Capacity Design Principles for the Ductile Behaviour of Conventional and High-Performance Steel Structures under Earthquake Shaking

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Summary

Steel lateral load-resisting systems offer a variety of design solutions to ensure a ductile behaviour of a structure during a seismic event, thereby achieving a relatively low earthquake-induced collapse risk, if designed and detailed properly according to current seismic design provisions. At the same time, latest developments in steel manufacturing and construction techniques can provide innovative solutions aiming to minimize life-cycle costs of steel buildings due to repairs in the aftermath of earthquakes. Structural steel systems also offer realistic solutions for the development of demountable buildings, where, structural members can potentially be re-used after the end of a building's life. This can be typically achieved by employing selected fuses (termed dissipative elements) to dissipate the seismic action, while maintaining the majority of the structural system damage-free; thus, member re-use can be promoted in an effort to meet environmental sustainability requirements and resilience challenges in urban areas of moderate to high seismicity.

This paper discusses key principles for the seismic design of conventional steel structural systems that provide adequate ductility during a seismic event to meet the life safety requirements. Moreover, seismic design concepts are presented for selected low-damage structural steel systems that potentially limit structural damage in replaceable dissipative elements. A brief discussion on innovative structural steel systems is also presented.

Conventional Structural Steel Systems

Conventional structural steel lateral load-resisting systems comprise steel moment resisting frames (MRFs) and steel frames with bracing members often termed as concentrically braced frames (CBFs). During a seismic event, both systems dissipate the seismic action through the inelastic response of pre-selected dissipative elements. This necessitates the consideration of local and global capacity design principles to ensure the desired ductility during earthquake shaking. Depending on the construction site and the country-specific design codes, such principles may vary. The subsequent sections attempt to provide a detailed discussion and overview of key seismic design principles of steel MRFs and CBFs including pertinent examples. Practical rules of "good" engineering practice are also presented for key structural elements that are likely to be adopted in prospective seismic design provisions worldwide.

Steel Moment Resisting Frame Systems

Steel MRF systems dissipate the anticipated seismic action through inelastic flexural yielding of their steel beam ends as part of full- or partial strength beam-to-column moment connections. This is illustrated schematically in Figure 1a, which shows a full-beam collapse mechanims. Referring to Figure 1b, dissipative action may be observed in the beam-to-column web panel through shear yielding. While web panels have large dissipative capacity, their participation into the energy dissipation should be balanced with that of the steel beams within a beam-to-column steel joint (Krawinkler et al. 1975). Due to the steel MRF deformation kinematics (i.e., sidesway movement) under lateral load, another location of anticipated inelastic deformations is the first storey steel column base as shown in Figure 1. On the other

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hand, steel MRF systems, which are designed without any capacity design considerations, are prone to soft-storey collapse mechanisms as shown schematically in Figure 1c.

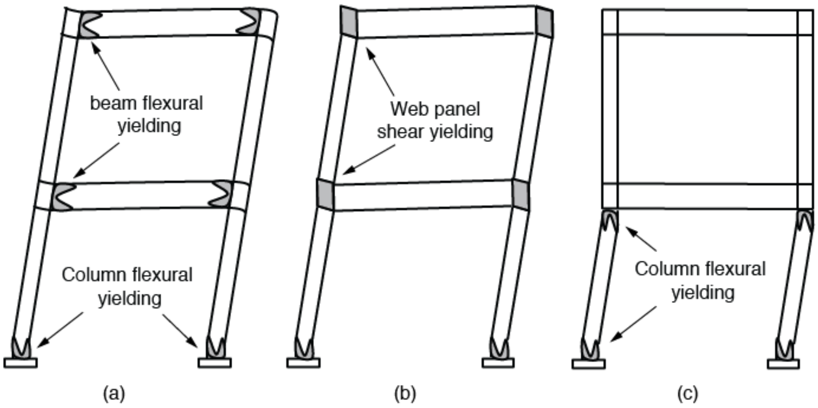


Figure 1. Steel MRF system under lateral load – Deformation kinematics and potential collapse mechanisms

Observations from past earthquakes (Nakashima et al. 1998; Mahin 1998; Okazaki et al. 2013) reveal that steel MRF systems designed prior to 1994 experienced unanticipated brittle failures in their beam-to-column connections. Figure 2 shows illustrative examples of such failure modes. These involved (a) fractures due to inadequate weld and base metal toughness requirements; (b) inadequate connection detailing due to stress concentrations arising from the weld access hole geometry; (c) notch effects since weld backing bars were not removed after the completion of complete joint penetration (CJP) welds; and (d) excessive panel web yielding causing fractures due to immoderate panel zone kinking at the bottom flange of steel beams.

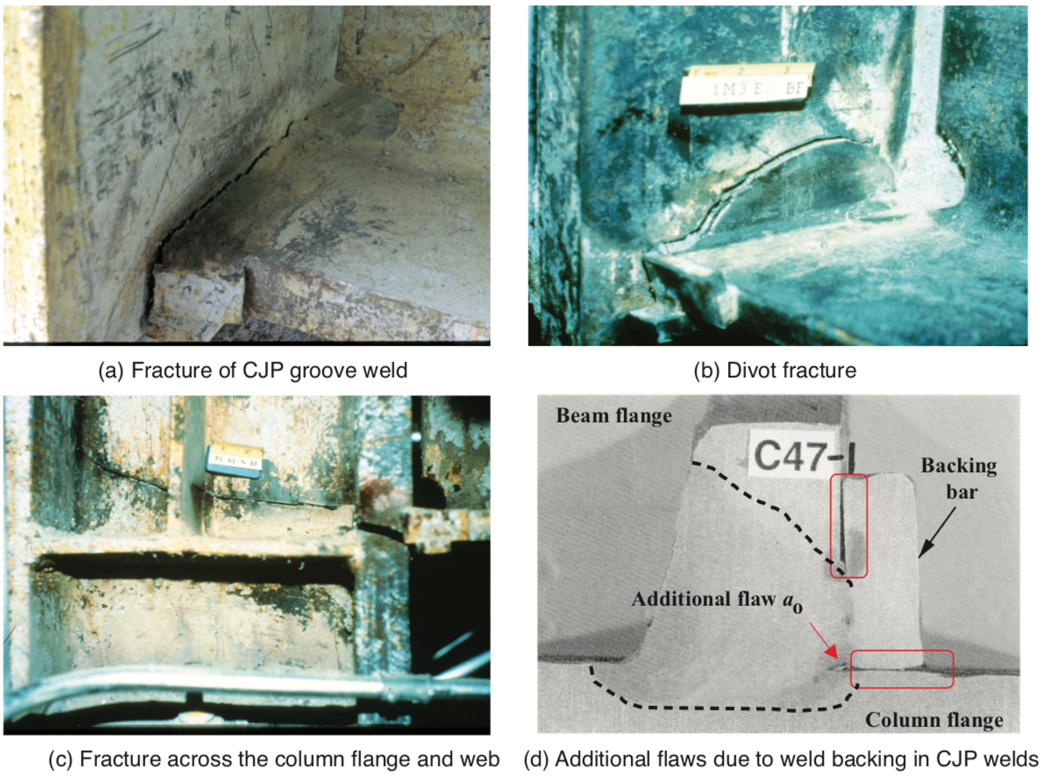
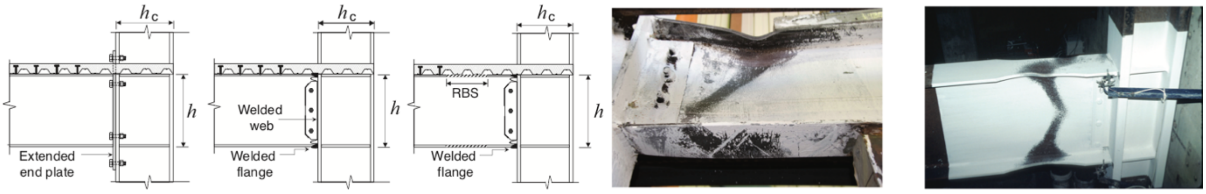


Figure 2. Brittle fracture failure modes in typical pre-Northridge beam-to-column connections (images “a” to “c” adopted from FEMA 2000; image “d” photo credit to Prof. Dr. J. Fisher, adopted from Paret 2000)

In response to the above problems, the steel industry in North America and Japan developed welded and bolted pre-qualified beam-to-column connections that alleviated most of the above challenges. The research was based on full-scale experimental testing, which was conducted as part of the SAC[2] Joint Venture program. These connections, including their steel fabrication details and requirements, are documented in AISC-358-16 (AISC 2016a). More recently, a similar effort was conducted in Europe as part of the EQUALJOINTS project (D'Aniello et al. 2018; Landolfo et al. 2018). The findings from this project will form the basis of European connection pre-qualification in steel MRF systems within the context of the next revision of Eurocode 8. Figures 3a to 3c shows a broad range of such connection configurations including typical end-plate bolted connections (see Figure 3a), welded unreinforced flange welded web (WUF-W) connections (see Figure 3b), as well as reduced beam section (RBS) connections (see Figure 3c). The latter involves cross-section weakening to concentrate the anticipated inelastic deformations away from the column face of a beam-to-column joint. Provided that the above connections comprise fracture-critical CJP welds, the inelastic action, during an earthquake, is concentrated on the beam end as shown in Figures 3d and 3f. The extent of nonlinear geometric instabilities may be controlled by the corresponding cross-section classification.



