Seismic Behavior and Design of Concentrically Braced Steel Frames

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ABSTRACT

This paper describes the main characteristics of the inelastic seismic response of concentrically braced steel frames. Current Canadian and U.S. provisions for the seismic design of these structures are presented and discussed, together with possible modifications that are currently being considered in view of recent research findings. Alternative bracing systems, which have been proposed over the years to improve the seismic response of concentrically braced steel frames, are also briefly described.

INTRODUCTION

Concentrically braced frames (CBFs) are among the most cost-effective systems to resist lateral loads in low- and medium-rise steel buildings. With this system, lateral stiffness and strength requirements can be easily met with minimum steel tonnage and the design can generally be performed or, at least, checked by hand, which makes it less prone to computation and modelling errors. The fabrication effort is also kept to a minimum as simple beam-column shear connections are most often used in conjunction with CBFs. Erection crews benefit from the fact that the final lateral bracing is being provided as the structure is installed and efficient field inspection can be readily performed due to the simplicity of the framework and the small number of key elements to verify in the lateral load resisting system.

CBFs have not been seen, however, as the best choice for resisting earthquake ground motions in the inelastic range. The limited energy dissipation characteristics of the bracing members, the inherent low redundancy of the system, and the likelihood of premature brace fracture under cyclic loading and brittle failure of brace connections are among the main concerns that have been raised against the use of CBFs in active seismic regions. In addition, the system is seen as being prone to soft-story response in multi-story structures due to its limited capacity to redistribute the inelastic demand over the height of the building.

Significant efforts have been devoted in the past 25 years to address these issues and develop solutions which have been implemented in modern building codes in North America. Structural engineers can now design robust, earthquake-resistant CBFs where high seismic demand is expected. These ductile CBF systems still are simple to design and build and are able to achieve the required strength and stiffness at low cost. These advantages also make them excellent candidates for the seismic strengthening of existing structures, especially when minimizing impact on normal building activities is an issue. Of course, research continually brings refinements to further improve code seismic design provisions and new bracing systems are being developed to enhance the inelastic response of CBFs while reducing the cost of the structure.

This paper summarizes the main characteristics of the inelastic response of CBFs and presents the key design provisions that have been included in current Canadian and U.S. Standards to achieve proper seismic performance under severe earthquake ground motions. Recent developments that have been considered for inclusion in the Canadian Standard are also discussed and some promising alternative bracing systems that have been proposed in the last few years are briefly described.

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SEISMIC INELASTIC BEHAVIOR OF CBFs

Seismic lateral loads prescribed in building codes for ductile systems are well below the expected elastic force level anticipated under the design earthquake level. Therefore, strategies must be adopted by the designer to ensure that a frame can undergo several cycles of inelastic loading without loss of structural integrity or excessive deformations (Bruneau, Uang, and Whittaker, 1998). In CBFs, lateral loads are primarily resisted by axial forces in the columns, beams, and diagonal bracing members of the braced frames and these members would buckle and yield in tension if they were loaded beyond their capacity. Buckling of beams and columns cannot represent acceptable means of dissipating seismic energy; as such response would endanger the gravity load carrying capacity of the structure. Hence, inelastic action under earthquakes must only take place in the diagonal bracing members and adequate detailing must be provided to ensure that the braces can go through the expected inelastic demand without premature fracture (Figure 1). The remaining elements of the lateral load resisting system must also have sufficient capacity to resist the maximum forces that will develop when inelastic response develops in the bracing members.

Inelastic Response of Bracing Members

Past experimental research (see References preceded by an *) identified the key characteristics of the inelastic response of bracing members to earthquake loading. Figure 2 shows typical hysteretic response (P versus δ plot) of a pin-connected brace to successive inelastic deformations cycles. When the brace is loaded first in compression (Figure 2a), it buckles at a load P equal to C_u (point B) and its compressive strength then rapidly decreases as a plastic hinge forms near its mid-length. Upon load reversal, elastic recovery takes place and the brace is straightened up through inelastic rotation in the plastic hinge (point D). Eventually, the brace tensile yield resistance (A_gF_y) is reached and the brace experiences inelastic extension while maintaining its yield load. During the second and subsequent cycles, the compressive resistance degrades significantly (point F) due to the Baushinger effect, residual out-of-plane deformations from previous cycles, and, possibly, local buckling of the cross section at the plastic hinge. At every cycle, the brace also accumulates permanent elongation and, hence, can only develop its full yield capacity if higher axial deformation is imposed in tension. The amplitude of the inelastic rotation in the plastic hinge increases as the brace elongates and the imposed deformation is increased. Local buckling eventually develops at the hinge location and fracture occurs shortly after, when the brace is stretched again in tension. As shown in Figure 2b, bracing members exhibit a similar response if loaded first in tension.

The energy dissipation capability of a member subjected to cyclic loading corresponds to the area enclosed by the hysteresis loops. Tests clearly showed that the energy dissi-

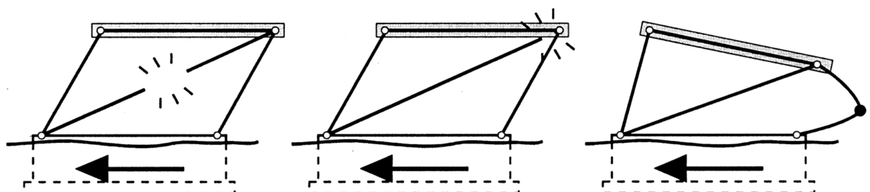


Fig.1. Unacceptable response of CBFs under seismic ground motion.

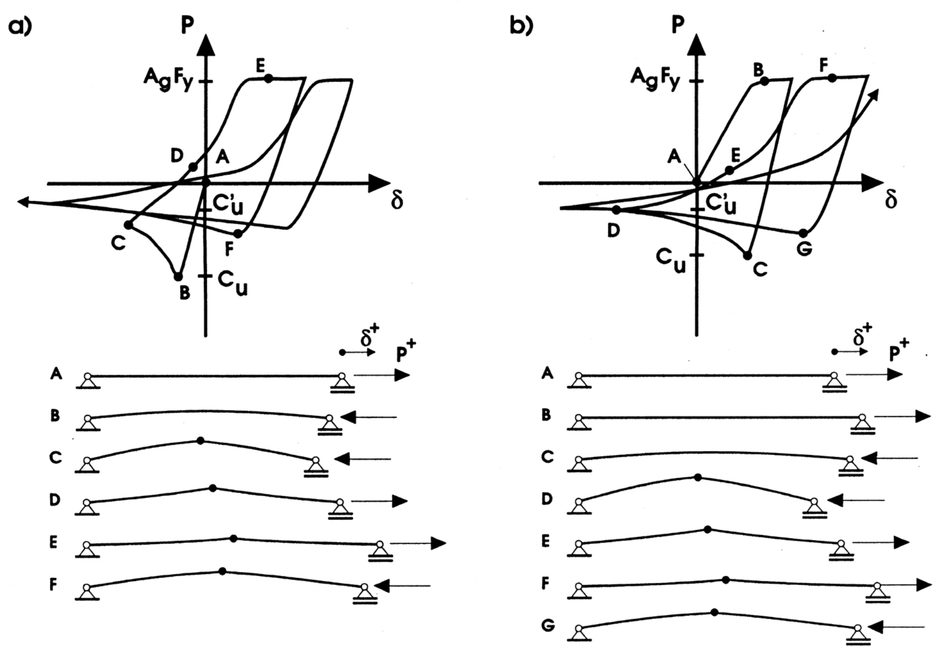


Fig. 2. Typical hysteretic response of bracing members.

pation decreases when the brace slenderness is increased—see Figure 3, taking

$$\lambda = (KL/r)\sqrt{F_y/\pi^2 E}$$

This is because inelastic buckling, with plastic deformations, only develops in stockier members. More severe inelastic response is therefore expected in less slender members, together with a higher likelihood of local buckling of the cross section elements and fracture at plastic hinge locations. Figure 4 illustrates the influence of the brace slenderness on the ductility level at fracture for rectangular HSS braces under stepwise increasing deformation cycles. Reducing the width-to-thickness ratio of bracing members has been shown to delay the apparition of local buckling and increase the fracture life of the members. The *b/t* ratio in the specimens of Figure 4 varied between 8 and 13.

