SEISMIC OVERSTRENGTH IN BRACED FRAMES OF MODULAR STEEL BUILDINGS

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ABSTRACT

The seismic behaviour factor, R, is a critical parameter in contemporary seismic design. It is 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, 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 depends on the structural system type. In this investigation, typical braced frames of Modular Steel Buildings (MSBs) are designed. Nonlinear static pushover analyses are conducted to study the inelastic behaviour of these frames. Structural overstrength and ductility are evaluated and their relationships with some key response parameters are assessed. The results show that the MSBs overstrength is greater than that prescribed by the Canadian code. It appears that R depends on building height, contrary to many codes prescribing single values for all buildings with a specific structural system. It is concluded that some unique detailing requirements of MSBs need to be considered in the design process to eliminate undesirable seismic response.

Keywords: Modular steel building, braced frame, seismic design, structural overstrength, ductility, capacity design, pushover analysis.

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1.0 Introduction

Contemporary seismic design of building structures involves reducing the forces obtained from an idealized elastic response spectrum by a ductility related force reduction factor R_d. The magnitude of such reduction primarily depends on the ability of the structure to undergo inelastic deformation without collapse. Furthermore, it is observed that structures usually possess a considerable amount of reserve strength. This extra strength is known to be one of the key characteristics, which influence seismic response of building structures. Many modern seismic design codes therefore permit further reduction of the design forces to account for the dependable portion of this reserve strength.

The 2005 edition of the National Building Code of Canada (NBCC 2005) and the New Zealand Earthquake Load Standard (NZS4203 1992) explicitly recognized this reserve strength by providing an overstrength related modification factor, R_o. Other codes such as the Uniform Building Code (UBC 1997) and the Australian Earthquake Standard (AS1170.4 1993) used a composite reduction factor to account for both overstrength and ductility. Many sources of overstrength can be easily identified but not all can be readily quantified. Sources that have been reviewed by Uang (1991), Mitchell and Paultre (1994), Rahgozar and Humar (1998), Bruneau et al. (1998) and Mitchell et al. (2003) include: material effects caused by higher yield stress compared with the nominal value R_{yield}; effect of using discrete member sizes and practical considerations that require provision of bigger sections for some elements R_{size}; strain hardening behaviour in steel R_{sh}; redistribution of internal forces in the inelastic range R_{mech}; difference between nominal and factored resistances R_{phi}; as well as code requirements for considering multiple loading combinations and contribution of non-structural elements.

Many experimental studies have been conducted to assess lateral overstrength of different structural systems (Bertero et al. 1984; Uang and Bertero 1986; Whitaker et al. 1989; Osteraas and Krawinkler 1989). Analytical procedures (Rahgozar and Humar 1998; Elnashai and Mwafy 2002; Balendra and Huang 2003; Kim and Choi 2005) have also been used extensively to estimate structural overstrength from capacity curves of different structural systems. Static nonlinear pushover analysis has been a reliable tool employed to produce these curves (Rahgozar and Humar 1998; Kim and Choi 2005). Based on results from pushover analyses of 2 to 30 storey concentrically braced frames designed using the same lateral load, Rahgozer and Humar (1998) observed that for concentrically steel braced frame structures, the main parameter that controls the reserve strength is the slenderness ratio of the bracing members. They also observed that the structural overstrength is almost independent of the height of the frame and the effect of building sway. The average observed reserve strength ratio for these frames accounting for only internal force redistribution R_{mech} was about 2.1. The NBCC (2005) recommends an R_o of 1.3 (includes R_{yield}, $R_{size,}$ $R_{sh,}$ $R_{mech,}$ and R_{phi}) for both moderately and limited ductile concentrically steel braced frame, regardless of the height of the building and the magnitude of the 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, and so for R_o given by the NBCC, design and detailing have to be in conformity with the Canadian standard (CSA 2001).

The analytical definition of structural overstrength is reasonably established. For many structural systems, a dependable source of reserve strength that can be reliably estimated is due to redistribution of internal forces in the inelastic range R_{mech}.

Considering a typical structural response envelope in Fig. 1, showing the relationship between base shear and roof displacement, the structural overstrength accounting for all possible sources can be defined by Eq. (1):

$$R_0 = \frac{V_y}{V_d} \tag{1}$$

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 only redistribution of internal forces, V_d would represent the load corresponding to the first significant yield. Structural ductility μ is defined in terms of maximum structural drift (Δ_u) and the displacement corresponding to the idealized yield strength (Δ_y) as

$$\mu = \frac{\Delta_u}{\Delta_y} \tag{2}$$

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.

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 facilities, and dormitories, with essentially repetitive units. A detailed description of the concept, process, advantages and limitations of this unique steel building technique was presented by Annan et al. (2005). Seismic design of this building type is performed using conventional methods, as no seismic performance studies for MSBs are presently available. The current study is part of an extensive research program that aims at providing basis for development and production of next generation seismic-resistant MSBs. The main purpose of this study is to assess structural overstrength in braced frames of MSBs. It also provides an investigation of their inelastic behaviour and ductility.

A two-dimensional (2-D) MSB braced frame that captures the behaviour of vertical connections of the units of MSBs is used to represent the seismic load resisting system. Braced frames of different heights of MSBs are designed considering moderate ductility in accordance with the Canadian standard (CSA 2001). The seismic design forces are determined based on the provisions of the NBCC (2005). The 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. Structural overstrength factors are extracted from the observed response curves and compared with that reported for regular steel braced frames.

2.0 MSB System

Fig. 2 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 of their members. Results of a study on the response of MSB floor framing system under gravity loading (Annan et al. 2005) showed that 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 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. 2). 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. 2). Only the outer faces of these columns that are accessible during assembling are welded. The floor beams are either set directly above the ceiling beams or at a specified clearance to allow for a fire protective layer to be installed. Lateral stability of the entire MSB is achieved by adding diagonal braces as shown in Fig. 2. 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.

The braced frame of MSBs is clearly different from regular braced frames and may respond differently in an event of any lateral movement due to earthquakes. 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.

Three heights of a typical modular steel dormitory are considered in the study: two-storey, four-storey and six-storey. The total heights of the three buildings are 6.8 m, 13.6 m and 20.4 m. They have the same overall plan dimensions of 21.6 m by 16.5 m (Fig. 3a). Each story is made up of six modules, 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. 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 to run along them. 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. 3a. These two frames are identical and so only one (centreline 7) is considered in this 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 the columns on centerlines A, B, C, D, E, and F. Fig. 3b shows the elevation of the braced frame of the four-storey MSB. A clearance of 150 mm was allowed between floor beams and ceiling beams. For ease of discussions in subsequent sections, all columns located on centerlines A and F 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.

3.0 Design of Modular Steel Braced Frames

For ductile 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 carefully 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), they must be provided with sufficient capacity to resist the maximum forces that might develop in them as a result of yielding and buckling of bracing members. They must also support gravity loads.

