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SEISMIC PERFORMANCE OF MODULAR STEEL BRACED FRAMES

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ABSTRACT

Capacity design procedure for regular steel braced frames is reasonably established. For concentrically braced frames, the general implication is to allow the diagonal braces to yield and buckle while protecting other frame members and components from inelastic deformation. These frame members and components are therefore designed to support induced forces due to yielding and buckling braces. In this study, conventional capacity design procedure is adopted for the design of typical braced frames of Modular Steel Buildings. The SRSS approach of accumulating brace induced forces as well as the "Direct Summation" approach are both considered in the capacity design of their columns. The frames are modeled and analysed by the nonlinear static pushover method. The analyses results are verified against the expected behaviour based on the design philosophy. It is concluded that some unique detailing requirements of modular steel buildings need to be considered during design in order to avoid undesirable seismic response.

INTRODUCTION

The concept of capacity design plays an important role in seismic performance and design. In contemporary seismic design codes, significant inelastic deformations are accommodated under severe earthquakes. The seismic behaviour factor, R, is widely utilised to reduce forces resulting from idealised elastic response spectra, which is representative of site seismicity. Capacity design then allows the designer to take advantage of regions or zones of considerable plastic deformation capacity. Once these regions are identified and detailed to enable the desired inelastic response, other regions in the frame are treated to respond elastically. The designer therefore has control over the failure mechanism of the frame and can dictate where inelastic deformation should and should not occur. For concentrically braced frames, inelastic deformation in the bracing members is the main source of dissipating seismic energy. These brace members are therefore designed to be capable of sustaining significant inelastic deformation in either compression or tension without significant loss of strength and stiffness. They are detailed to ensure that they go through the expected inelastic demand without premature failure. For other members and components (i.e. beams, columns and connections), sufficient capacity is made available to resist the maximum forces that would develop in them as a result of yielding and buckling of bracing members.

The Modular Steel Building (MSB) is fast evolving as an effective alternative to traditional on-site steel building. 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. MSBs make use of hot-rolled steel sections for enhanced strength and durability. They have been typically used for one-to-six storey schools, apartments, hotels, correctional

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facilities and dormitories, where repetitive units are required. Fig. 1 shows typical details for a MSB. A typical storey of a MSB structural frame 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. Results of a study on the response of MSB floor framing system under gravity loading (Annan et al. 2005) showed that the direct welding connections existing 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 horizontal connection (HC) between the units of a 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). Only the outer faces of these columns that are accessible during assembling are welded and the weld covers the full width of the columns. Lateral stability of the entire MSB is achieved by adding diagonal braces as shown in Fig. 1. Lateral loading on each floor is transferred through the horizontal connections to the modular braced frame and then through the vertical connections to the foundation.

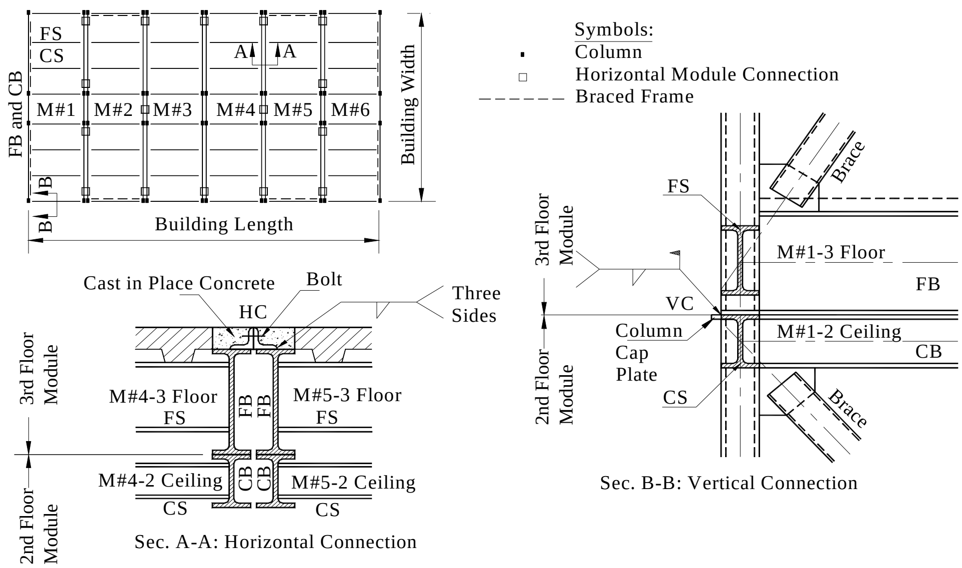


Figure 1. Typical details for a multi-story modular steel building (MSB)

