Achieving a Stable Inelastic Seismic Response for Multi-Story Concentrically Braced Steel Frames

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Recent seismic design provisions (CSA, 2001; AISC, 2002) for concentrically braced steel frames (CBFs) aim at dissipating seismic input energy through inelastic deformations in the bracing members. These deformations include tension yielding of the braces as well as plastic hinge rotation that develops upon buckling of the braces in compression and subsequent straightening of the braces when pulled in tension in the next loading cycle. Over the years, researchers have gained a good understanding of the hysteretic response of steel braces (Jain, Goel, and Hanson, 1980; Popov and Black, 1981; Astaneh-Asl and Goel, 1984; Astaneh-Asl, Goel, and Hanson, 1985) and detailing requirements such as maximum *b/t* and *KL/r* ratios are now prescribed in codes to achieve ductile brace response. Capacity design rules have also been introduced in codes to prevent premature failure of the brace connections and ensure that the response of the beams and columns will remain essentially elastic (Tremblay, 2001).

Even so designed, multi-story CBFs typically exhibit large variations in story drift and inelastic demand over their height when subjected to strong ground motions (Martinelli, Perotti, and Bozzi, 2000; Redwood, Lu, Bouchard, and Paultre, 1991). This behavior is mainly due to the degradation in brace compressive resistance that results from large compression inelastic excursions and/or successive compression load cycles beyond buckling. The story shear resistance diminishes at levels where brace buckling occurs first, promoting the development of larger story drifts at these floors. The severity of this phenomenon depends on several factors including brace properties, bracing configuration, number of stories, type of ground motions, etc. In most cases, this behavior leads to soft-story response, which has been known for long as being critical in multi-story structures. When significant gravity loads act on the laterally deformed stories, story drifts are generally amplified further and collapse of the frame by instability under seismic ground motion becomes a possibility.

P-delta effects and dynamic instability under seismic events have been studied by several researchers in the past (Jennings and Husid, 1968; Montgomery, 1981; Bernal, 1987 and 1992; Fenwick, Davidson, and Chung, 1992; MacRae, Priestley, and Tao, 1993; Tremblay, 1998; Tremblay, Cote, and Leger, 1999; Tremblay, Degrange, and Blouin, 1999; Gupta and Krawinkler, 2000; SAC, 2000a; Vian and Bruneau, 2001). When subject to several cycles of inelastic deformations, a structure tends to drift towards one direction, generally after a large inelastic excursion has taken place towards that direction. This "crawling" or "ratcheting" response is due to the gravity loads supported by the deformed structure; these gravity loads producing additional lateral loads pushing the structure further away from its initial position. This behavior can also be seen from a point of view of potential energy: as the structure deforms laterally, the columns sway and the story height diminishes slightly. Gravity loads then lose their potential energy. Under cyclic inelastic excursions, the structure naturally tends to migrate towards the direction associated with a reduction in potential energy rather than towards the undeformed position as the latter scenario requires lifting up the gravity loads back to their initial level.

Accounting for P-delta effects on the inelastic seismic response of structures still represents a challenging task as both material and geometric nonlinearities must be accounted for in the dynamic response of a structure to complex ground motion excitations. The amplitude of these effects depends on numerous factors: the dynamic characteristics of the structure (damping, periods, mode shapes, etc.), its lateral resistance relative to the ground motion demand, the hysteretic behavior of the energy dissipation elements in the structure, the geometry of the structures (number of stories, story height, etc.), the importance of the gravity loads, the characteristics of the ground motions, etc. As of today, no simple design method exists to safeguard against the potential for collapse by instability under seismic ground motions.

This paper describes the influence of P-delta effects on the inelastic seismic response of concentrically braced steel frames. A parameter is first defined to approximately characterize the significance of P-delta effects on inelastic structural behavior and key elements of typical brace hysteretic response are then briefly reviewed. Thereafter, the various parameters influencing the stability of CBFs under seismic events are examined through numerical examples. In the

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second part of the paper, solutions are proposed to improve the stability of the inelastic seismic response of CBFs. These vary from the selection of appropriate brace characteristics to the use of bracing configurations with enhanced capabilities in redistributing inelastic response.

DYNAMIC INSTABILITY IN CBFS

P-delta Effects in Seismic Design

Figure 1 shows the idealized lateral response of a single-story structure of height h and a lateral load-resisting system exhibiting elastic-perfectly plastic behavior characterized by its elastic stiffness, K, and strength, V_y. If a gravity load P is acting on the structure, the stiffness of the system is reduced by P/h, both in the elastic and the inelastic ranges. The decrease in stiffness does not affect much the seismic dynamic response in the elastic domain as only the period of vibration of the structure is influenced: $T_{P\text{-}\Delta} = T\ [K/(K\text{-}P/h)]^{1/2}$. P-delta effects on the inelastic response are generally more severe because the effective lateral strength of the structure is reduced by the force $P\Delta/h$. P-delta effects in the inelastic range are therefore better characterized using a stability coefficient, Ψ, defined herein as the loss in lateral resistance at maximum displacement, Δ_T, divided by the yield resistance, V_y. In seismic design, V_y is obtained by dividing the anticipated elastic lateral force, V_e, by a force or response modification factor, R, i.e., $V_y = V_e / R$, and the maximum anticipated drift under the design earthquake, Δ_T, is generally assumed equal to the drift obtained assuming elastic response, Δ_e, (equal displacement principle). Using these assumptions, the stability coefficient, Ψ, becomes:

$$\Psi = \frac{P\Delta_e}{hV_y} = \frac{PV_e}{hKV_y} = \frac{PR}{hK} \qquad (1)$$

A similar expression has been recently adopted in FEMA 350 (SAC, 2000b) to characterize P-delta effects for the seismic design of steel moment resisting frames. For this type of structure, P-delta effects are considered to be negligible if Ψ is less than 0.10 but dynamic instability becomes likely when Ψ exceeds 0.30. While convenient and simple to use, this coefficient can only give some indication of the anticipated gravity load effects relative to the lateral resistance of the structure. The inelastic dynamic response of structures to earthquakes is far too complex to fully capture P-delta effects on that response and, ultimately, assess the likelihood of dynamic instability, with such a simple parameter. For example, the value of Ψ at the verge of collapse under earthquakes can vary significantly among various structural systems or ground motions, as shown later in the paper, and it is clear that Ψ cannot be used alone to evaluate the importance of P-delta effects in design.

Hysteretic Response of Bracing Members

The inelastic behavior of braced steel frames under cyclic loading mainly depends on the hysteretic response of the bracing members and the bracing configuration used in the structure. Several experimental programs have been performed in the past to study the hysteretic behavior of steel braces and identify the key parameters influencing their inelastic cyclic response. Figure 2a shows a typical axial force-axial deformation (F-δ) hysteresis curve for a brace subjected to incremental cyclic displacements. In compression, buckling occurs in the first cycle at a load C_u and the compression strength reduces if further negative axial deformation is imposed. In the second and subsequent cycles, the compression strength also gradually decreases due to Baushinger effects, residual lateral deformation, and local buckling. In tension, braces can develop their yield capacity, AF_y, but they accumulate permanent elongation at each cycle and the full tension capacity upon tensioning in subsequent cycles can only be reached if a larger positive axial deformation is imposed.

The single most important parameter affecting the brace response is its effective slenderness ratio, KL/r. The buckling resistance, C_u and the post-buckling compressive

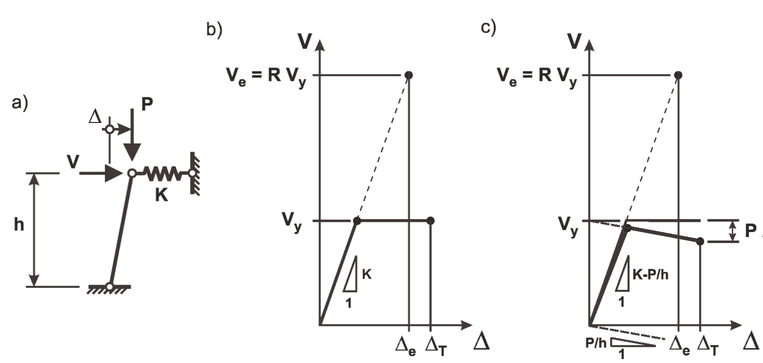

Fig. 1. P-delta effects on the inelastic lateral response of single-story structures: (a) structural model; (b) response with no gravity load; (c) response with gravity loads.

strength, C'_u, both depend on the brace slenderness. In Figure 2b, measured values of the residual compression strength at a ductility, μ, of 3.0 are plotted against the brace slenderness parameter, $\lambda = (KL/r)(F_y / \pi^2 E)^{1/2}$. Brace ductility, μ, is defined herein as the ratio of the peak axial deformation to the yield deformation. For a ductility of 3.0, a best fit approximation of the reduced brace resistance is given by $C'_u = AF_y\ (0.084 + 0.12\lambda^{-1.61}) \leq C_u$ (Tremblay, 2002). The predicted brace buckling strength, C_u, corresponding to the unfactored compression resistance as prescribed in the Canadian Standard S16-01, *Limit States Design of Steel Structures* (CSA, 2001), is also given in Figure 2b for reference: $C_u = AF_y\ (1+ \lambda^{2n})^{-1/n}$, with $n = 1.34$. As shown, the ratio C'_u / C_u is minimum for intermediate brace slenderness.

