PART TWO
CISC COMMENTARY ON CSA S16-14

Preface

This Commentary has been prepared by the Canadian Institute of Steel Construction in order to provide guidance on the intent of various provisions of CSA Standard S16-14, "Design of Steel Structures". This Commentary and the information contained in the references cited provide an extensive background to the development of the Standard and its technical requirements including the changes and new provisions introduced in the 2014 edition. The Preface to the Standard itself outlines the history of its development since the first edition in 1924.

CSA Standard S16-14 has been prepared by the Canadian Standards Association (CSA), an approved standards development organization of the Standards Council of Canada, according to the rules for development of consensus standards. The National Building Code of Canada 2015 has adopted CSA Standard S16-14 by reference.

The Institute gratefully acknowledges the efforts of the various members of the CSA Technical Committee on Steel Structures for their valuable contributions to the Commentary, especially, G. Grondin, R. Tremblay, R.G. Driver, J.A. Packer and A.F. Wong who helped to rewrite a significant portion of this edition. The contributors include many former members of the Committee, in particular, D.J.L. Kennedy who had chaired the committee, served as a key author of the Commentary and provided valuable background information pertaining to many requirements introduced in all previous editions of the Standard prior to S16-09.

The information contained in the Commentary is provided by the Institute. It is not to be considered the opinion of the CSA Committee, nor does it detract from that Committee's responsibility and authority insofar as interpretation and revision of the Standard are concerned. For information on requesting interpretations, see Note (5) to the Preface of CSA S16-14.

The Institute provides this Commentary as a part of its commitment to the education of those interested in the use of steel in construction. Neither the Institute nor the authors of this Commentary assume responsibility for errors or oversights resulting from the use of the information contained herein. Anyone making use of the contents of this Commentary assumes all liability arising from such use. All suggestions for improvements of this Commentary will receive full consideration for future printings.

Introduction

Since the Canadian Standards Association introduced the first limit states design standard for structural steel, S16.1-1974 "Steel Structures for Buildings–Limit States Design" in 1974, the Standard has undergone a number of technical improvements, but its major requirements have remained virtually unchanged. However, with the introduction of the 1989 edition, a number of more significant changes were introduced, in part reflecting the maturing of the Standard but also the acquisition of more detailed information on behaviour of steel structures. Specific seismic design requirements for ductile behaviour were provided, and a new ductile system, eccentrically braced frame, was introduced. The 1994 edition continued this process with the refining of some requirements and the addition of a new lateral load-resisting system, the plate wall. The 2001 edition of the Standard was reorganized in a more logical order. In addition, the Standard underwent a number of technical improvements reflecting the incorporation of research results, including a significant expansion of Clause 27, Seismic Design Requirements.

When CSA S16-09 arrived, 35 years after the birth of its first limit states design version, S16.1-1974 and 25 years after the official withdrawal of its last allowable stress design version, S16-1969, the title of the Standard was shortened to read "Design of Steel Structures". In S16-09, two Annexes were added: Annex K, a normative annex that outlines the requirements for structural design for fire conditions and Annex L, an informative annex that provides information on design against brittle fracture.

Notable changes and new provisions incorporated in CSA S16-14 are: explicit recognition of several ASTM structural steel grades, use of non-matching weld electrodes as permitted in W59-13, an optional approach for design of ductile connections in moment-resisting frames, design rules for modular links in eccentrically braced frames and Annex M, an informative annex on seismic design of industrial steel structures. Some specific changes introduced in CSA S16-14 are highlighted in the Preface of the Standard itself.

Background

To serve their intended purposes, all structures must meet the requirement that the probability of occurrence of various types of collapse or unserviceability be limited to a sufficiently small value. Limit states are those conditions of the structure corresponding to the onset of the various types of collapse or unserviceability. The conditions associated with collapse are the ultimate limit states (ULS); those associated with unserviceability are the serviceability limit states (SLS), and that associated with fatigue is the fatigue limit state (FLS).

In limit states design, the capacity or performance of the structure or its components is checked against the various limit states at certain load levels. For the ultimate limit states of strength and stability, for example, the structure must retain its load-carrying capacity up to factored load levels (at the ultimate limit states), with only an acceptably small probability of being exceeded. (A factored load is the product of a specified load and its load factor.) For serviceability limit states, the performance of the structure at service load levels must be satisfactory. (For most applications, the service loads consist of dead load and one variable load only; thus the service loads are the specified loads. The National Building Code of Canada (NBCC) provides guidance on the use of companion load factors for situations where a companion loads should also be accounted for.) Examples of the serviceability requirement include prevention of damage to non-structural elements and restrictions on deflections, permanent deformations, slip in slip-critical connections, and acceleration under vibratory motion. For the fatigue limit state, the stress ranges for critical elements due to the loads applied to the structure over its useful life must not exceed the prescribed stress ranges.

The loads acting on a structure as well as the resistance of a member can only be defined statistically. When considering the ULS, a load factor (α) is applied to the specified load to take into account the fact that loads have a statistical distribution and that loads higher than those anticipated may exist, and also to take into account approximations in the analysis of the load effects. A resistance factor (ϕ) is applied to the nominal member (or component) strengths, or resistances (R), to take into account that the resistance of the member due to variability of the material properties, dimensions, and workmanship may be different than anticipated, and also to take into account the type of failure and uncertainty in the prediction of the resistance. A major advantage, therefore, of limit states design is that the factors assigned to loads arising from different sources can be related to the uncertainty of their prediction, and the factors assigned to different members and components can be related to their reliability and to the different types of failure. Thus, a greater degree of consistency against failure can be obtained (Kennedy 1974; Allen 1975; Kennedy *et al.* 1976).

For the failure of structural steel members by yielding, the resistance factor is taken to be 0.90 (Kennedy and Gad Aly 1980). To maintain simplicity in design, the resistance formulas for buckling or other types of member failure have been adjusted so that a uniform resistance factor, $\phi = 0.90$, can be used, while providing the necessary safety required in the definition of the resistance factor. Several failure modes and applications justify the use of smaller ϕ-values. For example, smaller values for weld metals and bolts have been adopted in the Standard in order to promote a lower probability of failure for connectors.

Probabilistic studies (Allen 1975) show that consistent probabilities of failure are determined for all dead-to-live load ratios when a dead load factor of 1.25 and a live load factor of 1.50 for use and occupancy loads are used. The NBCC gives load factors for environmental loads such as those due to snow and rain, wind, and earthquakes. As well, importance factors are applied to building structures depending on their use and occupancy (building importance category), with the highest factors applied to post-disaster structures. For certain types of structures, if there is a high degree of uncertainty in the loads, the designer may elect to use larger load factors. However, in situations where the dead load and the live loads are counteractive, it is important that α_D be taken as 0.9 or less, as appropriate, except that when dead load counteracts earthquake effects, α_D is taken as 1.0 or less.

Kennedy (1974) and Allen (1975) provide considerably more information on the type of probabilistic, calibration, and design studies that were performed while developing the limit states standard. The NBCC 2015 contains a more extensive discussion on limit states design. Kennedy and Gad Aly (1980) and Baker and Kennedy (1984) provide information on the statistical determination of the resistance factors (ϕ).

In the Commentary clauses that follow, the numbers and headings refer to the relevant clause numbers and headings of Canadian Standards Association (CSA) Standard S16-14. This will be referred to simply as S16-14 herein and after.

1. SCOPE AND APPLICATION

This Standard applies generally to steel structures and structural steel components in other structures. The analysis, design, detailing, fabrication, and erection requirements contained in the Standard normally provide a satisfactory level of structural integrity for most steel structures.

Clause 1.2 states that requirements for some specific types of structures and members are given in other CSA Standards. Situations where additional requirements may be necessary are given in Clause 1.3. The Structural Commentaries to the National Building Code of Canada provide references to the technical literature on the topic of structural integrity.

Clause 1.3 describes types of structures that may need supplementary rules for design. Crane-supporting structures are included in this list.

Clause 1.4 prohibits the substitution of any other structural steel design standard (e.g. CSA S16-1969 or AISC) for S16-14. The treatment of a number of important technical issues relating to safety, such as notional loads, beam-columns, ductility of members, and connections for earthquake loads, is either not covered or is treated in a manner inconsistent with the intent of S16-14 or the NBCC.

Clause 1.4 permits the designer (subject to approval from the Regulatory Authority) to supplement the formulas given in the Standard by a rational method of design. It is required that the structural reliability provided by the alternative (as measured by the reliability index, for example) be equal to, or greater, than those in the Standard. An example of such a rational method would be the design of stub-girders using the method set out by Chien and Ritchie (1984) based on tests (Bjorhovde and Zimmerman 1980, Kullman and Hosain 1985, Ahmad *et al.* 1990). Since structural design is an inextricable part of the design-construction sequence, substitution of other standards or criteria for fabrication, erection, inspection or any combination thereof, unless specifically directed by S16-14, is prohibited.