The braced frame system of MSBs is clearly different from regular steel braced frames and may respond differently in an event of any lateral shaking due to an earthquake. 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, and 4) the connections between floor beams and columns and also ceiling beams and columns are achieved using direct welding which is unconventional for regular steel buildings. With these unique features identified, it is the aim of this investigation to evaluate the performance of typical braced frames of MSBs, designed for ductility using Canadian standards (CISC 2000) and following the most widely used capacity design procedures for regular steel braced frames.

Three heights of MSB are considered in the study: two-storey, four-storey and six-storey modular steel dormitories. The total heights of the three buildings are respectively 6.8 m, 13.6 m and 20.4 m. All the buildings have the same overall plan dimensions of 21.6 m by 16.5 m. Fig. 2a shows a floor plan of the MSBs considered in the study. Each story 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/ concrete composite floor. Similarly, 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 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 so only one (centreline 7) is considered in the study for each building type. 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 achieved only in the outer faces of all of the columns (i.e. on centerlines A, B, C, D, E, and F). Fig. 2b shows the elevation of the braced frame of the four-storey MSB. For ease of discussions in subsequent sections, all columns located on centerlines A and F as shown in Fig. 2b will be referred to as outer external columns and columns located on centerlines C and D will be referred to as inner external columns. Columns that are located on centerlines B and E will be referenced as internal columns. The MSB braced frames are designed for ductility. 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 MSBs. Non-linear push-over analyses are conducted to determine the ultimate lateral load resistance as well as the sequence of yielding/buckling events.

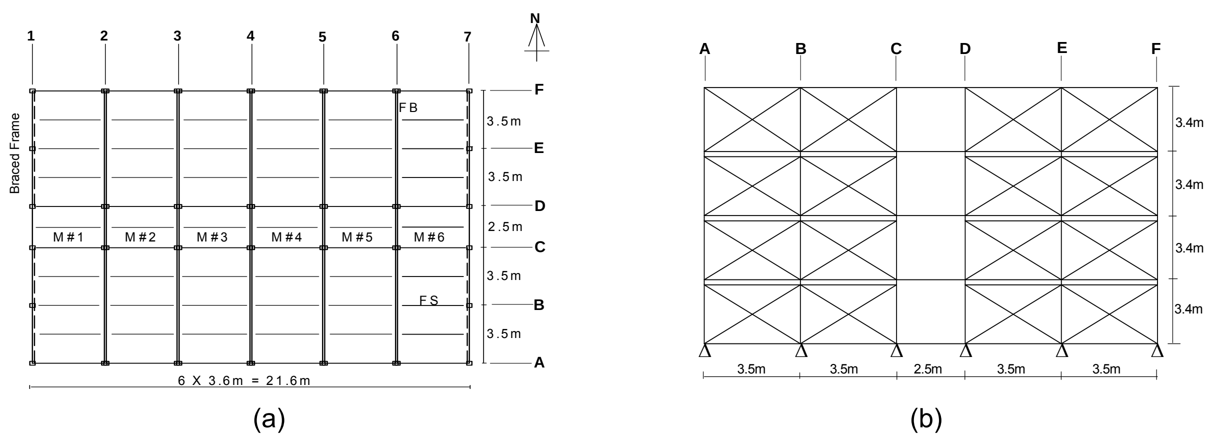


Figure 2. (a) Floor plan of MSBs in the study; (b) Elevation of a 4-story modular steel braced frame (frame on centerline 1 or 7)

DESIGN AND ANALYSIS OF MODULAR STEEL BRACED FRAMES

In the design of the MSB braced frames, 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 and floor 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 2000). 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/m^2 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/m^2 for the individual rooms and 4.8 kN/m^2 for the corridors. A snow load of 1.0 kN/m^2 was assumed for the roof. The seismic loading on each frame was based on the NBCC Equivalent Static Approach (NBCC 2005), which is based on uniform hazard values corresponding to a 2% in 50-year probability of exceedance. The location of the MSBs was selected as Vancouver, in 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 CAN/CSA-S16.1-94 standard (CISC 2000). 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. Demand/capacity ratios for axial, flexural and shear, based on factored loads and factored resistances were used as the criterion for the selection of optimal sections. Additionally, selected sections were modified to conform to more practical arrangements. Table 1 gives a summary of the resulting sections from the strength design for each member in the four-storey modular braced frame.