Influence of Bracing Configuration

The story shear-story drift response of frames with braces oriented in only one direction mimics the brace axial response described in the previous section. In Figure 3, the influence of the stability coefficient and the brace slenderness are studied for a single-story frame of that type when subjected to the 1940 El Centro earthquake record. An R factor of 3.0 was used in the design of that system. That value corresponds to the R factor specified in the CSA S16-01 Standard for Type MD (Moderately Ductile) CBFs. For comparison, values equal to 2.0 and 1.5 are assigned to Type LD (Limited-Ductility) CBFs and Ordinary Construction CBFs, respectively. It must however be noted that, unless otherwise specified, the R factors used in this paper are applied differently than in normal design practice in order to make the numerical examples more transparent and less dependent on specific building code regulations. In codes, the elastic lateral force, V_e, is obtained from smoothed design spectra that are established for a given probability level and adjusted to account for various factors. In the simulations presented herein, the elastic base shear, V_e, is determined using a linear time history dynamic analysis for the particular ground motion studied. That shear force is then divided by the R factor to obtain the design lateral load for the frame example. For instance, the factored compressive strength of the brace in the structure of Figure 3a, $C_r = \phi C_u$, with $\phi = 0.9$, was set equal to the brace compression force produced by V_e divided by 3.0. Also note that the nonlinear dynamic analyses performed on that structure as well as on the various structures presented in the paper were carried out with the Drain-2D computer program by Kanaan and Powell (1973). In these analyses, bracing members were modelled using the inelastic brace buckling element with pinned ends (Element No. 9) developed by Jain and Goel (1978). This element reproduces the key features of the hysteretic response shown in Figure 2a, including tension yielding, degradation of the compression strength and cumulative elongation.

In Figure 3b, the response of the frame is illustrated for three values of Ψ when a brace with intermediate slenderness (KL/r = 80) is used. The value of Ψ was varied by changing the load P applied to the structure. As shown, lateral deformations generally develop towards the direction corresponding to brace compression, due to the lower resistance offered by the brace in that direction, especially after buckling has occurred. The deformations increased and instability is eventually observed when Ψ is gradually increased. Figure 3c shows how the response varies with brace slenderness. In all three cases, the same brace was used assuming that the brace effective length factor K could be modified such that KL/r becomes equal to 40 and 120. So doing, the stiffness and the period are kept the same and only the amplitude of the ground motion is adjusted to maintain a ratio of 3.0 between elastic and design forces. As expected, the response improves when the brace becomes stocky or more slender due to the less severe degradation in compression strength (Figure 2b). Nevertheless, this brac-

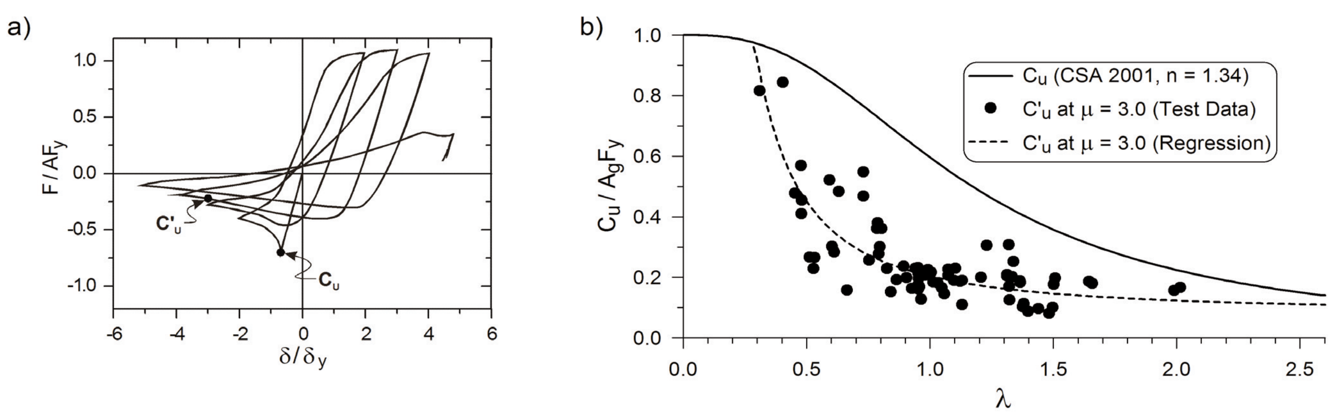

Fig. 2. Response of bracing members subjected to cyclic inelastic loading: (a) hysteretic behavior; (b) residual compressive resistance at a ductility of 3.0 (adapted from Tremblay, 2002).

ing configuration remains prone to dynamic instability and, hence, is excluded from Types MD and LD CBFs in the S16-01 Standard or from the Special Concentrically Braced Frame (SCBF) category in the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2002).

When two braces oriented in opposite directions are used, two design approaches can be used: tension-compression (T/C) design or tension-only (T/O) design. In the former, the story shear is shared between compression-acting and tension-acting braces. Because the system must resist loads in both directions, compression or slenderness requirements generally govern the choice of the brace cross-section. In T/O design, the compression braces are ignored and the story shear is assumed to be entirely resisted by the tension-acting braces. Typically, T/O design is used in lightly loaded frames with long brace effective length, *KL*, in which case braces acting in compression are not efficient. This is illustrated in Figure 4c where the brace cross-sectional area that is required according to each design approach is plotted against brace slenderness for a symmetrical X-bracing. As indicated, the figure applies to braces made of 350 MPa (50.8 ksi) steel and designed according to the CSA Standard. For this particular case, braces with *KL/r* greater than 90 ($\lambda \cong 1.20$) require less steel if designed only in tension. However, T/C bracing is specified for most seismic applications as it generally leads to better overall performance than T/O bracing. For instance, only T/C bracing is allowed in AISC for the SCBF category. In CSA, tension-only systems are permitted for Types MD and LD CBFs but building height limits apply, as discussed later. Therefore, the T/C system is discussed first, together with the influence of ground motion characteristics and building height. Thereafter, the stability of T/O systems will be commented on.

Figure 5 gives the variation of the peak story drift, Δ_T, as a function of the applied gravity loads for a single-story tension-compression X-bracing system. The frame was designed with R = 3.0 and was subjected to the El Centro

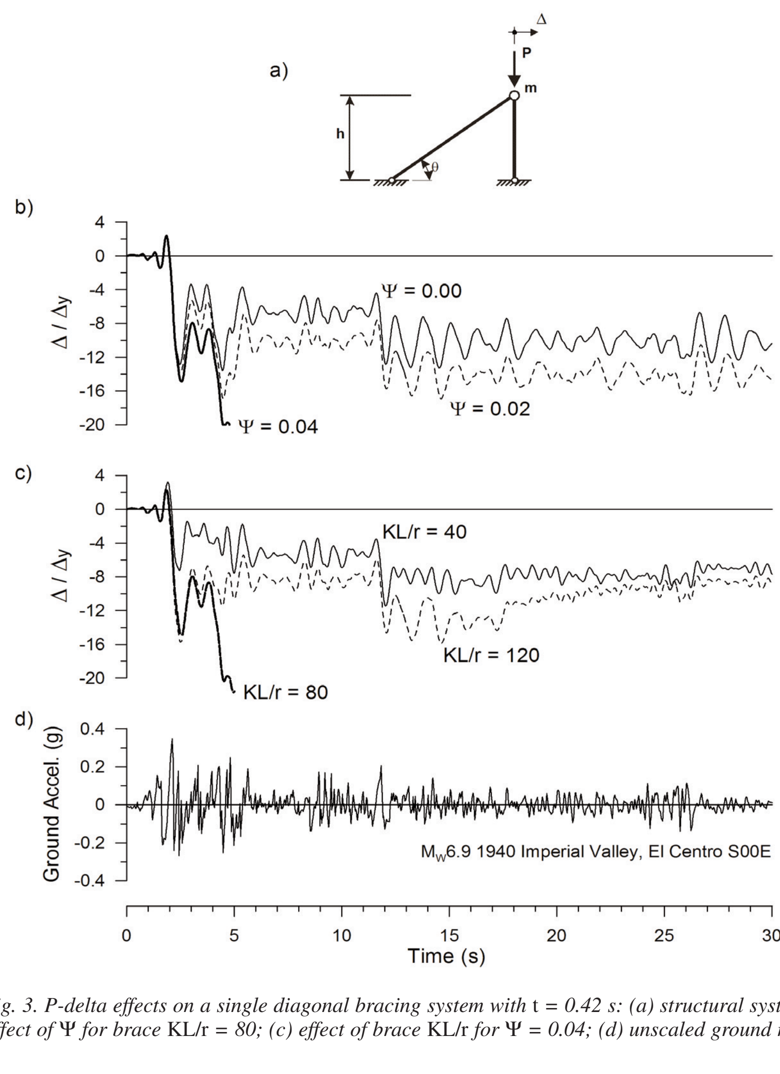

Fig. 3. P-delta effects on a single diagonal bracing system with t *= 0.42 s: (a) structural system; (b) effect of* Ψ *for brace* KL/r *= 80; (c) effect of brace* KL/r *for* Ψ *= 0.04; (d) unscaled ground motion.*

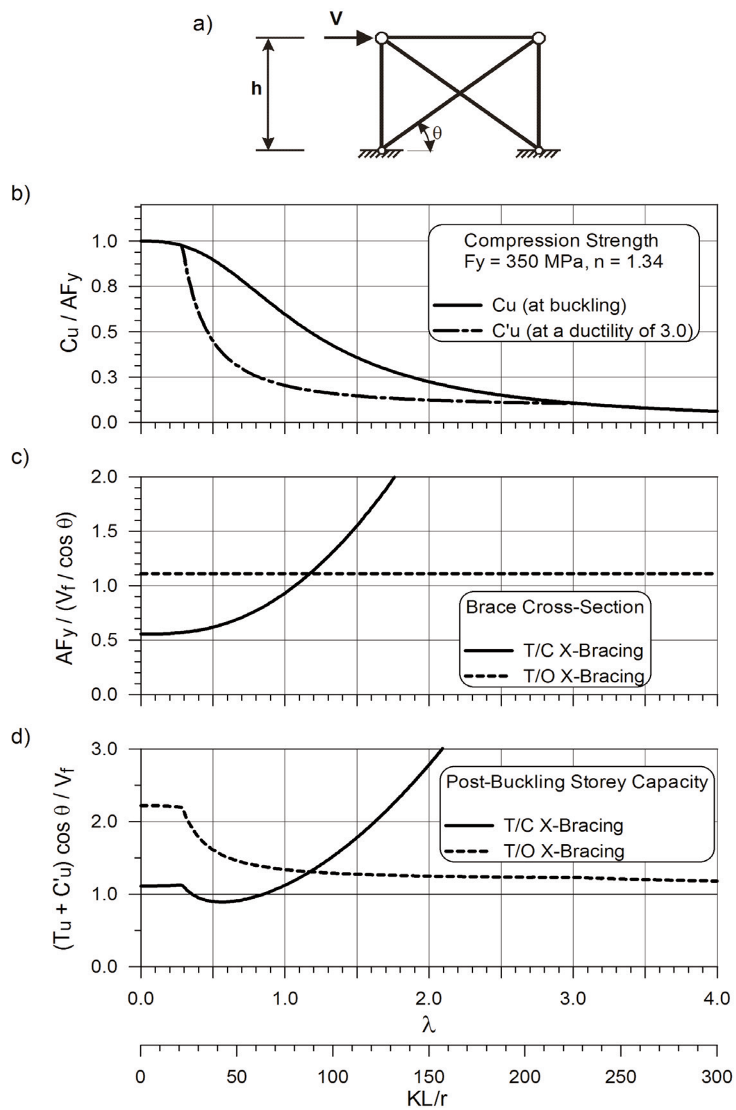

Fig. 4. Influence of brace slenderness on the story shear response of T/C and T/O bracing systems: (a) structural system; (b) brace compressive strength; (c) required brace cross section area; (d) post-buckling story shear capacity using C'_u *in Fig. 2b.*