In the design of the MSB braced frames, frame members are 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 are evaluated and modified, as necessary, according to ductility design requirements and capacity design procedures. The strength and ductility designs are based on the Canadian standard (CSA 2001). The dead load (DL) 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 are applied to account for additional loads on floor, roof, and ceiling, respectively. The live loads (LL) used for the design are based on the NBCC (2005) and are 1.9 kN/m^2 for the individual rooms and 4.8 kN/m^2 for the corridor. A snow load of 1.0 kN/m^2 is assumed for the roof. The seismic loading on each frame is 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 minimum lateral earthquake force, V, is given by Eq. (3):

$$V = \frac{S(T_a)M_v I_E W}{R_o R_d} \geq \frac{S(2.0)M_v I_E W}{R_o R_d} \tag{3}$$

where $S(T_a)$ is the design spectral response acceleration expressed as a ratio of gravitational acceleration for the fundamental lateral period of vibration of the building T_a; M_v is a factor to account for higher mode effects on base shear; I_E is an earthquake importance factor of the structure; W is the dead load plus 25% of the design snow load; R_d is a ductility related force modification factor; and R_o is an overstrength-related force modification factor. Observing that excessive design forces could result for short-period structures due to the steep nature of spectral shapes inherent in the NBCC (2005), the code limits the design base shear for framed structures with $R_d \geq 1.5$ by Eq. (4):

$$V \leq \frac{2}{3}\frac{S(0.2)I_E W}{R_o R_d} \tag{4}$$

The fundamental period, T_a, is obtained from an empirical expression $T_a = 0.05(h_n)^{3/4}$ (NBCC 2005), where h_n is the height of the framed structure. The location of the MSBs is selected as Vancouver in British Columbia, Canada. The buildings are assumed to be founded on a very dense soil (site class C) with an average shear wave 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 ductility factor of 3.0. The design base shears are distributed over the height of the building as per the NBCC (2005).

CISC Grade 350W steel with a specified yield stress, F_y, of 350MPa is used to design the beams, columns, and brace members. 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 used widely in the MSB industry. Wide flange sections (W shape) are 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 are used as the criterion for the selection of optimal sections. Additionally, selected sections are modified to conform to more practical arrangements. Table 1 gives a summary of the resulting sections from the strength design of the MSB braced frames considered in the study.

In this study, the bracing members are assumed to belong to class H (hot-formed or stress relieved) of the CAN/CSA-S16.1-01 standard (CSA 2001). Brace member capacities were calculated based on the same standard. The tensile yield strength, T_r, and the compressive yield strength, C_r, are respectively given by Eqs. (5) and (6):

$$T_r = \phi A F_y \tag{5}$$

$$C_r = \phi A F_y \left(1 + \lambda^{2n}\right)^{-1/n} \tag{6}$$

where the resistance factor ϕ = 0.9; A is the cross-sectional area of the member; n is a parameter for compression resistance, given as 2.24 for hollow structural sections, and λ is the slenderness coefficient. The buckling strength, C_r', of compression brace members is given in the code by Eq. (7):

$$C_r^{'} = \frac{C_r}{1+0.35\lambda} \tag{7}$$

The ductility provision by the Canadian code (CSA 2001) for the design of steel braced structures is based on the assumption that columns, beams and brace connections within the structure must be able to resist the resulting induced forces when braces reach their ultimate strength. For that purpose, the ultimate strength of brace members is taken as the nominal resistance. Specific requirements for brace members to ensure ductile behaviour during severe earthquakes are given in clause 27 of the CAN/CSA-S16.1-01 standard (CSA 2001) as follows: 1) the slenderness ratio of bracing members, *kl/r*, where k is the effective length factor, l is the unsupported length, and r is the radius of gyration, must be less than or equal to $1900/(F_y)^{1/2}$; 2) the width-to-thickness ratio, b/t, of bracing members must be less than or equal to $330/(F_y)^{1/2}$ for hollow structural sections; and 3) both tension and compression braces must be able to carry a minimum of 30% of shear in the storey. The effect of the reduction in compressive strength of the brace members due to repeated buckling (Jain and Goel 1978, Popov and Black 1981) is accounted for by checking the forces in the bracing members against the reduced brace compressive strength [Eq. (7)]. In the case where the tension brace in the same bent and at the same level has excess capacity to compensate for this reduction in compressive strength, the reduction factor, [1/(1+0.35λ)], need not be applied. In other words, if the tension brace in the same level and plane as the compression brace is found to possess sufficient reserve strength, the compression brace member is sized based on the resistance, C_r and not $C_r^{'}$. Columns 5 and 6 of Table 1 contain a summary of the brace member sections for ductile response of the MSB braced frames.

The column members obtained from the strength design are also reviewed to meet ductility requirements. According to the Canadian code (CSA 2001), 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 is 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. Fig. 4a shows a free body diagram for determining brace induced column actions in the four-story MSB frame based on the SRSS approach, including a calculation example at the second storey. The column loads that would result from a Direct Summation (DS) approach, where column actions are derived from a direct sum of vertical components of yielding and buckling brace forces are also shown in the figure. Clearly, this load accumulation approach results in much higher forces for columns located at lower levels of the braced frame. It is to be noted that induced forces are determined for external columns only, as they 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. Columns 5 and 6 of Table 1 contain a summary of the revised column sections in the MSB braced frames 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 variance is greatest for the six-storey MSB braced frame. It is noted that column sections at all levels of the six-storey MSB frame obtained from strength design are found to be inadequate for the required ductility when using the DS accumulation approach.

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 designing 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 beams. Fig. 4b shows free body diagrams for determining floor and ceiling (or roof) beam actions to support redistributed loads, including typical calculation examples. Here, it is assumed that a concrete slab would prevent instability but would not contribute to load carrying capacity. Only beams in braced bays can be considered in the determination of these brace induced beam actions, as this is the only case where such redistributed loads can be readily determined. The resulting section at any level is applied to beams in non-braced bays at the same level. A summary of the beam sections resulting from ductility design of the MSB braced frames is also contained in Table 1.

The brace end connections are designed 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. These connections 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 (CSA 2001) is used in the design of these welded connections.

4.0 Analysis of MSB Braced Frames

Elastic rigidities of steel structures can readily be computed on the basis of elastic material properties. Once the sectional geometry is established, member properties are computed analytically. Flexural rigidity EI, shear rigidity GA, and axial rigidity AE can be determined from cross-sectional properties, A and I (where A is the cross-sectional area and I is the moment of inertia), and material moduli, E and G (where E is the modulus of elasticity and G is the shear modulus). However, inelastic behaviour requires analysis techniques that may require substantially higher level of sophistication. Any nonlinear analysis procedure (static or dynamic) generally requires modeling of the complete load-deformation (or moment- curvature) characteristics to failure of each component of the structure.