The post-buckling resistance of braces under cyclic loading, referred to herein as C'_u, mainly varies with the amount of inelastic deformation experienced by the braces as well as with the brace slenderness ratio. Figure 5 gives the post-buckling strength at a displacement ductility of 3.0, as measured in 10 cyclic testing programs. For intermediate

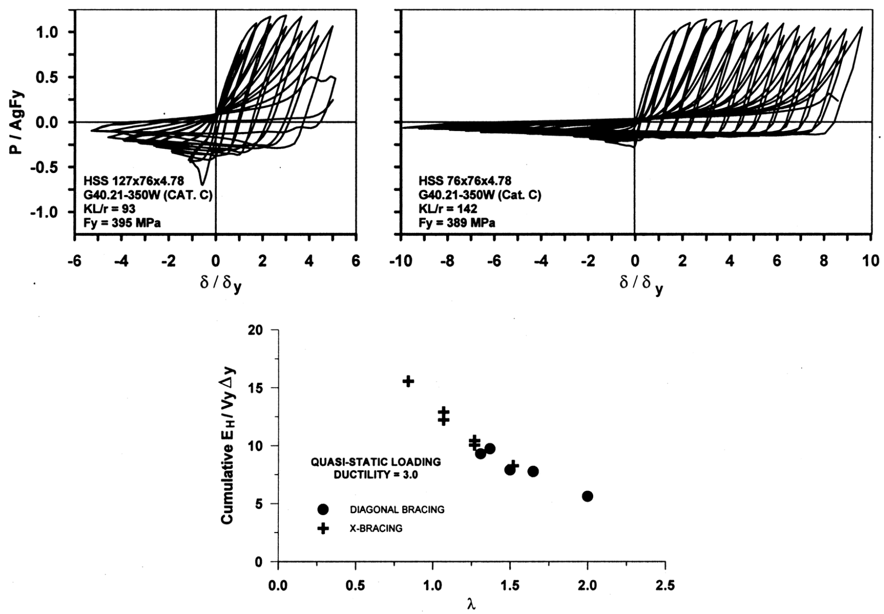


Fig. 3. Influence of brace slenderness on the energy dissipation capacity, E_H (Test results from Archambault et al. (1995)).

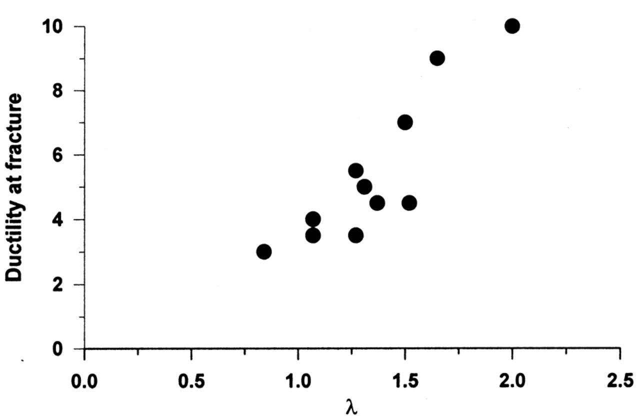


Fig. 4. Influence of the brace slenderness on the fracture life of HSS bracing members (Test results from Archambault et al. (1995)).

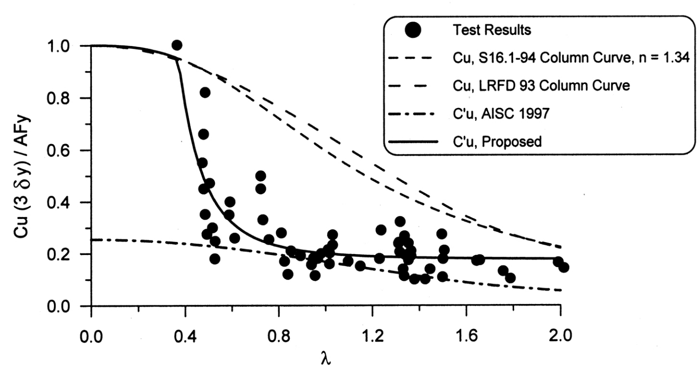


Fig. 5. Post-buckling strength of braces at a ductility of 3.0 (Test results from Higginbotham and Hanson (1976), Black, Wenger, and Popov (1980), Gugerli (1982), Lee and Goel (1987), Liu (1987), Perotti and Scarlassara (1991), Archambault et al. (1995), Leowardi and Walpole (1996), and Walpole (1996)).

brace slenderness, C'_u typically varies between 0.15 and 0.3 times the yield capacity of the braces. Less degradation is observed for both stockier and slender braces. The figure also shows the Canadian ($C_u = C_r/\phi$) and U.S. ($C_u = P_n$) column design curves, the post-buckling strength as suggested in the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 1997) equal to $0.3\phi P_n$, and a best fit solution of the test data for this ductility level:

$$C'_u = AF_y(0.176 + 0.024\lambda^{-3.51}) \leq C_u$$

End fixity influences the effective length of a brace and, thereby, its energy dissipation capacity. Additional plastic hinges will also form near the ends of the brace if the connections exhibit sufficient rotational strength and stiffness (Figure 6), which is expected to occur in-plane when using typical single gusset plate connections as shown in the figure. The type of brace cross-section also influences the brace inelastic response. For instance, it has been shown (Lee and Goel, 1987; Liu, 1987; Tang and Goel, 1987; Foutch, Goel, and Roeder, 1986; Sherman, 1996) that rectangular HSS shapes are more vulnerable to premature local buckling and early fracture. Singly-symmetric braces (T-shapes, double angle) which are susceptible to torsional-flexural buckling are less efficient than doubly-symmetric shapes such as W shapes and tubes (Black, Wenger, and Popov, 1980). Finally, research (Astaneh-Asl and Goel, 1984; Astaneh-Asl, Goel, and Hanson, 1985; Aslani and Goel, 1991) indicated that individual buckling of the components of built-up bracing members (e.g. double angle) results in a reduced compressive strength, energy dissipation and brace fracture life.

Inelastic Response of Bracing Configuration

Bracing configuration is also of prime importance in the inelastic seismic response of CBFs. Figure 7a shows various bracing schemes in which lateral loads in both directions are shared equally between tension- and compression-acting braces that are connected at beam to column joints. In these configurations, the bracing members are sized for compression but they can develop their full yield capacity in tension when large deformation cycles are

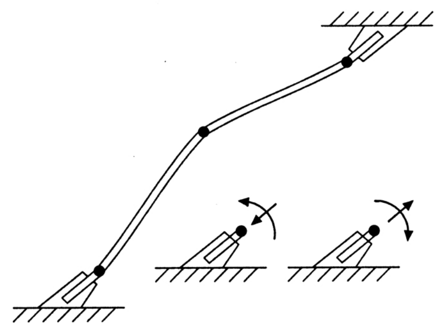


Fig. 6. Plastic hinges forming in a bracing member with fixed ends.

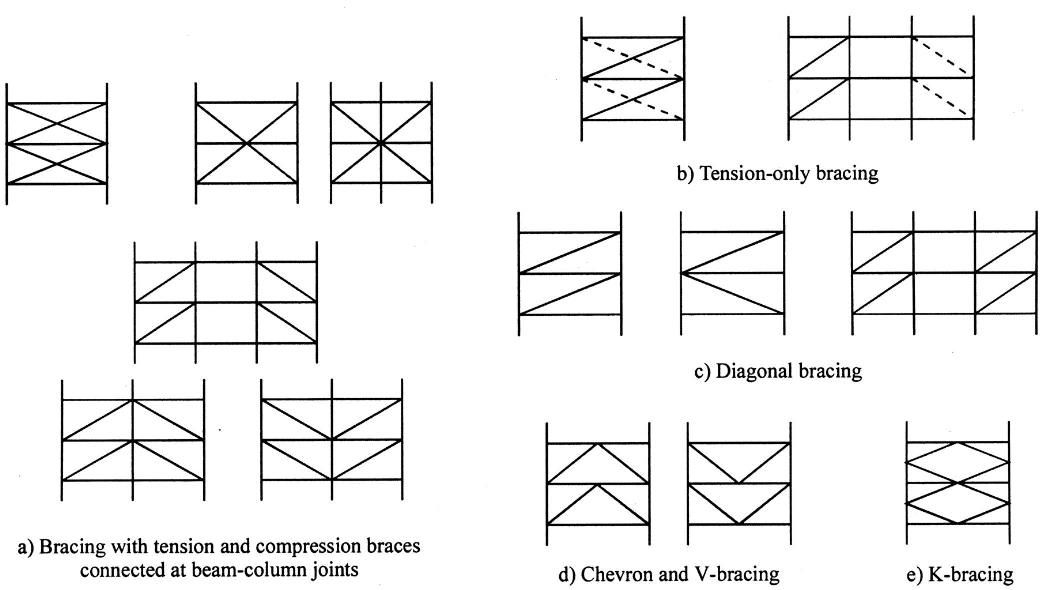


Fig. 7. Typical bracing configurations.

applied. Such frames exhibit similar response in both directions, as shown in Figure 8, with some degree of lateral overstrength arising from the extra-resistance provided by the tension-acting braces. The lateral resistance of these frames however decreases under cyclic loading due to the degradation of the brace compressive strength. In addition, pinching develops near the initial position after the braces have been permanently elongated. Such a phenomenon becomes more pronounced when the brace slenderness is increased.

For instance, Figure 9a shows the hysteretic response obtained from shake table testing of an X-braced frame in which slender braces had been designed to resist the entire story shear in tension only (Figure 7b). In this test specimen, both braces buckled elastically and yielded in tension several times during the ground motion. After each yield excursion, the braces could not provide any lateral strength or lateral stiffness over the range of plastic deformations experienced up to that time by the frame. Such a limited ability to dissipate energy generally results in relatively large lateral deformations (Tremblay and Filiatrault, 1997). In multi-story structures, this can lead to the formation of soft stories (concentration of the inelastic demand) in the lower or top floors (Figure 9b). Despite its rather poor behavior, such a braced frame system does maintain its full lateral capacity in both directions in case further deformation demand is imposed, as braces in tension can undergo considerable elongation (typically around 20 percent) before they fracture.

