Proceedings of the 9th U.S. National and 10th Canadian Conference on Earthquake Engineering
Compte Rendu de la 9ième Conférence Nationale Américaine et
10ième Conférence Canadienne de Génie Parasismique
July 25-29, 2010, Toronto, Ontario, Canada • Paper No 1768

SEISMIC DESIGN OF STEEL STRUCTURES
IN ACCORDANCE WITH CSA-S16-09

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ABSTRACT

Standard S16 "Design of Steel Structures" of the Canadian Standards Association (CSA) governs the design of the majority of steel structures in Canada. Clause 27 of the standard includes the earthquake design provisions for seismic force resisting systems for which ductile seismic response is expected. Technical changes and new requirements have been incorporated in the 2009 edition of CSA S16, including modifications of the expected material properties for HSS members, consideration of protected zones, definitions of brace probable compressive and tensile resistances for capacity design and special requirements for braces intersecting columns between floors for concentrically braced steel frames, new seismic provisions for buckling restrained braced steel frames, design and detailing requirements for built-up tubular ductile links in eccentrically braced steel frames, changes to the requirements for ductile steel plate walls and for plate walls with limited ductility, including allowances for perforations and corner cut-outs in infill plates, and special provisions for steel frames of the Conventional Construction category above 15 m in height. These modifications were developed in parallel with the 2010 National Building Code of Canada (NBCC). The paper summarizes the new CSA S16-09 seismic design requirements with reference to NBCC 2010.

Introduction

Standard S16 "Design of Steel Structures" (CSA 2009) of the Canadian Standards Association (CSA) governs the design of the majority of steel structures in Canada.. Clause 27 of the standard includes the earthquake design provisions for steel seismic force resisting systems (SFRS) for which ductile response is required to withstand earthquake forces. These systems are listed in Table 1, along with the ductility- and overstrength-related seismic force modification factors, R_d and R_o, respectively, and system restrictions that are prescribed in the upcoming 2010

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National Building Code of Canada (NBCC) (NRCC 2009). Seismic provisions are also specified in Clause 27 of CSA S16 for steel structures of the Conventional Construction category, i.e., structures for which only the inherent ductility of steel and other sources of energy dissipation present in ordinary steel frames (including friction and plastic deformation) are relied upon to dissipate the seismic energy input. System restrictions in NBCC 2010 depend on the short period spectral acceleration ratio, $I_E F_a S_a(0.2)$, and the one second spectral acceleration ratio, $I_E F_a S_a(1.0)$, where I_E is the importance factor, F_a and F_v are, respectively, the acceleration- and velocity-related site modification factors, and $S_a(0.2)$ and $S_a(1.0)$ are the uniform hazard spectral ordinates at periods of 0.2 and 1.0 s, respectively.

Several changes and new requirements have been incorporated in the 2009 edition of CSA S16, including modifications to the expected material properties for HSS members, consideration of protected zones, definitions of the compressive and tensile strengths of bracing members for capacity design checks and special requirements for braces intersecting columns between floors in concentrically braced steel frames, new provisions for buckling restrained braced frames, design and detailing requirements for built-up tubular links in eccentrically braced frames, changes to the requirements for ductile and limited ductility plate walls, including allowances for perforations and corner cut-outs in infill plates, and design provisions for industrial structures of the Conventional Construction category that exceed 15 m in height. The paper outlines the seismic provisions of CSA S16 with focus on the changes introduced in 2009.

Table 1: R-values and restrictions for steel SFRS designed and detailed according to CSA S16

Type of seismic force resisting system	R_d	R_o	Building height (m) limitations[1]				
			$I_E F_a S_a(0.2)$				$I_E F_v S_a(1.0)$
			< 0.2	≥ 0.2 to < 0.35	≥ 0.35 to ≤ 0.75	> 0.75	> 0.3
Moment-resisting frames							
Ductile moment resisting frames	5.0	1.5	NL	NL	NL	NL	NL
Moderately ductile moment resisting Frames	3.5	1.5	NL	NL	NL	NL	NL
Limited ductility moment resisting Frames	2.0	1.3	NL	NL	60	NP	NP
Moderately ductile concentrically braced frames							
Tension-compression braces	3.0	1.3	NL	NL	40	40	40
Tension only braces	3.0	1.3	NL	NL	20	20	20
Limited ductility concentrically braced frames							
Tension-compression braces	2.0	1.3	NL	NL	60	60	60
Tension only braces	2.0	1.3	NL	NL	40	40	40
Ductile buckling-restrained braced frames	4.0	1.2	NL	NL	40	40	40
Ductile eccentrically braced frames	4.0	1.5	NL	NL	NL	NL	NL
Plate walls							
Ductile plate walls	5.0	1.6	NL	NL	NL	NL	NL
Limited ductility plate walls	2.0	1.5	NL	NL	60	60	60
Conventional construction of moment-resisting frames, braced frames or plate walls							
Assembly occupancies	1.5	1.3	NL	NL	15	15	15
Other occupancies	1.5	1.3	NL	NL	60	40	40
Other steel SFRS(s) not defined above	1.0	1.0	15	15	NP	NP	NP