In this study, the SeismoStruct nonlinear computer program (SeismoSoft 2003) is employed in the modeling and analysis of the modular steel braced frames. For all the modular braced frames considered, two-dimensional models are developed based on centerline dimensions of the bare frames. This is chosen owing to the limited significance of torsional effects in the selected buildings and it 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 kN/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 to allow for 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 utilized 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 (i.e. different levels/stories of frame) 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 lower and upper frame units. This may lead to independent upper and lower rotations at the same joint. The model utilizes a number of beam-column elements and a joint element for this connection as shown in Fig. 5. The frame elements, J1-J2, J2-J3, J2-J4, J5-J6, J6-J7, and J6-J8 shown in the figure are modeled as rigid elastic elements. These elements are expected to capture the rigidity at the region of beam-column joints within the tube section columns. Their lengths therefore cover the depths of the floor beam and the ceiling beam of the two frame units being joined, and half the width of their columns. The internal element, J4'-J5, is taken as having the same geometric and mechanical properties as the column of the lower frame unit, C1. This element represents part of the lower unit column with height equal to the clearance between the base of the floor beam of the upper frame unit and the top surface of the ceiling beam of the lower frame unit. A pin joint element is defined to connect nodes J4 and J4' to capture conservatively the relative rotation expected between column members present at this vertical connection.

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 capacity of the bracing members plays an important role in seismic analysis of such frames. The physical phenomenon these brace members go through under cyclic loading can be complex. A sample hysteresis response for a brace component (rectangular hollow section) is shown in Fig. 6a, which is a plot of load (P) versus deformation (δ), normalized respectively by the yield capacity (P_y) and deformation at yield (δ_y). In the first cycle, the brace is subjected to tension and then reversed to compression until buckling occurs. Consequently, the compressive resistance decreases due to plastic hinge formation near the mid-length region of the brace. Subsequent cycles of loading result in further degradation of its compressive strength. In tension, the brace reaches its yield strength and it develops some strain hardening. Jain and Goel (1978) conducted an extensive series of experimental work and introduced, as a result, a model to simulate the buckling behaviour of a bracing member. Their proposed model is shown in Fig. 6b. According to the model, the buckling strength of the compressive brace members reduces significantly after the first cycle of loading and then becomes almost steady after a few cycles. This significant strength degradation may be due to residual out-of-plane deformations from previous cycles, the increase in length (i.e. accumulated permanent elongation), possible local buckling of the cross-section at the plastic hinge that induces high localized strains in the steel material, and the Bauschinger effect that lowers the elastic modulus upon reversed loading after previous yielding. Modern seismic design standards (SEAOC 1990; CSA 2001) have reasonably captured the post-buckling phenomenon by applying a buckling reduction factor to the compressive strength of bracing members. In the Canadian standard (CSA 2001), the reduced compressive strength is captured by Eq. (7).

In a pushover analysis, it is reasonable to assume that the design buckling strength is the reduced compressive resistance, C_r' (Rahgozar and Humar 1998). This is adopted in this study in representing the strength of the compressive brace members in the model. This is achieved by modifying the material yield strength, F_y, of compression brace members. Table 2 contains the modified F_y values for braces at different levels of the MSB braced frames considered in the study. The F_y value (=350 MPa) for the tension braces remains unchanged. The modeled MSB frames are subjected to static non-linear pushover analyses to estimate their lateral capacity. The gravity loads, lumped at nodal points, are 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.

5.0 Results and Discussions

5.1 Inelastic Response of MSB Frames

Figs. 7a, 8a and 9a show the order and distribution of plasticity in the six-, four- and two-storey MSB frames in which the SRSS accumulation approach is utilized to derive the brace induced column actions. The filled dots represent plastic hinge formation in the beams and columns. The brace members drawn with heavier lines are either buckled or yielded. The numbers associated with the dots and on the brace members describe the sequence of plasticity formation or yielding/buckling, 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 length of the four- and six-storey 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 may be affected by the brace sizes, slenderness ratio, frame configuration and analysis type. 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 storey. The analysis type might also affect the results as in static pushover analysis a lateral force is applied at each storey level while for dynamic analysis this lateral force would be distributed across the floor according to the distribution of masses.

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 storey is contained only in that storey. The distribution of shear in other stories is not affected. Thus, a yielding in the tension braces in one storey would result in the formation of mechanism in that storey and the structure consequently reaches its ultimate capacity. This implies that the reserve strength of the critical storey is also the global reserve strength of the frame.

In the six-storey MSB braced frame (Fig. 7), the brace member size is uniform in the first five stories and much smaller in the sixth storey. This is due to the distribution of design lateral forces along the height of the frame. Buckling of compression braces starts to occur in the smaller braces located at the sixth storey and then followed by braces in the first storey. Buckling then progresses up the height of the frame. At any same storey 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. Two tension braces at the first storey yielded before failure is reached for this frame. Similar trend is observed for the four-storey MSB braced frame (Fig. 8). 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 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 the frame (i.e. from the third to fourth and from the first to second stories). The second braced bay experienced early buckling of its compression braces as in the six-storey frame and two tension braces at the first story again yielded before failure is reached. In the two-storey MSB braced frame (Fig. 9), plasticization is not as well distributed within the frame as the other two frame heights. Failure of this frame is reached before any of the tension braces could yield.

It is observed in Figs. 7a, 8a, and 9a that plastic hinges form in some columns and beams before buckling/yielding of some bracing members and before failure is reached. For instance, some of the outer and inner external columns located at the lower storey levels of the 6- and 4-storey MSB braced frames experienced plasticization at the early stage of the inelastic response. In all the frames, plastic hinges form in the outer external column at the topmost storey level before failure. Some roof and top floor beams of the MSB frames also experienced plasticization early during the inelastic response. These 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 the 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 lower storey 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 DS approach that sums directly vertical components of yielding/buckling brace forces. This would always be true for any 2-storey braced frame due to the math involved in the SRSS accumulation (i.e. the square root of the square of a number equals the number). The variance in design loads for lower level columns from the two load accumulation approaches however increases significantly as the number of stories increases from 2-storey. Thus, in the six-story MSB braced frame, significantly less column actions result from the use of the SRSS approach compared to the DS approach, especially for the first two storey levels. These observations tend to raise questions about the appropriateness of the use of the SRSS approach for MSB braced frames, more so as it is evident from the analyses results that all the braces do not yield/buckle simultaneously, thereby validating the main assumption that governs its use. It therefore appears 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-storey and six-storey MSB braced frames were redesigned for ductility with brace induced column actions obtained from the DS accumulation approach. These frames were modeled and analyzed by the pushover method. Results of the analyses are shown on Figs. 10 and 11 respectively (i.e. sequence of yielding/buckling and horizontal capacity curves). The distributions of plasticity observed for these frames indicate no formation of plastic hinges in columns located at lower levels of these frames. The sequence of brace buckling/yielding are, however, almost similar to those of the four-storey and six-storey MSB braced frames in which the SRSS accumulation approach is utilized to determine column actions (Figs. 7a and 8a). The use of the SRSS approach for accumulating brace induced 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 roof and top floor beams and the outer external column of the top storey level, it is an indication of some limitation of the analysis method used in this study (i.e. the pushover 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 in this region. This sequence of events is more evident in the two-story MSB braced 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 both the four-storey and the six-storey MSB braced frame, where column actions are obtained from both the DS load accumulation approach (Figs. 7 and 8) and the SRSS approach (Figs. 10 and 11). This further suggests that the formation of plastic hinges as observed in beams and columns of this frame region is most probably a result of analysis limitations 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 storey level of the frame, although it could be carried over to members located at adjacent bays and even adjacent storey level. However, some other beams in non-braced bays may experience plasticization (i.e. in the region of mid-height of the six-storey 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 storey levels of the frame. In other words, redistribution of forces to attain equilibrium between two consecutive storey levels after more compression braces buckle in one storey 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. This will be possible only after conducting a complete non-linear analysis to failure. The requirements of the Canadian code (CSA 2001) 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.