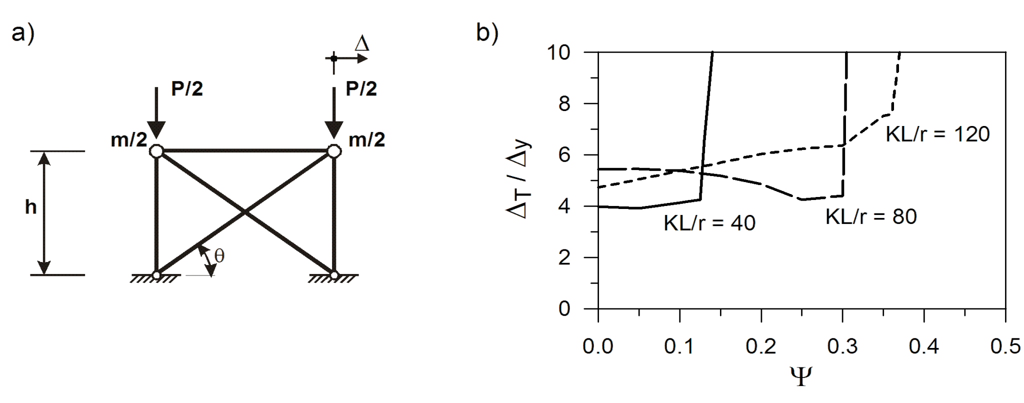

Fig. 5. Influence of gravity loads and brace slenderness on the stability of single-story T/C X-bracing under the 1940 El Centro Ground Motion Record (t = 0.42 s, R = 3.0).

ground motion. Using the same approach as before, the calculations were performed for *KL/r* = 40, 80, and 120. This time, the frame exhibits a more robust performance when more slender braces are used. The reason for this is that the braces are designed for compression and the difference between their yield tension capacity ($T_u = AF_y$) and their buckling strength (C_u) increases with the effective slenderness. A more slender brace thus possesses a larger reserve strength in tension after the compression brace has buckled and this results in a higher post-buckling story shear resistance, even if the compression brace loses some of its capacity upon buckling (see Figure 4d). Hence, more slender braces correspond to a larger story shear resistance prior to the formation of a complete story mechanism and, thereby, a more stable response. The example in Figure 5 also illustrates that T/C systems are by far less sensitive to dynamic instability than structures with single diagonal bracing. For example, collapse by instability with a brace slenderness of 80 develops at $\Psi = 0.30$ in Figure 5, whereas $\Psi = 0.04$ was sufficient to initiate the same behavior for a single brace system in Figure 3.

In the previous examples, the Ψ factor was modified by changing the amount of gravity load supported by the structure. It is interesting to note that a different response can generally be obtained if other parameters of Equation 1 are varied to produce the same value of Ψ. For example, the time history of the drift experienced by the single-story X-bracing of Figure 5 is shown in Figure 6 for a brace slenderness *KL/r* = 80 and $\Psi = 0.10$. In the figure, the response of the same structure is also given when the value of Ψ is increased by a factor of 2.0 by: 1) doubling *P*; 2) scaling up the ground motion by 2.0, i.e., multiplying Δ_e by 2.0; and 3) reducing the strength by a factor of 2.0. In each case, all

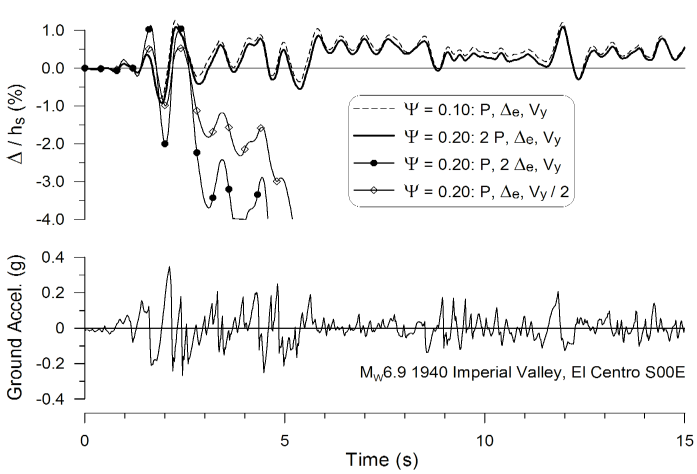

Fig. 6. Influence of varying P, Δ_e, *and* V_y *on the stability of single-story T/C X-bracing under the 1940 El Centro Ground Motion Record* (t = *0.42 s,* KL/r = *80).*

other parameters were kept unchanged. As illustrated, the response is not influenced much when the gravity load is increased but instability occurs when the other two parameters are varied. Reducing the strength typically is more critical because the lateral capacity of the system becomes smaller relative to both the P-delta and the inertia forces. However, a similar double effect also exists when the amplitude of the ground motion is amplified, as not only the seismic demand increases with respect to the structure strength, but also lateral deflections, and hence P-delta forces are also amplified.

The signature of the ground motion is another parameter which is not captured by the coefficient Ψ, as illustrated in Figure 7 for the same simple X-bracing. Three different ground motion time histories from intra-plate events are considered in the simulations shown: M_w 6.9 1940 Imperial Valley earthquake (El Centro #9, S00, PHA = 0.35g, PHV = 0.33 m/s, R = 12 km), Ms 5.7 1988 Saguenay earthquake (La Malbaie, N63, PHA = 0.12g, PHV = 0.047 m/s, R = 97 km), and M_w 6.7 1994 Northridge earthquake (Rinaldi, N228, PHA = 0.84g, PHV = 1.66 m/s, R = 7.5 km). Note that PHA and PHV correspond to the peak horizontal ground acceleration and velocity of the records, and R is the hypocentral distance of the recording station. All ground motions and response spectra in Figure 7 are normalized to their respective peak values, as only the shape of these plots is of interest here. The El Centro record is from a crustal event along the west coast of North America, while the Saguenay earthquake is typical for eastern North America with most of the released energy being concentrated in the high frequency (short period) range. The third signal is the fault-normal component of the ground motion recorded at a short distance from the epicentre of the Northridge earthquake. It contains a large acceleration pulse at $t = 2.5$ s and significant energy in the long period range. A simulated ground motion time history (Tremblay and Atkinson, 2001) representing long duration motions expected from Cascadia subduction earthquakes in the Pacific North West is also considered for comparison purposes (M_w 8.5 Simulated Cascadia ground motion, PHA = 0.10g, PHV = 0.17 m/s, R = 120 km). In the calculations, all records were scaled to produce elastic brace forces equal to three times the brace compression design forces.

As shown, the near field ground motion is by far the most critical, with instability developing during the acceleration pulse. This type of signal is known to amplify significantly inter-story drifts due to the large ground displacements (Filiatrault, Tremblay, and Wanitkorkul, 2001; Hall, Heaton, Halling, and Wald, 1995; Krawinkler, Parisi, Ibarra, Ayoub, and Medina, 2001; Mateescu and Gioncu, 2000), leading to significant P-delta forces. Lateral deformations under the El Centro record also develop during the main acceleration pulses of the signals, but the response of the structure remains stable as these pulses contain much lower energy and are more evenly distributed in the two opposite directions compared to the near-field event. High frequency ground motions in eastern North America typically produce relatively smaller structural displacements and shorter inelastic excursions (Tremblay and Atkinson, 2001). P-delta effects in structures subjected to these ground motions are expected to be smaller, especially when the period of the structure is much longer than the dominant period of the ground motion (Tremblay, Duval, and Leger, 1998). Conversely, long duration ground motions at plate boundaries contain a large number of cycles, which permits the gradual development of larger drifts (Tremblay and Atkinson, 2001; Tremblay, 1998) and, consequently, the potential for instability (Fenwick et al, 1992; MacRae et al, 1993; Tremblay et al, 1998). In this example, the structure collapsed after 30 s of shaking, when lateral deformations had progressively increased towards one direction ("ratcheting" response). Thus, the signature of the ground motions, the brace properties, and the bracing configuration are parameters that can significantly impact on the seismic stability of CBFs but which cannot be accounted for in the Ψ factor.