(a) Extended end-plate connection (b) Welded unreinforced flange welded web connection (c) Reduced beam section (RBS) (d) Damage pattern in WUF-W connection (f) Damage pattern in RBS connection

Figure 3. Typical full strength beam-to-column connection typologies and their damage patterns (images "a" to "c" adopted from El Jisr et al. 2019; image "d" adopted from Lignos et al. 2014; image "f" adopted from FEMA 2000)

While the North American and Japanese seismic design and fabrication standards have established rigorous specifications for the use of welded pre-qualified beam-to-column connections in seismic applications, the European standards are still evolving. In particular, welded beam-to-column connections require the use of fracture-critical CJP welds with minimum toughness requirements (27Joules at -30^{o} Celsius and 54Joules at 21^{o} Celsius) in accordance with AISC-358-16 (AISC 2016a). Special care should be put in weld access holes to avoid stress concentrations based on pertinent research, which was conducted right after the 1994 Northridge and 1995 Kobe earthquakes (e.g., Ricles et al. 2002, 2003; Zhang and Ricles 2006). Moreover, the removal of weld backing bars is recommended for weld inspection and reinforcement by means of fillet welds, if necessary, to prevent the propagation of potential weld flaws. These are typically traced by non-destructive testing.

In areas of high seismicity in Europe, most of the above fabrication requirements may be satisfied through the use of Execution Class 3 (EXC3) welds with quality level B for the acceptance criteria in accordance with (EN ISO 5817 2014; EN 1090-2 2018), respectively. In regions of low to moderate seismicity the use of modest weld specifications and acceptance criteria along with simple steel fabrication practices may suffice to ensure local ductility requirements in steel MRFs in the aftermath of earthquakes. However, this has not been contextualized within a European norm yet. In North America the above practice is followed in contemporary steel MRF designs in areas of low to moderate seismicity. The above

[2] Joint venture between Structural Engineers Association of California, Applied Technology Council, Consortium of Universities for Research in Earthquake Engineering

hypothesis for European regions of low to moderate seismicity requires further verification by means of experimental testing of representative connection typologies fabricated with lower quality welds with pre-qualified cyclic testing protocols (Krawinkler 1996; Clark et al. 1997).

Modern seismic codes (AISC 2016b; c) employ the strong-column/weak-beam (SCWB) check in an effort to control the potential formation of soft-storey collapse mechanisms in steel MRFs under lateral loading (see Figure 1c). It is worth noting that the SCWB check is employed as part of the design process when a certain value of the behaviour factor, q is assumed in the steel MRF design. According to CEN (2005a) when q > 1,5 is considered, the SCWB is mandatory. On the other hand, in North America (CSA S16 2014; AISC 2016b; c), the SCWB check is only considered in areas of high seismicity, in which the use of special moment frames is compulsory. The term "special" in this case implies special fabrication and design requirements to ensure the required ductility needed to dissipate the anticipated seismic action. Special moment frames (or Type D in Canada) are designed using relatively high behaviour factor values (ASCE 2016). According to the North American design provisions (AISC 2016c; b), the SCWB check is not mandatory for ordinary and intermediate steel MRFs, which are designed with smaller behaviour factors than those used in special moment frames. However, height limitations are placed in the use of ordinary and intermediate steel MRFs (CSA S16 2014; ASCE 2016) to avoid nonductile behaviour of these systems under lateral load.

Equation (1) indicates the SCWB check according to the current CEN (2005a) for steel MRF systems with two or more storeys,

$$\sum M_{Rd,c} \geq 1{,}3 \sum M_{Rd,b} \qquad (1)$$

Referring to Figure 4, the SCWB check should be performed at a beam-to-column joint intersection; $\sum M_{Rd,c}$ is the sum of the design values of the moments of resistance of the columns framing the joint reduced by the effects of compressive axial load demands; $\sum M_{Rd,b}$ is the sum of the design values of the moments of resistance of the beams framing the same joint. The additional flexural demands due to the shear demand, V_{Ed} (due to gravity and seismic loading), of the respective beams should be considered to conduct the SCWB check. The shear demand should be transferred to the joint centerline depending on the selected connection type. Referring to Figure 4, an illustrative example is shown for a bolted end-plate beam-to-column connection. In particular, the corresponding $\sum M_{Rd,b}$ should be calculated by assuming that a plastic hinge develops at a distance s_h away from the column face,

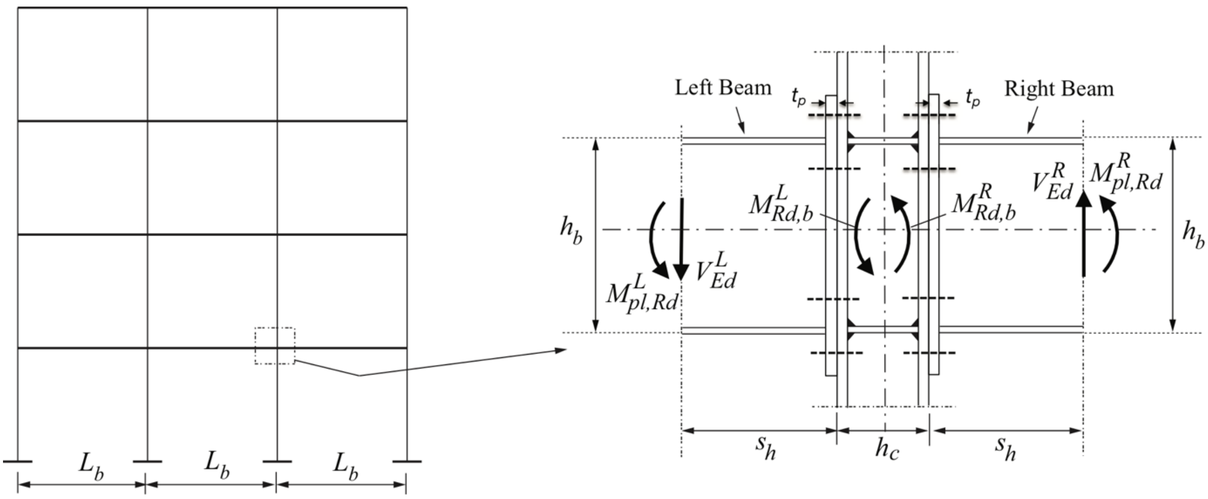


Figure 4. Strong-column/weak-beam ratio check in steel MRFs; illustration for bolted end-plate beam-to-column connections

Left beam,

$$M_{Rd,b}^{L} = M_{pl,Rd}^{L} + (s_h + h_c/2) \cdot (V_G + V_E^{L}) \tag{2}$$

Right beam,

$$M_{Rd,b}^{R} = M_{pl,Rd}^{R} + (s_h + h_c/2) \cdot (V_G + V_E^{R}) \tag{3}$$

where, V_G is the shear force due to gravity loading; V_E^L is the shear force due to seismic loading; s_h is the distance between the column face to the centre of the dissipative zone of the beam end. For bolted end-plate connections it is often assumed that $s_h = \min\{h_b/2, 3b_f\}$; b_f is the beam flange width; and h_c is the column depth,

$$M_{pl,Rd}^{L/R} = 1{,}1 \cdot \gamma_{ov} \cdot W_{pl,y}^{L/R} \cdot f_y \tag{4}$$

and

$$V_E^{L/R} = 2 \cdot M_{pl,Rd}^{L/R}/L \tag{5}$$

where, L is the length between the centre of two dissipative zones. Referring to Figure 4, the corresponding length, $L = L_b - 2 \cdot (s_h + h_c/2)$; γ_{ov} is the steel material overstrength. Historically, γ_{ov}=1,25 regardless of the steel material type. However, recent work within Europe (Braconi et al. 2013) suggests different values depending on the respective steel material type. This is more consistent with the AISC 341-16 (AISC 2016b) provisions with regards to the equivalent R_y values tied to specific steel material types. A comprehensive discussion regarding this matter can be found in El Jisr et al. (2019).

In composite steel MRFs, the $M_{pl,Rd}^{L/R}$ should be calculated by considering the sagging and hogging bending resistance of the composite beam according to the European (CEN 2005b) and Swiss SIA 264 (SIA 2014) norms. This is particularly important in steel MRF designs comprising steel beams with depths less than 450mm. For this range of beam depths, the presence of the slab can increase by more than 40% the sagging bending resistance of a composite steel beam relative to that of the equivalent bare steel cross-section (e.g., Nakashima et al. 2007). This may also depend on the degree of composite action as well as the respective slab details (El Jisr et al. 2019).