Table 1. Member sections resulting from strength and ductility designs of a 4-story 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

Brace member capacities were calculated based on the Canadian standard, CAN/CSA-S16.1-94 (CISC 2000). According to this code, the tensile yield strength, T_r, and the compressive yield strength, C_r, are respectively given by Eqs. 1 and 2:

$$T_r = \phi A F_y \tag{1}$$

$$C_r = \phi A F_y \left(1+\lambda^{2n}\right)^{-1/n} \tag{2}$$

where the resistance factor ϕ = 0.9, A is the cross-sectional area of the member, F_y is the yield strength of steel, n is a parameter for compression resistance and λ is a slenderness coefficient. The buckling strength, $C_r^{'}$, of compression brace members is given in the code by Eq. 3:

$$C_r^{'} = \frac{C_r}{1+0.35\lambda} \tag{3}$$

In this study, the bracing members are assumed to belong to class H (hot-formed or stress relieved) of the CAN/CSA-S16.1-94 standard with n = 2.24 for hollow structural sections.

The ductility provision by the Canadian code (CSA 1994) for the design of steel 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. For that purpose, the ultimate strength of brace members is to be taken as the nominal resistance. Specific requirements for brace members are given in clause 27 of the CAN/CSA-S16.1-94 standard (CISC 2000). The effect of the reduction in compressive strength of the brace members due to repeated buckling (Jain and Goel 1978, Popov and Black 1981) 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+0.35λ)], was not applied. In other words, if the tension brace in the same level and plane as the compression brace was found to possess sufficient reserve strength, then the compression brace member was sized based on the resistance C_r and not C_r'. Table 1 contains a summary of the brace member sections for ductile response of the four-storey modular steel braced frame.

The column members obtained from the strength design were also reviewed to meet ductility requirements. According to the Canadian code (CSA 1994), columns are to be proportioned to resist the gravity loads together with the forces induced by the brace connection loads. In order to meet this requirement, many 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-story frame, however, a widely used approach for column design for ductility 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; Redwood and Channagiri 1991) is preferred to a direct summation of the vertical components of yielding and buckling brace loads. The SRSS approach has been found to be reasonably conservative for regular braced frames. This approach was considered in the design of columns of the MSB braced frames. In the SRSS approach, the induced force in a column under consideration is taken as equal to the vertical brace components (nominal capacity) at the level of the column, plus the square root of the sum of the squares of all other brace load components at levels above the column under consideration. The resulting loads are combined with specified dead and live loads. Comparing this approach to the Direct Summation (DS) approach, where column actions are derived from a direct sum of vertical components of yielding and buckling brace forces, the latter results in much higher forces for columns located at lower levels of the braced frame. It is noted that induced forces are determined for only external columns, which are likely to be subjected to greater effective brace induced loads than internal columns. The resulting column section at one level is applied to all other columns on the same level in the frame. Table 1 also contains a summary of the revised column sections in the four-storey MSB braced frame obtained from the use of the SRSS accumulation approach as well as the DS approach. There is significant difference in sizes of columns located at lower levels of the frame resulting from the two load accumulation approaches and the difference is much greater for the six-story MSB braced frame. It is noted that column sections at all levels of this MSB frame obtained from strength design are found to be inadequate for the required ductility.

In the ductility design of the ceiling, roof and floor beam members, the effect of redistributed loads due to brace buckling or yielding are considered in the determination of beam actions 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. Redistributed loads due to brace buckling or yielding can only be determined readily 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 of the four-storey modular steel braced frame is also contained in Table 1.

The brace end connections are expected to remain elastic at all times so they could be at least as strong as the bracing member in order to maximize the energy dissipation capacity of the frame. They are therefore designed to support the full yielding brace resistance, given by the brace nominal tensile strength, A_gF_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 only one side (i.e. outside faces) of the connected columns in the MSB frame. The eccentricity of the force would impose bending stresses on the weld. The Canadian standard (CISC 2000) is used in the design of these welded connections. In this standard, reduction coefficients are provided based on the weld length and eccentricity for different weld configurations.