2. REFERENCE PUBLICATIONS

The Standards listed are the latest editions at the time of printing. When reference is made to undated publications in specific clauses of this Standard, it is intended that the latest edition and revisions of these publications be used. Two new Standards have been added.

3. DEFINITIONS AND SYMBOLS

In Clauses 3.1 and 3.2, new definitions and symbols have been introduced.

3.3 Units

All coefficients appearing in equations and expressions in this Standard are consistent with forces measured in Newtons and lengths in millimetres. While most coefficients are themselves non-dimensional, in Clause 17.9.10, the coefficient 2.76 has units of megapascals (MPa).

4. STRUCTURAL DOCUMENTS

4.1 General

The title "Structural Documents" reflects the fact that drawings are only part of a broadened range of structural documents that are currently used in the industry.

4.2 Structural Design Documents

4.2.1 Structural steel design documents, by themselves, should show all member designations, axis orientations, and dimensions needed to describe the complete steel structure. It should not be necessary, in order to ascertain information on structural steel components, to refer to documents produced for the use of other trades, as in some situations the fabricator may not be given, or have access to, the documents produced for other trades.

4.2.2 This list gives the minimum information to be included on the structural design documents in a logical order (much of it, no doubt, on Drawing S-1). By serving as a checklist, it will help insure that all the information the fabricator needs is provided and will help resolve disputes before they arise.

4.2.2(l) The development of adequate connections for structural members requires that the design engineer determine the shears, moments, and axial forces resulting from the governing load combinations for which the connection must be designed. For complex combinations, a useful presentation of this information may be to list the maximum value of each (e.g. shear, moment, and axial force), along with the values of the others which coincide with that maximum. The principle is to provide coexistent sets of forces so that free body diagrams can be identified to ensure that governing forces are transmitted through connections and panels.

4.2.2(m) Structural stability, a fundamental consideration of design, extends to the behaviour of elements within a member as well as to the functioning of members in total. Stabilizing components are needed to achieve both the correct local behaviour and the correct overall behaviour anticipated by the design. Therefore, the design engineer must define bracing, stiffeners, and reinforcement that are required to prevent failure due to instability. An example is web reinforcement in moment connections to prevent local instability. It may actually be more economical to use a heavier section and avoid the need for stiffeners or reinforcing detail material. This option should best be considered at the design stage.

4.2.3 The importance of proper recording of revisions on design documents, whether electronic files or paper, is emphasized. Control of documents is addressed in Steel Fabrication Quality Systems Guideline (CISC 2002) and in the CISC Code of Standard Practice, Appendix J, in Part 7 of this Handbook.

4.2.4 Architectural, electrical, and mechanical documents may be used for supplementary information, provided that the requirements in Clauses 4.2.1 and 4.2.2 for structural steel are shown on the structural documents.

4.3 Fabrication and Erection Documents

Although five types of documents are identified in the Standard, many structures which use pre-engineered connections from company or industry sources require only *shop details* and *erection diagrams*.

4.3.1 Connection Design Details

Connection design details, which often take the form of design brief sheets, typically show the configuration and details of nonstandard connections developed for specific situations. They are submitted to the design engineer for review to confirm that the structural intent has been understood and met, and they may be stamped by a professional engineer when appropriate. Drafting technicians use connection design details to prepare shop details.

4.3.2 Shop Details

Shop details frequently take the form of traditional shop drawings and are used to provide the fabrication shop with all the specific information required to produce the member. They are submitted to the design engineer for review to confirm that the structural intent has been understood and met. *Shop details* are not stamped by a professional engineer because they generally do not contain original engineering.

4.3.3 Erection Diagrams

Erection diagrams convey information about the permanent structure that is required by field personnel in order to assemble it. They are submitted to the design engineer for review, but are not stamped by a professional engineer because original engineering is generally not added by the fabricator.

4.3.4 Erection Procedures

Erection procedures outline methods and equipment, such as falsework and temporary guying cables, employed by the steel erector to assemble the structure safely. They may be submitted to the design engineer for review and may be stamped by a professional engineer when appropriate.

4.3.5 Field Work Details

Field work details are drawings which describe modifications required to fabricate members. The work may be done either in the shop or at the job site depending on circumstances. When extra material is involved, *field work details* effectively become *shop details*. They are submitted to the design engineer for review.

5. MATERIAL – STANDARDS AND IDENTIFICATION

The design requirements have been developed on the assumption that the materials and products that will be used are those listed in Clause 5. These materials and products are all covered by standards prepared by the Canadian Standards Association (CSA) or the American Society for Testing and Materials (ASTM).

The standards listed provide controls over manufacture and delivery of the materials, and products that are necessary to ensure that the materials and products will have the characteristics assumed when the design provisions of S16 were prepared. The use of materials and products other than those listed is permitted, provided that approval, based on published specifications, is obtained. In this case, designers should assure themselves that materials and products have the characteristics required to perform satisfactorily in the structure. In particular, ductility is often as important as the strength of the material. Weldability and toughness may also be required in many structures.

The values for yield and tensile strength reported on mill test reports are not to be used for design. Only the specified minimum values published in product standards and specifications may be used. This requirement was implicit in earlier editions of the Standard by definition of the terms F_y and F_u but was made explicit in more recent editions. Furthermore, when tests are done to identify steel, the specified minimum values of the steel, once classified, shall be used as the basis for design.

When, however, sufficient representative tests are done on the steel of an existing structure to be statistically significant, those statistical data on the variation of the material and geometric properties may be combined with that for test/predicted ratios available in the literature to develop appropriate resistance factors. This is by no means equivalent, for example, to substituting a new mean yield stress for a specified minimum value as the new reference value, and the bias coefficient must be established. It could well be that, although a higher mean value of the yield stress is established, the bias coefficient, depending as it does on the reference value, would be less. It would be expected that the coefficient of variation for the material properties in particular, derived for the steel in a single structure, would be less than for steel in general.

In Clause 5.1.3, both CSA and ASTM are referenced standards for structural steel. Because W-shapes are no longer produced by Canadian mills, mill test certificates will more often refer to ASTM A992/A992M or to ASTM A572/A572M. While ASTM A572 Grade 50 is comparable to G40.21 350W, ASTM A992 is a more restrictive version of A572 Grade 50 as it was developed specifically for seismic-resistant structures, but has become the most popular grade of wide-flange products available in North America. ASTM A913/A913M grades have been added to this Clause in S16-14. While Grade 65 (450) products are usually specified, Grade 70 (485) and Grade 50 (345) W-shapes are also produced.

The Standard requires that the design properties for ASTM A500/A500M HSS be determined from a wall thickness equal to 90% of the nominal wall thickness to account for the 10% under-tolerance for thickness permitted in ASTM A500/A500M and the lack of under-mass restriction. This requirement is consistent with the practice adopted in the CISC Handbook of Steel Construction. Grade C is the dominant grade for A500 HSS in Canada. ASTM A1085, a standard introduced in 2013, covers HSS that are produced to conform to a minimum average Charpy V-notch impact value. Other specific requirements include maximum yield stress and minimum corner radius controls. At the preparation time of this Commentary, users are advised to confirm availability prior to specifying A1085 HSS.

In Clause 5.1.7, ASTM Standards for bolts and bolt assemblies are referenced. ASTM F3125, a consolidation and replacement of six standards, A325, A325M, A490, A490M, F1852, and F2280, was published in January 2015. Since the name of each bolt standard becomes a bolt grade in this "umbrella" standard, F3125 (e.g. A490 becomes F3125 Grade A490), a seamless transition is anticipated.

6. DESIGN REQUIREMENTS

This clause clearly distinguishes between those requirements that must be checked using specified loads (the fatigue and serviceability limit states) and those which must be checked using factored loads (the ultimate limit states). Many of the serviceability requirements (deflections, vibrations, etc.) are stipulated qualitatively and guidance, in quantitative form, is provided in Annexes. Thus, the designer is permitted to use the best information available in order to satisfy the serviceability requirements, but is also provided with information that the Technical Committee on Steel Structures considers to be generally suitable, when used with competent engineering judgement.

6.1 General

6.1.2 Structural Integrity

A clause on structural integrity acts as a reminder that measures may be necessary to guard against progressive collapse as a result of a local incident. Being inherently ductile, steel structures have generally had an excellent record of behaviour when subjected to unusual or unexpected loadings. However, connection details are particularly important in achieving this ductile behaviour. Details which rely solely on friction due to gravity to provide nominal lateral force resistance may have little or no resistance to unanticipated lateral loads if subjected to abnormal uplift conditions and should be carefully evaluated for such an eventuality or completely avoided.