[1] NL = Not limited; NP = Not permitted

General provisions

The first section of Clause 27 includes a set of general provisions applicable to all steel SFRS, including capacity design requirements, continuity of the seismic load path, integrity of the gravity load carrying system, minimum requirements for material and bolted connections, expected material strength, stability issues, and protected zones.

Basic capacity design provisions are given in CSA S16 to ascertain that minimum strength hierarchy exists along the lateral load path such that the intended ductile energy dissipation mechanism is mobilized and the integrity of the structure is maintained under strong ground shaking. In the design process, the yielding components of the SFRS may be oversized compared to the specified design seismic forces, as would be the case when drift limits, minimum member sizes or non-seismic load combinations govern the design. In this case, it is specified both in NBCC 2010 and CSA S16 that the design forces in capacity-protected elements need not exceed those induced by a storey shear determined with $R_oR_d = 1.3$. This upper bound essentially corresponds to the elastic seismic force demand reduced by 1.3, recognizing that non-yielding components will likely possess minimum overstrength. This 1.3 reduction factor only applies if the governing failure mode is ductile, and $R_oR_d = 1.0$ must be used otherwise.

Recent surveys by Schmidt and Bartlett (2002) and Liu et al. (2007) revealed that the probable yield stress of circular and rectangular hollow structural sections (HSS) produced in North America can significantly exceed their nominal values. It is now stipulated in CSA S16 that the product R_yF_y be taken as not less than 460 MPa for all HSS types when assessing the probable member resistance in capacity design. This corresponds to $R_y = 1.31$ for 350 MPa steels, which represents an increase of 19% compared to the value $R_y = 1.1$ prescribed in previous editions of CSA S16. This change will mainly impact building systems using concentrically braced steel frames as the yielding element of these systems are bracing elements which are commonly fabricated from tube sections. For ASTM A500 HSS members, F_y varies between 290 and 345 MPa, depending on the steel grade (B or C) and the tube shape (rectangular or circular). The use of $R_yF_y = 460$ MPa for these HSS members translates into R_y factors ranging between 1.33 to 1.59, thus comparable to or greater than the 1.4 value specified in the 2005 AISC seismic provisions (AISC 2005a).

P-Δ effects in CSA S16 are determined using the design inter-storey drifts, including anticipated inelastic deformations. A clarification was included in S16-09 regarding stability effects in seismic design: notional loads and P-Δ force amplification must be considered only for the design of the yielding components of the SFRS such that the SFRS has sufficient lateral strength upon yielding to resist the design seismic loads in addition to the detrimental effects of initial structure out-of-plumb, premature yielding due to residual stresses and geometric imperfections, and gravity loads acting on the deformed structure. These stability effects need not be considered for the capacity protected components of the SFRS as the force demand on these components is bounded by the strength of the yielding components. When the yielding components are oversized and the upper bound seismic design forces apply, the system is expected to exhibit essentially elastic response and notional loads and P-Δ effects should then be considered in the design of all SFRS components.

The concept of protected zones was introduced in the 2005 AISC seismic provisions to prevent the ductility of the SFRS yielding components from being reduced below the level intended in design as a result of operations at the fabrication plant or construction site. The same concept was adopted in CSA S16-09 and protected zones are now defined for each SFRS. It is stipulated in S16 that protected zones must be identified on structural and shop drawings, but S16 does not provide specific means of conveying this information to other specialties involved in a construction project so that accidental alterations of critical SFRS elements is prevented.

Moment-Resisting Steel Frames

CSA S16 includes provisions for three categories of moment-resisting frames: Type D (ductile), Type MD (moderately ductile), and Type LD (limited-ductility) moment-resisting frames. These systems have been defined in the 2001 edition of S16, together with the applicable design and detailing rules, based on the extensive research and studies that followed the 1994 Northridge and 1995 Kobe earthquakes. No change was introduced in CSA S16-09 for moment-resisting frames, except for the definition of the protected zones.