In multi-story structures, the response becomes even more complex as several new factors come into play. These include, but are not limited to, the number of stories, the floor heights, the dynamic (modal) properties of the structure, the distribution of the lateral strength over the building height, the change in brace properties from one story to the next, the amount of gravity loading carried by the columns at each floor, and continuity of the columns. Of course,

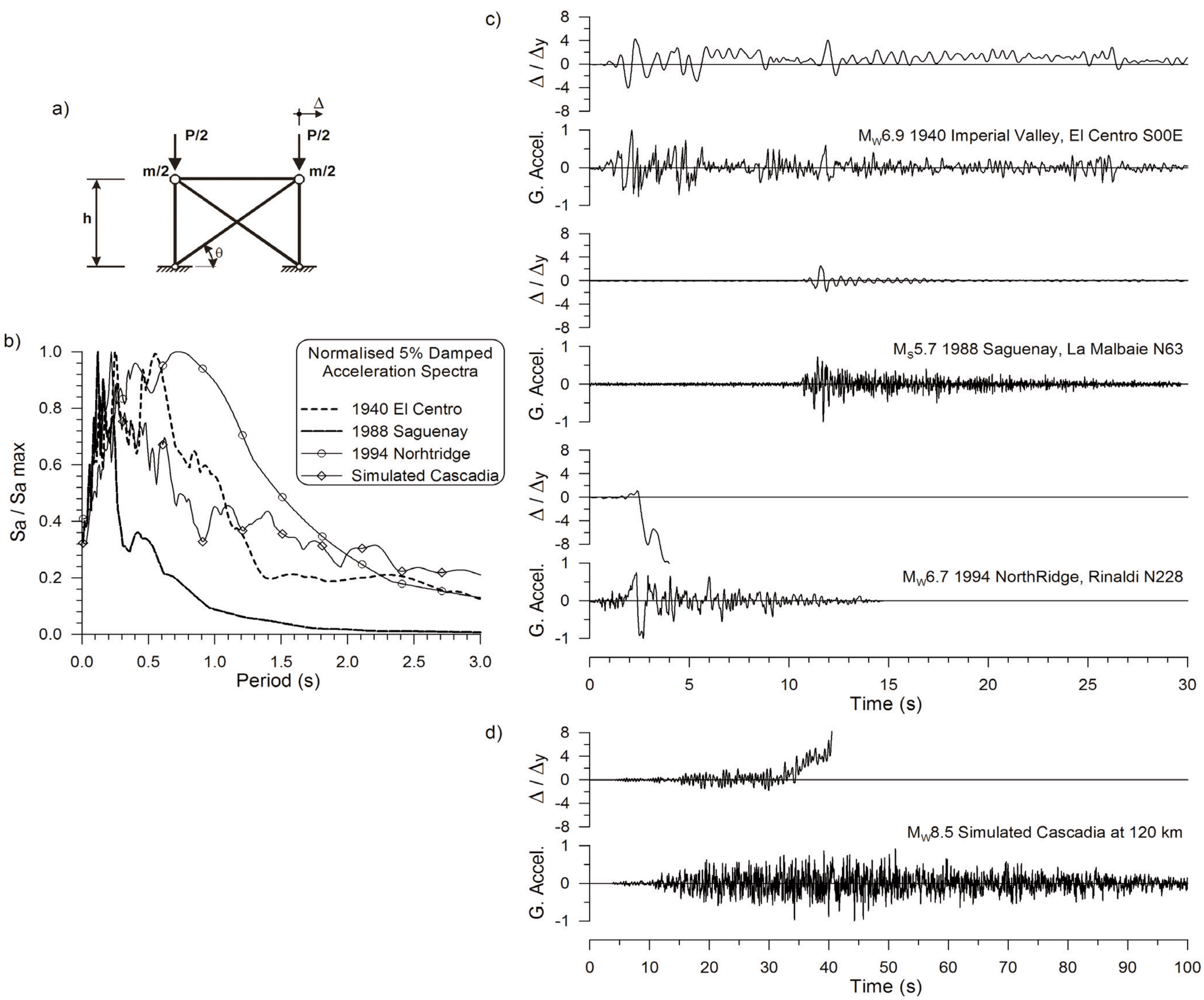

Fig. 7. Influence of ground motion on the response of single-story T/C X-bracing: (a) structural system (R = 3.0, KL/r = 80, t = 0.42 s, Ψ = 0.25); (b) normalized response spectra; (c) response to three intra-plate earthquakes; (d) response to an inter-plate earthquake.

brace slenderness, bracing configuration, and ground motions still remain key parameters influencing the response. In addition, it can be anticipated that the presence of vertical irregularities (mass, stiffness, geometry, strength, etc.) can be detrimental to the seismic stability of multi-story frames, but this issue still needs to be confirmed by further studies.

Figure 8a shows a plan view of a typical office building for which several bracing configurations are examined in the paper. Details on that structure can be found elsewhere (Tremblay, 2000; Tremblay and Robert, 2000). All beam-to-column connections are pinned and the lateral loads are resisted by the bracing bents only, as typically done in practice. The building is symmetrical in plan and a 2D model including half of the structure was used to evaluate the seismic response, as shown in Figure 8b for a T/C X-bracing acting in the N-S direction (note: for simplicity, only four-story frames are illustrated in the figure, although other building heights are examined). All gravity columns that are laterally braced by the braced frame studied are included in the model. This permits to correctly introduce all gravity loads and to capture the beneficial effect of column continuity when multi-story columns are used, as discussed later.

Figure 9 presents the response of an 8-story X-bracing. In that structure, the brace slenderness, *KL/r*, varied from 67 at the base to 98 at the top floor, with an average of 72. In Figure 9a, the vertical distribution of the story shear from a code equivalent static force procedure is compared to the supplied factored story shear resistance and to the elastic story shear demand from three ground motions. As shown, the supplied capacity matches nearly perfectly the design values, which represents optimum design conditions. However, the demand from the earthquakes deviates from the static force distribution, with relatively larger story shear forces in the upper floors due to higher mode effects.

Figures 9b and 9c give the envelope of the story drift computed under the El Centro and Northridge (Rinaldi) records, respectively. The calculations were performed for different amplitudes of ground motion, resulting in different R factors and, in turn, different values of Ψ. The parameter Ψ at the first floor, Ψ_1, is used to characterize the importance of the gravity load relative to the lateral strength of the structure. The results are presented for three values of Ψ_1. Under both ground motions, large deformations developed at the 7th floor due to the lack of capacity at this location. Under the El Centro record, dynamic instability developed at the 7th floor when Ψ_1 reached 0.16. Under the Northridge earthquake, the demand in the upper floors increased and then reduced as larger drifts developed at the base. Instability eventually took place at the bottom floors for the same

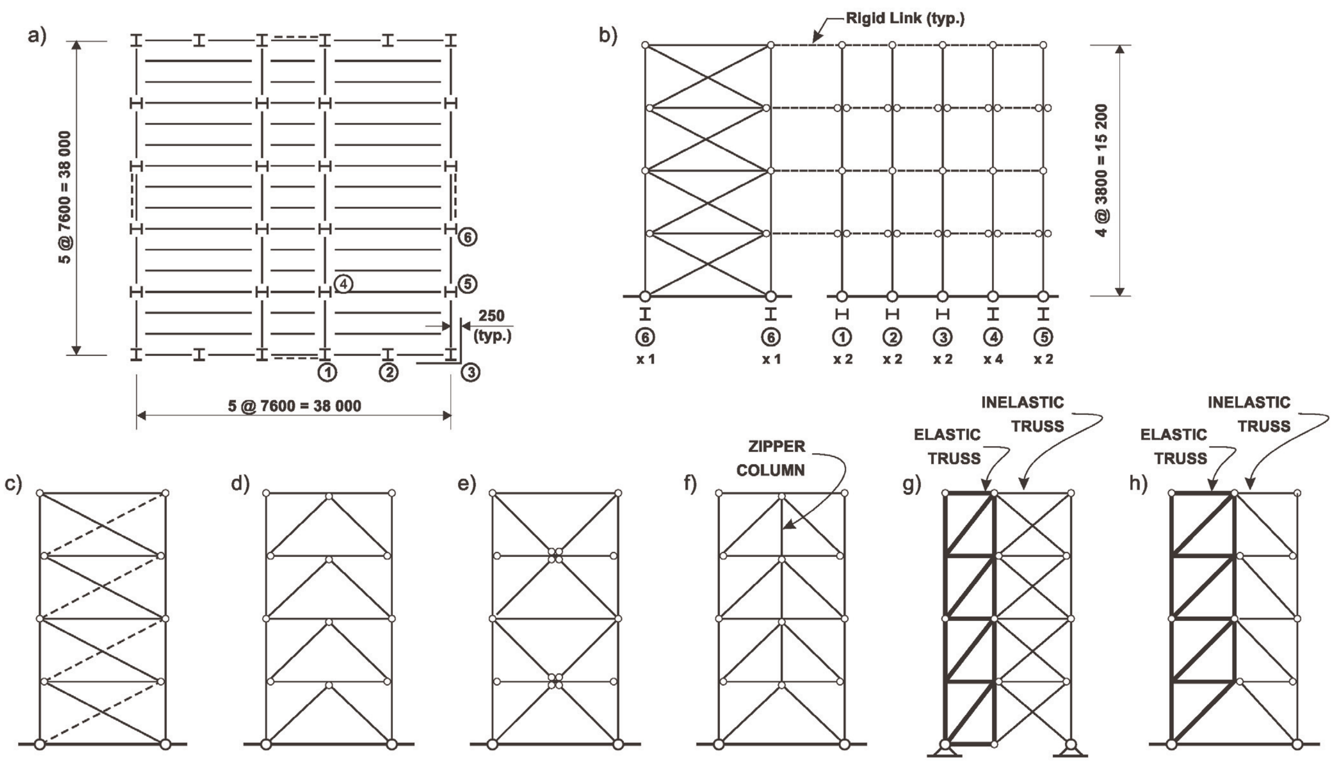

Fig. 8. Typical office building with various bracing configurations (4-story with 2-story column tiers shown): (a) plan view; (b) 2D model for a T/C X-bracing; (c) T/O X-bracing; (d) chevron bracing; (e) split-X bracing; (f) chevron bracing with a zipper column; (g) dual X-bracing; (h) dual chevron-bracing with BRB's.

value of Ψ_1 = 0.16. Based on elastic response, the Northridge record appeared to be more critical than El Centro at the top levels, but failure under the former eventually developed at the base, likely due to the large acceleration pulses acting directly on the floor carrying the heaviest gravity loads and exhibiting the lowest post-buckling reserve strength (lowest brace *KL/r*).

This example illustrates well key features related to dynamic instability in multi-story CBFs. Large variation in the inelastic response can be observed along the height of the building as there is no positive mechanism in the structure to help the propagation of yielding to the elastic portions of the frame. The inelastic demand typically concentrates where the elastic story shear demand is large compared to the supply (base of the building and upper floors). This leads to large story drift deformations developing at these levels, which in turn, result in significant P-delta forces that amplify further the story drifts. In addition, the concentration of the inelastic demand is accentuated at floors where a low story shear resistance is available after buckling of the braces, as this is the case when stocky braces are used (Figure 4d) or in chevron bracing (see below).