6.2 Loads

Dead loads are to include the additional mass of construction materials that will be built into a structure as a result of deflections of supporting members, such as a concrete floor slab placed to a level plane but supported by members that were not cambered and that deflect under the weight of the concrete.

6.3 Requirements Under Specified Loads

6.3.1 Deflection

6.3.1.2 Even though deflections are checked under the actions of specified loads, additional loading may result from ponding of rain on roofs, or the ponding of finishes or concrete, while in the fluid state, on floors or roofs. Such additional loads are to be included in the design of the supporting members under ultimate limit states as required by Clause 7. More information on ponding is available in the National Building Code of Canada (NBCC 2015).

6.3.3 Dynamic Effects

6.3.3.2 Additional information on vibrations of floor systems may be found in Allen (1974), Murray (1975), Allen and Rainer (1976), Rainer (1980), Allen *et al.* (1985), Allen and Murray (1993), Murray *et al.* (1997).

6.7 Requirements Under Fire Conditions

Background information on S16-14 Annex K, Structural Design for Fire Conditions, is available on the CISC Fire Protection webpage:

www.cisc-icca.ca/CommentaryS16AnnexK

6.8 Brittle Fracture

Annex L of S16-14 provides some design information to prevent failure of steel structures by brittle fracture. Annex L identifies the circumstances under which brittle fracture can occur and situations where brittle fracture should be considered as part of the design process. Annex L serves as a non-mandatory guide.

7. FACTORED LOADS AND SAFETY CRITERION

This clause sets forth the fundamental safety criterion (strength and stability) that must be met, namely:

Factored Resistance ≥ Effect of Factored Loads,

or

$$\phi R \geq \sum \alpha_i S_i$$

The factored resistance is given by the product ϕR where ϕ is the resistance factor and R is the nominal member strength, or resistance. The resistance factors of various types of members are given in Clause 13.1.

8. ANALYSIS OF STRUCTURE

Three types of construction are recognized, namely, "rigidly connected and continuous", "simple", and "semi-rigid (or partially restrained)". While semi-rigid construction was developed in the 1930's and 1940's, both in the USA and in the UK, and was previously a successful practice, it is now not in common use in North America.

With semi-rigid connections, because the angles between connected parts change under applied bending moments, the joint behaviour is non-linear and the moment/rotation response must be established by test, although many connection configurations have been tested and their moment/rotation responses have been compiled (Chen *et al.* 2011, Faella *et al.* 2000). Design of a semi-rigidly connected structure must take into account the effect of the "semi-rigid" connection stiffness on the stability of the structure. A second-order analysis is preferred because the non-linearities due to connection response and due to frame drift need to be assessed.

It is assumed that, if the connection has adequate capacity for inelastic rotation when subjected to the first application of factored gravity and lateral loading, under subsequent loading cycles the connection will behave elastically, although it will have a permanent inelastic deformation (Sourochnikoff 1950, Disque 1964). Such an assumption is valid except in joints where load fluctuation would create alternating plasticity in the connection (Popov and Pinkney 1969). With this form of construction, it is also important to consider the possibility of low-cycle, high-strain fatigue.

The use of open-web steel joists as connected members of these frames has been shown to be inadequate (Nixon 1981).

Clause 8 also permits the use of the two general methods of analysis – elastic and plastic analysis. Methods of elastic analysis are familiar to most designers.

8.3.2 Plastic Analysis

The use of plastic analyses at the factored load levels to determine the forces and moments throughout a structure implies that the structure achieves its limiting load capacity when sufficient plastic hinges have developed to transform the frame into a mechanism. As successive plastic hinges form, the load-carrying capacity of the structure increases above that corresponding to the formation of the initial plastic hinge until a mechanism develops. To achieve this, the members in which the hinges form before the mechanism develops must be sufficiently stocky (Class 1 sections) and well braced so that inelastic rotations can occur without loss of moment capacity.

Deflections at the specified load level are, of course, limited in accordance with Clause 6.3.1.1. Plastically designed structures are usually "elastic" at specified load levels, i.e. no plastic hinges have formed. Therefore, the deflections would generally be computed on the basis of an elastic analysis.

8.3.2(a) Material

The plastic method of analysis relies on certain basic assumptions for its validity (ASCE 1971). Therefore, restrictions are imposed to preserve the applicability of the plastic analysis theory. The basic restriction (Clause 8.3.2(a)) that the steel exhibit significant amounts of strain-hardening is required to ensure that satisfactory moment redistribution will occur (Adams and Galambos 1969). This behaviour should exist at the temperatures to which the structure will be subjected in service. Also, although not explicitly stated, plastically designed structures usually entail welded fabrication, and therefore the steel specified should also be weldable. At normal temperatures all the steels referred to in Clause 5.1.3 should be satisfactory except for CSA G40.21, 700 Q and 700 QT steels, for which $F_y > 0.85F_u$.

A reassessment of the stress-strain data (Dexter and Genticore 1997, Dexter *et al.* 2002) showed that the requirement that the yield strength not exceed 0.80 of the ultimate strength could be relaxed to 0.85 of the latter.

8.3.2(b) Width-to-Thickness Ratios

In order to preclude premature local buckling and thus achieve adequate hinge rotation to ensure sufficient moment redistribution to reach a plastic collapse mechanism, compression elements in regions of plastic moment must have width-to-thickness ratios no greater than those specified for Class 1 (plastic design) sections in Clause 11.2.

8.3.2(c) Lateral Bracing

The lateral bracing requirements are considerably more severe than those for structures designed on the basis of an elastic moment distribution because of the rotation needed at the location of the plastic hinges. Such requirements, as are needed to ensure adequate behaviour in earthquakes, are the basis for these new requirements. These equations were derived for non-cyclic plastic rotations of 3 and 4 times the elastic rotation at first yield following the procedure proposed by Bansal (1971) and summarized in Chapter 10 (Figure 10.27) of Bruneau *et al.* (1998). For traditional plastic design, case (a) is applicable, and to provide for the ductility demands implied for the three types of seismic moment frame categories, cases (a) or (b) are applicable as indicated. Test results on inelastic beams under moment gradient are reported by Lay and Galambos (1967).

Because the final hinge in the failure mechanism does not require rotation capacity, the bracing spacing limitations of this clause do not apply, and the elastic bracing requirements of Clause 13.6(a) may be used.

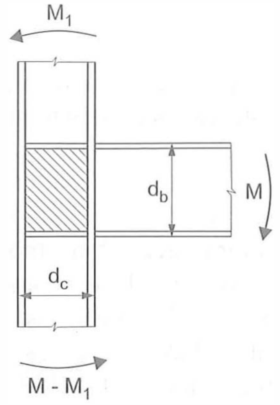
Lateral bracing is required to prevent both lateral movement and twisting at a braced point. Lateral bracing is usually provided by floor beams or purlins that frame into the beam to be braced. These bracing members must have adequate axial strength and axial stiffness to resist the tendency to lateral deflection. These requirements are given in Clause 9.2. Further information on the design of bracing members is given in Lay and Galambos (1966) and Chapter 12 of Ziemian (2010). When the bracing member is connected to the compression flange of the braced member, the brace should possess bending stiffness to resist twisting of the braced member. Some information on the bending stiffness of braces is given in Essa and Kennedy (1995).

A concrete slab in which the compression flange is embedded or to which the compression flange is mechanically connected, as in composite construction, or metal decks welded to the top flange of the beam in the positive moment region, generally provide sufficient restraint to lateral and torsional displacements. When the lateral brace is connected to the tension flange, provision must be made for maintaining the shape of the cross-section and for preventing lateral movement of the compression flange. This can be accomplished with either diagonal struts to the compression flange or adequately designed web stiffeners.

8.3.2(d) Web Crippling

Web stiffeners are required on a member at a point of load application where a plastic hinge would form. Stiffeners are also required at beam-to-column connections where the forces developed in the beam flanges would either cripple the column web or, in the case of tension forces, distort the column flange with incipient weld fracture. The rules for stiffener design are given in Clause 21.3 (Kennedy *et al.* 1998). See ASCE (1971) for further details of stiffeners and Fisher *et al.* (1963) for special requirements pertaining to tapered and curved haunches.