5.2 Overstrength for MSB Frames

Primarily, overstrength is a direct consequence of design simplification, especially in terms of the redistribution of internal forces due to redundancy in a structure. Generally, frame member sections are designed for critical loading conditions and results are applied to other non-critically loaded members in the frame, which may add to the redundancy.

Some of the requirements of the design code for ductility may also result in some redundancy on the braced system. An example is the limitation imposed by the Canadian standard (CSA 2001) on the effective slenderness and the compactness of brace member cross-section to ensure ductile response under strong earthquake. However, the main simplification in the design procedure for concentrically braced frames is related to the treatment of buckling and post-buckling behaviour of compression brace members. For tension-compression braced frames, overstrength arises once buckling of the compression braces has occurred and additional force is required to develop yielding in the tension braces. The redistribution of lateral force from a compression brace to a tension brace allows such structures to carry significantly higher lateral force than at compression buckling.

For ductile concentrically braced frames, the overstrength is generally identified as the difference between the strength corresponding to the first buckling of any 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. Figs. 7b, 8b, 9b, 10b and 11b indicate that in all of the MSB braced frames considered in the study, the base shear force, V, which corresponds to the first buckling of a compression brace member, is higher than the design base shear. Since nominal material properties are utilized in this study, the only dependable sources of extra strength are therefore caused by redundancies in the bracing system (i.e. design and system redundancies) and brace member ductility capacities. The overstrength factor (R_0=1.3) provided by the Canadian code (NBCC 2005) accounts also for the difference between actual and nominal material properties. In this code, the overstrength factor accounting for the braced system's ability to mobilize full capacity before collapse (i.e. due to redistribution of internal forces), R_{mech}, is conservatively set to unity in view of the strength degradation of compression braces under reversed cyclic loading (Mitchell et al. 2003). Thus, experimental investigations that yielded overstrength factor of the order of 2.4 – 2.8 for regular six-story braced steel frame (Uang and Bertero 1986; Whitaker et al. 1989) as well as an analytical study that estimated overstrength factor due to internal force redistribution in the range of 1.5 to 2.1 for ten-story X-braced frames under different design earthquake forces (Rahgozar and Humar 1998) both suggest that the Canadian code's provision is rather conservative.

The capacity envelopes obtained from the pushover analyses were used to estimate the reserve strength ratio. The base shear force versus lateral roof drift for each of the MSB frames (six-, four-, and two-storey) in which the SRSS accumulation approach was used to determine brace induced column actions are depicted in Figs. 12a, 12b, and 12c. The roof drift is defined as the ratio of the top displacement to the height of the MSB frame. It is observed that, in all these frames, failure is caused by formation of a collapse mechanism when the frames are no longer able to carry additional loads. Table 3 contains a summary of the calculated overstrength factors, obtained from the ratio of the ultimate load to the design load. The results indicate that lateral forces 90 – 150% greater than those considered during design are necessary to trigger failure of the frames. The overstrength factors for the six-, four-, and two-stories are respectively 1.9, 2.2 and 2.5, indicating that a decrease in height of the MSB results in an increase in overstrength. This variation of the reserve strength ratio with the height of the MSB is significant compared to observations by Rahgozar and Humar (1998) that the height of ductile concentrically braced frames contributes very little or nothing at all to the frame's reserve strength. For example, if the MSB frame is decreased from four-storey to two-storey or from six-storey to four-storey, there is an increase of about 15% in overstrength. This observation is also not accounted for by the 2005 edition of the NBCC, which requires the use of the same overstrength factor irrespective of the height of the steel braced frame being designed. Figs. 13a and 13b respectively depict the base shear force versus lateral roof drift for the four-storey and six-storey MSB braced frames in which the DS accumulation approach was used to determine brace induced column actions. It is observed from these figures and from Table 3 that, for the same frame height, overstrength of MSB resulting from the use of both the DS accumulation approach and the SRSS approach is the same. This further suggests that overstrength in braced frames is more sensitive to brace sectional properties than it is to columns as it results primarily from redistribution of lateral force from compression braces to tension braces. The reserve strength of a critical story is also the global reserve strength of the frame. It has to be emphasized that the overstrength factors observed for the MSB braced frames above were obtained from the use of nominal material properties and the actual overstrength accounting for other sources identified in section 1.0 could be higher. Thus, the use of R_0 given by the Canadian code (NBCC 2005) is conservative for the design of MSB frames.

5.3 Ductility of MSB Frames

Figs. 12a, 12b, and 12c provide some evidence that the two-storey MSB frame shows a relatively more ductile behaviour, followed by the four-storey and then the six-storey

MSB frames. Structural ductility is defined as the ratio of the ultimate structural drift to the displacement corresponding to the idealized yield strength. 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, the ductility values given in Table 3 for the MSB braced frames considered in the study were obtained by this simplified method for the purpose of assessing the effect of the height of MSB frames on structural ductility. According to the results, structural ductility of the frames considered ranges from 1.8 to 4.6. Ductility also increased slightly in the range of 3-6% for the four-storey and six-storey MSB braced frames when brace induced column actions are derived from the DS accumulation approach instead of the SRSS approach.

6.0 Conclusions

MSBs are fast evolving as an alternative to conventional onsite steel buildings but knowledge on 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 MSB braced frames designed using conventional methods. Canadian standards were used in the strength and ductility designs. The SRSS approach, widely used for accumulating brace induced forces in capacity design of columns for regular braced frames, as well as a Direct Summation approach were considered during ductility design. The MSB braced frames were modeled and pushover analyses were performed to obtain their capacity curves. Overstrength factors for different heights of the MSB frame were evaluated. Also, structural ductility for these frames was estimated. The results were compared with code-specified overstrength values as well as experimentally and analytically determined values for regular braced frames.