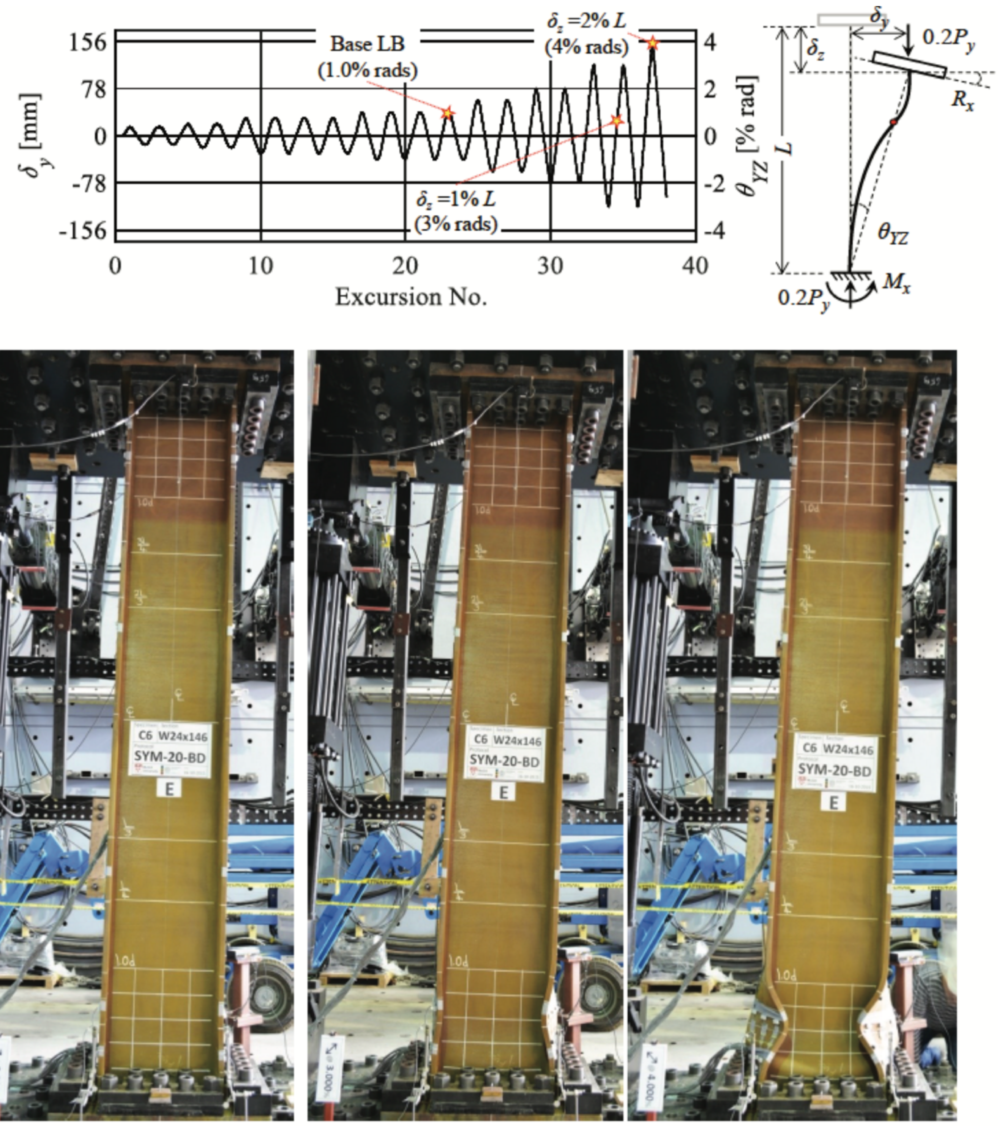
Research with emphasis on the collapse risk quantification of steel MRFs under seismic loading (Ibarra and Krawinkler 2005; Lignos et al. 2011, 2013; Nakashima et al. 2013; Elkady and Lignos 2014, 2015; Tsitos et al. 2018; Bravo-Haro et al. 2018) has shown that in highly seismic regions, the SCWB check may have to be raised to at least 2,0 to ensure the prevention of the soft storey collapse mechanisms in steel MRFs. Figure 5a shows a full-scale test of a 4-storey steel MRF building conducted at the world's largest shake table at E-Defense. This steel MRF was designed with a SCWB > 1,5 according to the Japanese seismic design practice (BCJ 2011). During the design basis event (10% probability of exceedance over 50 years of building life expectancy), the plastic hinge sequence mostly involved the inelastic behaviour of steel beams and the first storey column bases as intended. However, when the steel MRF was subjected to a low-probability of occurance seismic event, a soft storey formed as shown in Figure 5a. While the SCWB ratio was respected, the observed collapse mechanism shifted to a soft storey due to several reasons, including (a) the redistribution of forces and material hardening once inelastic behaviour initiated in the steel beams; (b) the inherent flexibility of the column bases despite the fact that they were designed to be ideally fixed (Zareian and Kanvinde 2013); (c) the role of the composite action that was not considered in the SCWB check; and (d) the associated steel material variability (Lignos et al. 2013). Recent studies (Inamasu et al. 2017, 2019) focus on the ductility of steel columns interacting with concrete footings to further comprehend the role of column bases on the seismic design and behaviour of steel MRFs. Referring to Figure 5b, in more recent work, the 18-storey steel MRF building, which was also tested at the E-Defense facility in Japan (Nakashima et al. 2013), collapsed with a formation of a collapse mechanism involving the first few steel MRF stories. This confirms earlier findings regarding the seismic behaviour and global stability of both steel (Gupta and Krawinkler 2000a; Elkady and Lignos 2014) and concrete MRFs (FEMA 2009) designed in highly seismic regions.

(a) (b)

Figure 5. Collapse tests of steel MRF buildings under ground motion shaking – bottom storey collapse mechanisms (image "a" is adopted from Lignos et al. 2013; image "b" is adopted from Nakashima et al. 2013)

While the seismic design practice of steel MRF systems with full- and partial-strength beam-to-column connections is fairly well established, design provisions regarding the steel MRF column stability and ductility are still evolving. Recent experimental work (Suzuki and Lignos 2015; Ozkula et al. 2017; Elkady and Lignos 2018a; b) and corroborating finite element analyses (Elkady and Lignos 2015, 2018b; Fogarty et al. 2017) characterized the hysteretic behaviour of steel MRF columns under multi-axis cyclic loading. The findings demonstrate that depending on the column member slenderness ratio, L_b/i_z (L_b unbraced length of the column; i_z is the ratio of gyration of the column cross section), as well as the column cross-section web slenderness ratio, h_1/t_w (h_1 and t_w are the web height and thickness, respectively, of the column cross-section) first storey steel MRF columns may exhibit coupled local and/or member nonlinear geometric instabilities associated with local and lateral torsional buckling. These could potentially compromise the collapse resistance of steel MRFs under strong ground motion shaking.

Figure 6 shows an I-shape steel column subjected to coupled compressive axial load, *N* and cyclic-symmetric lateral loading. The results suggest that the steel column exhibits appreciable axial shortening, δ_z (Elkady and Lignos 2018a). The onset and progression of cross-sectional local buckling may be controlled with the cross-section classification, depending on the selected behaviour factor, q, for ductile design (CEN 2005a; SIA 2013). However, it is less obvious how a designer may control the amount of column axial shortening depending on the selected q-factor during the design process. Column axial shortening primarily depends on the web local slenderness, h_1/t_w, the imposed compressive axial load, N, and the imposed lateral loading history (MacRae et al. 1990; Elkady and Lignos 2018a). Elkady and Lignos (2018b) proposed a practical rule to control column axial shortening and subsequently the steel MRF column ductility. In particular, they proposed to limit $h_1/t_w \leq 37$ for a maximum gravity axial load demand, $N_G = 0{,}30 \cdot N_{pl,Rd}$. Moreover, the member slenderness ratio L_b/i_z should be restricted to 80 for steel MRFs designed in highly seismic regions. The above limits along with other related proposals (e.g., Wu et al. 2018) are expected to form the basis for the development of coherent seismic design rules of steel MRFs in areas of moderate and high seismicity in Europe and worldwide. Notably, the Canadian specifications (CSA S16 2014) have already integrated some of the above limits for the seismic design of Type D (Ductile) steel MRFs.



(a) 1st cycle, 2% drift amplitude

(b) 1st cycle, 3% drift amplitude

(c) 1st cycle, 4% drift amplitude

Figure 6. Damage progression of steel MRF columns under cyclic loading (image adopted from Elkady and Lignos 2018a)

Capacity design principles for steel MRFs involve the use of continuous steel columns spliced every two stories at the mid-height of a storey (point of zero moment). The stabilizing effects of gravity columns and conventional shear tab connections spanning these columns have been well documented in prior-related studies associated with the global stability of steel MRFs under seismic loading (Gupta and Krawinkler 2000a; Flores et al. 2014; Elkady and Lignos 2015; Del Carpio et al. 2019). Although the gravity system comprising columns and shear tab connections are only designed to resist the gravity loads of a structure, they have an appreciable lateral stiffness that typically assists to better distribute the storey shear force demand on the primary lateral load-resisting system (Gupta and Krawinkler 2000b; Elkady and Lignos 2015). As a first approximation, the elastic stiffness of the gravity load system (as a percentage of the elastic stiffness of the brace frame) should exceed the stability coefficient value (Gupta and Krawinkler 2000b). Others propose the combination of steel MRFs with a rocking wall system for the same purpose (Makris and Aghagholizadeh 2017; Aghagholizadeh and Makris 2018).