When the shear force is excessive, additional stiffening may be required to limit shear deformations. The capacity of an unreinforced web to resist shear is taken to be that related to an average shear yield stress based on the Huber-Henckey-von Mises criterion of $F_y/\sqrt{3}$. For an effective depth of the web of a rolled shape of about 95% of the section depth, Clause 13.4.2 gives:



WITHIN HATCHED ZONE

$$w_c \geq \frac{M}{0.8\phi\, d_c\, d_b\, F_s}$$

Figure 2-1
Web Thickness at Beam-to-Column Connections

$$V_r = 0.95\phi\, w d F_y / \sqrt{3} = 0.55\phi A_w F_y = 0.8\phi A_w F_s$$

At beam-to-column connections, when the shear force exceeds that permitted above, the excess may be carried by providing doubler plates to increase the web thickness or by providing diagonal stiffeners (Figure 2-1). The force in the beam flange that is transferred into the web as a shear is approximately

$$V = M / d_b$$

Equating this to the shear resistance as given in Clause 13.4.2 (where now, $w = w_c$ and $d = d_c$), and solving for the required web thickness,

$$w_c \geq \frac{M}{0.8\phi\, d_c\, d_b\, F_s}$$

If the actual web thickness is less than w_c, the required area of diagonal stiffeners may be obtained by considering the equilibrium of forces at the point where the top flange of the beam frames into the column. Using a lower bound approach, the total force to be transmitted ($V = M/d_b$) is assumed to be taken by the web and the horizontal component of the force in the diagonal stiffener:

$$V = M/d_b = 0.8\phi\, w_c d_c F_s + \phi F_y A_s \cos\theta$$

where

A_s = cross-sectional area of diagonal stiffeners

$\theta = \tan^{-1}(d_b/d_c)$

The required stiffener area is therefore

$$A_s = \frac{1}{\cos\theta}\left(\frac{M}{\phi F_y\, d_b} - \frac{0.8\, w_c d_c F_s}{F_y}\right)$$

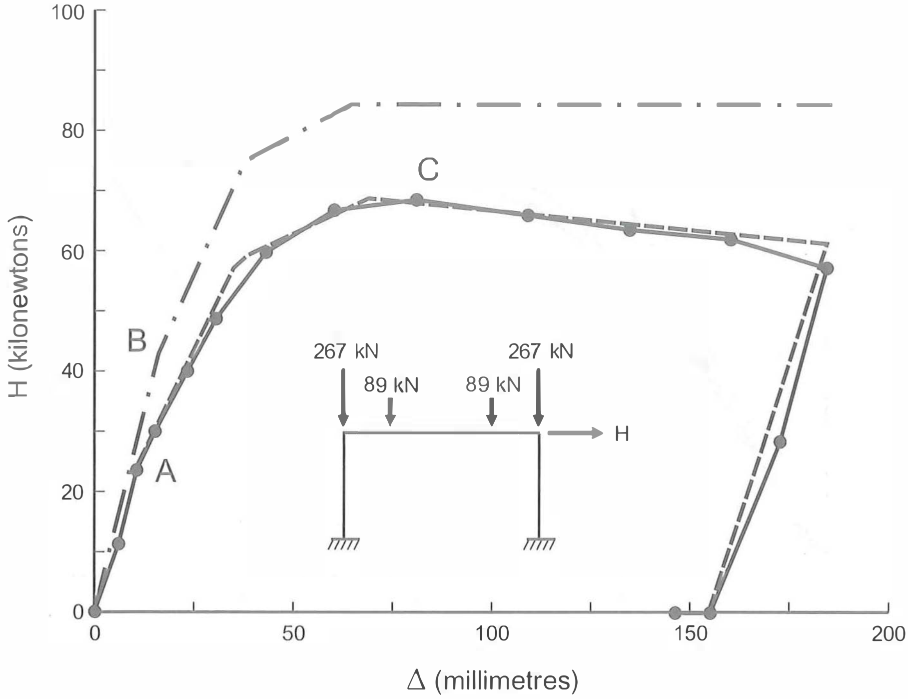


Figure 2-2
Observed and Predicted Load-Deflection Relationships

8.3.2(e) Splices

The bending moment diagram corresponding to the failure mechanism is the result of moment redistribution that occurred during the plastic hinging process. For example, points of inflection in the final bending moment distribution may have been required to resist significant moments to enable the failure mechanism to have developed (Hart and Milek 1965). To ensure that splices have sufficient capacity to enable the structure to reach its ultimate load capacity, a minimum connection requirement of $0.25\,M_p$ is specified in Clause 8.3.2(e). Also, at any splice location, the moments corresponding to various factored loading conditions must be increased by 10% above the computed value. The splice is then designed either for the larger of the moments so increased or for the minimum requirement of $0.25\,M_p$.

8.3.2(f) Impact and Fatigue

The use of moment redistribution to develop the strength of the structure corresponding to a failure mechanism implies ductile behaviour. Members that may be subjected repeatedly to heavy impact and members that may be subject to fatigue should not be designed on the basis of a plastic analysis because ductile behaviour cannot be anticipated under these conditions. Such members, at least for the present, are best proportioned on the basis of elastic bending moment distribution.

8.3.2(g) Inelastic Deformations

For continuous beams, inelastic deformations may have a negligible effect on the strength of the structure. For other types of structures, in particular multi-storey frames, these secondary effects may have a significant influence on the strength of the structure (ASCE 1971).

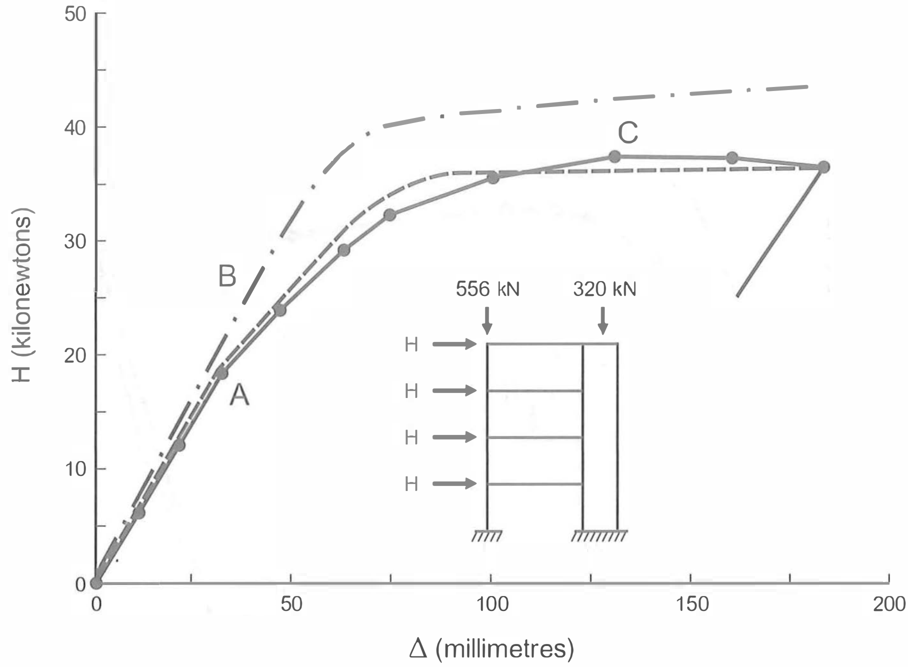


Figure 2-3
Load-Deflection Relationships

In the structure shown inset in Figure 2-2, the secondary effects have reduced the lateral load-carrying capacity (while maintaining the same vertical load) by approximately 25% (ASCE 1971; Adams 1974). The first plastic hinge formed at stage A in this structure, while the ultimate strength (considering moment redistribution) was not attained until stage C. The inelastic deformations between these two stages have reduced the overall strength of the structure. Clause 8.4 requires that the sway effects produced by the vertical loads be accounted for in design. Therefore, Clause 8.3.2(g) requires that, in a structure analyzed on the basis of a plastic moment distribution, the additional effects produced by inelastic sway deformations be accommodated. In most cases the actual strength of the structure can only be predicted by tracing the complete load-deflection relationship for the structure or for selected portions (Beedle *et al.* 1969). Methods are available to perform this type of design. For braced multi-storey frames, however, simpler techniques have also been developed (AISI 1968).

8.4 Stability Effects

Clause 8.4 recognizes that all building structures, whether unbraced or braced, are subjected to sway deformations. The vertical loads acting on the deformed structure produce secondary bending moments in the case of a moment-resisting frame, or additional forces in the vertical bracing system, in the case of a braced frame. These additional moments or forces (the stability effects) reduce the strength of the structure, as shown for a moment-resisting frame in Figure 2-2. In addition, bending moments and deflections, which exceed those predicted by a first-order analysis, are produced at all stages of loading (Adams 1974). Similar effects are produced in structures containing a vertical bracing system, as shown in Figure 2-3 where the steel frame is linked to a shear wall (Adams 1974).

8.4.1 Within the context of elastic analysis, there are essentially two general categories of procedures used to assess the stability of frames, namely, effective length approaches and notional load approaches. In S16.1-M89, the effective length approach in use prior to that time was abandoned because of the complexity involved in getting the approach to yield the correct solution. The notional lateral load approach makes use of the actual column length ($K = 1.0$) and was adopted in 1989. It has been used for the design of beam-columns in Canada since then (MacPhedran and Grondin, 2007).