In Type D and Type MD frames, seismic energy is expected to be essentially dissipated through plastic rotation at the beam ends. The engineer can also choose to activate shear yielding in column joint panel zones, but the extent of plastic deformation at these locations is limited. Plastic hinging is also permitted at the bottom of columns rigidly connected at their bases, or at the upper column ends in single-storey structures. The same design and detailing rules apply to both frame categories, except that relaxations are permitted for Type MD structures in view of the lower anticipated ductility demand resulting from the smaller value of R_d (= 3.5) used to determine the design seismic loads, compared to $R_d = 5.0$ for Type D frames (see Table 1). For instance, beam-to-column connections have to be demonstrated by means of physical testing as satisfying cyclic deformation performance up to 0.03 radians for Type MD frames versus 0.04 radians for Type D structures. When plastic hinging is expected in columns, the column axial load is limited to delay the occurrence of local buckling. The limits are set to 50% and 30% of the column axial yield strength (squash load) for Types MD and D frames, respectively. Pre-qualified connections are permitted to be used, as described in CISC (2004) and AISC (2005b). Comprehensive capacity design requirements equally apply to both frame categories to achieve the intended yielding mechanism.

Type LD moment-resiting frames were introduced in CSA S16-01 to propose a simpler moment-resisting solution for low- and mid-rise applications, especially when member sizes need to be increased to satisfy drift limits, resist wind effects or control P-Δ effects. The design seismic loads are obtained with a R_d factor of 2.0 and the resulting limited yielding is accepted in beams, columns or joints. The frames are, however, restricted to 60 metres and 30 metres in height in regions of moderate and high seismicity, respectively. Strength hierarchy need not be verified in most cases, except that strong-column/weak-beam design must be applied in high seismic regions or in any buildings taller than 60 metres, as permitted in low seismicity regions. Beam-to-column joint details must be capable of accommodating an inter-storey drift angle of 0.02 radians. Alternatively, traditional joints with complete penetration groove welds at the beam flanges, using matching electrodes, can be adopted for beams framing into column flanges, provided that the beam web is connected by a welded joint and that the steel backing bars and run-off tabs at the beam flanges are removed and repaired with reinforcement fillet welds.

Braced Steel Frames

Concentrically Braced Steel Frames

Two categories of concentrically braced steel frames are described in CSA S16-09: Type MD (moderately ductile) and Type LD (limited ductility) braced frames. Energy dissipation in both systems is achieved through brace yielding in tension and, to a lesser extent, by flexural hinging in braces upon brace buckling in compression and subsequent brace straightening upon reloading in tension. A strict capacity design procedure is enforced for both braced frame categories and the same design and detailing rules apply to both framing systems with the exception that some relaxations are permitted for the Type LD system.

Once brace buckling has occurred under strong ground shaking, the inelastic seismic response of concentrically braced steel frames becomes strongly influenced by the braces acting in tension. Therefore, to achieve symmetrical inelastic response, it is required in CSA S16 that the braces along any braced line be arranged in such a way that the storey shear resistance provided by the tension-acting braces in opposite directions is comparable. This requirement allows design engineers to use tension-compression or tension-only braced frame design, whichever represents the most cost-effective solution. As indicated in Table 1, height limits are prescribed in NBCC that vary according to the braced frame type adopted in design. Lower limits are prescribed for tension-only bracing, in view of the limited energy dissipation capacity exhibited by the slender braces typically resulting from this design strategy. Conversely, taller buildings are permitted when Type LD frames are used, recognizing that these frames will sustain lower inelastic demand and, hence, will be less prone to soft-storey response.

Brace effective slenderness, KL/r, is limited to 200 for all systems, with the exception that the limit is extended to 300 for tension-only Type LD braced frames in one- and two-storey building applications. Past experimental studies on bracing members have shown that the fracture life of braces subjected to cyclic inelastic axial deformation tends to diminish as the brace slenderness is reduced (Tremblay 2002). The limits on the brace cross-section width-to-thickness ratios specified in CSA S16 reflect this observation with stringent limits when the brace KL/r is equal to or less than 100, and limits that linearly increase towards less critical values when KL/r is between 100 and 200. Recent test programs (e.g., Uriz 2005, Fell et al. 2009) have revealed that HSS bracing members with low slenderness may not possess sufficient ductility capacity to sustain the anticipated inelastic demand. In CSA S16-09, it is now stipulated that HSS braces must have a minimum effective slenderness of 70 to mitigate the risk of premature fracture. In S16, it is required that the effective brace slenderness be used in these verifications, with proper consideration of actual brace end fixity and intermediate support conditions. When performing capacity design, SFRS components other than the bracing members must resist the forces that develop when the braces reach their probable axial resistance. New definitions have been introduced in S16-09 for the probable brace tensile resistance, $T_u = AR_yF_y$, where A is the brace cross-sectional area, the probable brace compressive resistance, C_u, which is taken as equal to the lesser of AR_yF_y and 1.2 times the brace buckling resistance determined with the resistance factor $\phi = 1.0$ and the probable yield stress, R_yF_y, and the probable post-buckling brace compressive resistance, C'_u, taken as equal to the lesser of 0.2 AR_yF_y and C_u.