Column splices in capacity-designed steel MRFs feature fracture-critical CJP groove welds to develop their flexural strength of the column in *lieu* of field observations from past earthquakes (e.g., Kobe 1995) as well as experimental evidence on low toughness in the base and/or weld material of the splice itself (Bruneau and Mahin 1990). Partial joint penetration welds with at least 60% degree of penetration may also be permitted provided that minimum toughness-rated weld electrodes are used (Shaw et al. 2015). These feature a minimum Charpy V Notch energy of 27J at -18^{o}C and additionally, a CVN energy of 54J at 21^{o}C in accordance with AISC 341-16 (AISC 2016b). Moreover, beam-to-column web panels should be designed with a balanced design procedure to control the inelastic deformations between the steel beam, which acts are the primary dissipative element, and the web panel (Krawinkler et al. 1975; Krawinkler and Mohasseb 1987).

It should also be stressed that the non-dissipative structural components within a steel MRF should be designed for the amplified seismic action due to system overstrength, Ω. This is attributed to a variety of sources such as, the difference between the design force demand and theoretical member resistance; the effects of gravity loads on the member resistances; the material hardening and redistribution of internal forces when a structure exhibits inelastic behaviour; the discrete choices of member sizes as well as the member overstrength due to stiffness requirements (drift-controlled versus force-controlled steel MRF designs). It is common for steel MRFs above 4-stories to be drift-controlled, thereby exhibiting appreciable system overstrength. Past earthquakes have demonstrated that global collapse of steel structures may occur if overstrength is not appropriately considered in the design actions of non-dissipative members of steel MRFs, such as columns and foundations (Osteraas and Krawinkler 1989). The system overstrength, Ω can be directly measured by means of conventional nonlinear static analysis (pushover) with a predefined lateral load pattern (Krawinkler and Seneviratna 1998; Lignos et al. 2015). Typical values for contemporary steel MRF systems designed in highly seismic regions can be found in NIST (2009) and Elkady and Lignos (2015).

Steel Braced Frame Systems

Steel CBF systems dissipate the anticipated seismic action through inelastic tensile yield and post-buckling deformation of steel braces. These are acting as the primary dissipative elements within a CBF. This is shown in Figures 7a and 7b. Referring to Figure 7c, steel braces exhibit a highly asymmetric hysteretic behaviour under tensile and compressive loading excursions. Once they reach their flexural buckling resistance in compression they deteriorate in axial strength and they typically attain a residual plateau of about $0{,}3 \cdot N_{pl,Rd}$. This value is often used in the design of non-dissipative elements of CBFs to accommodate the unbalanced loads arising from the inelastic behaviour of bracing members during an earthquake. Past studies indicate that the above value depends on the steel brace geometric characteristics among other parameters (Tremblay 2002).

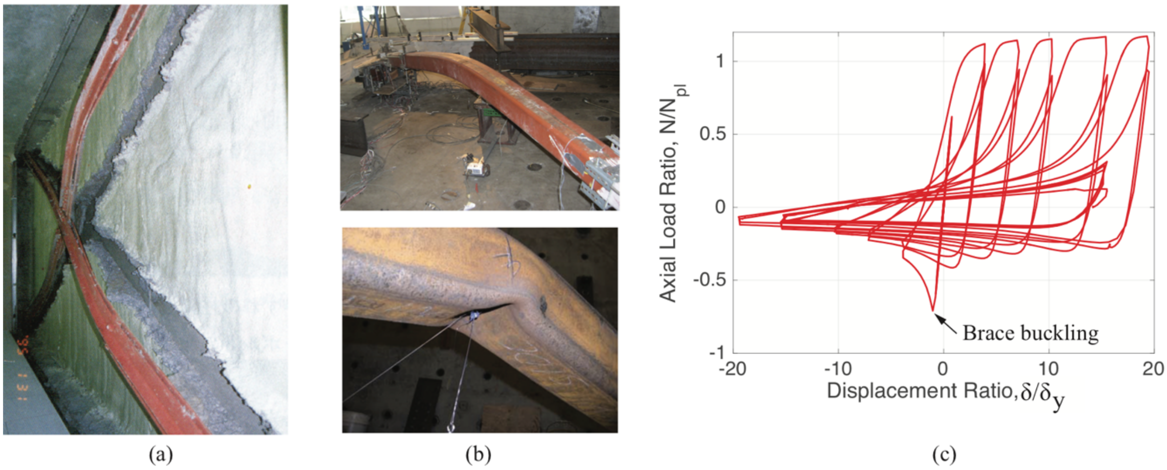


Figure 7. Anticipated failure modes in steel braces to ensure ductile behaviour of steel braced frames under cyclic loading (image “a” courtesy of Prof. Dr. M. Nakashima; image “b” adopted from Lehman et al. 2008; image “c” experimental data adopted from Tremblay et al. 2008)

Vis-à-vis the above discussion, it is practically not attainable to achieve a uniform distribution of demand-to-capacity ratios along the height of a steel CBF. Thus, inelastic storey drift demands often tend to concentrate into individual storeys once brace buckling occurs, thereby increasing the earthquake-induced collapse risk of steel CBF buildings (Karamanci and Lignos 2014; Hwang and Lignos 2017b). This is one reason why steel CBFs are designed with smaller behaviour factors, q, than those used in steel MRFs regardless of the employed seismic provision around the world. Table 1 summarizes for reference the assumed behaviour factors, q, per lateral load-resisting system according to SIA 263 (SIA 2013). The cross-section classification requirements are also summarized on the same table depending on the selected q-factor.

Table 1. Behaviour factor, q, for ductile design of steel lateral load resisting systems - values adopted from SIA 263 (SIA 2013)

Type of lateral load resisting system	Cross-section classification		
	Class 1	Class 2	Class 3
Steel MRFs	$q = 5$	$q = 4$	$q = 2$
Steel frames with X-bracings	$q = 4$	$q = 4$	$q = 2$
Steel frames with V-bracings	$q = 2{,}5$	$q = 2{,}5$	$q = 2$

Steel CBFs feature a broad range of structural configurations including X- chevron V-, inverted V- or multi-storey X-braced configurations. Steel CBFs typically comprise angles (L- or U-shape), square hollow sections (RRK according to SZS 2005), round hollow sections (ROR according to SZS 2005) as well as I-shape cross-sections. Angles are preferred in low-rise steel CBFs due to easiness in connection detailing and maintenance. Ordinary steel CBFs according to the American provisions (AISC 2016b) are often designed as tension-only systems. Therefore, the lateral stiffness and strength of the compressive brace is disregarded. Although this assumption may be rational in regions of high seismicity, it often creates appreciable overstrength in steel frames with bracings (Elghazouli 2010).

Recent work (Kanyilmaz 2017) suggests that exploiting the post-buckling resistance of compressive diagonals may increase the economic efficiency of frames with X-bracings designed in low to moderate seismicity regions. However, a hierarchy of failure modes should be established to eliminate connection out-of-plane bending/buckling. These deformations could ultimately lead to unanticipated bracing connection failure modes (Davaran et al. 2015). Figure 8a shows a typical bolted connection at brace intersection in X-bracing configurations with a single shear lap splice. This is a common structural detail in steel CBFs designed in regions of low to moderate seismicity. The connection eccentricity and the associated bending stiffness of the connecting plates at the mid- and end- bracing connections strongly influences the compression capacity of the bracing members. In particular, plate bending may occur as shown in Figure 8b. In turn, this could cause connection fracture as shown in Figure 8c. Connection fracture is common due to the formation of a double plastic hinge mechanism that forms on the respective steel plate as shown in Figure 8c. This structural deficiency is consistent with field observations from past earthquakes (Okazaki et al. 2013).

(a) (b) (c)

Figure 8. Bracing connection bending and fracture (images adopted from Davaran et al. 2015).

In tension-compression steel CBF systems, rectangular or square hollow sections are preferred since they are stored relatively easily on site compared to ROR. However, ROR and I-shape braces are inherently more resilient to ultra-low cycle fatigue than HSS braces. The latter exhibit crimping at the cross-section corners that greatly amplifies the local plastic strain demand and ultimately leads to fracture (Fell et al. 2009).