The SeismoStruct nonlinear computer program (SeismoSoft 2003) is employed in the modeling and analysis of the MSB braced frames. For all the modular braced frames considered, two-dimensional models are developed based on centerline dimensions of the bare frames. 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 X 10^3 N/mm^2. Inelastic beam-column frame element, which employs a cubic shape function (Izzuddin 1991), is used to represent all structural frame members. This element type accounts for geometric and material non-linearities. 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. In such elements, the sectional stress-strain state is obtained through the integration of the nonlinear uniaxial stress-strain response of the individual fibres in which the section has been subdivided. The element response (curvatures and stress/strain peak values) is assembled from contributions at two gauss points, where the cross sections can be discretized into a number of monitoring points. 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.

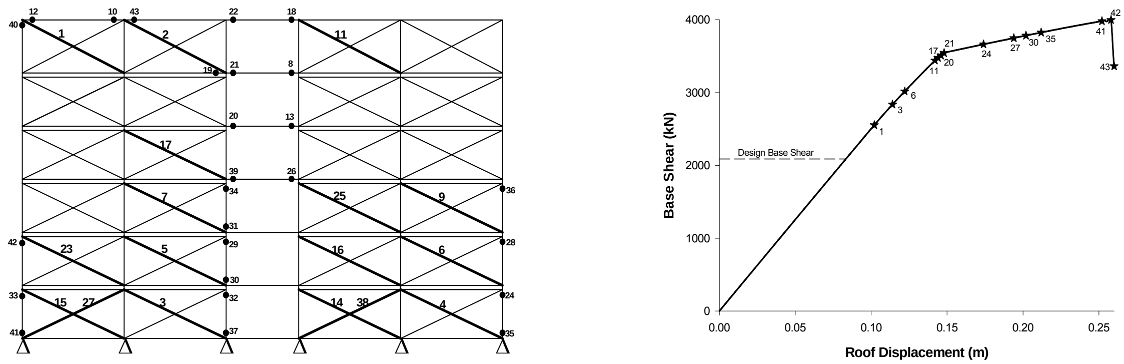
The model of the vertical connection of different frame units is likely to influence the lateral response of the entire frame. These vertical connections typically involve welding one face (i.e. the outer face) of the columns of a lower and upper frame units. This may lead to independent upper and lower rotations at the same joint. The model utilises a number of beam-column elements, which are modeled as rigid elastic elements and they are expected to capture the rigidity within the adjoining columns in the region of the beam-column connections. The lengths of these elements therefore cover the depths of the floor beam and the ceiling beam of the two frame units being connected, and half the widths of their columns. A joint element that captures the relative rotation expected between column members present at this vertical connection is defined to connect the two units together.

During a strong earthquake, a brace member in a concentrically braced frame 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 (Jain and Goel 1978, Papov and Black 1981). In a pushover analysis, it is reasonable to assume that the design buckling strength of the compression brace members is the reduced compressive resistance, $C_r^{'}$ (Rahgozar and Humar 1998). This was adopted in the model of compression bracing members. 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

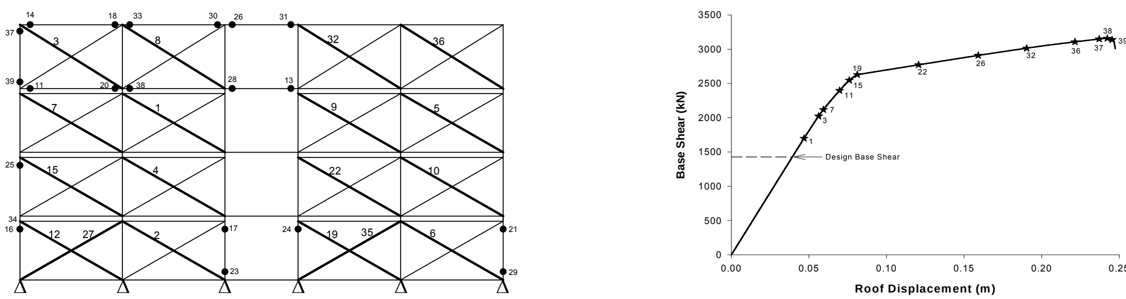
Figs. 3a, 4a and 5a show the order and distribution of plasticity in the six-, four- and two-story MSB frames, where the SRSS approach is utilized in the capacity design of their columns. The filled dots represent plastic hinge formation in the beams and columns. The numbers associated with the dots and on the brace members describe the sequence of plasticity formation or yielding/buckling events, with the number one (1) representing the first member to buckle or yield. It is observed that there is a good distribution of energy dissipation along the height and across the width of the four- and six-story frames. The two-story MSB frame tends to concentrate plasticity distribution in only one-half of the entire frame.