Figure 7c shows diagonal bracing configurations in which the story shear at every story is resisted by only one brace or by braces oriented in a single direction. Although the braces are suitably designed to sustain the prescribed lateral loads in compression and tension, the difference in the inelastic brace response in each direction (strength and stiffness degradation in compression versus full yield capacity available in tension) results in a structure that crawls towards the direction corresponding to compression in the braces. This behavior may lead to excessive lateral deformation and, ultimately, to instability as shown in Figure 10. This phenomenon is more pronounced if slender braces are used because the difference between tension and compression brace properties accentuates when the brace slenderness is increased.

Inverted V (chevron) bracing (Figure 7d) is among the most popular configuration in CBFs: it is very effective in resisting lateral loads and its geometry allows openings to be easily created within the bracing bents. Under severe earthquake ground motions, however, the braces buckle and lose their compressive strength. The beams are then pulled downward due to the combined action of the gravity loading and the tension acting braces (see Figure 11). Unless the

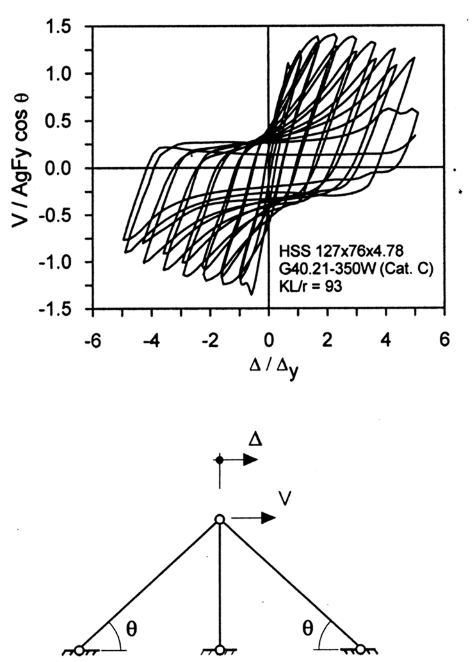


Fig. 8. Symmetrical response of symmetrical tension-compression braced frames (Figure 7a).

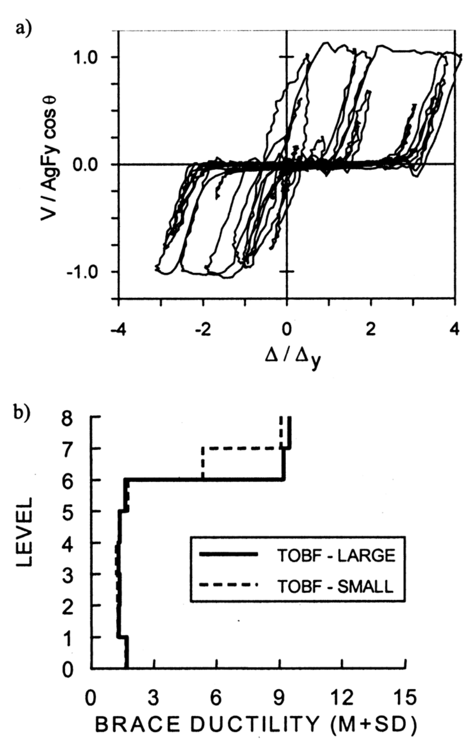


Fig. 9. Tension-only brace frame (TOBF): a) experimental story shear-story drift response (Tremblay and Filiatrault (1996)); b) Computed peak ductility demand in two 8-story buildings (Tremblay and Filiatrault (1997)).

beams are designed to carry this net vertical load together with the axial loads that develop from the braces, a plastic hinge eventually forms at mid-span of the beams before the tension braces reach their yield tensile capacity. This behavior results in a sudden drop in lateral resistance, as shown in Figure 11, with large drifts and possible soft story response in multi-story frames.

A recent study (Robert and Tremblay, 2000; Tremblay and Robert, 2001a; Tremblay and Robert, 2001b) indicates that typical chevron braced frames located in moderate seismic areas can exhibit unstable inelastic dynamic response beyond four stories in height. This study and others (Khatib, Mahin, and Pister, 1988; Remennikov and Walpole, 1997) revealed, however, that strengthening the beams significantly improves the seismic performance of these braced frames. Figure 12 illustrates the impact of the beam strength on the performance of chevron bracing (the parameter "*v*" on the graphs is the vertical deformation at mid-span of the beam). V-braced frames exhibit similar response but gravity loads on the beams help in straightening the buckled bracing members.

K-bracing (Figure 7e) behaves in a manner similar to V-bracing except that the unbalanced brace load now applies horizontally to the columns (Figure 13). The formation of a plastic hinge in the columns is unacceptable, however, as this would likely lead to failure of the columns by instability.

ACHIEVING A DUCTILE RESPONSE THROUGH A PROPER DESIGN STRATEGY

CSA-S16.1, the standard for the design of steel structures in Canada (CSA, 1994), provides seismic design requirements for two categories of CBFs: Ductile CBFs (DBFs) and CBFs with Nominal Ductility (NDBFs). The prescribed seismic loads for the second category are 50 percent greater than those specified for DBFs. In the U.S., the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 1997) also includes provisions for two concentrically braced frame systems: Special Concentrically Braced Frames (SCBFs) and Ordinary Concentrically Braced Frames (OCBFs). The seismic loads prescribed for SCBFs are comparable to those for DBFs and, hence, both systems are expected to experience a similar degree of inelastic response across the border. OCBFs are designed for slightly higher loads than SCBFs (20 percent in BSSC (1997)).

Both codes specify the allowable bracing configurations for each category, detailing requirements to achieve a ductile inelastic response in the bracing members, and capacity design provisions to ensure elastic response of the remaining components of the lateral load resisting system. These provisions are reviewed and compared herein for the DBF and SCBF categories. Recent research work and possible directions for future code developments are also discussed.

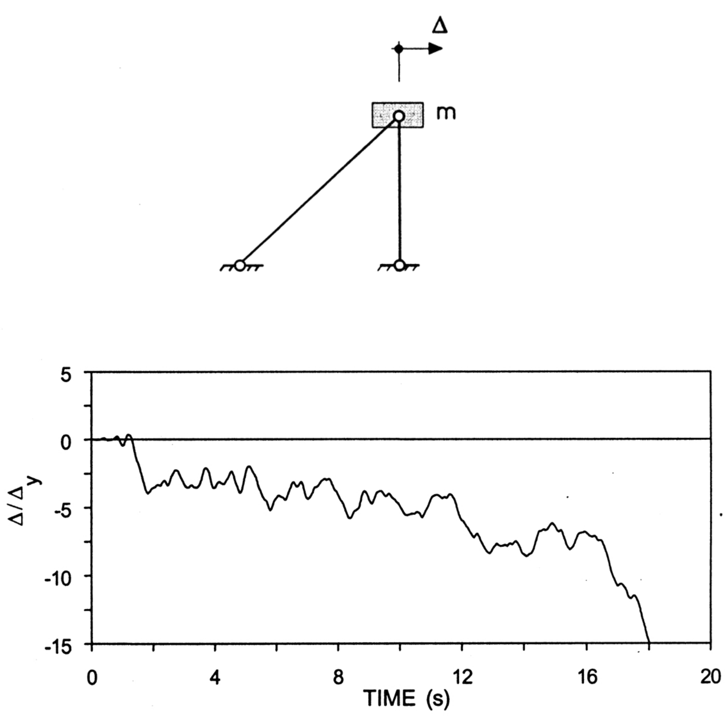


Fig. 10. Unsymmetrical response of diagonal bracing.

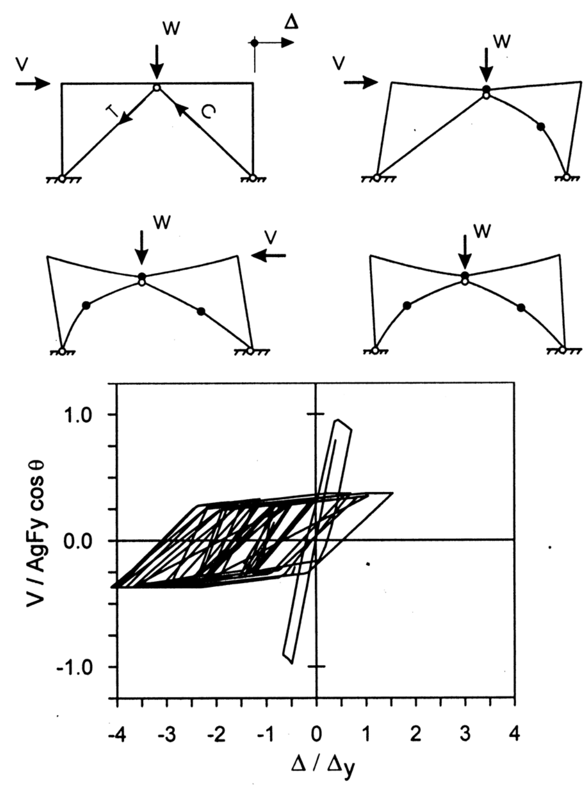


Fig. 11. Typical inelastic response of inverted-V chevron bracing.