While for steel frames with bracings up to two stories, there is no limit on the normalized brace slenderness (CEN 2005a; SIA 2013),

$$\overline{\lambda_k} = \sqrt{A_g \cdot f_y / N_{cr}} \tag{6}$$

in all other cases, $\overline{\lambda_k} \leq 2{,}0$, in order to avoid elastic flexural buckling of the brace diagonals (CEN 2005a; SIA 2013). In Equation (6), A_g and f_y are the gross area and yield stress, respectively, of the bracing member; $N_{cr} = \pi^2 EI/(l_k^2)$ is the Euler load of the bracing member; E is the modulus of elasticity of the steel material; I is the moment of inertia of the cross-section with respect to the axis that flexural buckling may occur. Figure 9 provides illustrative examples on how to calculate the characteristic length, l_k of a bracing member depending on its end restraints. Often times, the centerline length, L is used for simplicity in the calculations.

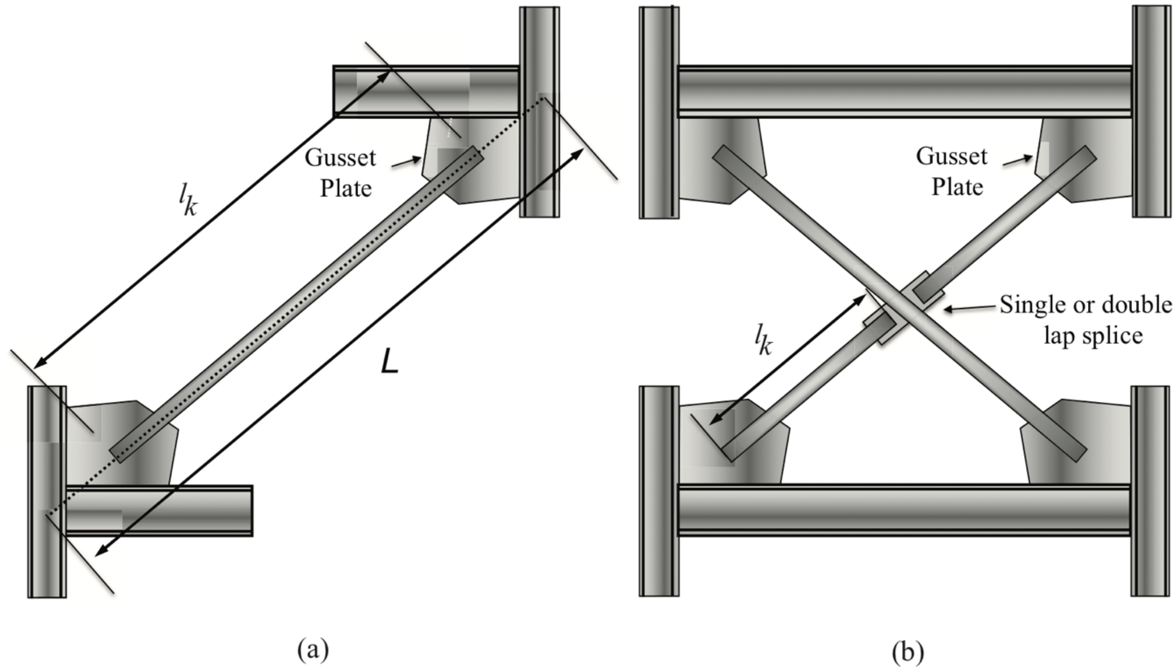


Figure 9. Characteristic length for flexural buckling resistance calculations

A lower limit on brace slenderness (i.e., $\overline{\lambda_k} \geq 1{,}3$) is often used in steel frames with X-bracings, in an effort to guarantee a minimum ductility of the bracing element (Tremblay 2002; CEN 2005a).

Surveys on steel frames with bracings from past earthquakes (Tremblay et al. 1996; Nakashima et al. 1998; Okazaki et al. 2013) demonstrate that the ductile behaviour of steel CBFs is ensured when bracing connections and other non-dissipative members are properly designed to avoid premature failure modes. Referring to Figure 10, common structural deficiencies result into net section fracture (see Figure 10a and b) as well as block shear (see Figure 10b).

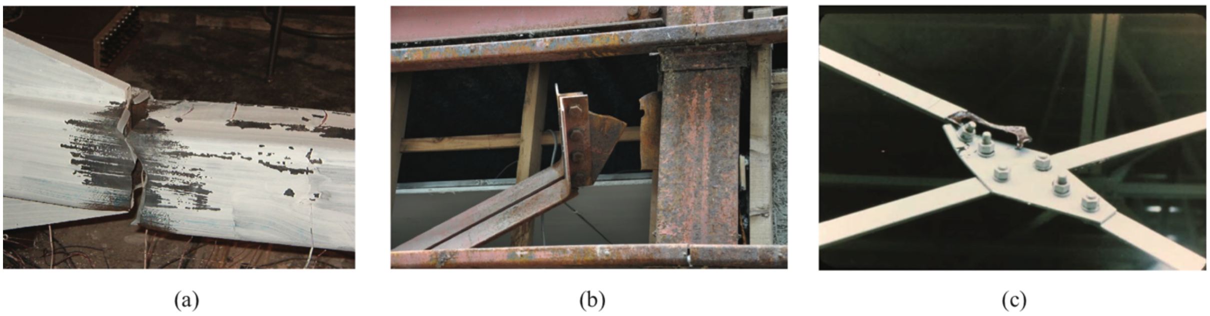
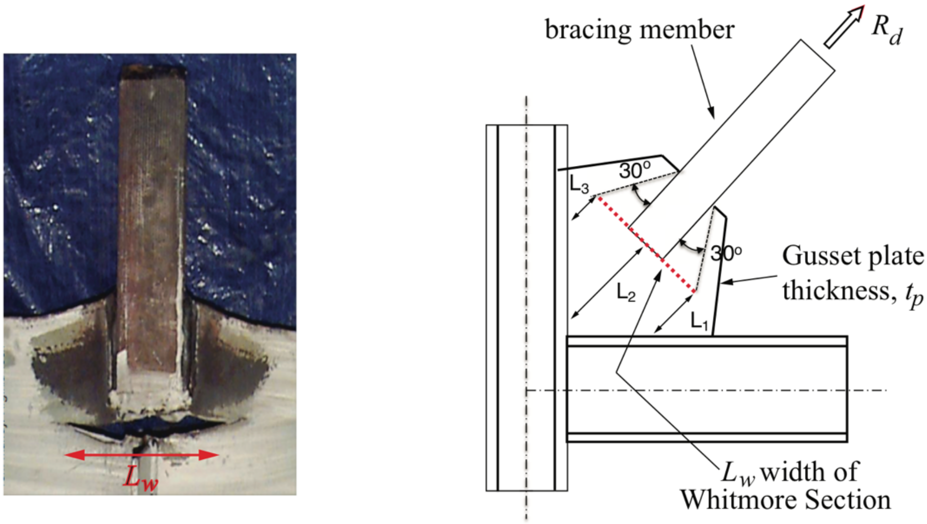


Figure 10. Net section fracture and block shear in typical bracing connections (image “a” adopted from Yang and Mahin 2005; image “b” courtesy of Prof. Dr. D. Lignos; image “c” courtesy of Prof. Dr. R. Tremblay)

For this purpose, bracing connections in CBFs designed in highly seismic regions, involves a local capacity design check. In particular, bracing connections should be typically designed for a tensile resistance,

$$R_d \geq 1.1 \cdot \gamma_{ov} \cdot N_{pl,Rd} \tag{7}$$

Where, $N_{pl,Rd}$ is the tensile plastic resistance of the bracing member. A common practice in North America (AISC 2016b) to ensure ductile behaviour of gusset plate connections is to size them according to the width, L_w of the Whitmore section (Whitmore 1952) that forms over a 30° angle with respect to the bracing member centerline as shown in Figure 11.



(a) (b)

Figure 11. Whitmore section for gusset plate design to resist tensile and compressive axial forces (image a courtesy of Prof. Dr. T. Murray)

The tensile resistance of the gusset plate should be calculated as follows,

$$N_{t,Rd} = \frac{1}{\gamma_{M2}} \cdot \left[f_y \cdot \left(L_w \cdot t_p\right)\right] \geq R_d \tag{8}$$

Where, t_p is the gusset plate thickness. The gusset plate buckling resistance, $N_{b,Gusset}$, should also be verified for plate buckling according to an equivalent buckling length, L_b. In particular,

$$N_{b,Gusset} = \frac{\pi^2 E I_W}{(kL_b)^2} = \frac{\pi^2 E L_w t_p^3}{12(kL_b)^2} \tag{9}$$

Referring to Figure 11b and Equation (9), $L_b = (L_1 + L_2 + L_3)/3$. Referring to Equation (9), the effective length factor k typically ranges from 0,65 to 1,00; whereas the moment of inertia of the Whitmore cross-section, $I_w = L_w \cdot t_p^3/12$; E is the modulus of elasticity of the steel material. The equivalent buckling length, L_b, may be calculated accordingly depending on the gusset plate geometry. These calculations are not only relevant for seismic applications. In particular, after the I35W bridge collapse in Minneapolis, USA, several transportation agencies are now evaluating and rating their inventory of steel truss bridge gusset plate connections in existing steel bridges (Higgins et al. 2010). Therefore, checks of the compressive capacity of bracing connections are pertinent. Higgins et al. (2010) provides a comprehensive comparison of various checks regarding the above issue.