Since 2001, CSA S16 has included special provisions to account for the flexural demand imposed on continuous columns of multi-storey structures that arises from variations in storey drifts between adjacent floors during an earthquake (Fig. 1a). These bending moments cannot be predicted from linear elastic analysis, as typically performed in design. In S16, columns in bracing bents must be designed as beam-columns assuming an additional bending moment equal to 20% of the column plastic moment. This requirement need not be applied to gravity columns as these columns possess sufficient capacity to carry the reduced gravity loads considered in earthquake load combinations together with the seismic-induced bending moments. However, the splice connections in all columns of the building must be designed for the shear forces associated with the expected flexural demand. Column continuity has positive impacts on the distribution of brace inelastic response over the building height and it is required in S16 that all columns in multi-storey Type MD structures be continuous and of constant cross-section over a minimum of two levels for tension-compression systems and over the full building height for tension-only bracing.
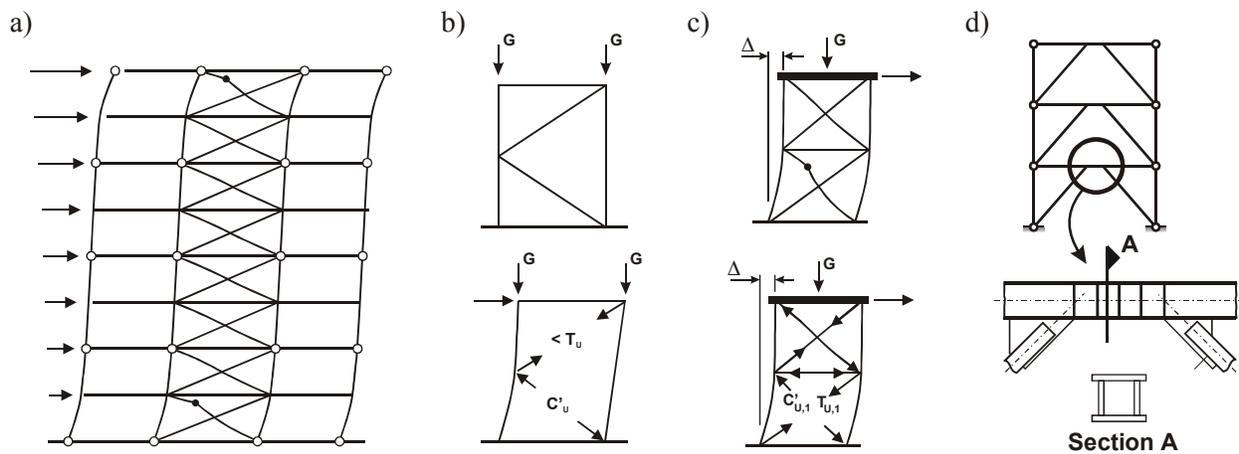
Figure 1 a) Variation in inter-storey drifts and column bending along the height of CBFs; b) Post-buckling response and flexural demand on columns in K-bracing; c) Type LD single-storey CBF with braces meeting on one side of columns between floors; and d) Eccentrically braced steel frames with built-up tubular links.