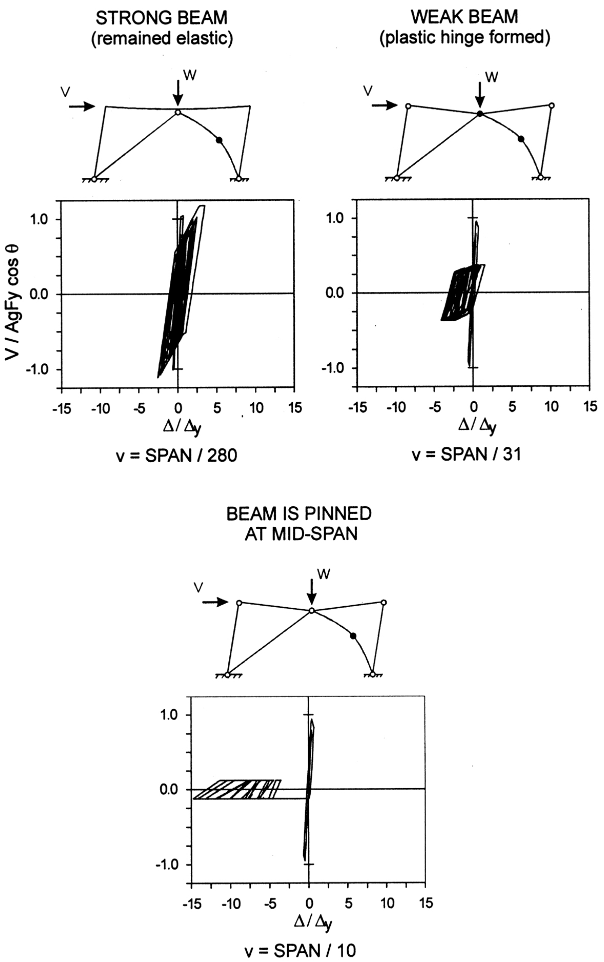


Fig. 12. Influence of beam flexural strength on the inelastic response of chevron bracing.

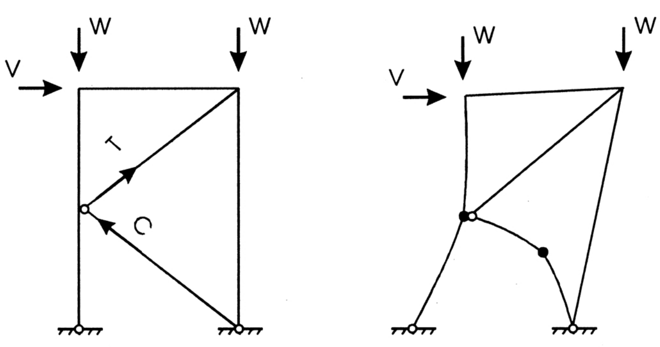


Fig. 13. Inelastic response of K-bracing.

In the AISC provisions, the brace capacity must be calculated using the expected yield strength of the steel, R_yF_y, with $R_y > 1.0$, when assessing the force demand imposed on connections and surrounding members during an earthquake. A similar requirement is being considered for the next CSA-S16.1 edition but does not exist in the current edition. For simplicity, reference to R_y is generally omitted in the paper and the reader should refer to the applicable code provisions to determine when R_y needs to be included.

Bracing Configuration

In order to achieve symmetrical seismic response, only the systems in which the story shear is shared between similar tension- and compression-acting braces are permitted in the DBF and SCBF categories. The requirement is that compression braces must not carry less than 30 percent but no more than 70 percent of the story shear, which can be met using the bracing schemes shown in Figure 7a. Tension-only bracing (Figure 7b) and diagonal bracing (Figure 7c) do not satisfy this provision and, therefore, are excluded from the two CBF categories. This rule applies along every line of bracing to prevent unsymmetrical response of individual braced frames that can lead to undesirable torsional response of the whole structure.

As shown in Figure 5, the compressive strength of a bracing member degrades significantly under cyclic loading and this reduction varies considerably with the brace slenderness. In fact, except for very stocky braces, the influence on the story shear resistance of the braces acting in compression becomes marginal after buckling has occurred. Therefore, it appears that symmetrical seismic response would be more likely achieved if a comparable story shear resistance was provided in both opposite directions with only tension-acting braces in each direction being considered in the calculations. Such a criteria is currently being examined for inclusion in the next CSA-S16.1 code, as it would capture more closely the inelastic response of CBFs while being simpler to verify. This approach would also be applicable to tension-only bracing, when compression braces are ignored in design. Tension-only bracing often represents a more economical solution than tension-compression bracing (in the case of lightly loaded and/or long braces, for instance) but its use must be restricted to low-rise buildings in view of the higher likelihood of soft-story response. Building height limitations will be discussed later.

In view of its poor inelastic response, K-bracing configuration is not permitted in DBFs and SCBFs. In the current CSA-S16.1 Standard, V-type and inverted-V-type bracing are also excluded from the DBF category. In the AISC provisions, however, these two configurations are permitted provided that the beams are designed to sustain their tributary gravity loading together with the vertical unbalanced force imposed by the braces responding in their inelastic range (Figure 14a). As shown, the tension brace is assumed to yield while the compression brace develops its post-buckling strength (= $0.3\phi P_n$ in AISC). As shown in Figure 5, the latter appears to be adequate for intermediate slenderness but on the conservative side for stockier members. Although not explicitly required in the code, the beams should be checked as beam-columns resisting the bending moment due to the vertical loads in combination with the axial forces associated with the imposed brace loads. In addition, beam connections should be capable of sustaining the applied forces shown in Figure 14a.

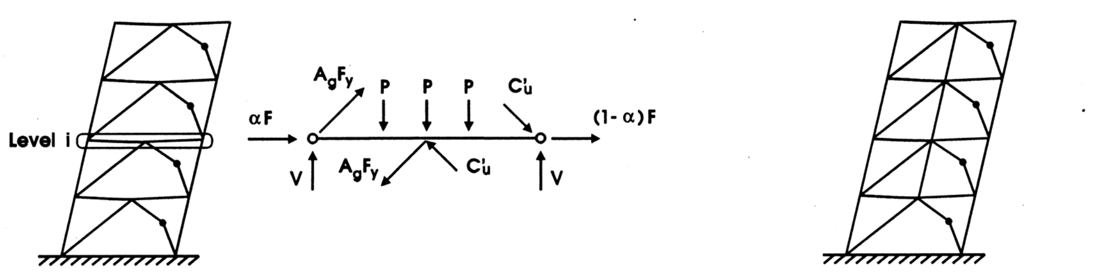
Beams so designed are found to be generally very large in size (Tremblay and Robert, 2001; Sabelli, Wilford, Abey, Pottebaum, and Hohbach, 1998) and this may offset the advantage of using the lower seismic loads prescribed for SCBFs. For inverted V bracing, composite beams may be used to reduce beam sizes and adding a "zipper column" (Figure 14b) allows spreading of the vertical force demand among all beams (Khatib et al., 1988; Remennikov and Walpole, 1997; CSA, 1994; BSSC, 1997; Sabelli et al., 1998). Selecting a two-story X (split-X) configuration, as shown in Figure 7a, represents another efficient means of achieving the desired behavior. In this configuration, brace sizes can be tuned to force a two-story truss mechanism (Sabelli et al., 1998). Recent studies (Robert and Tremblay, 2000; Tremblay and Robert, 2001a; Tremblay and Robert, 2001b; Remennikov and Walpole, 1997) have also indicated that allowing some degree of plastic hinging in the beams of low-rise chevron braced frames does not alter significantly their overall performance. In design, such a limited beam inelastic response could be achieved by specifying that only a fraction of the yield load in the tension braces be considered when sizing the beams. For instance, a brace tension load equal to 60 percent of the brace yield strength is being examined for the next edition of CSA-16.1 for structures up to four stories in height.

Provisions for Ductile Response of Bracing Members

Bracing members are selected to resist axial loading induced by the various combinations of gravity and lateral loads, as obtained from a linear elastic analysis or manual calculations. In both the Canadian and the U.S. codes, a maximum brace slenderness ratio is specified to achieve minimum energy dissipation capacity. In the current CSA Standard, the limit is $\lambda = 1.35$, which corresponds approximately to the transition between elastic and inelastic buckling. Higher slenderness is permitted in the AISC provisions ($\lambda = 1.87$), recognizing that a longer brace fracture life can be obtained with more slender braces.