The order and distribution of plasticity in braced frames can be affected by the brace sizes, slenderness ratio and the frame configuration. If the brace sizes are uniform along the height of the frame and the braces have the same slenderness ratio, buckling would most likely occur first in the first story. In ductile concentrically braced frames of regular buildings, the global ultimate strength is controlled by the formation of structural mechanism in one storey. This is because redistribution of internal forces in one story is contained only in that story. Thus, a yielding in the tension braces in one-story results in the formation of mechanism in that story and the structure consequently reaches its ultimate capacity. In the case of the six-story MSB braced frame (Fig. 3), the brace member size is uniform in the first five stories and much smaller in the sixth story, due to the distribution of designed lateral forces along the height of the frame. Buckling starts to occur in the smaller braces located at the sixth story and then followed by braces in the first story. Buckling then progresses up the height of the frame. At any story level, compression braces in the second and fourth braced bays (i.e. counting from the left hand side of the frame) experienced earlier buckling than those in other braced bays. Also, two tension braces at the first story yielded before failure is reached for this frame. Similar trend is observed for the four-story MSB braced frame (Fig. 4). In this frame, the first two stories have the same brace member size and the top two stories also have the same brace size but smaller than that of the lower stories. Buckling of compression braces initiates from the lowest level with the smaller brace section (i.e. the third floor in this frame). For the same brace size, buckling progresses up the height of this frame. The second braced bay experienced early buckling of its compression braces as in the six-story frame and two tension braces at the first story again yielded before failure is reached. In the two-story MSB braced frame (Fig. 5), plasticization is not as well distributed within the frame as the other two frame heights. Failure of this frame is therefore reached before any of the tension braces could yield.



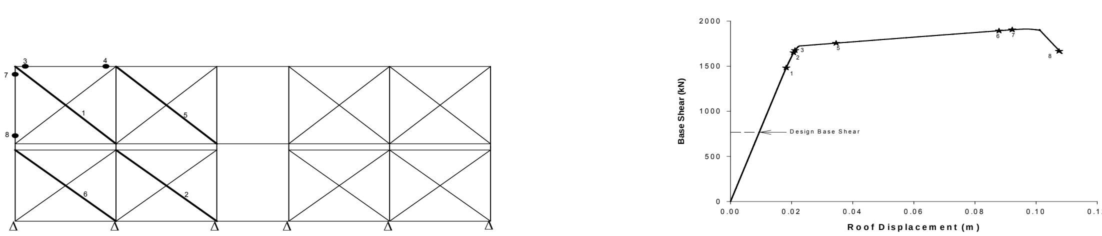
(a) (b)

Figure 3. Sequence of yielding/buckling of the 6-story MSB braced frame (column design by SRSS approach): (a) sequence shown on frame; (b) sequence shown on capacity curve



(a) (b)

Figure 4. Sequence of yielding/buckling of the 4-story MSB braced frame (column design by SRSS approach): (a) sequence shown on frame; (b) sequence shown on capacity curve



(a) (b)