K-bracing where braces meet on one side of columns between floors is not permitted in Type MD and Type LD frames as the large unbalanced horizontal load that will be imposed on the column at the brace intersection point after buckling of the compression brace occurs may lead to column instability (Fig. 1b). In cases of tall single-storey structures such as large warehouses, airplane hangars or heavy industrial applications, it is not uncommon, and sometime unavoidable, to have two or more X-bracings stacked between the ground level and the roof (Fig. 1c). For this configuration, it is expected that brace buckling and yielding under strong ground motions will concentrate in the bracing panel where inelastic response is triggered first, as the capacity of this panel will likely degrade after brace buckling to create a fuse that will prevent inelastic response from developing in the remaining panels. As a result, large inelastic deformations will concentrate in just one pair of braces and significant flexural demand will be imposed on the columns (Fig. 1c). In S16-09, that bracing configuration is now permitted, but only for Type LD braced frames as this system category is expected to sustain only a limited

degree of inelastic response. Horizontal struts must be provided between the columns at each brace intersection point such that the tensile capacity of the braces can be mobilized after brace buckling, without imposing concentrated unbalanced horizontal loads on the columns at brace-to-column connections points (Fig. 1c). The capacity of the columns must then be verified when subjected to the gravity loads together with the axial loads and bending moments that are induced when the design storey drift, i.e., R_oR_d times the elastic roof lateral displacement, is reached at the roof level assuming that brace yielding in tension develops in any one panel along the storey height. The struts must also be designed for the axial loads that will develop in that deformed configuration, i.e., essentially the horizontal component of the brace probable tensile strength. Out-of-plane forces and deformations may develop upon brace buckling and an out-of-plane force equal to 10% of the member compression loads must be considered in the analysis at brace-to-column intersection points.

The full length of the bracing members and the connections between the braces and beams and columns are considered protected zones in Type MD and Type LD frames.

Eccentrically Braced Frames

Provisions for eccentrically braced steel frames have been modified only slightly since the 1994 edition of CSA S16. The main technical change in 2009 is the introduction of design provisions for ductile link beams with built-up rectangular tubular cross-section as an alternative to the traditional I-shaped link beams (Fig. 1d). The main advantages of this new design are that the links can be tailored to closely match the strength demand and the tubular links, when adequately proportioned, do not need any lateral bracing due to their higher out-of-plane and torsional stiffnesses. The supporting research work on this new link detail was originally performed for bridge structure applications (Berman and Bruneau 2008a), where link lateral bracing is difficult to achieve. In building structures, the system can then represent an attractive solution when it is not possible to provide link lateral bracing, as would be the case along an exterior column line, or next to an elevator or stairway shaft. Provisions for the design and detailing of EBFs with built-up rectangular tubular links are given in CSA S16-09. Protected zones in EBFs extend one-half the beam depth beyond the link ends.

Buckling Restrained Braced Frames

Provisions for the seismic design of buckling restrained braced frames (BRBF) were introduced for the first time in CSA S16 in 2009. BRBFs are concentrically braced frames in which the braces are specially designed and detailed such that they yield both in tension and compression, without buckling. These bracing members were introduced in Japan in the late 1980s to act as hysteretic dampers designed to dissipate energy and control drifts in steel moment-resisting frames (Xie 2005). A typical BRB member design is illustrated in Fig. 2a. It consists of a steel core plate with a reduced cross-section inserted in a steel tube. The volume between the tube and the core is filled with mortar or concrete. At both ends of the brace, the core plate extends outside of the tube and is connected to the framework. The core is covered with an un-bonding material prior to filling the tube, so that it can freely deform axially and resist most of the applied axial load. The unsupported projections of the core at the two ends of the brace are typically made larger in size and/or are stiffened to avoid buckling in compression.

Under seismic loading, yielding is expected to take place both in compression and tension in the reduced segment of the core, leading to a nearly symmetrical hysteretic response (Fig. 2b).
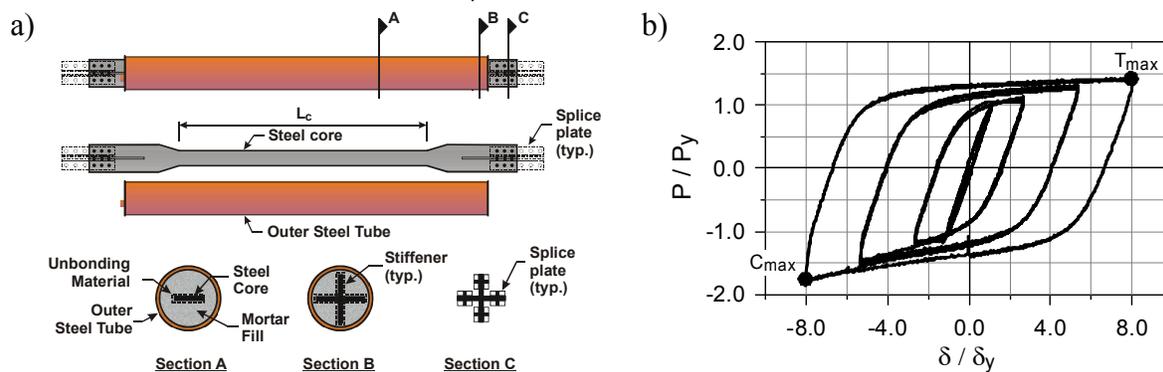
Figure 2   Buckling restrained bracing member: a) Typical design; b) Hysteretic response.