The two hysteresis plots in the upper part of Figure 3 have been obtained from tests on bracing members with slenderness ratios that nearly correspond to these two limits ($\lambda = 1.31$ and 1.99 for the left-hand side and right-hand side diagrams, respectively). When examining these plots, one could question the ability of these braces to dissipate any significant amount of energy during a severe earthquake, especially when the limit is set to $\lambda = 1.87$. Fortunately, for tension-compression bracing, the lack of inelastic performance of slender braces is compensated by a larger available story shear resistance: when compression governs the design of the braces, lateral overstrength is introduced because of the difference that exists between the brace resistance in tension and compression. This difference increases with brace slenderness and it is found that interstory drift and brace ductility demand generally decrease if braces with a higher slenderness are selected in design for a given specified lateral load. This is illustrated in Figure 15 for a simple single-story X-bracing in which both the slenderness and cross-sectional area of the braces are varied in such a manner that the horizontal component of the brace compression capacity is equal to 0.5 times the same force V_y.

As shown, the frame with stockier braces exhibits stable and fatter hysteresis loops with superior energy dissipation



a) Design forces for the beams

b) Addition of a zipper column

Fig. 14. Chevron (inverted V) bracing with enhanced performance.

but the response of the frame reduces as brace slenderness is increased. Similar results are obtained in multi-story structures (Tremblay, 2000). This shows that energy dissipation becomes less critical when lateral overstrength is created by using slender braces. In addition, as mentioned earlier, slender braces experience lower inelastic demand and can therefore survive a longer duration of strong shaking. The drawback, however, is that larger brace forces are expected to develop in tension and such high forces have to be resisted by the brace connections and the other components of the lateral load resisting system, as will be described later.

When applying the brace slenderness limits, it is also important to recognize that it is the effective slenderness ratio, *KL/r* that governs the behavior of bracing members, not *L/r*. In X-bracing, for example, several studies (Archambault, Tremblay, and Filiatrault, 1995; El-Tayem and Goel, 1986; Sabelli and Hohbach, 1999) revealed that a *K* factor equal to 0.5 can be used both for in-plane and out-of-plane buckling when the brace end connections are pinned. For double angle bracing members connected to a single vertical gusset plate, *K* values equal to 0.5 and 1.0 have been proposed (Astaneh-Asl et al., 1984; Astaneh-Asl et al., 1985) for in-plane and out-of-plane buckling, respectively. When assessing *KL/r*, the length *L* can also be taken equal to the clear length of the brace.

In CSA-S16.1, a reduced design compression strength is specified for the braces to account for the anticipated degradation of their compressive resistance under cyclic loading. A similar provision existed in previous U.S. codes but this reduction is no longer required for SCBFs in AISC (1997) because it has been shown to have little effect on the overall inelastic response of braced frames. No reduction is being considered in the future edition of the Canadian code.

Stringent width-to-thickness ratio limits are specified in both CSA-S16.1 and AISC provisions to prevent (delay) local buckling response at brace plastic hinges. The prescribed limits are particularly severe for rectangular HSS members due to their tendency to fracture shortly after local buckling has developed. The *b/t* limit for angles is also rather stringent and often difficult to meet. In the case of double angle bracing members, it is suggested (Astaneh-Asl et al., 1984; Astaneh-Asl et al., 1985) to apply the specified limit only to the outstanding legs if buckling occurs about the axis of symmetry of the cross section and, otherwise, to the back-to-back legs.

In order to prevent individual buckling of elements in built-up bracing members, stitch spacing closer than normally required is specified in both Canadian and U.S. codes. In addition, when shear forces are induced in the stitches upon overall buckling of built-up members, minimum shear resistance is required for the stitches to avoid premature stitch failure when tension yielding develops in a brace that has buckled in a previous loading cycle. AISC provisions also require that no bolted stitches be located in the vicinity of plastic hinges because the presence of a bolt hole in a hinge can lead to premature brace fracture. When applying the latter two requirements, the engineer must correctly predict the governing buckling mode and location of plastic hinges based on the actual brace length and end conditions (see Figures 2, 6, 15).

Capacity Design

Capacity design implies that the components of the lateral load resisting system other than the bracing members must be capable of resisting the maximum forces and accommodating the maximum deformations that are anticipated during the design earthquake. Of course, structural integrity of

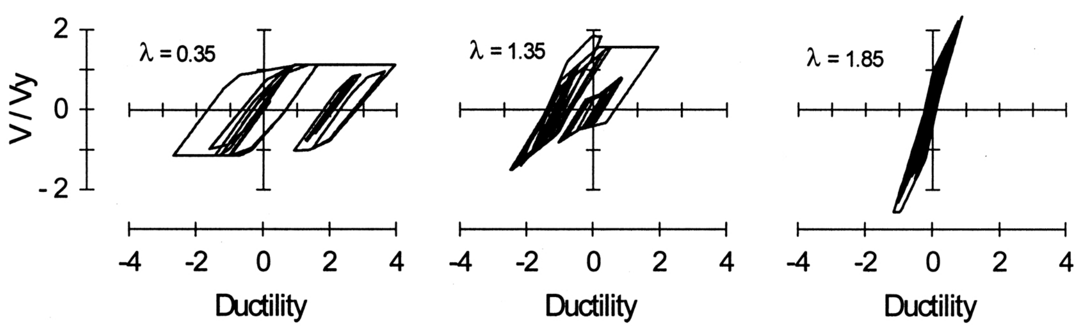


Fig. 15. Influence of brace slenderness on the response of tension-compression X-bracing (adapted from Tremblay (2000)).

the gravity load carrying system must also be maintained while the bracing members undergo cyclic inelastic response.

Brace connections are the first elements to be considered in that process. Several occurrences of brittle brace fracture in tension at brace connections or failure of gusset plates due to cyclic buckling have been observed in past earthquakes (Tremblay, Bruneau, Nakashima, Prion, Filiatrault, and DeVall, 1996). Hence, brace connections must be designed to sustain the full tensile yield resistance and the expected buckling strength of the bracing members. As mentioned earlier, the brace capacity must be calculated using the expected yield strength of the steel (= R_yF_y in AISC provisions, with $R_y > 1.0$). The addition of reinforcing plates is therefore not uncommon to develop yielding of the braces in tension prior to fracture on net area, especially when shear lag is expected in the connection. Due consideration to the as-built end conditions and clear length of the braces must also be given when assessing the brace C_u. In addition, it must be recognized that the value of C_u as obtained from common design column curves represents a conservative estimate of the brace compression strength and, hence, that some amplification factor could be applied to come up with a more likely loading condition for the gusset plates.

Large design connection loads can be obtained in tension when slender braces are used or in tension and compression when braces are oversized to meet other design requirements, such as width-to-thickness ratio limits, drift provisions, etc. In the CSA-S16.1 provisions, a cut-off on the brace loads, which corresponds to twice the specified seismic brace loads is specified for low seismic zones. In active seismic zones, reduced brace connection loads can also be used if the engineer can demonstrate that such lower loads are adequate. In AISC, brace connection loads need not exceed the maximum forces that can be generated in the system, as indicated by analysis. For the latter two situations, nonlinear dynamic analysis can be used to evaluate the maximum anticipated forces under design level ground motions.

Brace connections must also be designed and detailed to allow brace end rotation if pinned end conditions are assumed, or permit a plastic hinge to develop at the brace ends when connection fixity is anticipated. Excellent cover-

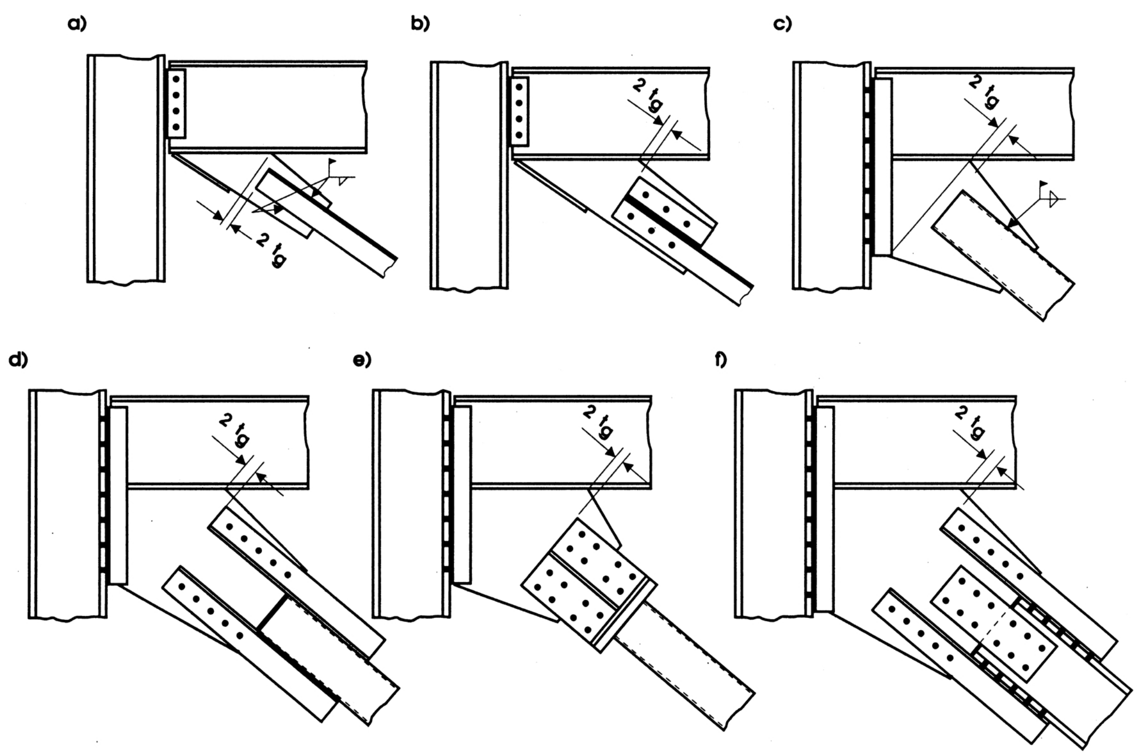


Fig. 16. Connection detail allowing rotation of the brace ends upon out-of-plane buckling (adapted from Astaneh-Asl et al. (1985)).

age of this topic, together with useful design guidelines, is given in Astaneh-Asl (1998). Figure 16 shows examples of a detail proposed to accommodate rotation in single gusset plate connections when out-of-plane buckling is expected (Astaneh-Asl et al., 1985). In this detail, a clear distance equal to two times the thickness of the gusset, t_g, is left at the end of the bracing member or connecting elements in order to allow the formation of a hinge in the gusset plate along a line perpendicular to the member longitudinal axis. Tearing of the gusset plate will rapidly develop if this geometry is not carefully met.