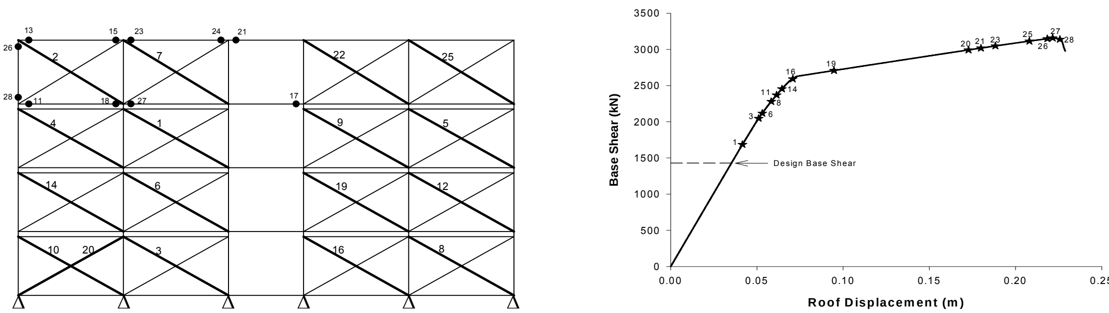
Figure 5. Sequence of yielding/buckling of the 2-story MSB braced frame (column design by SRSS approach): (a) sequence shown on frame; (b) sequence shown on capacity curve

It is observed in Figs. 3a, 4a, and 5a that plastic hinges form mainly in some of the outer and inner external columns located at the lower story levels and also in some outer external column at the topmost story level. Some roof and top floor beams and also some floor beams in non-braced bays of the MSB frames also experienced plasticization. This occurred notwithstanding the design philosophy to prevent yielding or buckling of columns and beams before all of the braces. The internal columns are, as expected, not affected by plasticization because of design simplification that assigns sections resulting from the design of more critical external columns to these internal columns.

The formation of plastic hinges in the columns at the lower levels is more pronounced in the six-storey MSB braced frame than the four-storey and none is present in the two-storey frame. In the two-story MSB braced frame, the SRSS approach utilized for capacity design of its columns results in the same column actions/sections as the direct summation, DS, approach that sums directly vertical components of yielding/buckling brace forces. The variance in design loads for lower level columns from these two load accumulation approaches increases significantly as the number of stories increases from the two-story. In the six-story MSB braced frame, significantly less column actions result from the use of the SRSS approach compared to the DS approach. These observations tend to raise questions about the appropriateness of the use of the SRSS approach for MSB braced frames, especially as it is evident in the analyses results that all the braces do not yield or buckle simultaneously thereby validating the main assumption that governs the use of the SRSS approach. It appears from these observations that the unique vertical connection requirements of different units of the MSB seem to impose an additional demand on the columns, especially those at lower levels. The four-story and six-story MSB braced frames were redesigned for ductility with columns actions obtained from the DS approach and these frames with the resulting member sections were also modeled and analysed by the pushover method. Results of the analyses (i.e. sequence of yielding/buckling) are shown on Figs. 6 and 7 respectively. The distributions of plasticity observed for these frames indicate no formation of plastic hinges in columns located at lower levels. The sequence of brace buckling/yielding are, however, almost similar to those of the four-story and six-story MSB braced frames in which the SRSS approach is utilised to determine column actions (i.e. Figs. 3a and 4a). The use of the SRSS approach for determining column actions in capacity design, therefore, does not appear conservative for MSB braced frames. Rather, the DS approach seems to be yielding the desired response for these frames.

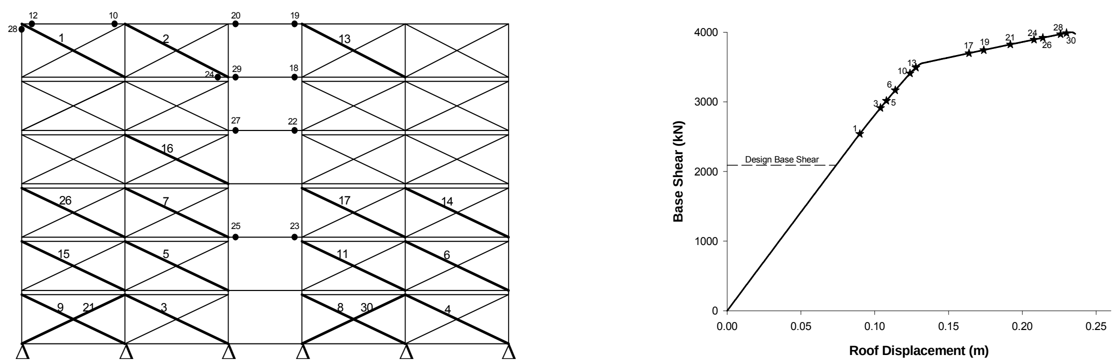
For the hinges formed in some of the roof and top floor beams and the outer external column of the top story, it is an indication of some limitations of the analysis method used in this study (i.e. the pushover analysis method). These members are left to carry much greater loads than they are designed for after buckling of some of the brace members located at the top story level. Once the compression brace member in the first braced bay at this story level buckles and no longer able to support further loading, the design lateral load at this level is directly carried by the roof beam in this bay and the outer external column and consequently columns and beams making up this braced bay and in its vicinity are subjected to load levels that have not been accounted for in their designs, so they begin to fail. In effect, the bracing action for supporting lateral load is lost. This sequence of events is more evident in the two-story frame where the compression brace in the top story of the first braced bay buckles early. This may also have contributed to the poor distribution of plasticity within this frame. The order of events, described above, is evident in the four-story and the six-story MSB braced frames, where column actions are obtained from both the DS approach (i.e. Figs. 6 and 7) and the SRSS approach (i.e. Figs 3 and 4). This further suggests that the formation of plastic hinges as observed in these beams and columns is most probably a result of analysis limitation rather than a design deficiency. This situation is likely to be avoided in reality during earthquake with mass being distributed over the entire floor.

