic buckling or cyclic end moments and buckling, *Report UMEE 78R6*, Department of Civil Engineering, University of Michigan, Ann Arbor, USA.
8. Kim, J. and Choi, H. (2005), "Response modification factors of chevron-braced frames", *Engineering structures*, 27, 285-300.
9. Khatib, I. F., S. A. Mahin, and K. S. Pister, 1988. Seismic behavior of concentrically braced steel frames, *Report UCB/EERC-88/01*, Earthquake Engineering Research Center, University of California. Berkeley, CA, USA.
10. NBCC, 2005. *National Building Code of Canada*, Institute for Research in Construction, National Research Council of Canada, Ottawa, Ontario, Canada.
11. Rahgozar, M. A., and J. L. Humar, 1998. Accounting for overstrength in seismic design of steel structures, *Canadian Journal of Civil Engineering*, 25, 1-15.
12. SEAOC (2005), "Recommended lateral force requirements and commentary", *Structural Engineers Association of California, Seismology Committee*, Sacramento/ San Francisco/ Los Angeles, USA.
13. SeismoSoft, 2003. SeismoStruct - A computer program for static and dynamic nonlinear analysis of framed structures, Available from URL: <http://www.seismosoft.com>.
14. Uang, C. M. (1991), "Establishing R (or R_w) and C_d factors for building seismic provisions", *J. Struct. Eng. ASCE*, 117, 19-28.
15. Uang, C.M. and Bertero, V.V. (1986), "Earthquake simulation tests and associated studies of a 0.3-scale model of a 6-story concentrically braced steel structure", *Report No. UCB/EERC-86/10, Earthquake Engineering Research Center; Univ. of California, Berkeley, CA, USA.*