The concept of notional lateral loads is an internationally recognized technique for transforming a sway buckling problem into a bending strength problem. It accounts for the effect of initial out-of-plumb in the columns and for partial yielding at factored load levels. Following the recommendation of Kennedy (1995), the notional load is applied to all design load combinations. Thus, the factored lateral force to be used in establishing the value of Δ at the various levels of the building is the summation of the applied lateral force and the notional load and the horizontal reaction to prevent sway from gravity loads. Since the notional loads are applied for the only purpose of accounting fully for the P-Δ effects on the overturning moment without the necessity of incorporating the initial out-of-plumb and inelastic effects in the analysis of the structure, they do not need to be considered for shear design. These notional shear forces do not exist when equilibrium of the structure is considered on the structure in its deformed configuration.

The magnitude of the notional lateral load, applied at each storey, is taken as 0.005 times the sum of the factored gravity loads contributed by that storey. While there is variation in international standards regarding the magnitude of the notional load coefficient (Bridge *et al.*, 1997), Clarke and Bridge (1992, 1995) have shown that $0.005\Sigma P$, established conservatively for a flagpole column (Kennedy *et al.*, 1990b), is an appropriate value that results in an adequate prediction of strengths in comparison with "exact" plastic zone analyses (Kanchanalai, 1977). There may be, as stated above, some conservatism in applying this magnitude of notional load to all load combinations in buildings where double-curvature bending of the columns predominates.

The use of the notional lateral load fulfills several important functions. The applied notional loads transform a bifurcation problem of sway buckling into a bending strength problem. Second, because it accounts for the P-Δ moments directly, the use of effective length factors greater than 1.0 is obviated, and its use allows effective lengths equal to the actual length to be used. At best the effective lengths used for sway buckling analyses are based on elastic analyses that are not appropriate for use with beam-column interaction equations that take into account inelastic material behaviour. Third, when equilibrium is formulated including the notional loads, the girders and beams restraining the columns are designed for the increased P-Δ moments that must exist in them for equilibrium just as the columns are. The use of effective lengths only accounts for increased moments in the columns and then only in an approximate manner with assumed elastic behaviour. Thus, although there may be some slight conservatism in using a notional load of $0.005\Sigma P$ compared to a lesser value, this is more than offset by the three advantages enumerated above.

It is noted that the flagpole column is bent in single curvature, whereas many columns in actual structures have some degree of double curvature. Consider now a sway column with complete fixity at both ends. It has very significant double curvature and an effective length of L. The sway buckling strength is now equal to the bending strength of a pin-ended column of the actual length with no notional lateral load because the effective length for buckling is equal to the actual length, L. These two cases show that the notional load required to transform the bifurcation problem of sway buckling into a bending strength problem depends on the end conditions in the actual structure and is greater when the degree of restraint is less. On the average, therefore, the notional load should be less than $0.005\Sigma P$, but Clarke and Bridge (1992, 1995) deem it to be the appropriate value.

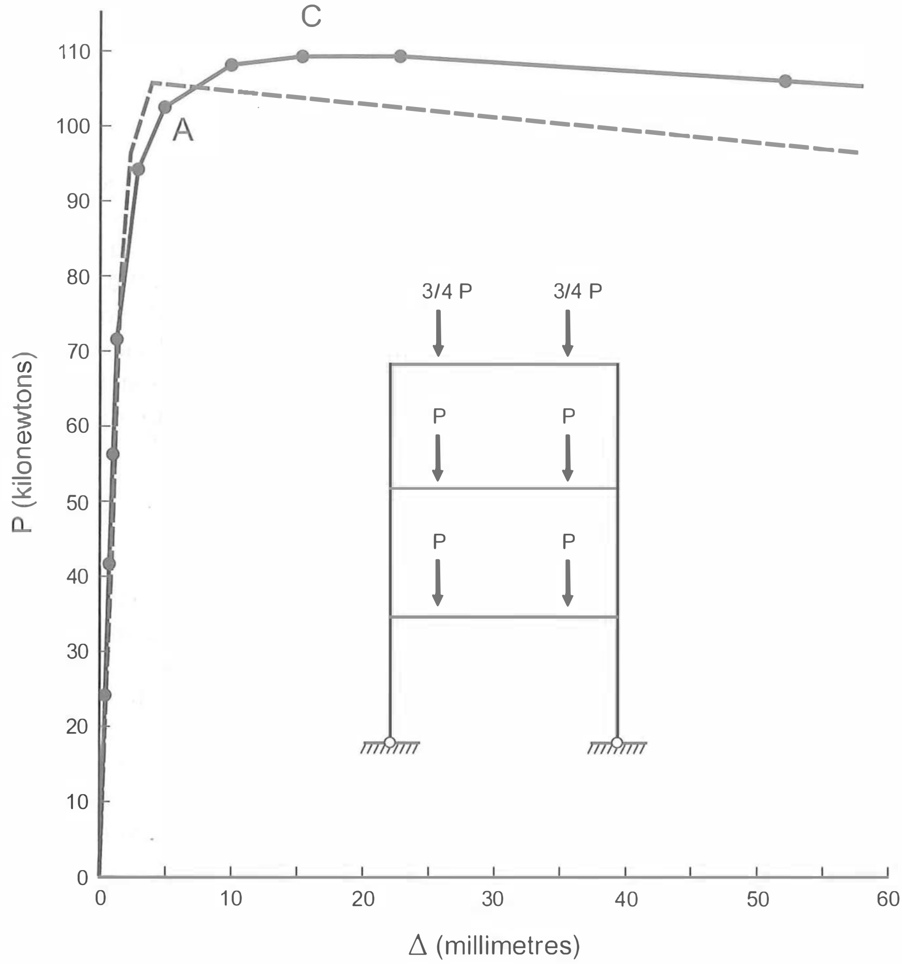
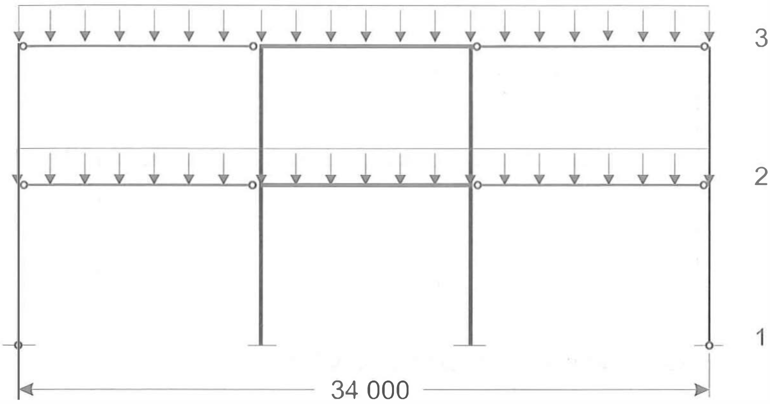


Figure 2-4
Load-Deflection Relationship – Vertical Load Only

The use of the notional lateral load remains of particular importance for structures subject to gravity loads only that may have insignificant lateral deflections and may only fail by elastic or inelastic sway buckling. Figure 2-4 shows a frame subject to vertical loads only. As the loads are increased, the effects of the vertical loads acting on the initial imperfections resulting from fabrication and erection lead to failure through instability, much the same as for the combined load case shown in Figures 2-2 and 2-3. The notional lateral loads of 0.005 times the factored gravity loads acting at each storey, as required by clause 8.4.1, simulate this condition. Figure 2-5 shows, for a frame loaded with gravity loads only, the notional lateral loads that would be used to calculate the translational moments and forces for this load combination.

When either the gravity loads or the structure or both are asymmetric, horizontal reactions at floor levels are obtained when computing M_{fg}, defined as the first-order moment under factored gravity loads determined assuming that there is no lateral translation of the frame as shown in Figure 2-6. These horizontal reactions, when released by applying sway forces in the opposite direction, produce translational effects and must be considered for all valid load combinations, in addition to the notional lateral loads or the actual lateral loads as appropriate.

8.4.2 Since the introduction of S16.1-M89, the designer must account for the sway effects directly. This is done by (1) performing a second-order geometric elastic analysis for the moments



Loads	Specified		Factored Gravity			Notional Lateral Load
	DL	LL	DL	LL	Total	
Level 3	10.8	22.5	13.5	33.75	47.25	0.005(47.25 x 34) = 8.03 kN
Level 2	18.0	18.0	22.5	27.00	49.50	0.005(49.5 x 34) = 8.42 kN

Note: For complete analysis of this frame, see Kennedy, *et al*, 1990.