In North America, the BRB members were introduced in the late 1990s, essentially as a substitute to conventional bracing members in concentrically braced steel frames, not as a damping device. In Canada, buckling restrained braces have typically been produced by local steel fabricators using simple designs such as that proposed by Tremblay et al. (2006). In view of the stable brace hysteretic response, a ductility-related factor $R_d = 4.0$ is specified for the Type D (ductile) BRBF system in the NBCC, the same value as for ductile eccentrically braced steel frames (Table 1). Typically, the core plate of BRB members is cut to exactly achieve the required factored axial resistance for the brace, taking into account the measured steel properties as obtained from mill test certificates or coupon tension tests. Hence, the dependable system overstrength, as defined by Mitchell et al. (2003), arises only from the difference between nominal and factored brace resistances and the strain hardening of the core steel when subjected to large inelastic deformations, and a relatively low overstrength-related force modification factor, $R_o = 1.2$, is specified in the NBCC for BRBFs. In Canada, buckling restrained braced frames are typically built with non-moment-resisting beam-to-column joints, which makes them sensitive to concentrations of inelastic demand over the frame height. Height limits have therefore been introduced for BRBF applications in moderate and high seismic regions to minimize the risk of soft-storey response (see Table 1).

The seismic provisions for Type D BRBFs in S16-09 are similar to those introduced in the 2005 AISC provisions (Sabelli 2004). It is assumed that BRB members can develop the same factored axial resistance in tension and compression, based on the achievement of the full yield strength of the core. The core plate must be made from a single piece and, when made of thick plates, meet minimum toughness requirements. Qualifying cyclic testing is specified in CSA S16-09 to ensure that the braces perform as intended in design. Individual brace tests and sub-assemblage tests must be performed up to axial deformations corresponding to two times the design inter-storey drifts including inelastic response. Sub-assemblage testing aims at verifying that the braces can sustain the imposed axial deformations together with the end rotational demand that is imposed as the frame deforms laterally during an earthquake. In capacity design, the maximum brace compression and tension forces that are determined in the tests must be used to determine the loads to be considered for the design of beams, columns, connections, etc.

Plate Walls

In 2001, seismic design provisions for unstiffened steel plate walls were incorporated in CSA S16 for two plate wall categories: Type D (ductile) and Type LD (limited-ductility). The S16-01 requirements were developed based on the pioneering research work conducted in Canada since the late 1970s on the system. For both wall categories, energy is mainly dissipated by tension yielding of the infill plates. For Type D walls, the beams must be rigidly connected to the columns to form moment-resisting frames that are designed such that energy is also dissipated through plastic hinges forming at the beam ends and at the column bases. The beams need not be moment-connected to columns in Type LD frames, which results in pinched hysteretic response and lower energy dissipation under cyclic inelastic loading. This difference in behaviour translates into two different ductility-related force modification factors that were then adopted for Types D and LD walls: 5.0 and 2.0, respectively. Limited capacity design provisions were prescribed in CSA S16-01 for Type D walls, with emphasis put on column design, whereas no special requirements applied to Type LD walls. The latter was, however, limited to 60 m in height in moderate and high seismic regions.

As shown in Table 1, the same two plate wall categories, R_d factors, and system restrictions have been retained in NBCC 2010, but more comprehensive seismic requirements have been introduced in S16-09 for both systems. For Type D walls, the total seismic storey shear must be resisted by the infill plates acting alone, as was done before, but a minimum additional lateral strength of 25% of the total storey shear must be supplied by the moment-resisting frame. This can be satisfied by selecting beams that can develop the required storey shear resistance assuming a beam plastic hinge mechanism. Simplifications have been introduced in Clause 20 of S16-09 for the calculation of the angle of inclination of the infill plate tension field. When the width-to-height ratio of the infill plate is between 0.6 and 2.5, which covers most applications encountered in practice, the angle can be taken as equal to 40°, without further calculations. This simplification is based on observations by Shishkin et al. (2009) of the insensitivity of the overall system behaviour to the angle selected. If the angle is determined from the wall properties, the average angle can be used over the entire wall height, instead of different angles at every floor, which greatly simplifies the calculations and computer modelling. A minimum column stiffness is prescribed to ensure that a sufficiently uniform tension field forms in the infill plates. In S16-09, a new stiffness parameter has been introduced to address the same issue for the top and bottom panels, based on the work by Dastfan and Driver (2008).