When a plastic hinge is expected in the brace, near its ends (Figure 6), the connection must be designed to develop 1.10 times the full expected plastic moment of the brace about the buckling axis. The 10 percent increase accounts for the possibility of strain hardening in the hinge. In this verification, there is no need to consider brace axial loads in combination with the bending moment, as these loads are relatively small when plastic hinge rotation develops in the hinge (see Figure 2). No explicit requirements for flexural strength are included in the Canadian code and engineers are encouraged to follow U.S. practice to ensure proper behavior.

Columns, beams, and their connections must then be checked to ensure that they can support gravity loads together with the lateral loads that correspond to the actual brace capacity. Member forces for this verification can be determined by removing the braces and applying the gravity loads and brace induced loads to the remaining structure. The brace induced loads are taken equal to the forces used in the design of the brace connections, unless a more severe condition exists. This is illustrated in Figure 17 for the design of the columns of a two-bay CBF.

In this figure, W refers to the gravity loading (shown only at the roof level for clarity); C_u is the brace compressive strength; C'_u is the post-buckling strength of the braces; and A_gF_y is the brace tensile capacity. In the U.S., R_yF_y must be used for the calculation of C_u and the brace yield load. The load pattern in Figure 17b must be considered for the design of the exterior columns and their foundations. Maximum compressive loads in these columns develop when the braces buckle in the first loading cycle. Conversely, the most critical condition for the interior column is after degradation of the compressive strength of the braces has occurred. Figure 18 illustrates this loading condition for a simple single-story frame.

In this numerical example, the brace capacity is set equal to A_gF_y = 2,160 kN in tension and C'_u is taken as 0.3 C_u = 346 kN. Therefore, the maximum expected compressive load in the column, C_{fmax}, is equal to 2,130 kN, i.e., the gravity load (1000 kN) plus the net vertical resultant of the brace loads (A_gF_y and C'_u). The results of a nonlinear dynamic analysis performed on that frame indicate that this value is actually reached several times during the ground motion. Such a behavior cannot be predicted by a linear elastic analysis. For this particular frame, such an analysis would tell the designer that the gravity load induces a compressive force of 821 kN in the column and that the lateral loads do not produce any column forces. That column would have most likely failed during an earthquake had it been designed on the basis of an elastic analysis (821 kN versus 2,130 kN).

In the calculation of column forces, the loads induced by the bracing members need not exceed those used in the design of the brace connections (maximum or cut-off brace loads). In multi-story structures, the model described in Figure 17 suggests that all the bracing members reach their capacity at the same time, which tends to be more unlikely as the number of stories above the level under consideration becomes large. Current Canadian and U.S codes do not account for this situation but statistical accumulation of brace loads have been proposed in the literature (Khatib et al., 1988; Redwood and Channagiri, 1991).

Inelastic response in multi-story braced frames is not uniform over the building height, as it typically tends to concentrate in the bottom and upper floors of the structure

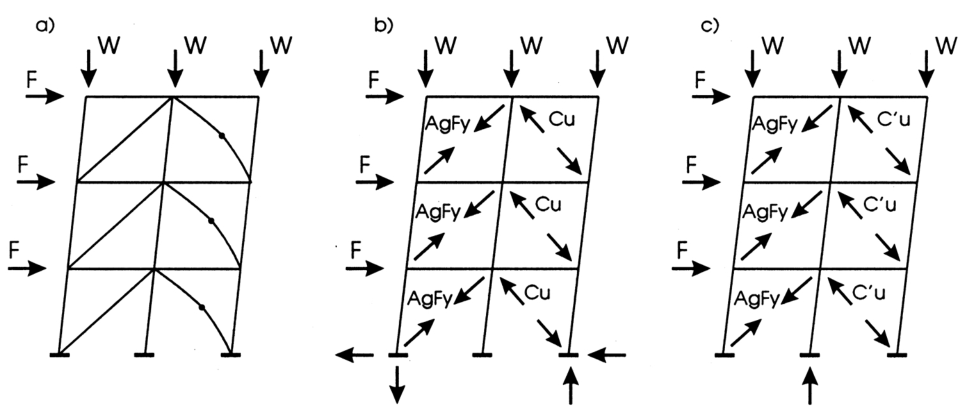


Fig. 17. Determining member forces for capacity design of columns.

(Perotti and Scarlassara, 1991; Tremblay and Robert, 2001; Martinelli, Perotti, and Bozzi, 2000; Redwood, Lu, Bouchard, and Paultre, 1991). Because the columns in multi-story buildings are most often continuous over two or more stories, significant bending moments may develop in the columns when the structure experiences different inter-story drift angles in adjacent floors (Tang and Goel, 1987; Tremblay and Robert, 2001; Tremblay, 2000; Hassan and Goel, 1991) (see Figure 19). Current CSA-S16.1 specifications do not explicitly address this issue while AISC provisions require that the columns of the bracing bents have minimum compactness and that minimum shear and flexural capacity be provided for their splices.

Column continuity can be very beneficial in preventing soft-story response in multi-story structures (Tremblay, 2000; Hassan and Goel, 1991; Tremblay and Stiemer, 1994) and it has been proposed for the future CSA-S16.1 code that every column in braced frame structures be continuous over a minimum of two stories. Additional provisions also require that the columns be compact sections and be checked as beam-columns with an applied bending moment equal to 20 percent of their plastic moment. The latter requirement should not govern for the gravity columns because of the lower load factors applicable to gravity loading acting in combination with seismic loading. It may, however, affect columns in the bracing bents. Finally, col-

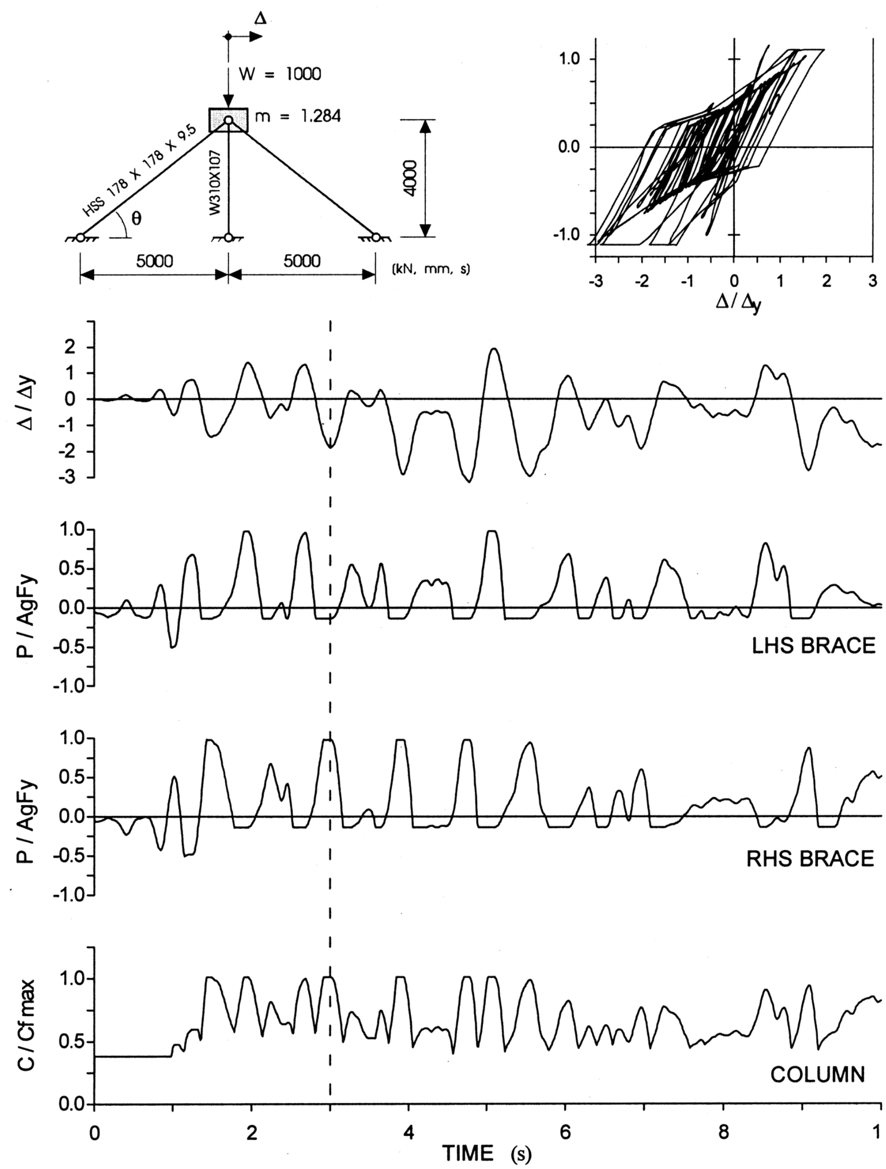


Fig. 18. Time history of the story drift and member forces in a single-story CBF.

umn splices would have to resist shear forces consistent with the design moments assuming double curvature response over the story height.