In the example, two-story column tiers with simple (pinned) splice connections were assumed in the model, as shown in Figure 8b. Such continuity of the columns contributed up to a certain extent in redistributing the demand along the building height, as illustrated in Figure 9d where the peak story drifts computed with and without column continuity are compared for a particular case. This beneficial impact comes with a price, however, as bending moments are imposed on the columns (see Figure 9e) that must be accounted for in the design in order to protect the integrity of the gravity load system (Sabelli, 2001; Tremblay, 2000; Tremblay and Robert, 2001; Tremblay and Stiemer, 1994).

The potential for dynamic instability typically increases with the number of stories, mainly because: 1) the inelastic demand tends to concentrate in a relatively smaller portion of the frame height, which results in higher demand at these locations; 2) stockier braces are used in taller frames, and those braces possess a lower reserve capacity in tension; and 3) the magnitude of the gravity loads relative to the lateral resistance becomes more important because taller frames have longer periods and, therefore, are designed with lower seismic coefficients. This tendency is reflected by the lower value of Ψ at collapse for the 8-story example in Figure 8 (= 0.16) compared to the value of 0.30 for *KL/r* = 80 ($\cong$ 72) in Figure 4.

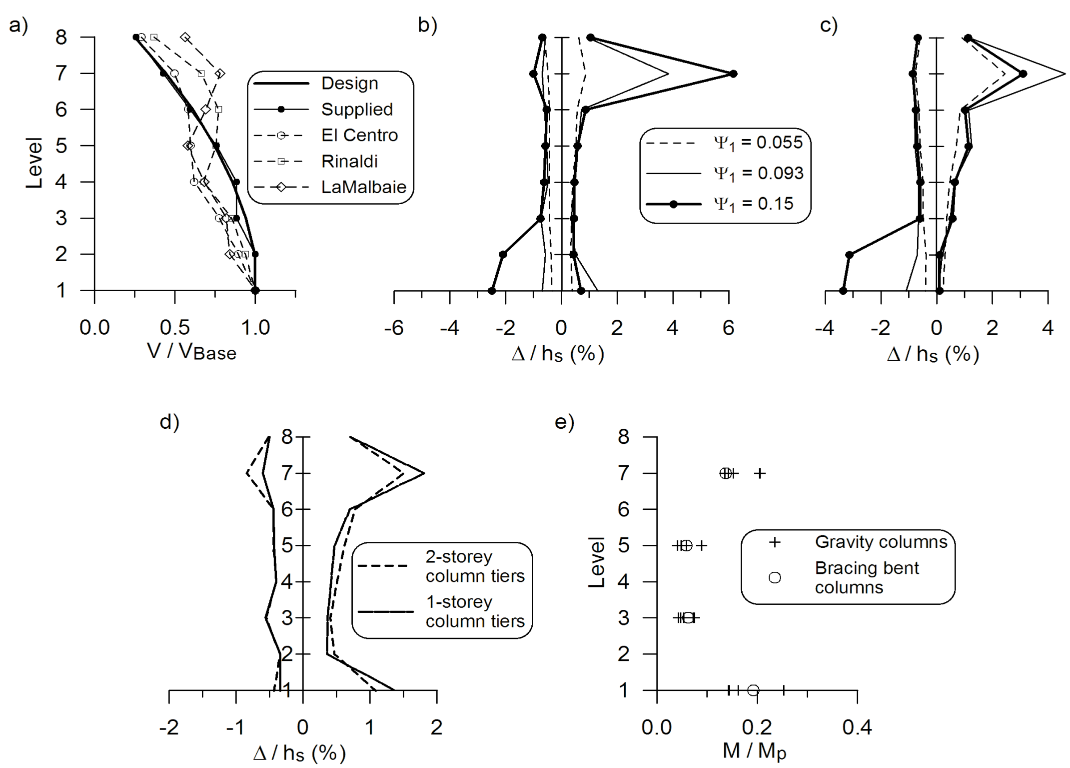

Fig. 9. Response of an 8-story X-bracing (T_1 = *1.95 s*)*: (a) normalized story shear demand vs supplied; (b) envelope of story drifts under 1940 El Centro; (c) envelope of story drifts under 1994 Northridge; (d) effect of the continuity of the columns on peak story drifts under 1940 El Centro* (Ψ_1 = *0.075*)*; (e) bending moments in two-story tiered columns* (Ψ_1 = *0.075*)*.*

In Figure 10, the effects of brace slenderness and number of stories on peak brace ductility demand are examined when T/C and T/O X-bracing systems are used for the building shown in Figure 8. Values of the brace slenderness parameter, λ, were set for each system and kept constant over the building height. For the T/C system, λ varies from 0.35 to 2.65 ($KL/r \cong 26$ to 200 for F_y = 350 MPa) and the brace cross-section area was determined to develop the required compressive strength using the specified value of λ. For the T/O design, the braces were sized to resist in tension the total story shear and the so-designed braces were assigned the prescribed λ value in the numerical model. As shown, λ varies from 1.35 ($KL/r \cong 100$ for F_y = 350 MPa) to infinity (very slender flat bars or rods).

All structures in Figure 10 were subjected to an ensemble of six earthquake records and the mean plus one standard deviation (M+SD) of the peak ductility demand in the braces is presented in the figure. As expected, the tendency to develop high localized inelastic deformations in T/C bracing increases with the number of stories and when the brace slenderness is decreased. The building height influences the response of T/O frames in the same manner but increasing the brace slenderness produces the opposite effect. In the T/O system, the compression braces are ignored in design but they can provide additional story shear resistance in the completed CBF. The contribution of the compression braces and the energy dissipation capacity of the system become more important when the brace slenderness is decreased (Figure 4d) and, hence, the response of T/O bracing generally improves when stockier braces are selected.

Figure 10 presents the results for 6-story and lower buildings only. Dynamic instability was observed in both systems for taller structures (Tremblay, 2000). Also worth mentioning is the fact that brace slenderness typically increases along the building height of T/C bracing. Hence, building structures with very stocky braces at every floor are unlikely to be encountered in practice and cases such as the one for λ = 0.35 in Figure 10 stands on the conservative side. Conversely, very slender braces are possible over the full height of a multi-story building in T/O design and that type of structure would typically be more prone than T/C systems to the development of large story drifts and dynamic instability.

Critical situations also exist when the story shear resistance of the frame degrades during inelastic cyclic response, as in chevron, or inverted-V bracing (Figure 8d). After the compression brace at a given floor has buckled, it gradually loses its capacity upon further lateral displacement and the tension brace starts to pull down on the beam until a plastic hinge forms near mid-span of the beam. Unless the beams are sized to resist the forces induced after buckling of the braces, a story mechanism typically forms before yielding develops in the tension brace, at a story shear force much lower than the story shear that was required to initiate brace buckling in the first place. Therefore, contrary to X-bracing, the reserve strength of the tension braces cannot be mobilized in this type of chevron bracing, and the resulting post-buckling story shear resistance is minimum when braces with intermediate KL/r are used (C'_u/C_u is minimum) and/or the beams carry significant gravity loads (Khatib, Mahin, and Pister, 1988; Tremblay, 2000; Tremblay and Robert, 2001; Tremblay and Stiemer, 1994).

Figure 11 illustrates the collapse response of an 8-story chevron frame designed with R = 2.0 for the building shown in Figure 8. For this particular structure, the design base shear was calculated using the National Building Code of Canada (NRCC, 1995). The frame is subjected to the El Centro record and the figure shows snapshots of the response together with the time history of story shear forces

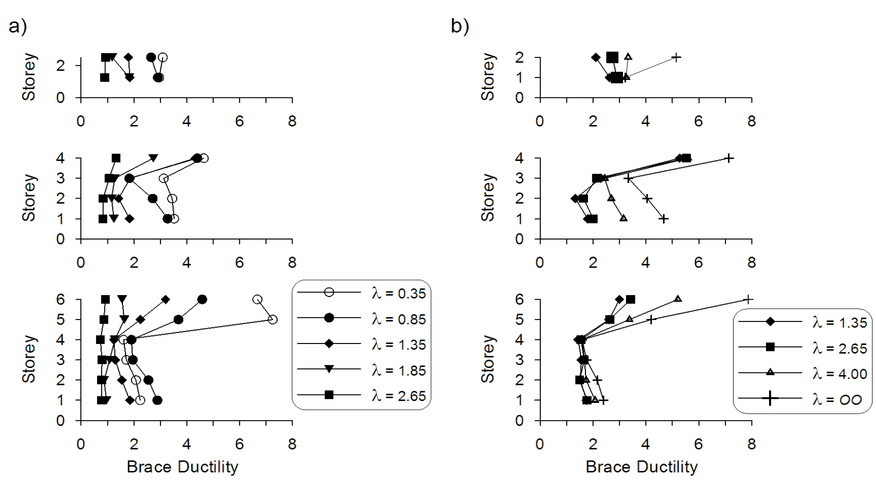

Fig. 10. Influence of brace slenderness and building height on peak brace ductility demand for R = *3.0 X-bracing: (a) T/C system; (b) T/O system (adapted from Tremblay, 2000).*

at the first floor. Three contributions to the base shear are given: the story shear resulting from the brace axial loads, the sum of the shear forces acting in half of the columns of the buildings (see Figure 8b), and the shear due to the gravity loads acting on the deformed structure (P-Δ shear). For clarity, the contribution of the columns is added to that of the braces. Up to a time t = 2.27 s, the structure remains elastic, as illustrated by the snapshot at t = 1.70 s. Brace buckling occurs at the 8th floor at t = 2.27 s and at the bottom floor at t = 2.62 s. Up to the buckling of the first story brace, the columns at the first floor do not carry any significant story shear and P-delta shears are very low because the story drift remains small. After buckling of that brace, all columns in the first tier slightly bend and some column shears then develop. In the time period between 3.31 s and 5 s, the ground motion is such that the story drift at the first floor reduces and remains small. After t = 5 s, the building is pushed again towards the left (negative drift) and very large deformations concentrate at the first floor due to the lower resistance offered by the buckled brace and the yielding beam. Significant bending then develops in the columns until buckling of the second floor brace occurs at t = 5.63 s, forming a 2-story mechanism which offers little lateral resistance. After t = 6.1 s, the total story shear resistance of the braces and columns at the first floor is less than the P-delta story shear, which leads to the collapse of the structure.