The formation of plastic hinges in some of the beams of non-braced bays of the frames may be a result of the load transfer mechanism, described above, which develops from the analysis methodology. The effect of this limitation is most likely present in the region of the first braced bay at top story level of the frame, although it could be carried over to members located at adjacent bays and even adjacent story level. However, some other beams in non-braced bays may experience plasticization (i.e. in the region of mid-height of the six-story frame), which could be caused by unbalanced forces that are transferred onto these beams as a result of buckling of compression braces in different degrees at two consecutive story levels of the frame. In other words, redistribution of forces to attain equilibrium between two consecutive story levels after more compression braces buckle in one story than the other may leave these beams with load magnitudes that have not been accounted for in their designs. This can only be identified and quantified if the order/sequence of plasticization of brace members is known a priori. This will be possible only after conducting a complete non-linear analysis to failure. The requirements of the Canadian code (CSA 1994) to consider the effect of redistributed loads due to brace buckling or yielding in the determination of beam actions is therefore vague when such beam members in non-braced bays are under consideration.



(a) (b)

Figure 6. Sequence of yielding/buckling of the 4-story MSB braced frame (column design by DS approach): (a) sequence shown on frame; (b) sequence shown on capacity curve



(a) (b)

Figure 7. Sequence of yielding/buckling of the 6-story MSB braced frame (column design by DS approach): (a) sequence shown on frame; (b) sequence shown on capacity curve

CONCLUSIONS

Modular steel buildings are fast evolving, as an effective alternative to conventional onsite steel buildings but knowledge of their behaviour is limited at this time. There is also no record on the performance of MSB under past earthquakes since it is a relatively new technique. This paper has highlighted some unique features of the MSB and has assessed the inelastic behaviour of typical braced frames of a MSB designed for ductility using conventional codes and capacity design procedures. The SRSS approach, widely used for accumulating brace induced forces in capacity design of columns in regular braced frames, as well as a Direct Summation approach were considered during design. The MSB braced frames were modeled and pushover analyses were performed to ultimate capacity. The sequence of yielding/buckling events was studied and compared to the expected behaviour based on the design philosophy.

The results showed that the use of the SRSS approach in the determination of column actions due to yielding/buckling braces may not be conservative for MSB braced frames due to the system's unique detailing requirements. The main assumption that governs the use of this approach may hold for this frame type but special vertical connections of units of the MSB frame seem to impose additional demand on columns located at lower levels of the frame. It is shown that the use of the direct summation approach, where vertical components of yielding/buckling brace forces are added directly to determine brace induced column actions for design may compensate this additional demand. The analysis also revealed that care must be taken in the ductility design of beams in braced frame configurations with non-braced bays. For such beams within non-braced bays, the effect of redistributed loads due to brace buckling or yielding cannot be reliably accounted for in their designs unless the complete failure mechanism of the entire frame including the sequence of plasticization is known. This can be possible only after a complete nonlinear analysis to failure is conducted. Assigning these beams with sections obtained from the capacity design of beams in braced bays, although appear convenient, may lead to undesirable response of the entire frame since such beams could be more critical and govern the design of floor beams at any level.

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