Capacity design of beams is similar to that of the columns. Typical brace induced loads for an X-bracing are shown in Figures 20a and 20b and those corresponding to the two-bay braced frame of Figure 17 are given in Figure 20c. For clarity, gravity loading is not shown in Figure 20 but must be included in the beam check. Again, R_yF_y must be used in the U.S. for the calculation of C_u and the brace tensile yield load.

Beam axial forces depend upon the brace loads acting above and below the level under consideration. A large variety of brace load combinations may occur during an earthquake. For design purposes, however, it can be assumed that two consecutive stories will deform in the same direction and that the braces in these two stories will reach simultaneously their capacity. Both the cases where C_u or C'_u acts in the compression braces must also be examined to determine the most critical condition. The sum of the forces F_1 and F_2 is obtained by working out the equilibrium of the beams in the horizontal direction. The ratio of F_1 to F_2 depends on how the lateral loads are fed into the bracing bent at the level under consideration. Once F_1 and F_2 are known for a given brace load condition, the axial forces in the beam can be determined by horizontal equilibrium at each joint and the beam can then be checked as a beam-column with the bending moment produced by the tributary gravity loads. Again, this calculation must be performed manually to capture the nonlinear behavior of the frame.

If V- and inverted V-bracing are used, as permitted in SCBFs, out-of-plane stability of the beams is critical to achieve a stable response when the bracing members buckle in compression. AISC provisions require that lateral bracing be provided at both beam flanges at the point of intersection of the bracing members, as shown in Figure 21 for inverted V bracing.

Forces in braces, columns, and beams obtained as described in this section form consistent sets of forces that can be used for the design of beam-to-column connections, for pass-through forces, etc. They can also be used in the design of roof and floor diaphragms, anchorage of the frame to the foundations, the foundations themselves, etc.

Building Height Limitations

As stated earlier, soft-story response can develop and eventually lead to collapse by dynamic instability in multi-story CBFs. This situation is more critical in taller frames, when higher R values are used in design (higher inelastic response), and when poor energy dissipation is anticipated such as in chevron and tension-only bracing systems (Robert and Tremblay, 2000; Tremblay and Robert, 2001b; Tremblay, 2000). The steel structures Standard in New Zealand (SNZ, 1997) prescribes building height limits which are in line with those findings. For example, CBFs in the most ductile category must not exceed 8, 4, and 2 stories in height when an X, a chevron, and a tension-only bracing are used, respectively. Higher limits are permitted when higher seismic loads are considered in design.

Current Canadian provisions do not impose any limitation on the building height for steel CBFs. In the U.S., height limits are prescribed in building codes and these are generally more severe for the less ductile systems. For instance, the building height limits specified in ICBO (1997) are 73 m and 49 m for the SCBF and OCBF systems, respectively, when located in seismic zones 3 and 4. It is likely that more stringent limits will be prescribed in the near future and the design engineer should stay alert of any changes in that direction. Such limitations are currently being considered for the coming CSA-S16.1. In addition, caution must be exercised in regions where severe near-field events are expected, as the associated pulse-type ground motions can be more critical for P-delta collapse (Gupta and Krawinkler, 2000).

NEWLY PROPOSED BRACING SYSTEMS

Concurrent with the development of seismic design provisions for conventional braced steel frames, alternative bracing systems have been proposed over the years which generally offer the key advantages of CBFs (simplicity, efficiency in resisting lateral loads, etc.) while eliminating their main drawback, i.e., the rather poor inelastic response of bracing members under cyclic loading. Most of these systems then exhibit a more stable and symmetrical hysteretic behavior with superior energy dissipation capacity. This generally results in smaller lateral deformations and longer fatigue life. In addition, the brace overstrength due to the difference between tension and compression capacity is

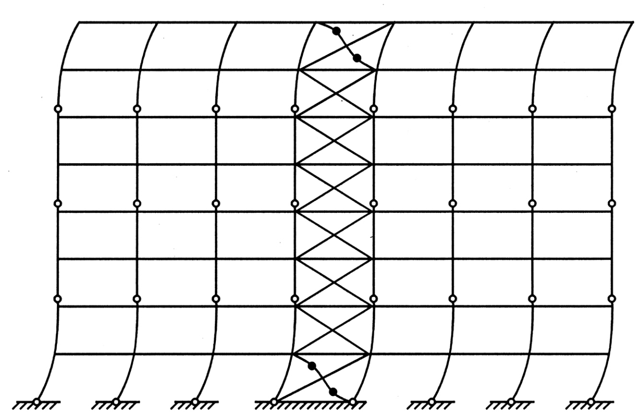


Fig. 19. Bending of multi-story column tiers due to non-uniform story drift.

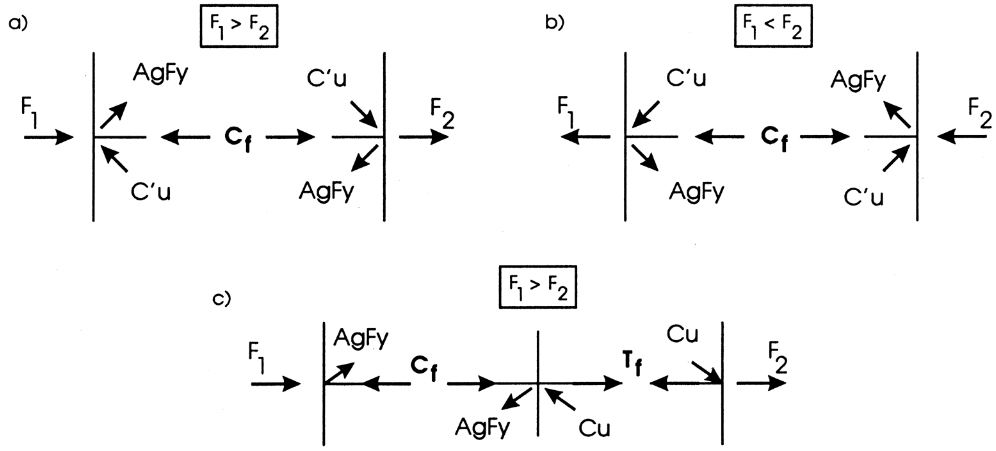


Fig. 20. Brace induced load for capacity design of beams.

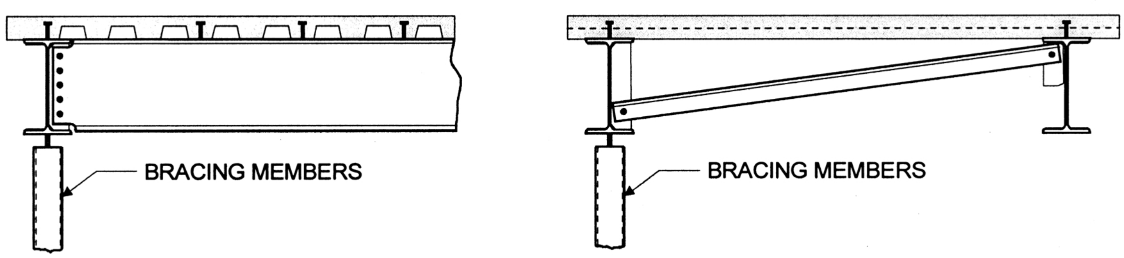


Fig. 21. Lateral bracing of beams in V-bracing.