The stability factor Ψ_1 for that particular frame was equal to 0.08, based on the factored story shear capacity prior to brace buckling. Collapse at such a low value confirms the sensitivity of this bracing system to seismic instability. For

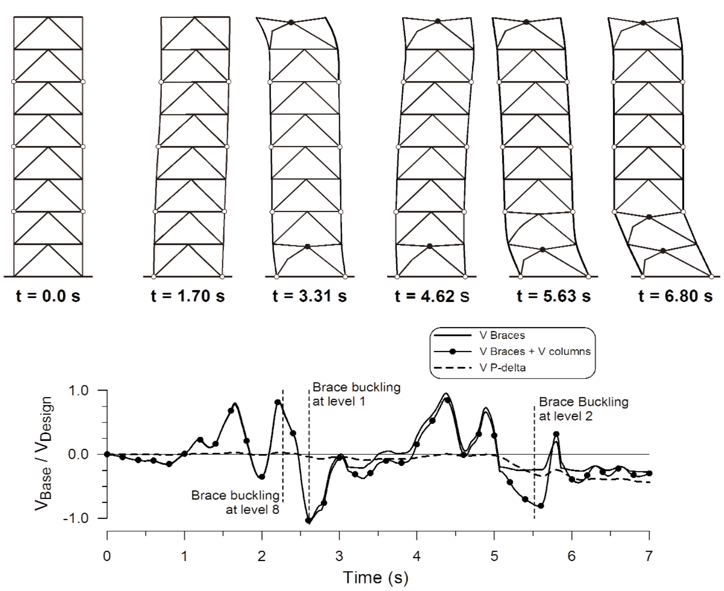

Fig. 11. Response of an 8-story chevron bracing (t_1 = 2.02 s, Ψ_1 = 0.080) Under 1940 El Centro ground motion (adapted from Tremblay and Robert, 2001).

this particular frame, only 25 percent of the initial story shear capacity is left after formation of the story mechanism. The stability factor then becomes equal to 0.32 if such a reduced value were used in Equation 1. Chevron braced frames of this type are not permitted in the AISC SCBF and CSA Type MD categories. They can be used only if the beams are designed to resist the forces corresponding to the development of the full yield strength of the tension braces, forcing a response similar to that observed for the T/C X bracing system (Khatib et al, 1988; Remennikov and Walpole, 1998; Sabelli, 2001; Tremblay and Robert, 2001). Chevron bracing of CSA Type LD must meet the same requirement if taller than 4 stories, as discussed next.

SOLUTIONS

System Limitations

As demonstrated in the previous examples, it is not appropriate in design to rely only on the stability coefficient Ψ to determine whether or not the seismic response of CBFs will be stable as many other parameters must also be considered. One approach that can then be used is to limit the range of application of the various bracing configurations within limits such as number of stories, seismic zone, design lateral resistance (R factor), etc. for which adequate performance has been demonstrated. This simple method also has the advantage of providing clear indications to the designer on which systems are likely to exhibit a more robust response. The procedure can be extended further by specifying ranges for parameters such as Ψ factor, brace slenderness, etc. for which better performance is expected for each system.

This approach has been adopted in the CSA S16-01 Standard as several provisions have been introduced to minimize the potential for dynamic instability of CBFs under seismic loading. For instance, all concentrically braced frames must be proportioned such that the total lateral resistance at all levels of every planar frame, as provided by the tension-acting braces only, is similar in the two directions of loading. In most braced structures, the brace slenderness is such that a large portion of the story lateral resistance is provided by the tension braces after brace buckling has occurred. Therefore, it is believed that this requirement is more efficient in mitigating the development of unsymmetrical inelastic response than specifying limits on the proportion of the story resistance provided by the compression and tension braces, as was done in previous Canadian code editions and still specified in AISC provisions.

S16-01 also now includes building height limitations that depend on the bracing configuration and the lateral resistance of the system (R factor). These limits, summarized in Table 1, are independent of the seismic zone in which the

Table 1 . CSA S16-01 Building Height Limitations

Bracing systems	R	Maximum number of stories
T/C X-bracing & Chevron bracing with strong beams[(a)]	3.0 2.0	8 12
T/O X-bracing	3.0 2.0	4 8
Chevron Bracing	2.0	4
[(a)] Beam designed to resist gravity + post-buckling brace forces.		

building is located. As shown, taller frames are permitted when larger seismic forces are used in design, i.e., when a lower level of inelastic demand is anticipated. This follows from the analytical observations described earlier. Also, more stringent height limits are specified for tension-only systems as well as for chevron bracing with beams that are not designed to sustain forces that develop after buckling of the braces. A similar trend is found in the New Zealand NZS 3404 Standard (NZS, 1997).

In S16-01, all columns in multi-story structures must be continuous over a minimum of two stories (full building height in T/O systems designed with R = 3.0) to control further the formation of soft-story response. Columns in braced bays must be Class 2 (compact) sections and be designed as beam-columns assuming that a uniform moment equal to 20 percent M_{pc} is acting concomitantly with maximum compression axial load. Minimum shear resistance is also prescribed for all column splices in multi-story structures. For load combinations involving earthquake loads, gravity columns are deemed to possess sufficient reserve axial strength to accommodate the flexural demand expected under the design earthquake. Class 3 (noncompact) sections are specified for these columns, however, to provide minimum curvature ductility capacity in case of temporary overloading during the ground motion.

P-delta effects are limited to 40 percent for all systems in CSA S16-01. In this Standard, however, the stability coefficient is computed using the factored base shear, not V_y, and the lateral resistance must be increased to account for P-delta effects, which results in an effective limit Ψ = 0.26 when using Equation 1. Brace slenderness, KL/r, must be equal to or less than 200 for all CBFs, except that KL/r up to 300 is permitted for single- and two-story T/O systems. Satisfying these limits, together with the prescribed maximum number of stories and column continuity requirements, should lead to a stable seismic response in most situations. However, it is recommended that caution be exercised in situations where lateral loads are small compared to gravity loads or when the brace slenderness approaches values that are critical for the selected bracing configuration.

Exceeding System Limitations

In S16-01, it is permitted to exceed the building height limitations given in Table 1 or to use other bracing systems, provided that stable inelastic response can be demonstrated. This can be achieved by performing nonlinear dynamic analysis of the structure using appropriate site specific ground motions for the return period adopted in design for ultimate or near-collapse limit states. Guidance on modelling assumptions, analysis procedures, and selection of ground motion time histories can be found in the literature (ATC, 1997; SAC, 2000a; Somerville, Smith, Punyamurthula, and Sun, 1997; Tremblay and Atkinson, 2001; Tremblay and Robert, 2001) and numerical tools exist that include inelastic bracing elements (Jain and Goel, 1978; Firmansjah, Goel, and Rai, 1996; Ikeda and Mahin, 1984; Taddei, 1995). Particular attention must be paid to ensure that the portion of the gravity load system which is stabilized by the braced frames studied is properly included in the model. If torsional effects are anticipated, a three-dimensional model of the whole structure will be more suitable to better capture the distribution of the inelastic demand between the different lateral load resisting elements of the building. Nonlinear dynamic analyses can also be advantageous for capacity design of tall frames, as the probability that all braces reach their maximum capacity simultaneously, generally assumed equal to 100 percent in manual calculations, diminishes with the number of stories. In particular, substantial savings can be expected for the columns and the foundations of the braced bays.

A first step towards achieving stable response in tall frames is to select the best of the bracing systems described earlier with proper detailing of the bracing members, i.e., a T/C bracing in which tension braces can develop their full yield strength and where the brace slenderness ratio is kept high enough so that every floor possesses some reserve strength after buckling of the braces. In several cases, however, this approach alone will not suffice because of the inherent deficiencies and shortcomings of conventional CBFs responding inelastically to earthquakes. Design engineers will then need to come up with alternative solutions involving bracing members that exhibit superior inelastic response, with no strength nor stiffness degradation, and/or bracing configurations that encourage a more uniform distribution of the inelastic demand over the building height.

Innovative bracing members have been developed recently that are specially detailed to yield both in tension and compression, without buckling (Chen, Chen, and Liaw, 2001; Clark, Kasai, Aiken, and Kimura, 2000; Iami, Yasui, and Umezu, 1997; Iwata, Kato, and Wado, 2000; Ko, Tajiran, and Kimura, 2001; Morino, Kawaguchi, and Shimokawa, 1996; Rezai, Prion, Tremblay, Bouatay, and Timler, 2000; Sabelli, 2001; Tremblay et al, 1999; Watanabe, Hitomi, Saeki, Wada, and Fujimoto, 1988). Such bracing members exhibit a nearly perfect and very stable bi-linear response under reversals of inelastic loading and can then be used very effectively in a chevron bracing configuration, as in Figure 8d (Sabelli, 2001; Tremblay and Bouatay, 2002). Figure 12 shows the envelope of the peak story drifts computed for an 8-story frame of this type used to brace the building of Figure 8 in the E-W direction under the 1940 El Centro ground motion. The braces are made of 300 MPa (43.5 ksi) steel and three levels of strain hardening have been considered in the analyses: 0 percent, 5 percent, and 10 percent. The value of Ψ_1 was also varied by modifying the ground motion amplitude and the results are presented for $\Psi_1 = 0.10$ and 0.20.

As shown, the response varies with the strain hardening level assumed for the braces, the demand being more uniform over the building height when higher strain hardening was considered. Once yielding has initiated at a level, the story shear resistance continues to increase if strain hardening is present, which encourages yielding to eventually develop in adjacent floors. This has an important effect on the stability of the structure. For that particular frame example, collapse occurred at $\Psi_1 = 0.11$, 0.32, and 0.46 for strain hardening equal to 0 percent, 5 percent, and 10 percent, respectively. Hence, significant improvement of the response can be obtained compared to that of a conventional chevron bracing (Figure 11) if sufficient strain hardening is provided. If not, the structure remains vulnerable to soft-story response, in spite of the better brace hysteretic response.