Modern gusset plate design of steel frames with bracings in highly seismic regions requires the use of a minimum yield line that spreads inelastic deformations of the gusset plate within a plastic hinge location as shown in Figure 12a. This is because of the expected out-of-plane bending of the gusset plate connections once the bracing member buckles in flexure to dissipate the seismic action. Referring to Figure 12a, a plastic hinge zone equal of $2t_p$ typically leads to a relatively good spread of plasticity within the anticipated plastic hinge location of the gusset plate (AISC 2016b). An example from an actual steel frame with bracings is shown in Figure 12b after the completion of the building construction. Referring to Figure 12c, the same connection is able to guarantee a ductile behaviour of the bracing member once it buckles in flexure under cyclic loading. In recent work (Lehman et al. 2008), a balanced design procedure was proposed in which the gusset plate design promotes the distribution of inelastic deformations between the steel brace and the gusset plate connection through an elliptical fold line as shown in Figure 12d. This improves the seismic performance of gusset plate connections within a steel CBF. Moreover, bracing connections typically feature thin steel plates in this case, thereby leading to more economic steel CBF designs.

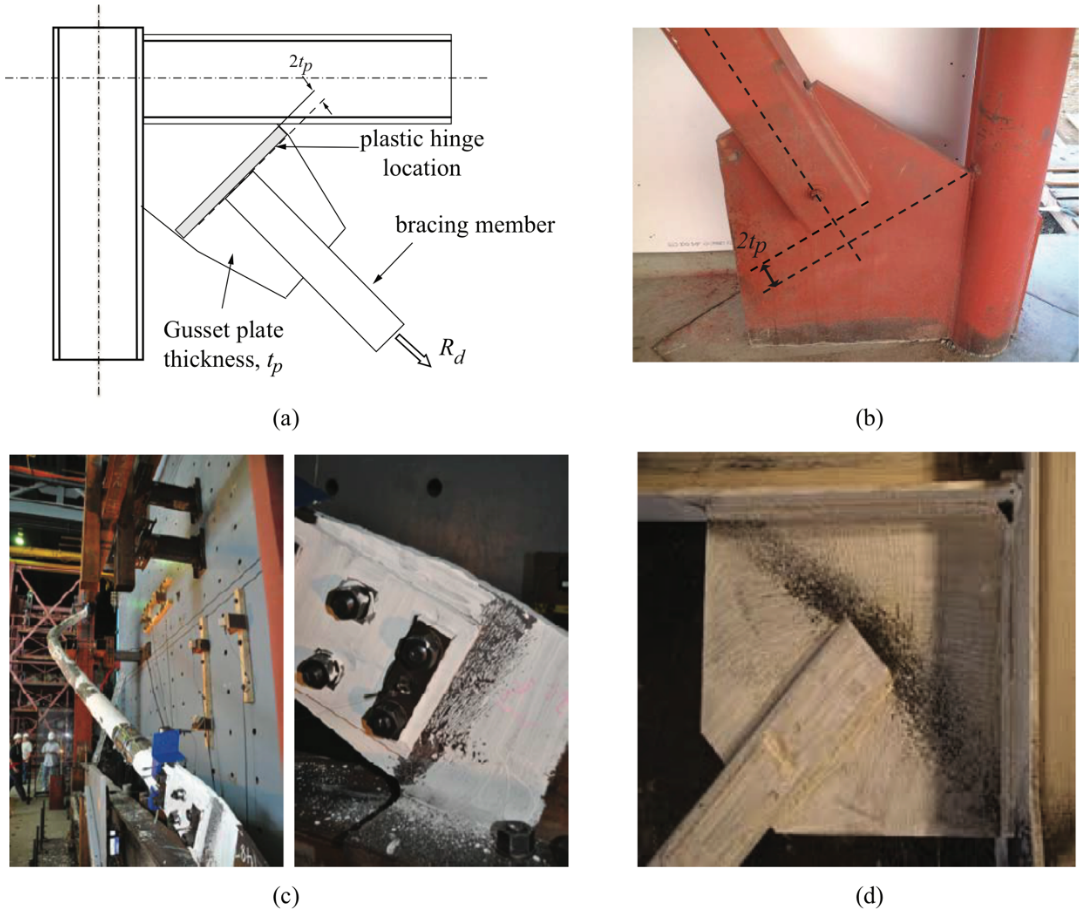


Figure 12. Typical gusset plate detailing to ensure ductile behaviour of bracing connections (image “b” courtesy of Prof. Dr. R. Tremblay; image “c” from de Oliveira et al. 2011; image “d” from Lehman et al. 2008)

While in areas of high seismicity robust capacity design rules have been developed to ensure the ductile behaviour of bracing connections and other members, in areas of low to moderate seismicity such rules are still evolving. Available test data (Davaran et al. 2014, 2015) suggest that by establishing a certain hierarchy of local failure modes within a bracing connection, the acquired ductility for conventional steel frames with bracings designed with a $q = 1{,}5$ can be appreciable. However, further research is required to established simple design rules for engineering use along with commonly employed connection and fabrication details particularly in X-bracing configurations.

Non-dissipative elements, such as steel beams and columns, within steel CBFs may be subjected to drift-induced bending moments coupled with high compressive axial load demands due to unbalanced loading. This occurs after the onset of brace buckling to dissipate the seismic action. The resulting load redistributions should be safely transferred to the non-dissipative elements. Figure 13 shows some practical rules that may be employed in order to calculate the unbalanced loads in tension-compression bracing systems without conducting static analysis with a structural analysis software. In particular, the non-dissipative elements (i.e., steel beams and columns) can be designed by simply calculating the anticipated seismic demands, once the bracing members buckle, by considering the full axial tensile resistance of the bracing member in tension ($N_{pl} = A_g \cdot f_y$) and the residual compressive resistance, $N_{b,r}$, of the bracing member after it experiences flexural buckling ($N_{b,r} = 0{,}3 \cdot A_g \cdot f_y$). This value is adopted in CEN (2005a). Slightly different values are adopted in the North American standards (CSA S16 2014; AISC 2016b) regarding this matter. The above considerations facilitate easiness in the seismic design process of low-rise steel CBFs when the equivalent lateral force method is employed.

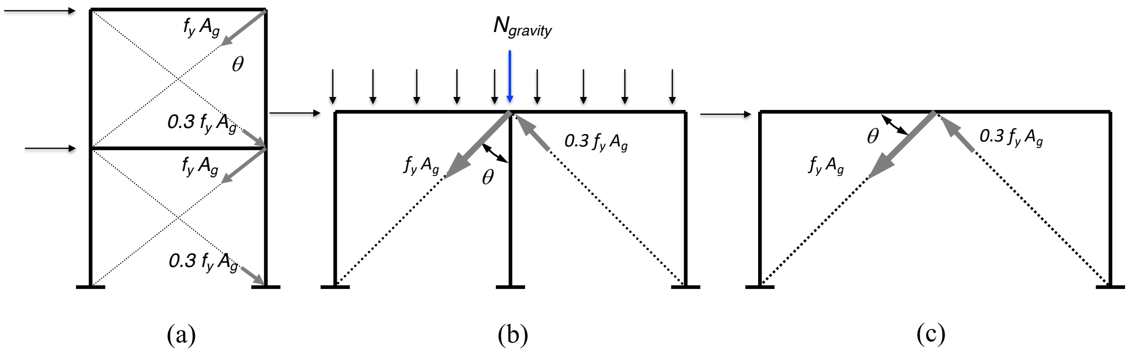


Figure 13. Earthquake forces within a tension/compression steel bracing system

In industrial applications featuring tall open spaces, it is common to use a multitiered braced frame with X-, chevron, V-, or single-diagonal bracing configurations as shown in Figure 14a. Recent research (Imanpour et al. 2016b; Toutant et al. 2017) suggests that brace buckling tends to cause drift concentration in the bottom tier of multi-tiered braced frame systems. This, in turn, causes in-plane flexural demands on the steel columns. These coupled with high compressive axial loads may lead to column flexural and/or lateral torsional buckling. The damage sequence is shown in Figure 14b. Practical rules to mitigate such instability modes have been integrated in recent versions of the AISC provisions (AISC 2016b) based on recent experimental and numerical studies (Imanpour et al. 2016a; b; Toutant et al. 2017).