Figure 2-5
Notional Lateral Loads for a Frame Subject to Gravity Loads

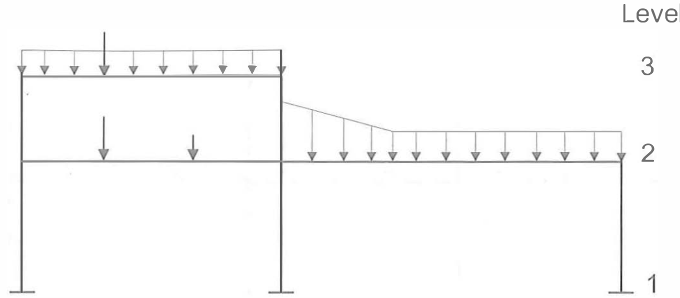
and forces, or (2) accounting for these effects by amplifying the first-order elastic translational moments by the factor U_2. The notional lateral loads (discussed in Clause 8.4.1) must be included in both of the above methods of analysis.

Computer programs are now commonly available to perform second-order elastic analyses based on equilibrium of the deformed structure. With these types of programs, the additional moments or forces generated by the vertical loads acting on the displaced structure (the so-called P-Δ effect) are taken into account directly and this method of analysis is the preferred method in Clause 8.4.2. In addition, most second-order programs also account for the change in column stiffness, caused by their axial loads (Galambos 1968).

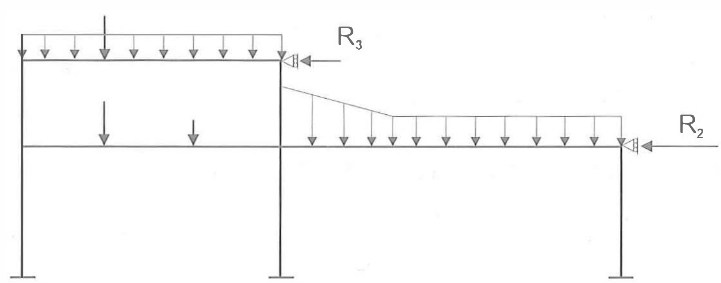
The second approach in Clause 8.4.2 is simply to amplify the results of a first-order analysis to include the P-Δ effects. With this "amplification factor method", it is necessary to do two first-order analyses, one for gravity loading and the other for translational loading. From the horizontal displacements produced by the factored lateral loads, the amplification factor U_2 may be established. The factored moments or forces, including the effects of side-sway, may then be computed from:

$$M_f = M_{fg} + U_2\, M_{ft} \quad \text{or from} \quad T_f = T_{fg} + U_2\, T_{ft}$$

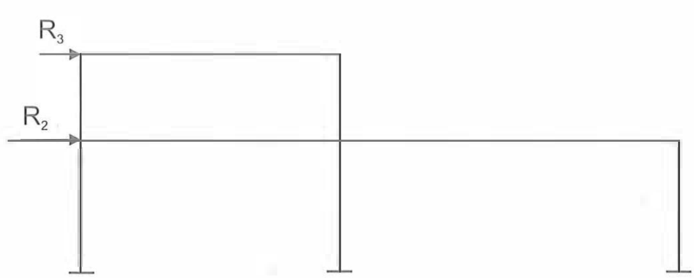
where $U_2 = \dfrac{1}{1 - \dfrac{\sum C_f \Delta_f}{\sum V_f h}}$



a) Asymmetrical frame with gravity loading



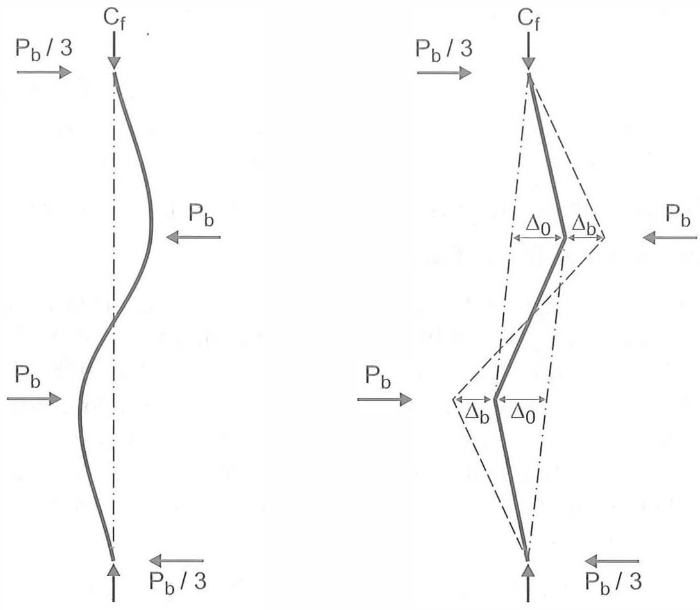
b) Computation of M_{fg}



c) Computation of M_{ft} and Δ_f

Figure 2-6

Starting with the 2001 Standard, the upper limit of 1.4 on the amplification factor U_2 was removed. The 1.4 limit was removed because the strength predictions for beam-columns compare well with the results of "exact" plastic zone finite element analyses when notional loads are applied to all load combinations. Nevertheless the designer is cautioned against designing structures that have excessive lateral deformations not only for the ultimate limit state of stability but also for serviceability considerations.



The solid line represents the initial misalignment. The dotted line represents the final displaced configuration due to all the forces acting on the system.

Figure 2-7
Δ_0 and Δ_b – Two Braces

9. STABILITY OF STRUCTURES AND MEMBERS

9.1 Stability of Structures

Emphasis continues to be placed on the designer's responsibility to ensure stability of the structure and of the individual members. Clause 8.4 requires the structure as a whole to resist the P-Δ effects.

The stability of the column-girder assembly and the girder web, when a girder is continuous over a column, requires careful assessment. The column, girder web, and the girder flange are all in compression, creating a condition of inherent instability. Stability can be achieved by providing lateral support to the girder-column joint or by properly designed web stiffeners restraining the rotation of the joint. See also the commentary on Clauses 16.5.11.1, 13.6 and the references cited therein.

9.2 Stability of Members

This clause applies equally to columns, the compression chord of joists and trusses, and the compressed portion of beams. For the last, it is only necessary to compute the maximum factored compressive force in that portion. The basic equation for the stiffness of the brace (Winter 1958) is derived on the premise that the brace or braces force the member to buckle into a series of half-sine waves of length, L, the distance between bracing points, with nodes at the bracing points. For this to occur, the braces must provide both strength and stiffness.

Most bracing assemblies in buildings have inherent torsional resistance. Normally, header connections provide sufficient torsional restraint at supports; however, Cheng *et al.* (1988), Cheng and Yura (1988) and Yura (1995) note that beams with deep or extended copes should be given special consideration.

Massey (1962) examined lateral bracing forces for beams, while Zuk (1956) and Lay and Galambos (1966) considered requirements for structures analysed plastically. Ziemian (2010) summarized many of the design requirements for bracing assemblies.

For additional discussion on member bracing, refer to Chapter 12 of Ziemian (2010).

9.2.1 Initial Misalignment at Brace Point

This requirement has changed in this edition of S16. Winter also showed that a critical parameter in designing the bracing is the initial out-of-straightness Δ_0 at the brace point. Based on S16 tolerances (Clause 29.3.3), a value for Δ_0 of no more than 0.001 times the distance between brace points may be used with Winter's model. A common construction technique used to reduce the initial misalignment is to pull the structure within tolerance at brace locations, bringing all column sections into compliance with the aforementioned Clause 29.3.3 for plumbness. Thus, when the structure is pulled into alignment, one brace point at a time, the Δ_0 that results is the erection tolerance.

Figure 2-7 shows the critical values of Δ_0 when two brace points exist.

9.2.2 Displacement of Bracing Systems

Δ_b is the displacement of the member being braced at the brace point perpendicular to the member caused by the force P_b and any other external forces. This deflection may be the result of axial shortening or elongation of the bracing or its flexural displacement depending on whether the bracing resistance is provided axially or by bending. In addition to the brace deformation, the brace connection deformation and the brace support displacement must be included.

The Simplified method of analysis is premised on a displacement Δ_b not greater than Δ_0, and therefore Δ_b shall not exceed Δ_0. When justified, this limit may be exceeded in either of the detailed methods.

In the case of girts bracing columns in the plane of the wall, the girts are the bracing members that deform. There could be deformation in the connections, and the shear deformation of the cladding is the displacement at the brace support.

In the case of a brace angle bracing the lower flange of a beam and connected to the upper flange of a secondary flexural member, the brace angle deforms axially; the connection to the brace angle may deform and the supporting secondary member may deflect flexurally to contribute to the deflection of the brace point perpendicular to the axis of the member.