Reducing the length of the yielding portion of these buckling restrained braces can contribute in increasing the strain hardening level of the system. However, this approach will also lead to an increase in the inelastic demand on the braces and low-cycle fatigue and fracture under cyclic loading can become a problem. In addition, brace strain hardening typically results in larger brace forces that need to be resisted by the rest of the structure. Alternatively, story shear strain hardening response can be obtained through frame action in the structure, by introducing partially restrained or rigid connections between beams and columns. Such dual braced frame-moment frame action has proven to be very efficient in improving the response of CBFs (Hassan and Goel, 1991; Whittaker, Uang, and Bertero, 1990) but this solution also imposes a penalty as beam and column sizes must be increased to resist the moments induced by the frame action.

Another strategy to achieve a stable seismic response is to adopt a bracing configuration capable of distributing more uniformly the inelastic demand over the building height during earthquakes. Figures 8e and 8f illustrate two such configurations: the two-story X (split-X) system and the zipper column system (Khatib et al, 1988). The former aims at developing a 2-story collapse mechanism, as shown in Figure 13 for the same 8-story frame example. The benefits obtained from that system can be minimal, however, depending upon the properties of the bracing members and the flexural capacity of the beams (Tremblay and Lacerte, 2002). This is the case for the sample frame of Figure 13, which became unstable at $\Psi_1 = 0.11$, i.e., the same value as for the buckling restrained braces with zero strain hardening behavior.

In zipper-braced frames, a column is added at the centre of a chevron bracing to force inelastic response in the stories below or above the level where inelastic action has initiated. When a brace buckles, the beam to which it is connected is pulled down by the adjacent tension brace. As shown in Figure 14a, the zipper column then forces a similar response to develop in the stories above (by tension),

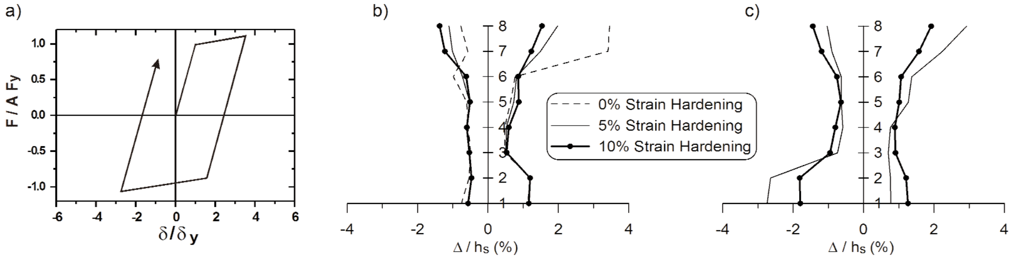

Fig. 12. Response of an 8-story chevron bracing with buckling restrained braces ($t_1 = 1.94$ s) under 1940 El Centro: (a) brace hysteresis; (b) $\Psi_1 = 0.10$; (c) $\Psi_1 = 0.20$.

when brace buckling occurs in the lower floors, or in the stories below (by compression), if inelastic action develops first in the top part of the structure. This system is very efficient in controlling story drifts at the critical floors in the early stage of ground motions. Once all braces in the structure have been forced to buckle, however, the system loses most of its advantages as its lateral strength is reduced, allowing large story deformations to develop. Collapse involving a global mechanism can then occur, as illustrated in Figure 15 for the 8-story building example. In that case, the zipper column was very efficient in forcing all beams to undergo the same vertical mid-span deflection at all floors but the lateral resistance of the building as a whole became too low after buckling of the braces. Stability of that frame can only be improved by increasing the flexural strength of the beams. However, if the beams are made strong enough to carry alone the forces induced by the tension braces after brace buckling, the zipper column becomes much less efficient as the beam must deflect at a given floor to mobilize the capacity of the adjacent floors. For instance, the addition of a zipper column has no effect on the response of the chevron frame with buckling restrained braces discussed earlier because there is no beam deflection in that type of frame.

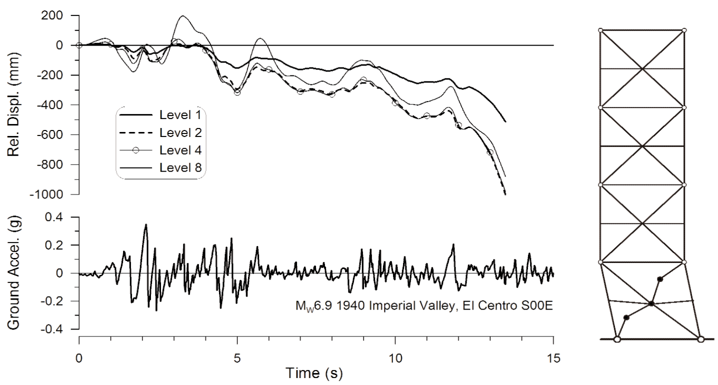

Fig. 13. Two-story collapse mechanism in an 8-story split-X bracing system (t_1 *= 1.82 s,* Ψ_1 *= 0.11).*

Elastic response of the zipper column is essential to develop the full mechanism shown in Figure 15, but the design of that column poses a problem as the column becomes really active only after inelastic response has been triggered in the braces, meaning that the force demand depends on how inelastic response develops along the height of the structure. Realistic estimates of the peak axial forces can only be obtained by performing a series of nonlinear dynamic analyses under different earthquake ground motions to capture the various possible sequences of brace buckling and beam yielding in the structure. Alternatively, manual calculations can be performed by assuming various story yielding scenarios and assessing the force required in the zipper column to initiate brace buckling and beam yielding in adjacent floors (Tremblay and Tirca, 2003). Khatib et al (1988) proposed a method in which the zipper column loads are determined from a statistical summation of the unbalanced load that can develop at every floor. Sabelli (2001) suggests that zipper columns possess the same compression and tension strengths as the braces located at the level below and be detailed with the expectation of inelastic demand. The last two approaches do not ensure that buckling or yielding of the zipper column will be prevented and caution must be exercised if they were adopted in design.

Two other bracing configurations are proposed in Figures 14b and 14c to mitigate soft-story response in CBFs. In both cases, the bracing bents include two vertical trusses: one in which energy will be dissipated through inelastic response of the bracing members and one in which the members are designed to remain essentially elastic. The elastic truss forces the adjacent stories in the frame to undergo similar lateral deformations during inelastic response and, hence, prevents the formation of story mech-

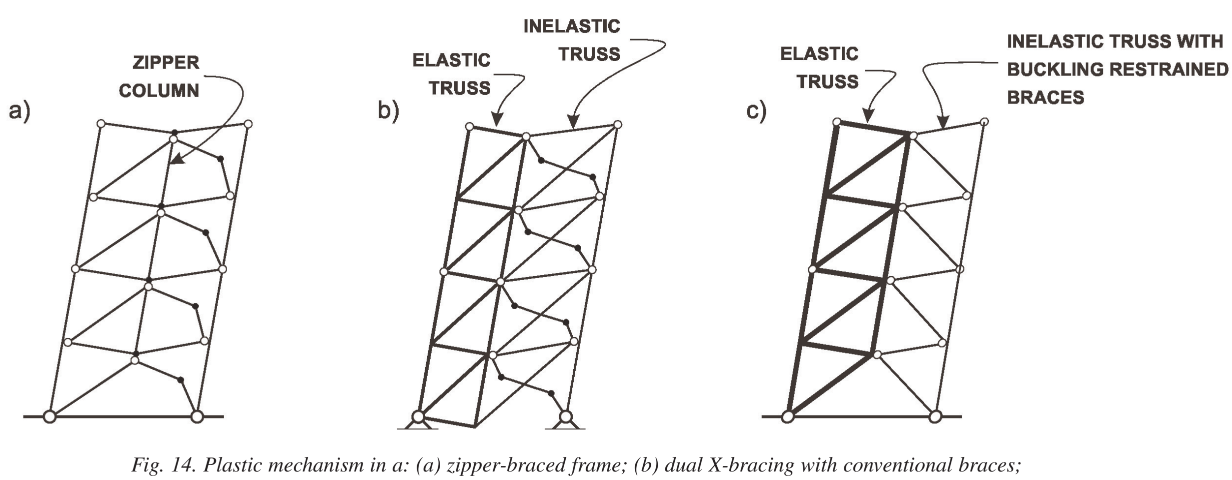

Fig. 14. Plastic mechanism in a: (a) zipper-braced frame; (b) dual X-bracing with conventional braces; (c) dual CBF with buckling restrained braces.

anisms. This mimics the role of the columns in weak beam/strong column steel moment resisting frames or that of the ties or trusses that are added in eccentrically braced steel frames (Ghersi, Neri, Rossi, and Perretti, 2000). In the two dual framing configurations shown in Figure 14, the elastic truss is kept simple with only one diagonal member per floor and one interior vertical member. The latter is part of both the elastic and the inelastic trusses. In the first system (Figure 14b), the inelastic truss is made of conventional steel braces arranged in an X-bracing configuration. A T/C system will generally be the preferred choice because of its higher energy dissipation capability but T/O bracing is also possible (Tremblay, Robert, and Filiatrault, 1997). In the second configuration, buckling restrained braces or other devices exhibiting symmetrical hysteretic response can be used in the inelastic truss.

In Figure 16, the envelope of story drifts and the envelope of axial forces in the elastic truss braces are given for the two systems when used in the 8-story frame example subjected to the El Centro ground motion. For the dual X-bracing system (Figure 16a), a T/C design with brace slenderness *KL/r* varying between 58 and 74 was considered. For the chevron framing with buckling restrained braces, the calculations were performed for two levels of brace strain hardening: 0 percent (Figure 16b) and 5 percent (Figure 16c). The value of Ψ was increased stepwise by scaling up the amplitude of the ground motion. The elastic brace forces in Figure 16 are normalized with respect to the brace forces due to the design seismic lateral loads, F_E, as obtained from elastic analysis.