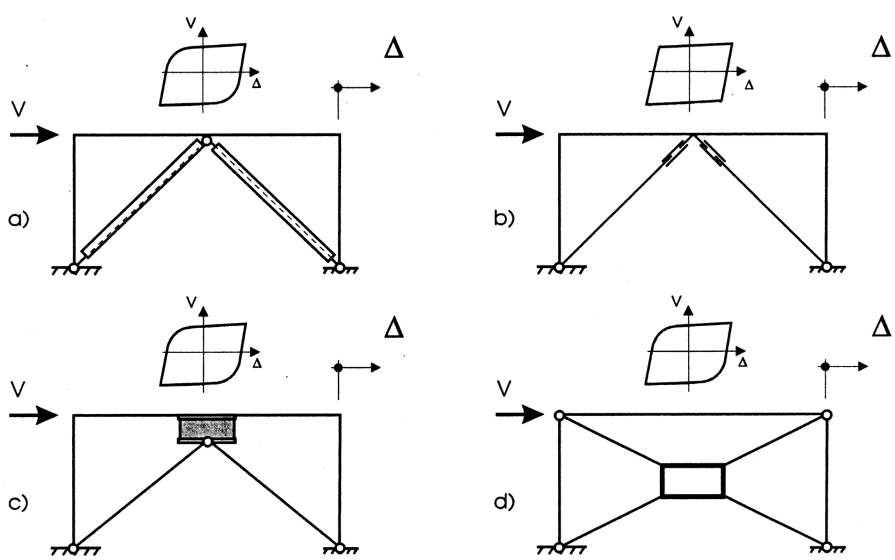


Fig. 22. Alternative bracing systems.

generally eliminated, which contributes in reducing the design loads for brace connections and the other components of the lateral load resisting system.

Eccentrically braced frames probably represent the most well known achievement resulting from this development effort. This system is now well described in current code provisions and is not discussed further in this paper.

The other systems can be grouped into four categories, which are schematically illustrated in Figure 22. The first group (Figure 22a) includes systems in which buckling of the bracing members is prevented, allowing the braces to yield both in tension and compression (Chen and Lu, 1990; Morino, Kawaguchi, Ito, and Shimokawa, 1996; Reina and Normile, 1997; Maeda, Nakata, Iwata, and Wada, 1998; Shimokawa, Ito, Kamura, Morino, and Kawaguchi, 1998; Tremblay, Degrange, and Blouin, 1999; Clark, Aiken, Kasai, Ko, and Kimura, 1999). One of these systems is the Buckling Restrained Brace system in which the bracing members are made of steel plates. The plates are inserted in steel tubes, which are filled with concrete to prevent buckling of the plates over their entire length. An example of such a brace is illustrated in Figure 23. In this example, a polyethylene membrane was used to allow the plate to slide freely in the grout and both ends of the plate have been reinforced to avoid buckling or fracture in these regions. Alternatively, the grout can be eliminated by sizing the steel plates and the tubes in such a manner that the steel plates fit tightly into the tubes.

Figure 22b illustrates systems in which energy is dissipated through friction at the brace ends by means of bolted connections or specially designed devices (Pall and Marsh, 1982; Filiatrault and Cherry, 1987; FitzGerald, Anagnos,

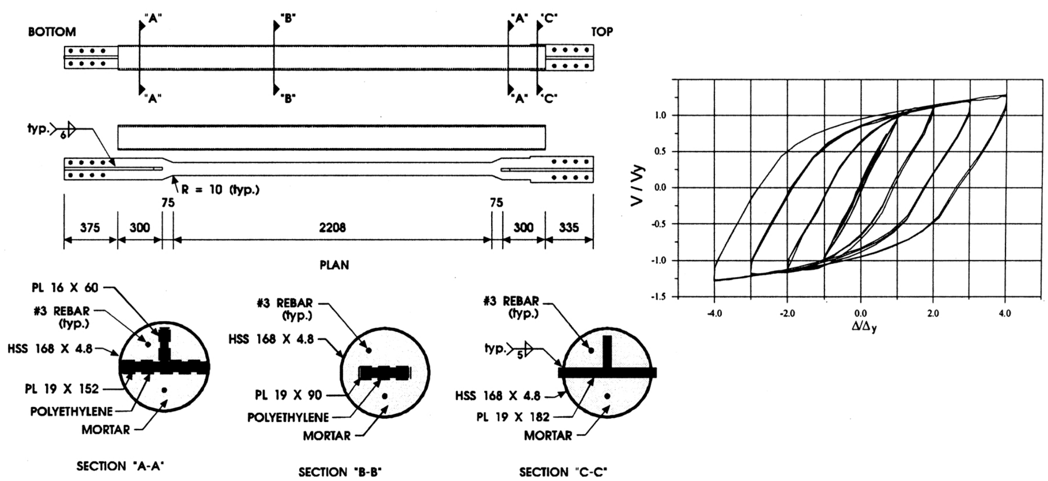


Fig. 23. Buckling restrained braces with concrete filled tubes (Tremblay et al. (1999)).

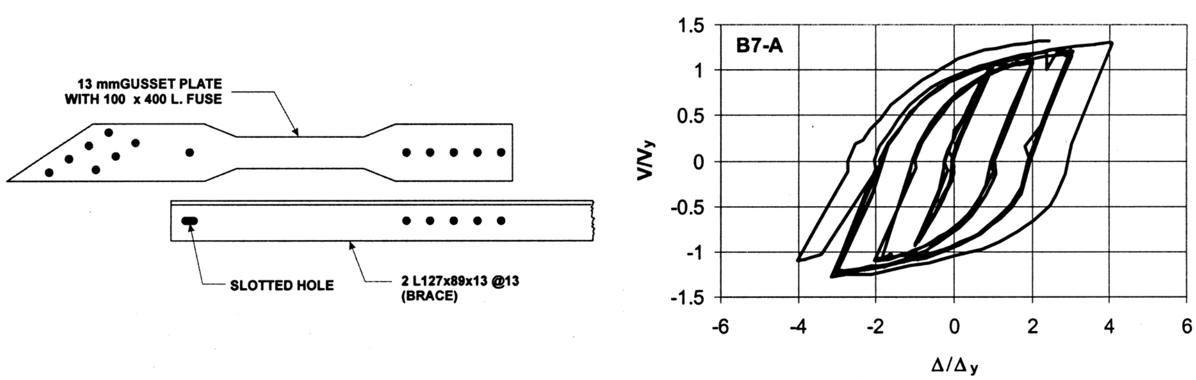


Fig. 24. Brace fuse detail (Tremblay and Bouatay (1999)).

Goodson, and Zsutti, 1989; Grigorian and Popov, 1993; Tremblay and Stiemer, 1993; Kullmann and Cherry, 1996; Filiatrault, Tremblay, and Kar, 1999). In addition to the enhanced hysteretic response, these systems have the advantage that the bracing members remain intact after an earthquake. Also part of this group is the brace fuse system (Tremblay and Bouatay, 1999; Rezai, Prion, Tremblay, Bouatay, and Timler, 2000) and the "weak gusset-strong brace" design (Walbridge, Grondin, Cheng, 1998; Cheng and Grondin, 1999). An example of the former is illustrated in Figure 24 where a fuse plate is introduced at one end of a double angle bracing member. As shown, the brace load is transmitted to the fuse plate before the end of the bracing member. Buckling of the fuse is prevented by extending the angles beyond their connection to the fuse, which results in a hysteretic response that nearly matches that of plain steel. In the weak gusset design, the gusset is sized to buckle at a lower load than the bracing members. Tension yielding can develop either in the brace or the gusset plate. Experimental and analytical studies performed in the last decade show that buckling in the gusset plate rather than in the bracing members results in better energy dissipation characteristics.

In Figure 22c, energy-dissipating elements are introduced at the apex of chevron bracing. These elements may consist of steel plates, which yield in shear (Seki, Katsumata, Uchida, and Takeda, 1988; Ohi, Lin, Nishida, Lee, and Tanaka, 1997; Takayama, Tsujii, Ogura, Izumi, and Tsujita, 1997) or a series of triangular steel plates that yield in bending (Whittaker, Bertero, Alonso, and Thompson, 1989; Tsai, Li, Hong, Chen, and Su, 1993; Wada, Huang, Yamada, 1997). Systems in Figure 22d are those in which steel members yielding in flexure are introduced in the braced frame. These members can be assembled to form a rigid frame inserted in the middle of an X-bracing (Jurukovski and Simeonov, 1988) or a knee brace located at the beam-column joints (Aritizabal-Ochoa, 1986; Balendra, Lim, and Liaw, 1997). Analytical studies on these two systems have been performed in Sugeng, Moss, and Carr (1988).

CONCLUSION

Considerable progress has been made over the last two decades in the understanding of the seismic inelastic response of Concentrically Braced Frames for steel structures. This development resulted in a set of comprehensive, while simple to use, seismic design provisions that permit the design and building of reliable and cost effective CBFs in active seismic regions. Alternative CBF systems have also been recently developed to achieve better overall performance through enhanced hysteretic response. Both types of CBFs can now be used in new construction as well as in seismic retrofit projects.

Design codes could still be improved, however, by stating more explicitly the various capacity design requirements for beams, columns, and their connections. Minimum seismic performance levels should also be included in codes for further development and approval of alternative bracing systems. Further research should concentrate on the development of cost effective solutions, together with quantitative design guidelines, for the mitigation of soft-story response in CBFs. Several promising avenues have already been proposed but very limited guidance is currently available for practicing engineers. The effects of the various types of ground motions (long duration subduction earthquakes, near-field pulse type motions, eastern versus western events, etc.) on CBFs should be examined in order to tailor seismic provisions to achieve a more uniform level of safety across North America.

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