When braces are supported by a truss system, the deformation of the truss between its points of support (assuming these are on the same line as the support of the members being braced) is the displacement of the brace supports. Figure 2-7 shows values of the displacements Δ_0 and Δ_b for a member braced at two locations.

9.2.4 Twisting and Lateral Displacements

The possibility of twisting of a member at brace points should be investigated and the bracing provided if necessary to prevent this.

The top (tension) flange at a cantilever, if not braced, can deflect laterally more than the bottom flange and therefore bracing of the cantilever end tension flange should be considered.

Torsional bracing can also increase the buckling load of cantilever beams.

The distortional buckling of steel beams in cantilever-suspended-span construction was examined by Albert *et al.* (1992), and Essa and Kennedy (1995) investigated torsional restraint stiffness provided by open-web steel joists. In this type of construction it is essential to analyze potential lateral displacements at the tops of supporting columns, because the beam web is also in vertical compression.

An inflection point cannot be considered a brace point (Ziemian 2010). Header connections normally provide sufficient torsional restraint at supports; however, Cheng *et al.* (1988), Cheng and Yura (1988) and Yura (1995) note that beams with deep or extended copes do not have similar torsional stiffness and should be given special consideration. This also applies for extended shear tab connections. Sherman and Ghorbanpoor (2002) indicate that bracing near the connection will compensate for the low torsional stiffness of the connection, and Thornton and Fortney (2011) give some design examples on calculating the torsional stiffness of an unbraced connection.

Simply supported beams in single curvature typically require only lateral bracing at the compression flange.

9.2.5 Simplified Analysis

The simplified analysis permitting a brace to be designed conservatively for a force equal to $0.02C_f$ has been reintroduced but with the qualification that the resulting deflection Δ_b shall not exceed the initial misalignment Δ_0, which is consistent with Winter's original provisions. As the long history of successful use has shown, this provides a brace of such strength that both the stiffness and strength requirements are generally satisfied.

9.2.6 Detailed Analysis

9.2.6.1 Second-Order Method

To begin this solution manually, a deformed configuration is assumed and bracing forces determined by statics in terms of the deflections. With these calculated forces, the resulting bracing deflections are computed and compared to the deformations initially assumed. The process is repeated until satisfactory convergence is achieved. In checking a design, if the calculated deformations are less than the assumed deformed configuration, the conditions of strength and stiffness are satisfied and there is no need for further calculations, unless further optimization is desired. Alternatively, brace forces and deformations can be obtained from a computerized second-order analysis that accounts for P-Δ effects provided that the structure is modelled with the most critical initial misalignment condition.

Iterations to determine the forces and deflections are performed on the most critical deformed configuration. Typical deformed configurations to be investigated include those shown on Figure 2-7. When hinges are assumed in the braced member at the brace points, a slightly conservative solution is obtained.

The second-order method is useful in checking as-built conditions.

9.2.6.2 Direct Method

The design brace force is given directly in the expression for P_b, where the factor β from Winter (1958) depends on the number of equally spaced braces by assuming that the displacement of the bracing system Δ_b is equal to the initial misalignment Δ_0. The required brace stiffness is $P_b/(\Delta_b + \Delta_0)$.

The initial assumption that Δ_b does not exceed Δ_0 must be confirmed.

By defining the maximum compressive force, C_f, as the maximum compression force in the segments bound by the brace points on either side of the brace point under consideration, the situation where a brace occurs near a point of contraflexure is accounted for. For trusses this applies when there is a significant change in the force in the chord at a panel point.

Consideration shall be made for cantilevered beams and beams bent in double curvature. Yura (1993) gives an amplification factor $C_d = [1 + (M_s/M_l)^2]$ where M_s and M_l refer to the smaller and larger moments, respectively. This yields a maximum value of 2 when $M_s = M_l$.

For loads applied above the shear centre, brace forces may be amplified. Yura (1993) gives a factor $C_d = (1 + 1.2/n)$ where n is the number of braces. Braces counterbalance this effect.

Calibration of the bracing requirements with finite element analyses suggest that in some cases twisting of chords in trusses may result in brace forces 25% higher than those predicted by the direct method.

9.2.8 Accumulation of Forces

When an element in a structure must resist the bracing forces from more than one member, the average maximum out-of-straightness of the members should be used to compute the bracing forces. Provided that member misalignment is independent among members, it can be shown statistically that the average maximum out-of-straightness is a function of the maximum out-of-straightness of one member divided by the square root of the number of members (Kennedy and Neville 1986). The expression given in the standard is a conservative empirical equation that applies the statistical reduction to only 0.80 of the initial misalignment. In the design of such bracing systems it must be recognized that the (axial) displacement of the in-line brace increases from the location where the brace is affixed or restrained to the most remote member, and the force in the in-line brace increases in the opposite direction. Beaulieu and Adams (1980) provide more guidance in selected cases.

In many cases two parallel frames or members are brought into alignment, and whatever misalignment remains is reflected in the initial position of the remaining members. The statistical reduction in the initial misalignment does not apply, and all members have essentially the same Δ_0.

9.2.9 Torsion

Because the shear centre of a monosymmetric or an asymmetric section does not coincide with the centroid, these sections may be loaded so as to (unintentionally) produce torsion and biaxial bending. Both the connections and the members providing reactions should be checked.

10. DESIGN LENGTHS AND SLENDERNESS RATIOS

10.1 Simple Span Flexural Members

For design purposes, it is usually convenient to consider the length of a member as equal to the distance between centres of gravity of supporting members. In most instances the difference resulting from considering a member to be that length rather than its actual length, centre-to-centre of end connections, is small. In some cases, however, there is sufficient difference to merit computing the actual length. Regardless of the length used for design, the actual connection detail may cause an eccentric load, or moment, to act on the supporting member, and this effect must be taken into account.

10.3 Members in Compression

10.3.1 General

The unbraced length and the effective length factors may be different for different axes of buckling. Information about effective lengths is given in Ziemian (2010) and Tall *et al.* (1974).

Further guidance is provided in Annexes F and G of the Standard. The second-to-last sentence of Clause 10.3.1 introduces the concept that effective length factors depend on the potential failure mode – how the member would fail if the forces (and moments) were increased sufficiently – as discussed in subsequent clauses.

10.3.2 Failure Mode Involving Bending In-Plane

When the end moments and forces acting on a beam-column have been determined for the displaced configuration of the structure, that is to say, the sway effects have been included as required by Clause 8.4, the in-plane bending strength of the beam-column can be determined by analyzing a free-body of the member isolated from the remainder of the structure. In-plane displacements between the ends, which contribute to failure, arise from the end-moments and forces acting on the actual length. When the actual member length and the actual (or at least approximate) deflected shape are used, the analysis of the free-body will yield close to the correct member strength. Recourse to effective length factors is neither necessary nor appropriate.

When the actual member length is used together with the interaction expressions of Clause 13.8, the analysis is approximate and the in-plane member bending strength obtained will tend to be conservative. This simply arises because the value of the compressive resistance inherent in the interaction expression by using a length equal to the actual length (a K factor of 1.00) is that corresponding to single curvature buckling. For any other deflected shape, having accounted for sway effects, the compressive resistance is greater because the points of inflection of the deflected member shape are less than the member length apart. Under these circumstances, a better estimate of the strength, as is indeed permitted under Clause 1.4, can be obtained when the compressive resistance is based on the actual distance between points of inflection. Inelastic action of the member in the structure, however, may make this determination onerous. Therefore the relatively simple but sometimes conservative approach given in the Standard which obviates the use of effective length factors is presented as the usual procedure.

10.3.3 Failure Mode Involving Buckling

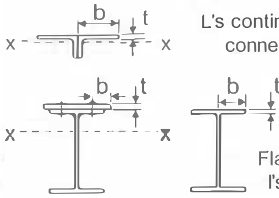
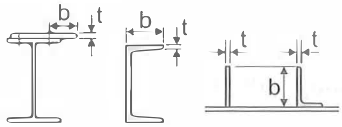
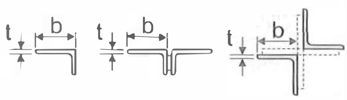
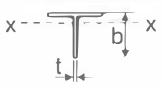
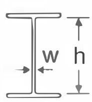
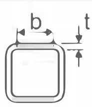
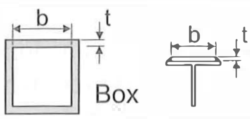
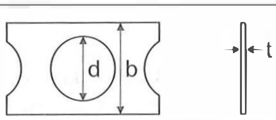
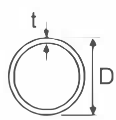
The compressive resistance of an axially loaded column depends on its end restraints, as does the out-of-plane buckling resistance of a beam-column under uniaxial strong-axis bending. The failure is a bifurcation mechanism.