The comparison of the story drifts in Figures 16a and 9a shows that the concentration of inelastic demand has been eliminated by adding the elastic truss to the X-bracing system. In addition the use of a dual X-braced frame increased the safety against dynamic instability as collapse for that frame only occurred at $\Psi_1 = 0.29$ under El Centro compared to $\Psi_1 = 0.17$ for the frame in Figure 9. Similar improvement is observed for the dual systems with buckling restrained braces when compared to the system described in Figure 12. When elastic-perfectly plastic braces (zero strain hardening) are used, dynamic instability occurs at $\Psi_1 = 0.28$ versus 0.11 if no elastic truss is present. When braces can develop 5 percent strain hardening, the value of Ψ_1 at collapse increases to 0.37 versus 0.32 for the frame in Figure 12. The gain is smaller for the latter case as brace strain hardening had already improved significantly the stability of the frame without the elastic truss.

As for the zipper column in chevron bracing, significant forces in the elastic truss members of a dual system only develop after initiation of inelastic response in the structure. Therefore, nonlinear dynamic analysis is required to assess the design forces. As shown in Figure 16, the demand on the

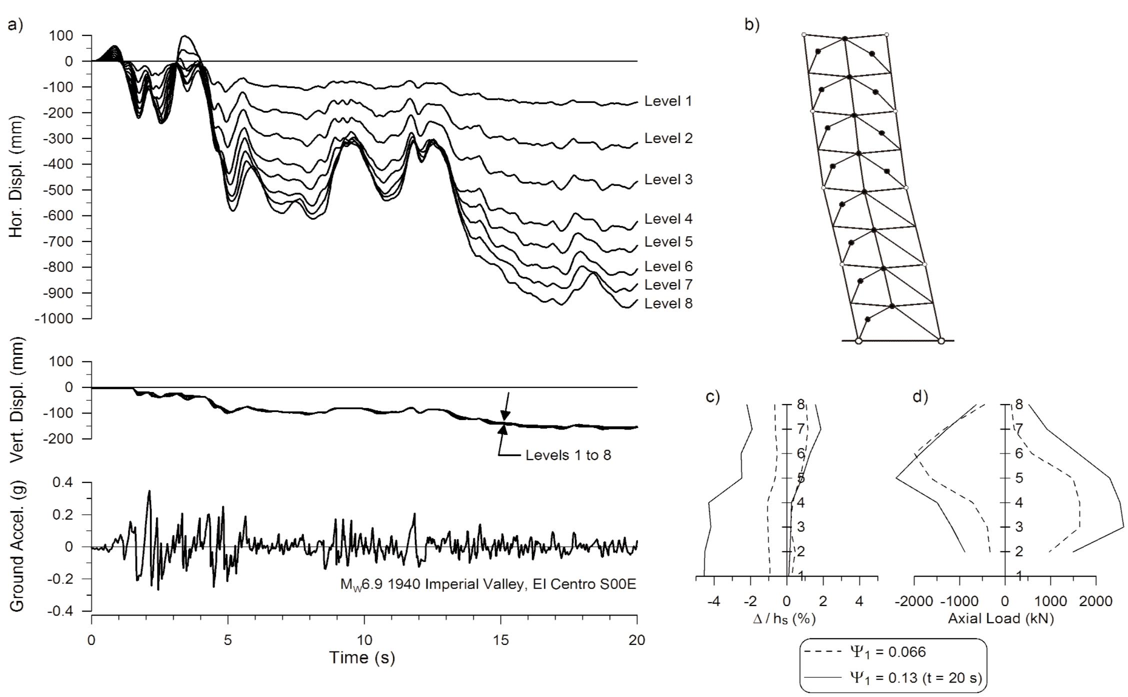

Fig. 15. Response of an 8-story chevron bracing with a zipper column under 1940 El Centro (t_1 *= 2.00 s): (a) time history of the horizontal and vertical displacements at mid-span of the beams for* $\Psi_1 = 0.13$*; (b) deformed shape at* t *= 20 s for* $\Psi_1 = 0.13$*; (c) peak story drifts; (d) peak axial loads in the zipper column.*

elastic trusses is relatively higher near the building base and at the upper floors, which typically correspond to the regions of highest inelastic demand in CBFs (Figure 9). Forces in the elastic trusses also increase with the factor Ψ as the structure is more prone to concentration of inelastic response when the seismic demand to strength ratio or the gravity loading is increased. In design, the elastic truss should be proportioned to guard against dynamic instability with a sufficient margin to cope with uncertainties associated with the assessment of ground motion effects, structural strength, and gravity loads.

Such dual systems would generally involve additional costs compared to a traditional CBF construction but they represent a reliable approach to minimize the potential for dynamic instability in multi-story structures. Optimum inelastic response is also achieved with these systems as the story drift demand becomes uniform over the building height and the entire energy dissipation potential of the structure can be mobilized. In addition, braces are shorter and, therefore, more effective in the dual X-bracing system and the proposed chevron dual braced frame requires only one buckling restrained brace per floor. Furthermore, dual systems provide minimum redundancy in case of defects or malfunction in the system, which is not the case for most conventional CBFs.

CONCLUSIONS

Concentrically braced steel frames are particularly vulnerable to dynamic instability under seismic ground motions compared to other lateral load resisting systems due to the inherent poor hysteretic response of the diagonal bracing members and the tendency to develop story mechanisms in multi-story applications. The stability of CBFs responding in the nonlinear range to earthquake ground motions is a complex problem that depends on several parameters including the properties of the bracing members, the brac-

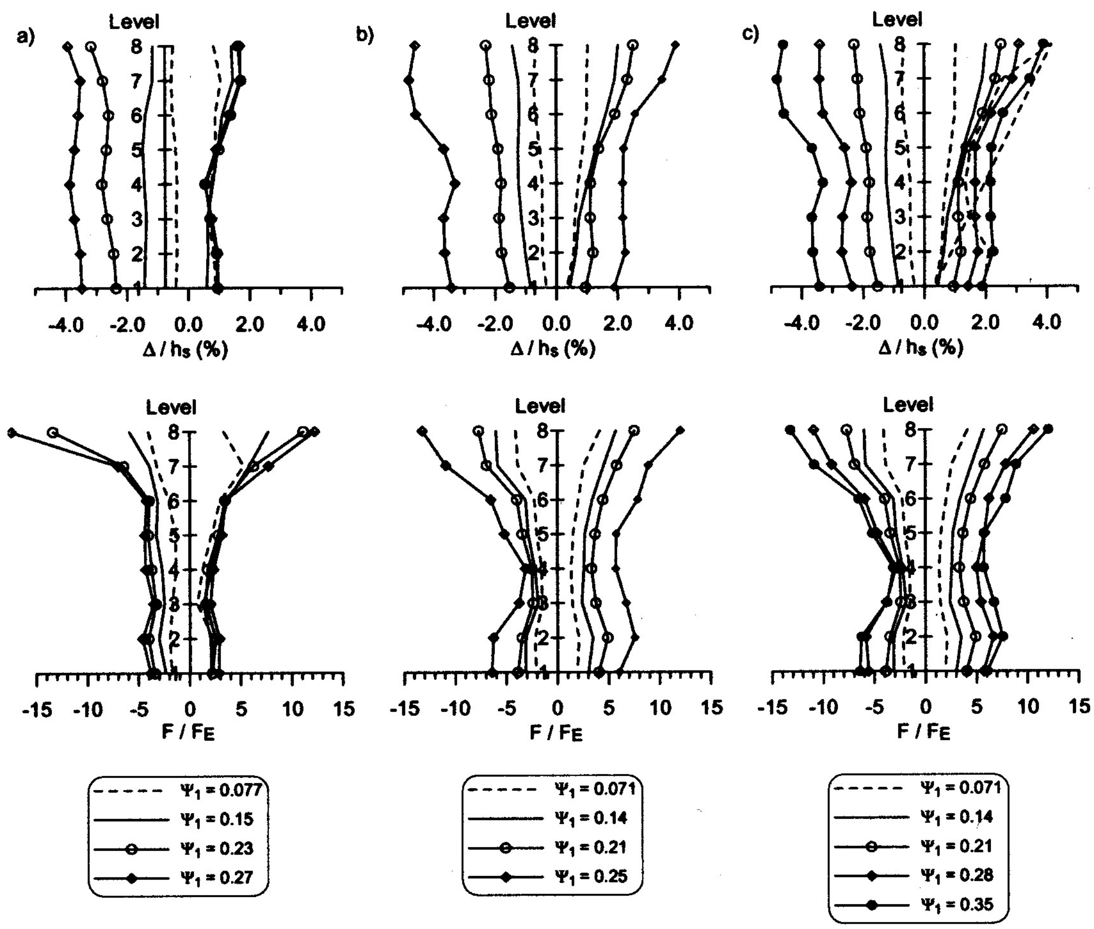

Fig. 16. Envelopes of peak story drifts, D, and axial load in elastic braces, F, in 8-story dual bracing systems under 1940 El Centro Earthquake: (a) dual X-bracing (t_1 = 1.69 s); (b) dual chevron bracing with buckling restrained braces exhibiting no strain hardening (t_1 = 1.82 s); (c) dual chevron bracing with buckling restrained braces exhibiting 5 percent strain hardening (t_1 = 1.82 s).

ing configuration, the characteristics of the ground motions, the lateral resistance of the structure relative to the seismic demand and the amount of gravity loads supported by the structure. In multi-story frames, the potential for collapse by instability is also influenced by other parameters including the number of floors and the distribution of the lateral resistance over the building height. As of today, no simple method has been proposed for design purposes to adequately capture the combined effects of all these parameters.

Past studies have shown, however, that stable CBF seismic response can be anticipated if structural characteristics such as the lateral resistance, the number of floors, the amount of gravity loads, etc. are kept within limits that are dependent upon the bracing configuration used. This approach has been adopted in the latest edition of the CSA S16-01, the standard for the design of steel structures in Canada. For other CBF applications, the design engineers must demonstrate that the proposed system can exhibit a stable behavior. Nonlinear dynamic analytical studies can be performed to evaluate the potential for dynamic instability for such structures. The engineers must then pay great attention to the modelling of the structure and to the selection and scaling of the ground motions. A successful CBF design will require the choice of a proper bracing system and various possibilities involving different bracing arrangements as well as bracing member types have been proposed and discussed in the paper. For the most critical situations, dual bracing systems that include an elastic truss appear to be a very promising solution. Further studies are still needed, however, to develop specific design guidelines for these systems and to propose other innovative solutions that will permit a safe and efficient use of braced steel frames for multi-story structures located in active seismic zones.

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