In actual applications, the thickness of the infill plates is often selected to meet minimum workability limits, which may result in large forces in capacity design checks. One solution in this case is to lower the strength of the plate by introducing uniformly distributed circular perforations that are aligned according to the tension field inclination (Fig. 3a). Design equations and detailing requirements are now given in S16-09 for perforated infill plates, essentially based on the work by Purba and Bruneau (2009) and Vian et al. (2009). These perforations can also be useful for passing through mechanical and electrical equipment. Corner cut-outs reinforced with an arch can also be implemented in the infill plates for that purpose (Fig. 3b). This alternative is now accepted in S16-09, provided that the arch has sufficient strength to develop tension yielding in the infill plates and resist the forces that are likely to develop when the frame deforms laterally (Vian et al. 2009).

Once the infill plates are sized, beams and columns must be designed for the forces associated with full yielding of the plates, as illustrated in Fig. 3c. In particular, large shear forces and bending moments are induced in the columns by the horizontal component of the tension yield forces in the plate. The columns must also be designed for the axial loads induced by the gravity loads and the vertical components of the infill plate induced tension, together with the vertical shear forces and bending moments that develop when plastic hinging develops at the beam ends (Berman and Bruneau 2008b). Similarly, beams must be designed for the vertical forces induced by gravity loads and the unbalanced plate tension forces, acting in combination with the compression axial loads imposed by the columns that are pulled inwards by the infill plate. CSA S16 also contains provisions for the design and detailing of the beam-to-column joints.

In S16-09, Type LD plate walls must now satisfy all provisions prescribed for Type D walls, with some relaxation that is permitted in view of the lower expected ductility requirement. The main difference is that rigid beam-to-column joints are not required in Type LD frames, which considerably simplifies the calculations and fabrication. It is noted, however, that a thorough capacity design check must still be performed to avoid undesirable behaviour, such as column yielding or buckling, from occurring before yielding of the infill plate is achieved. Protected zones in both wall categories include the infill plates and rigid beam-to-column joints when applicable.
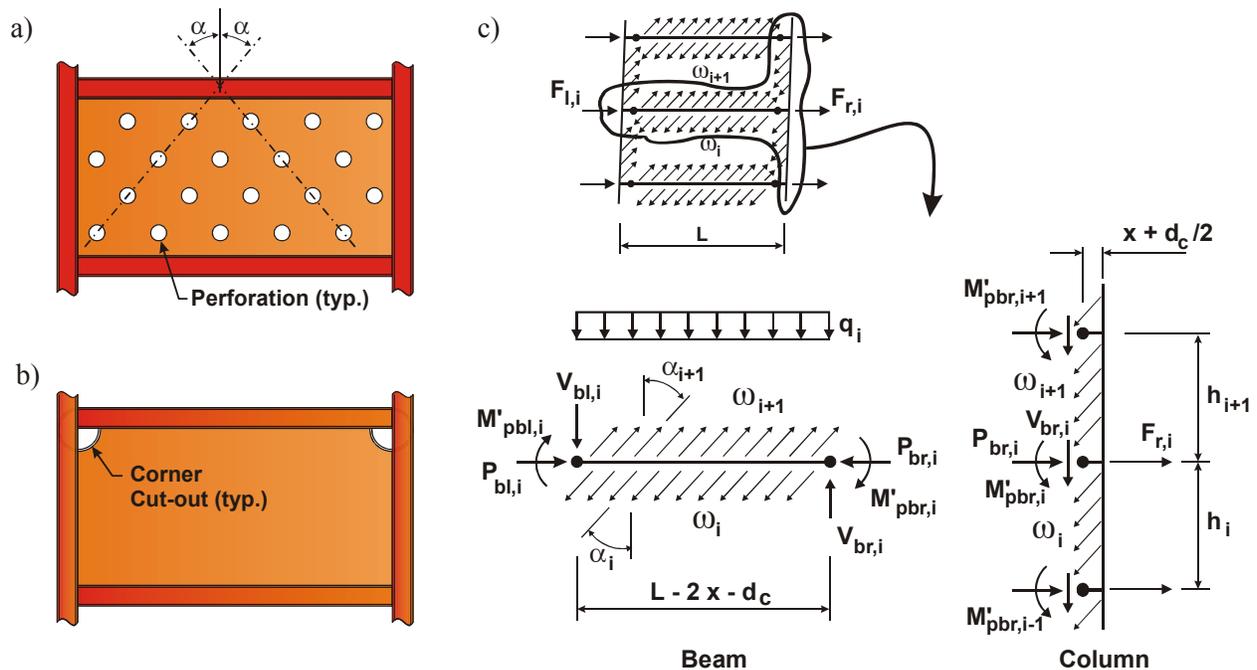
Figure 3   Steel plate walls: a) Perforated infill plates; b) Circular corner cut-outs in infill plates; and c) Forces imposed on beams and columns upon yielding of the infill plates (x = distance from the beam plastic hinge to the column face, d_c = column depth).