The results showed that the use of SRSS approach in the determination of brace induced column actions in capacity design is not conservative for MSB braced frames due to the system's unique detailing requirements. The main assumption that governs the use of this approach might 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 (DS) approach in which vertical components of yielding/buckling brace forces are added directly to determine column actions for design may compensate this additional demand. The analysis results also revealed that MSB frames possess considerable overstrength due to the intrinsic redundancies in the frame system. Overstrength factors for the frame heights (i.e. two to six-storey MSB braced frames) considered in this study range from 1.9 to 2.5 compared to 1.3 given by the Canadian code (NBCC 2005) for regular braced frames. For the same MSB frame height, there was no difference in overstrength obtained from the use of the SRSS and the DS accumulation approaches to determine brace induced column actions. The use of the code's value for the design of MSB is, thus, shown to be conservative. The results also show significant ductility in the MSB frame system, especially in the two-storey MSB braced frame. Furthermore, overstrength and ductility in MSBs appear to depend on building height contrary to many seismic design codes prescribing a single value for all buildings with a specific structural system. The reserve strength ratio was found to increase with decrease in the height of the frame and ductility also increased with a reduction in frame height.

The analysis also revealed that care must be taken in the ductility design of beams in braced frame configuration with non-braced bays. For such beams in 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 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 may appear convenient but may lead to undesirable response of the entire frame since they could be more critical and govern the design of floor beams at any level.

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Table 1. Member sections from strength and ductility designs of MSB braced frame

Number of stories	Frame Member	Storey / Floor #	Strength Design	Ductility Design (column design by SRSS approach)	Ductility Design (column design by DS approach)
2-storey MSB braced frame	Braces	2	HS 89X89X6	HS 89X89X6	
		1	HS 89X89X6	HS 89X89X6	
	Columns	2	HS 89X89X6	HS 127X127X5	
		1	HS 127X127X5	HS 178X178X8	
	Beams	Roof	W100X19	W100X19	
		Floor 2	W100X19	W100X19	
		Floor 1	W100X19	W100X19	
		Ceiling	W100X19	W100X19	
4-storey MSB braced frame	Braces	4	HS 76X76X5	HS 76X76X6	
		3	HS 76X76X5	HS 76X76X6	
		2	HS 89X89X6	HS 89X89X6	
		1	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	
		Floor 4	W100X19	W100X19	
		Floor 3	W100X19	W100X19	
		Floor 2	W100X19	W100X19	
		Floor 1	W100X19	W100X19	
		Ceiling	W100X19	W100X19	
6-storey MSB braced frame	Braces	6	HS 76X76X5	HS 76X76X5	
		4	HS 102X102X5	HS 102X102X6	
		4	HS 102X102X5	HS 102X102X6	
		3	HS 102X102X5	HS 102X102X6	
		2	HS 102X102X5	HS 102X102X6	
		1	HS 102X102X5	HS 102X102X6	
	Columns	6	HS 89X89X5	HS 102X102X6	HS 102X102X6
		5	HS 127X127X6	HS 178X178X6	HS 178X178X6
		4	HS 178X178X10	HS 178X178X10	HS 203X203X10
		3	HS 203X203X10	HS 203X203X10	HS 305X305X10
		2	HS 254X254X10	HS 254X254X10	HS 305X305X13
		1	HS 305X305X10	HS 305X305X10	HS 305X305X13
	Beams	Roof	W100X19	W100X19	
		Floor 6	W250X33	W250X33	
		Floor 5	W250X33	W250X33	
		Floor 4	W250X33	W250X33	
		Floor 3	W250X33	W250X33	
		Floor 2	W250X33	W250X33	
		Floor 1	W250X33	W250X33	
		Ceiling	W100X19	W100X19	

Table 2. Modified F_y values for compression brace members of MSB braced frames

Number of stories	Storey Braces	Slenderness, λ	$B(\lambda) = (1+\lambda^{2n})^{-1/n}$	$k = B(\lambda) / (1+0.35\lambda)$	kF_y (MPa)
2-storey MSB brace frame	2	0.98	0.75	0.56	195.7
	1	0.94	0.78	0.58	204.7
4-storey MSB brace frame	4	1.16	0.62	0.44	153.9
	3	1.12	0.65	0.47	163.6
	2	0.94	0.78	0.59	204.6
	1	0.94	0.78	0.59	204.6
6-storey MSB brace frame	6	1.13	0.64	0.46	161.0
	5	0.81	0.86	0.67	234.8
	4	0.81	0.86	0.67	234.8
	3	0.81	0.86	0.67	234.8
	2	0.81	0.86	0.67	234.8
	1	0.81	0.86	0.67	234.8

Table 3. Overstrength factor and structural ductility of MSB braced frames

Number of Stories	Overstrength Factor, R_0		Structural Ductility, μ	
	SRSS Approach	DS Approach	SRSS Approach	DS Approach
6	1.91	1.91	1.84	1.89
4	2.20	2.20	3.30	3.48
2	2.49		4.62	