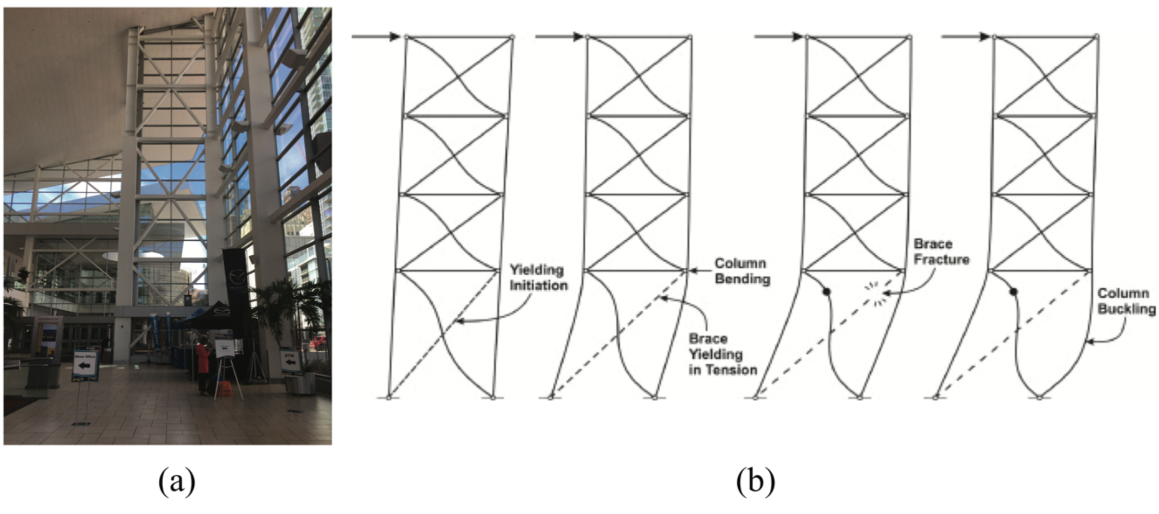


Figure 14. Multi-tiered braced frame (image "a" courtesy of Prof. Dr. D. Lignos; image "b" adopted from Imanpour et al. 2016a)

Innovative Steel Structural Systems

Recent earthquakes have shown that conventional structures complying to current seismic design requirements exhibit considerable economic losses due to repairs in structural and non-structural building content. Earthquake-induced economic losses in frame buildings is a fundamental challenge to be addressed in earthquake-resilient cities. When a building is not functional it results into considerable downtime and business disruption, thereby, augmenting the financial losses in the aftermath of earthquakes. Building-specific economic loss estimations for conventional steel MRF and CBF structures can be found in Hwang and Lignos (2017a; b). These studies suggest that during frequently-occurring seismic events, losses in steel MRF and CBF buildings are governed by non-structural component damage. However, in design-basis and low-probability of occurrence seismic events, losses in conventional structural steel systems are governed by either structural damage and/or demolition due to excessive residual storey drifts. To address these challenges, the steel industry has proposed innovative structural steel solutions featuring replaceable dissipative fuses, which may be easily replaced after an earthquake. The subsequent sections intend to review key features of the seismic behaviour of selected innovative structural steel systems along with basic design rules. These include eccentrically braced frames and buckling-restrained braced frames. A brief summary of other novel solutions is also presented.

Eccentrically braced frame systems

Eccentrically braced frames (EBFs) concentrate the inelastic action during a seismic event into replaceable ductile dissipative fuses so-called EBF links (Popov and Engelhardt 1988; Popov et al. 1989). The EBF links exhibit shear and/or flexural yielding depending on their length, e. It has been shown that EBF systems can ensure a highly ductile behaviour (similar to MRFs). At the same time, they can provide a large lateral stiffness (similar to CBFs) to resist the anticipated lateral load demands. Unlike the first generation of EBF links (Ricles and Popov 1989; Okazaki et al. 2009), more recent work (Mansour et al. 2011; Ji et al. 2016, 2017; Ioan et al. 2016) promotes the use of bolted EBF links that can be easily decoupled from the floor system. This offers a tremendous advantage to expedite repairs and restore a building's functionality in the aftermath of earthquakes provided that residual storey drifts are small.

(a) (b) (c)

Figure 15. 3-story steel frame building with EBF system and free-body diagram of EBF link (photos courtesy of Prof. Dr. D. Lignos)

Although seismic design rules for steel EBF systems were fairly well developed since the late 1980s (Popov et al. 1989; Okazaki et al. 2005), these systems gained much attention in recent years during the reconstruction of urban areas that experienced major earthquakes. These areas experienced considerable downtime due to building functionality disruption (MacRae et al. 2015). A notable example is that of the 2011 earthquake series in Christchurch, New Zealand. Figure 15 shows a 3-story steel frame building with replaceable EBF links after the reconstruction of Christchurch. The bolted EBF link shown in Figure 15b allows for the fabrication of the rest of the braced frame in a shop as a conventional truss system. Therefore, the erection process on the construction site can be significantly expedited.

Referring to Figure 15c, for a given instance of time during earthquake loading, the free body diagram of an EBF link indicates by force equilibrium that, $e = 2M/V$. Shear yielding occurs when the shear force becomes equal to the plastic shear capacity of the EBF link,

$$V = V_{pl,link} = \frac{f_y}{\sqrt{3}} \cdot (h - t_f) \cdot t_w \quad (10)$$

On the other hand, flexural yielding transpires when the flexural demand within the EBF link becomes equal to the plastic bending resistance of the EBF cross-section,

$$M = M_{pl,link} = f_y \cdot W_{pl,y} \quad (11)$$

Shear and flexural yielding occur simultaneously when, $e = 2{,}0M_{pl,link}/V_{pl,link}$. Shear yielding will occur when, $e < 2{,}0M_{pl,link}/V_{pl,link}$, whereas flexural yielding will occur when $e > 2{,}0M_{pl,link}/V_{pl,link}$. Longer links possess less strength, stiffness and ductility because local buckling governs their hysteretic behaviour at modest storey drift demands. Accrording to current seismic provisions (CEN 2005a; AISC 2016b), a link is considered to be long when, $e > 3{,}0M_{pl,link}/V_{pl,link}$, whereas an EBF link is considered shear-critical when $e \leq 1{,}6M_{pl,link}/V_{pl,link}$. In all other cases the EBF link is considered to be intermediate. While in steel MRF beams the axial load demand is fairly small, EBF links may be subjected to axial load demands due to the axial restraining effect of adjacent members (e.g., steel braces). Depending on the axial load demands within the EBF link, current seismic provisions (CEN 2005a; AISC 2016b) entail the reduction of the shear and bending resistance of the link itself due to the axial load-shear and axial load-bending interactions.

Representative experimental data on EBF links (e.g., Okazaki et al. 2005; Kazemzadeh Azad and Topkaya 2017) suggest that for a given loading protocol, shorter links exhibit, on average, larger overstrength values than longer ones. This is due to the associated steel material hardening prior to the onset of nonlinear geometric instabilities causing shear strength deterioration of the EBF link. Particularly, an overstrength of 1,5 is common in shorter links, whereas an overstrength of 1,2 is more appropriate for longer ones (Okazaki et al. 2005). The above values are particularly important for the sizing of the non-dissipative elements within an EBF system.

The European (CEN 2005a) and American seismic provisions (AISC 2016b) impose an upper limit of the link rotation angle, which is 0,08rads for shear-critical EBF links. The corresponding limit for long EBF links is 0,02rads. A value determined by linear interpolation between the above two values should be used in all other cases. Referring to Figure 14b, it is common to design EBF links with intermediate web stiffeners, which are fillet welded to the EBF link web. This is done in an effort to achieve a stable hysteretic behaviour at the targeted EBF link rotation angle. In short EBF links, web stiffeners are needed to prevent web tearing shortly after shear buckling of the EBF web.

Buckling-restrained braced frame systems

Buckling restrained braced frames (BRBFs) comprise bracing members that dissipate the seismic energy through axial yielding both in tension and compression without experiencing member buckling; thus exhibiting stable tension-compression yield cycles (Clark et al. 1999). Component and system-level experiments (Fahnestock et al. 2003, 2007; Black et al. 2004; Kasai et al. 2009) have demonstrated that BRBFs possess high seismic performance, thereby transcending several limiting features of the seismic performance of conventional CBF systems discussed in a previous section. Furthermore, their design principles are fairly simple to implement (Sabelli et al. 2003; Sabelli 2004). Figure 16 shows illustrative examples of steel buildings featuring BRBs with two common types of bracing connection detailing. These involve either a pinned or bolted connection. Buckling restrained braced frames have been the primary lateral load resisting system together with EBF systems for several of the buildings that were reconstructed in Christchurch after the 2011 earthquake series in New Zealand (Bruneau and MacRae 2018).