Conventional Construction

Many steel buildings are designed using Type CC (Conventional Construction) structures for seismic resistance, and rely only on the inherent ductility of steel and other energy dissipation mechanisms present in these steel structures to withstand seismic ground motions in the inelastic range. In moderate and high seismic zones the use of Conventional Construction is precluded by NBCC for post-disaster facilities.

Design seismic forces for Type CC structures are determined using a ductility-related force modification factor $R_d = 1.5$. Prior to 2001, no special detailing requirements were prescribed for these structures except that a 3-storey height limit was specified in the NBCC for moderate and high seismic zone applications. In S16-01, a requirement was added to increase the design forces for connections by 1.5 times for these frames, without exceeding the expected yield strength of the members being joined, unless the connection's governing failure mode was ductile. This change was introduced on the basis that most of the damage to steel structures in past earthquakes was associated with brittle connection failures.

In NBCC 2005, the height limit for Type CC frames was changed from 3-storeys to 15 m. This modification, although insignificant for most regular office or residential buildings, had an impact on the steel industry, as Type CC designs were no longer permitted for tall single-storey steel structures such as steel mills and aircraft hangars that often greatly exceed 15 metres in height. For these structures, implementing one of the ductile SFRSs described in Clause 27 may pose a problem due to their complex geometry or the necessity to build a structure that can be easily modified as the needs evolve over the years.

As shown in Table 1, the 15 m height limit is extended to 40 or 60 m in NBCC 2010, depending upon the seismic demand, except for assembly occupancy structures such as schools, stadia, exhibition halls, arenas, convention centres and other similar structures, which must still satisfy the 15 m height restriction in view of the greater consequences in case of structural collapse. The height limit relaxation for the other structures came with several new requirements that have been introduced in S16-09 to avoid concentration of inelastic demand or brittle failure modes. In particular, the seismic design loads must be increased linearly by 2% per metre of height above 15 m, without exceeding forces corresponding to $R_d R_o = 1.0$, and the seismic forces and deformations must be determined using the Dynamic Analysis Procedure described in the NBCC. Steel and weld metals used in these structures must meet minimum requirements for ductility and all members of the SFRS must meet plastic or compact cross-section limits to delay the occurrence of local buckling. Columns and connections must be designed for amplified earthquake loads to further protect their integrity. In addition, the connections must be detailed such that their governing failure mode is ductile. Also, minimum factored seismic forces are prescribed for diaphragms to control inelastic diaphragm response and the members of the SFRS that intersect at an unbraced location are required to minimum have out-of-plane transverse resistance. It is anticipated that the increased height restrictions will benefit mostly the industrial type buildings where floor-to-floor heights and restrictions in process configuration have precluded the use of more ductile seismic systems.

Conclusions

Several new seismic design requirements have been introduced into CSA S16-09 as a result of recent research results (e.g., minimum slenderness for HSS braces, enhanced capacity design provisions for plate walls), the availability of new systems or designs (e.g., BRBFs or EBFs with built-up tubular links), or the need for simplified or more effective systems (calculation of the tension field inclination in plate walls, use of perforated infill plates or corner cut-outs in infill plates of plate walls or extension of the 15 m height limits for Type CC structures). These changes will undoubtedly provide design engineers with a wider and better choice of cost-effective solutions for resisting seismic ground motions. It is expected that this evolution will continue in future editions of S16, with the inclusion of improved design or detailing provisions for current SRFSs, as more information becomes available on their seismic behaviour. It is also expected that S16 will eventually provide guidance for the use of more effective systems such as plate walls with composite columns or innovative systems with replaceable fuse elements or re-centering capabilities for enhanced overall seismic performance, with consideration of cost associated to repair and disruption time.

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