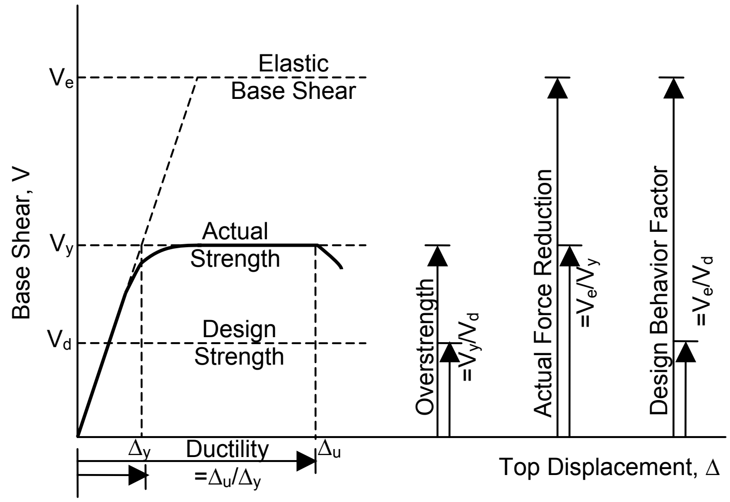


Figure 1. Typical structural response envelope

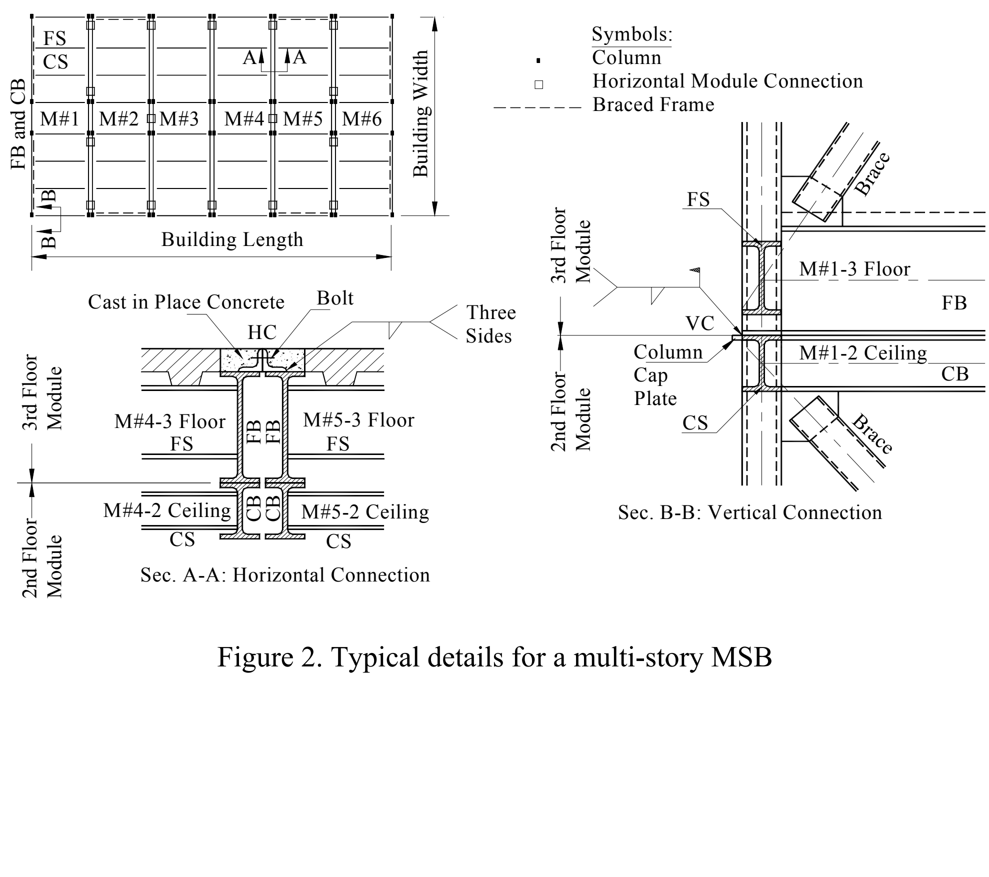
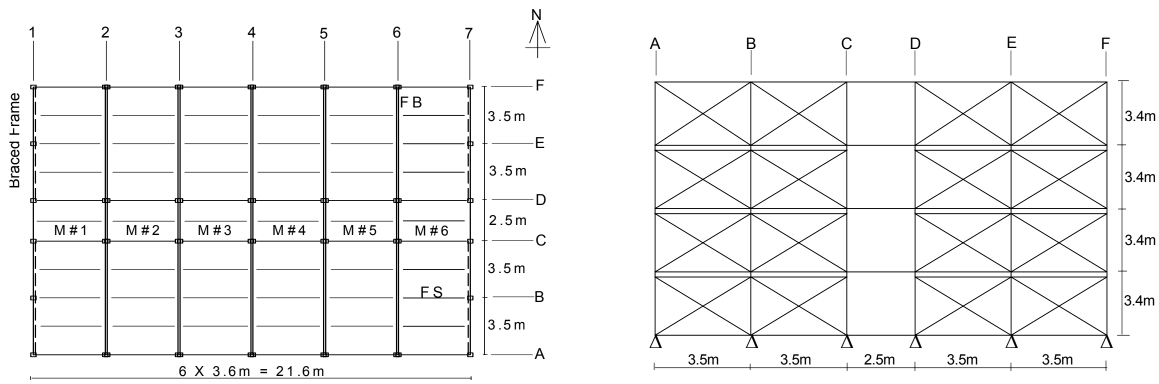


Figure 2. Typical details for a multi-story MSB



(a) Floor Plan (b) Elevation (centerline 1 or 7)

Figure 3. 4-storey modular steel braced frame

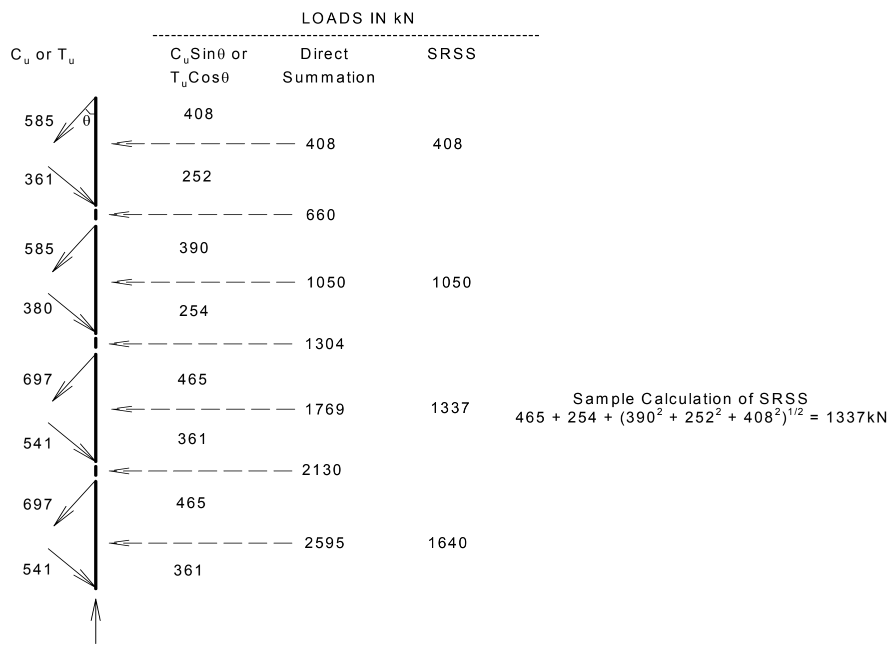


Figure 4a. Cumulative brace loads for determining column actions in 4-story MSB

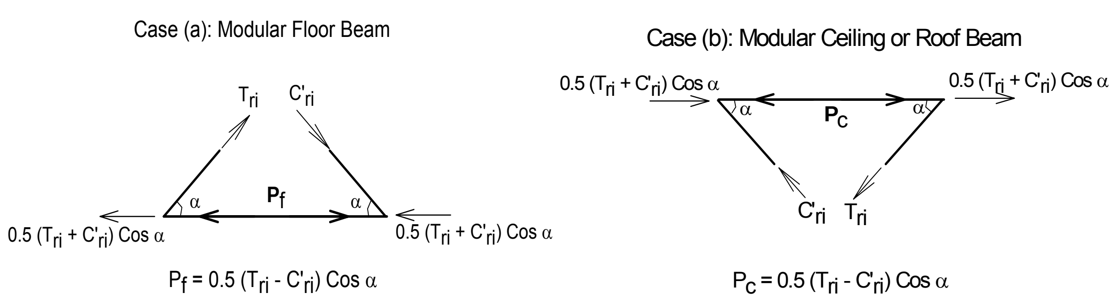


Figure 4b. Free body diagrams of beam forces to support redistributed brace loads