Figure 16. Buckling-restrained braced frames including typical bracing connections (photos from University of Canterbury campus, New Zealand, courtesy of Prof. Dr. D. Lignos)

Referring to Figure 17a, BRBs comprise a steel core, which is usually made of low yield stress steel (i.e., S235 steel) that is encased in a steel tube filled with concrete. The steel core is usually wrapped with a thin layer of debonding material that eliminates the shear transfer during the steel core elongation under tensile axial load and contraction under compressive axial load. In recent years, the all-steel BRB has also been developed (Dehghani 2016). The yielding segment of the brace has a cross-section smaller than the cross-section of the end connection region. Figure 17b illustrates the axial force-axial displacement relationship of such a brace. Its elastic stiffness is simply the in-series sum of the individual stiffnesses of the segments shown in Figure 17a. Particularly,

$$K_{total} = \frac{1}{\frac{1}{K_i} + 2\frac{1}{K_{CON}}} \tag{12}$$

Where $K_i = EA_i/L_i$ is the elastic axial stiffness of the yielding segment; $K_{CON} = EA_{CON}/L_{CON}$ is the elastic axial stiffness of the connection portion. In the inelastic regime, the steel brace delivers very stable hysteretic behaviour up to relatively large inelastic displacement demands. Typically, a qualification test is needed for a BRB verification. This test can accommodate the deformation and rotational demands associated with the respective design. Based on the BRB test the maximum expected tensile and compressive forces can be deduced such that the bracing connections and non-dissipative elements of the BRBF system can then be designed. Such tests are documented in Annex K of AISC-341-16 (AISC 2016b) including specific information regarding the test control, loading sequence, material testing requirements along with the acceptance criteria. In particular, a tension strength adjustment factor, ω, is calculated based on the maximum tensile axial load, T_{max}, as shown in Figure 17b for a given BRB qualification test. This factor accounts for material overstrength and strain hardening due to plastic deformations of the steel core. Similarly, the compressive strength adjustment factor, β, is calculated based on the maximum compressive axial load, P_{max} from the same qualification test.

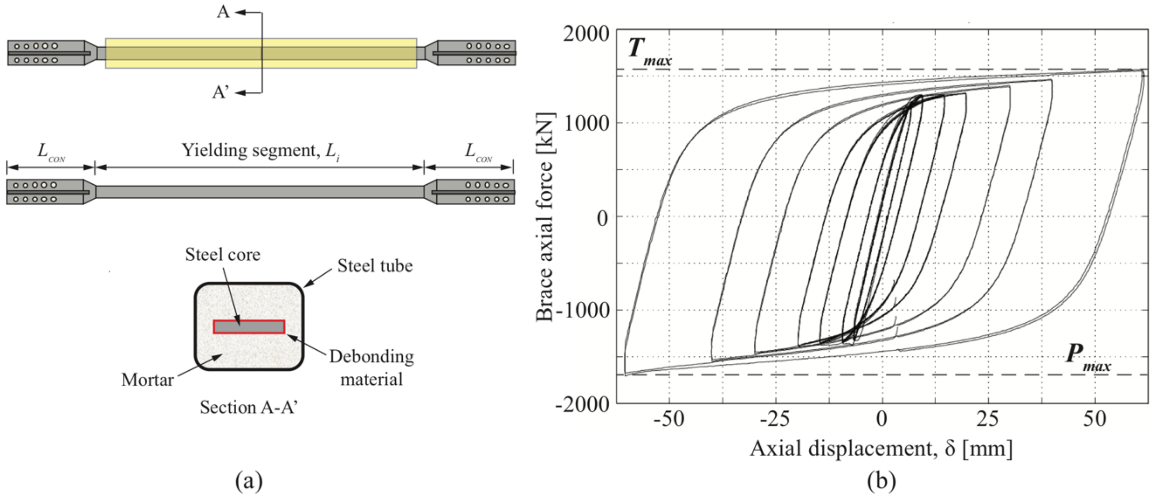


Figure 17. (a) Buckling-restrained brace including its main components; (b) typical hysteretic behaviour of a buckling-restrained brace under symmetric cyclic lateral loading history (experimental data from Black et al. 2004b)

In particular,

$$\omega = \frac{T_{max}}{f_y A} \tag{13}$$

and

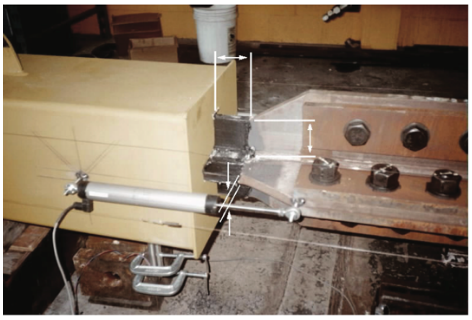
$$\beta = \frac{P_{max}}{f_y A} \tag{14}$$

Where, P_{max} is the maximum compression force and T_{max} is the maximum tension force within deformations corresponding to 200% of the considered design storey drift. The acceptance criteria for testing require that values of β and ω shall be greater or equal than 1,0 (Sabelli 2004). These values are then used to size the bracing connections and other non-dissipative members so as they remain elastic during an earthquake.

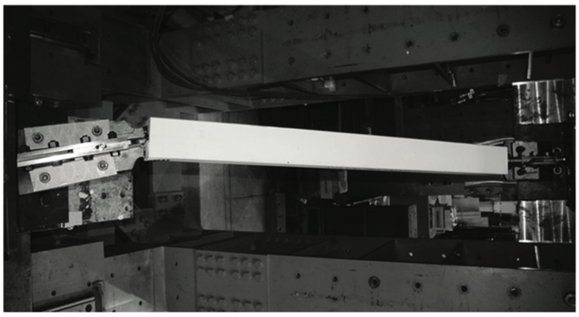
Although BRBs are expected to exhibit stable hysteretic behaviour under cyclic loading, past experiments suggest that some care is needed in the BRB connection detailing (Takeuchi et al. 2010, 2014a; b; Hikino et al. 2013). In particular, the stability of gusset-plate connections may be very critical if the surrounding connections are not properly stiffened. Figure 18a shows how the unsupported length of the brace while it elongates at maximum deformation, whereas Figure 18b shows the out-of-plane instability of the bracing connection due to insufficient flexural stiffness of the bracing connection relative to that of the casing. Comprehensive testing programs (Hikino et al. 2013; Takeuchi et al. 2014a) provide guidelines on how to properly stiffen the gusset plate connections to prevent the aforementioned failure modes.

Other innovative structural steel systems

Several studies (Midorikawa et al. 2002; Roke et al. 2006; Sause et al. 2010; Wiebe et al. 2013; Eatherton et al. 2014) have realized the rocking concept in controlled rocking (self-centering) steel concentrically braced frame systems (SC-CBFs). In this case, each column base of a SC CBF is allowed to lift off its foundation during a seismic event. Through post-tensioning, the rocking load is controlled, and the peak response is limited through energy dissipation devices. More recently, Gray et al. (2014) proposed a cast steel yielding brace system that dissipates seismic energy through the yielding fingers of a specially engineered cast steel connector. This brace has a very stable hysteretic behaviour that is characterized by a stiffening effect due to geometric hardening of the yielding fingers at large deformation demands. Hsiao et al. (2016) proposed the concept of the naturally buckling brace that combines high-strength and low-yield steels within a steel brace with a specified initial eccentricity that provides very high post-yield stiffness. Others (Christopoulos et al. 2008; Miller et al. 2012) developed self-centering steel braces that combine the advantages of a BRB with self-centering capabilities.



(a)



(b)

Figure 18. (a) brace at maximum deformation showing unsupported length (image adopted from Black et al. 2004b); (b) out-of-plane instability of the bracing connection (image adopted from Takeuchi et al. 2014a)

Summary Remarks

This paper attempts to provide a comprehensive overview of practical rules, which are currently used for the engineering of conventional structural steel systems, such as moment-resisting frames (MRFs) and frames with bracings (CBFs) to resist the lateral load demands during earthquakes. These systems, when they are designed to the current code specifications, generally meet the life-safety requirements since a local and/or global failure mode hierarchy is established. Therefore, unanticipated failure modes that could potentially compromise the global stability of a building are controlled. While the above design concepts seem to be fairly well established for conventional steel structures designed in highly seismic regions, practical rules are still evolving for the seismic design and fabrication of steel MRFs and CBFs in regions of low to moderate seismicity. Aspects related to weld and base metal toughness requirements, fabrication detailing should be benchmarked for commonly used welded beam-to-column connections, bracing connections with single lap splices. The above challenges are particularly important when a certain level of ductility is required depending on the employed behaviour factor, q.

On the other hand, in line with the concept of seismic resilience, several innovative lateral load-resisting systems have been developed. These systems typically concentrate the inelastic action in few selected dissipative fuses that are mostly characterized by a stable hysteretic behaviour. These fuses are intended to be easily replaced after an earthquake. The main systems that were covered are eccentrically braced frames (EBFs) with replaceable links, buckling-restrained braced frames (BRBFs) as well as alternative systems that leverage the beneficial aspect of rocking to improve the seismic performance of structures under earthquake shaking. The general consensus is that most of these systems minimize the structural damage and potential residual deformations that control decisions for building demolition in the aftermath of earthquakes. The general consensus is that special care should be put in the development of robust bracing connection detailing in an effort to eliminate potential problems with the stability of the connection itself.

All-in-all, steel materials and structures offer an interesting alternative to provide innovative solutions that potentially promote sustainability, reduce life-cycle costs over the building's service and have assisted in major reconstruction of urban cities that suffered from major earthquakes.

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