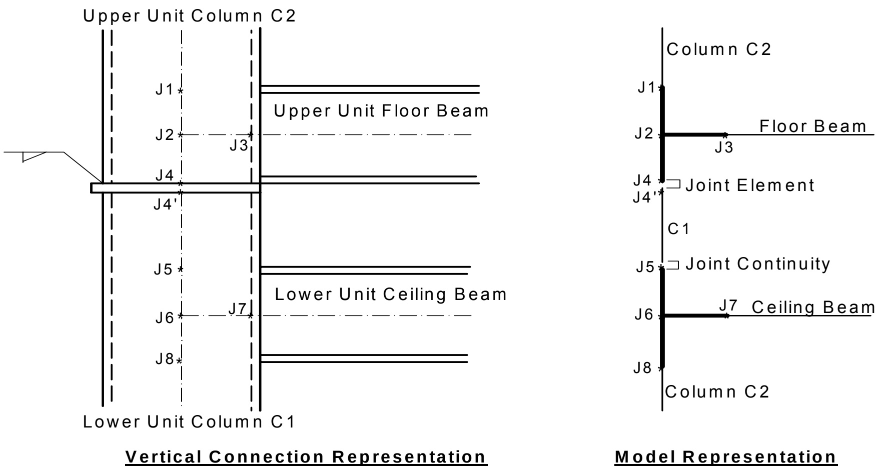
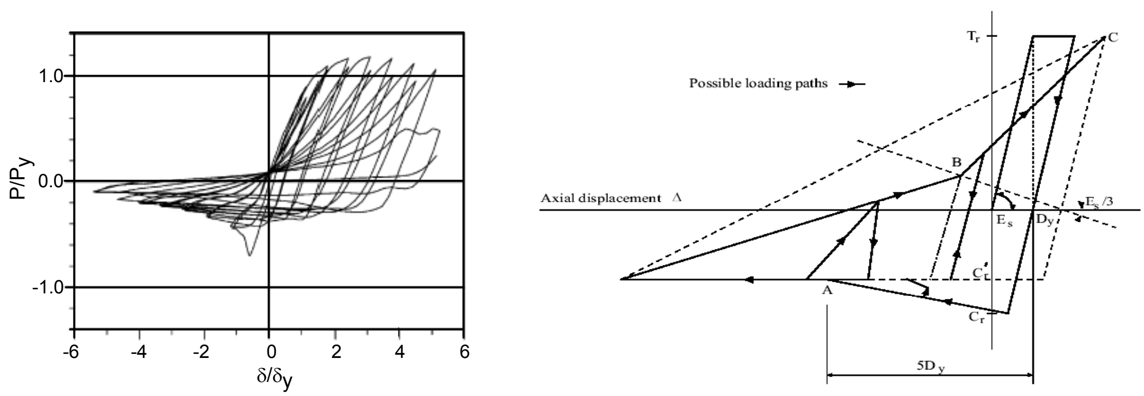
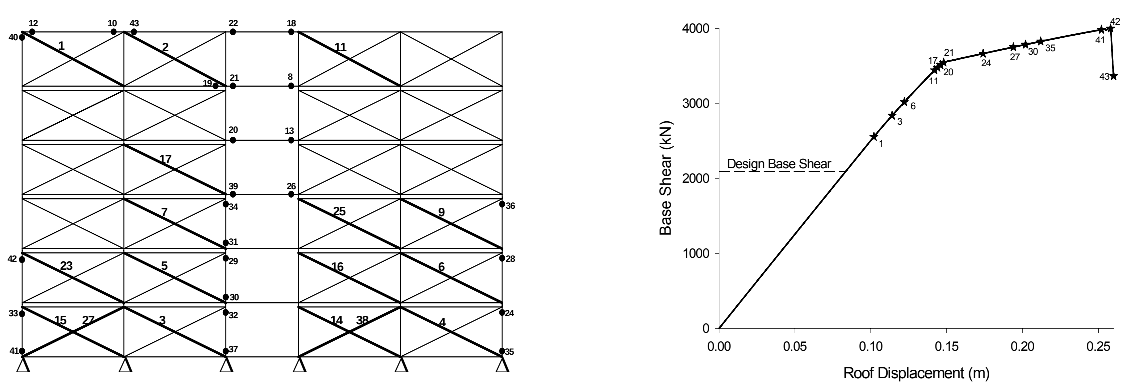


Figure 5. Model of vertical connection of units of MSB braced frame



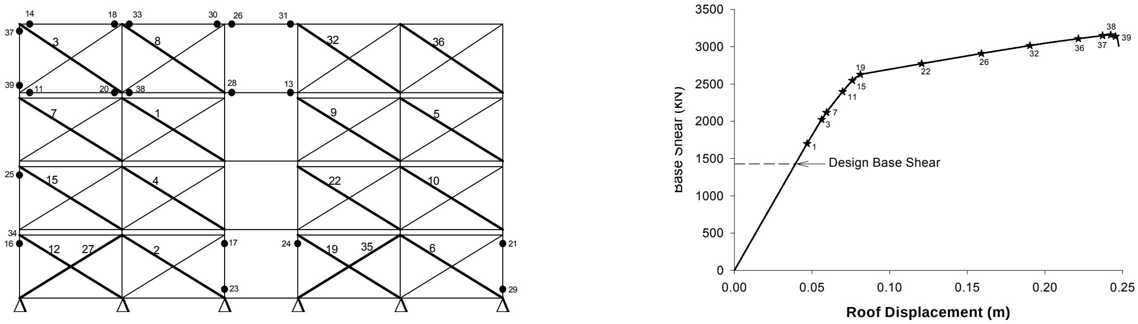
(a) (b)

Figure 6. (a) Typical hysteresis of a steel bracing member under cyclic load (Tremblay 2002); (b) Model hysteresis of steel brace proposed by Jain and Goel (1978)



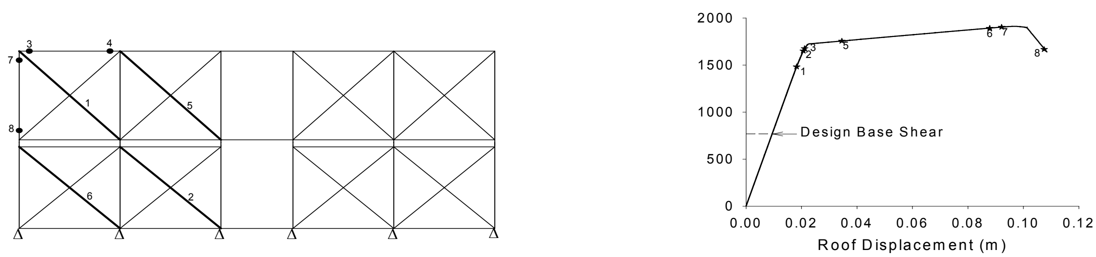
(a) Frame (b) Capacity curve

Figure 7. Sequence of yielding/buckling of the 6-storey MSB braced frame (brace induced column actions by SRSS accumulation approach)



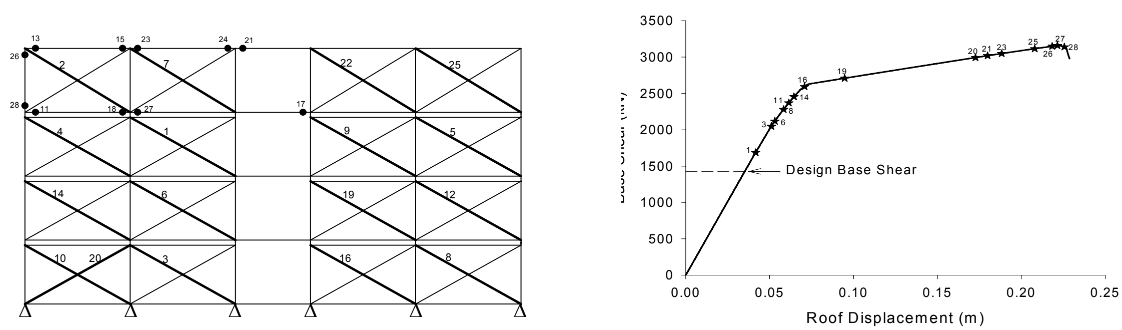
(a) Frame (b) Capacity curve

Figure 8. Sequence of yielding/buckling of the 4-storey MSB braced frame (brace induced column actions by SRSS accumulation approach)



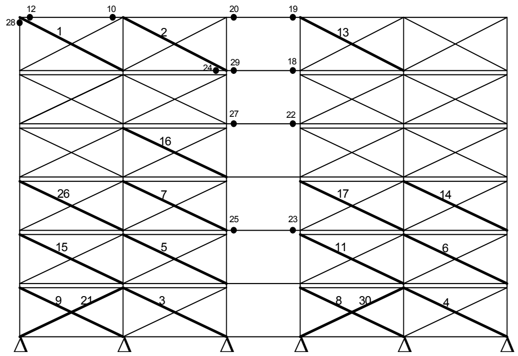
(a) Frame (b) Capacity curve

Figure 9. Sequence of yielding/buckling of the 2-storey MSB braced frame (brace induced column actions by SRSS accumulation approach)

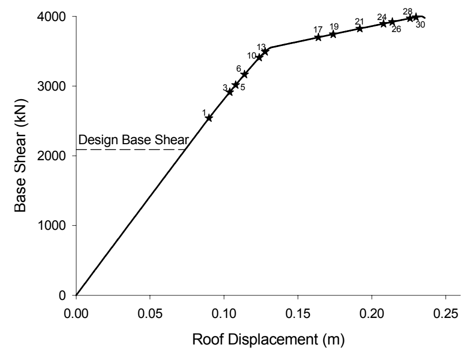


(a) Frame (b) Capacity curve

Figure 10. Sequence of yielding/buckling of the 4-storey MSB braced frame (brace induced column actions by DS accumulation approach)

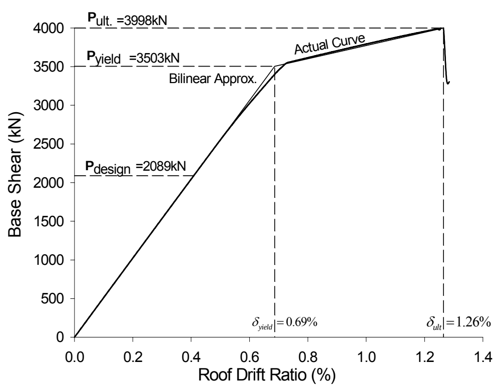


(a) Frame

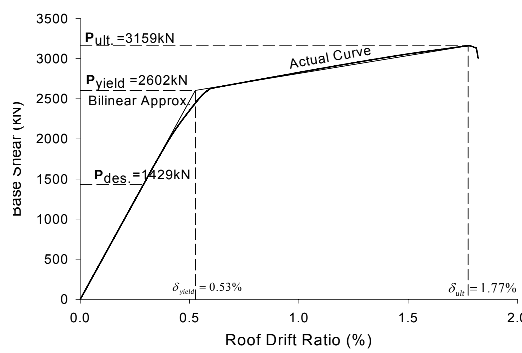


(b) Capacity curve

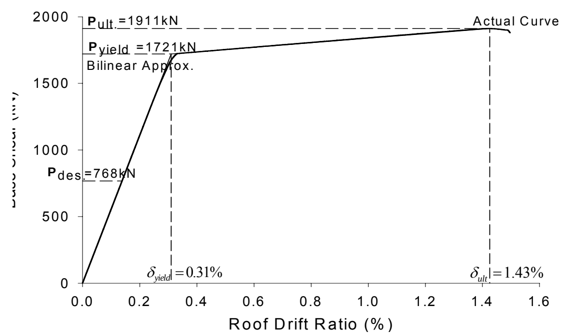
Figure 11. Sequence of yielding/buckling of the 6-storey MSB braced frame (brace induced column actions by DS accumulation approach)



(a) 6-storey

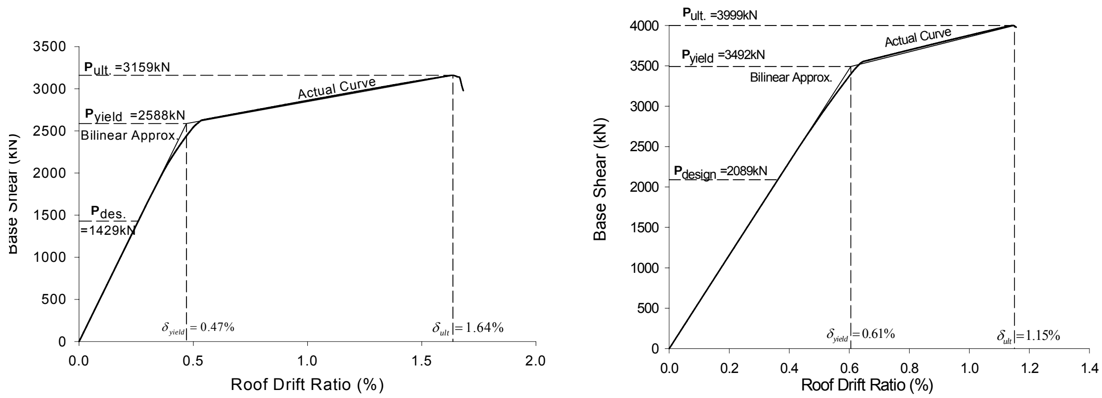


(b) 4-storey



(c) 2-storey

Figure 12. Horizontal capacity curve of MSB braced frames (brace induced column actions by SRSS accumulation approach)



(a) 4-storey (b) 6-storey

Figure13. Horizontal capacity curve of MSB braced frames (brace induced column actions by DS accumulation approach)