10.4 Slenderness Ratios

The maximum slenderness ratio of 200 for compression members, stipulated as long ago as the 1974 Standard, has been retained in S16-14 for the reason that strength, or resistance, of a compression member becomes quite small as the slenderness ratio increases and the member becomes relatively inefficient.

For considerations of strength, no limiting slenderness ratio is required for a tension member and, indeed, none is applied to wire ropes and cables. However, a slenderness ratio limit of 300 is given with permission to waive this limit under specified conditions. The limit does assist in the handling of members and may help prevent flutter under oscillating loads such as those induced in wind bracing designed for tension loads only. Tension chords of trusses and joists have more stringent slenderness ratios (see commentary on Clauses 15 and 16).

Members whose design is governed by earthquake loadings may be subject to more stringent slenderness ratios, depending on the ductility requirements of the lateral load-resisting system. See Commentary on Clause 27.

Detail	Class 1	Class 2	Class 3
 L's continuously connected; Flanges of I's or T's	$\frac{b_{el}}{t} \leq \frac{145}{\sqrt{F_y}}$ †	$\frac{b_{el}}{t} \leq \frac{170}{\sqrt{F_y}}$ †	$\frac{b_{el}}{t} \leq \frac{200}{\sqrt{F_y}}$ Flanges of I's in minor-axis bending $\frac{b_{el}}{t} \leq \frac{340}{\sqrt{F_y}}$
	——	——	Flanges of C's, asymmetric cover plates, plate girder stiffeners $\frac{b_{el}}{t} \leq \frac{200}{\sqrt{F_y}}$
	——	——	L's not continuously connected $\frac{b_{el}}{t} \leq \frac{250}{\sqrt{F_y}}$
 Stems of T's	$\frac{b_{el}}{t} \leq \frac{145}{\sqrt{F_y}}$ †	$\frac{b_{el}}{t} \leq \frac{170}{\sqrt{F_y}}$ †	$\frac{b_{el}}{t} \leq \frac{340}{\sqrt{F_y}}$
	Bending only $\frac{h}{w} \leq \frac{1100}{\sqrt{F_y}}$ Axial compression ——	Bending only $\frac{h}{w} \leq \frac{1700}{\sqrt{F_y}}$ Axial compression ——	Bending only $\frac{h}{w} \leq \frac{1900}{\sqrt{F_y}}$ Axial compression $\frac{h}{w} \leq \frac{670}{\sqrt{F_y}}$
 HSS	$\frac{b_{el}}{t} \leq \frac{420}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \leq \frac{525}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \leq \frac{670}{\sqrt{F_y}}$
 Box	$\frac{b_{el}}{t} \leq \frac{525}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \leq \frac{525}{\sqrt{F_y}}$	$\frac{b_{el}}{t} \leq \frac{670}{\sqrt{F_y}}$
	——	——	$\frac{b_{el}}{t} \leq \frac{840}{\sqrt{F_y}}$
	Bending only $\frac{D}{t} \leq \frac{13\,000}{F_y}$ Axial compression ——	Bending only $\frac{D}{t} \leq \frac{18\,000}{F_y}$ Axial compression ——	Bending only $\frac{D}{t} \leq \frac{66\,000}{F_y}$ Axial compression $\frac{D}{t} \leq \frac{23\,000}{F_y}$

† Symmetric about plane of bending or including asymmetry effects in analysis

Figure 2-8
Width-to-Thickness Ratios for Compression Elements

11. WIDTH (OR DIAMETER)-TO-THICKNESS – ELEMENTS IN COMPRESSION

Clause 11 emphasizes the distinction between elements in axial compression and elements in flexural compression by placing the maximum width-to-thickness ratios for these elements in Tables 1 and 2 respectively.

Clause 11.1.1 identifies four categories of cross-sections, Class 1 through Class 4, based upon the width-thickness ratios of the elements of the cross-section in compression that are needed to develop the desired flexural behaviour. With the ratios given in Table 2 of Clause 11 for Classes 1, 2, or 3, the respective ultimate limit states will be attained prior to local buckling of the plate elements. These ultimate limit states are: Class 1 – maintenance of the plastic moment capacity (beams), or the plastic moment capacity reduced for the presence of axial load (beam-columns), through sufficient rotation to fulfill the assumption of plastic analysis; Class 2 – attainment of the plastic moment capacity for beams, and the reduced plastic moment capacity for beam-columns, but with no requirement for rotational capacity; Class 3 – attainment of the yield moment for beams, or the yield moment reduced for the presence of axial load for beam-columns. Class 4 – have plate elements that buckle locally before the yield strength is reached.

Elements in Flexural Compression

The requirements given in Figure 2-8 for elements of Class 1, 2, and 3 sections in flexural compression, particularly those for W-shapes, are based on both experimental and theoretical studies. For example, the limits on flanges have both a theoretical basis (Kulak and Grondin 2014; ASCE 1971; Ziemian 2010) and an extensive experimental background (Haaijer and Thurlimann 1958; Lay 1965; Lukey and Adams 1969). For webs in flexural compression the limits $1100/\sqrt{F_y}$, $1700/\sqrt{F_y}$ and $1900/\sqrt{F_y}$ for Class 1, 2 and 3, respectively, when $C_f/\phi C_y = 1.0$ come from both theory and tests on Class 1 sections (Haaijer and Thurlimann 1958) but mostly from test results for Class 2 and 3 sections (Holtz and Kulak 1973 and 1975).

For circular hollow sections in flexure, see Stelco (1973) for the requirements for Class 1 and Class 2 sections and Sherman and Tanavde (1984) for Class 3.

Elements in Axial Compression

The distinction between classes based on moment capacity does not apply to axially loaded members as the plate elements need only reach a strain sufficient for the plate elements to develop the yield stress. This strain is affected by the presence of residual stresses, but there is no applied strain gradient across elements of the cross-section as there is for members subject to flexure. The width-thickness limits for the various plate elements are not dependent on the Class of the section and are only a function of the residual stress pattern and the edge conditions. Thus for webs, from Table 2 in the Standard for each of Classes 1, 2 and 3 when $C_f/\phi C_y = 1.0$, the limit on h/w is the same value of about $670/\sqrt{F_y}$ as given in Table 1. The width-thickness limit for the flanges of axially loaded columns, based on the same argument, is the same as for Class 3 beam flanges, i.e., $200/\sqrt{F_y}$ (Dawe and Kulak 1984). As well the limit on the D/t ratio of $23\,000/F_y$ (Winter 1970) for circular hollow sections in axial compression is the same irrespective of the Class.

Elements in Compression Due to Bending and Axial Load

In Figure 2-9, the requirements for webs in compression ranging from compression due to pure bending to that due to pure compression are plotted. Because all of the web is in compression for columns and only one-half for beams, the depth-to-thickness limits vary as a function of the amount of axial load. The results presented here reflect the research results of Dawe and

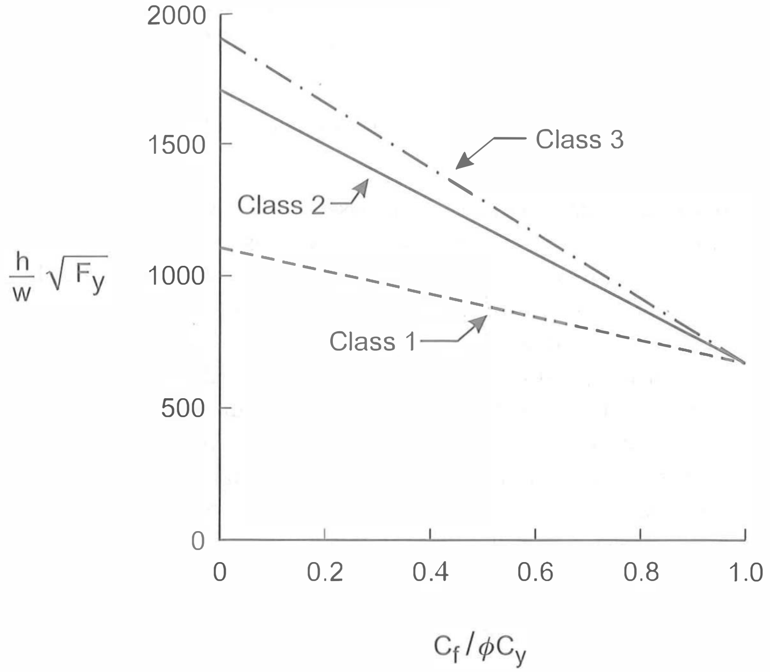


Figure 2-9
Width-to-Thickness Ratios for Webs

Kulak (1986), and are significantly more liberal than previous limits (Perlynn and Kulak 1974; Nash and Kulak 1976). The expressions for the variation of web h/w ratios have been corrected such that the maximum value of C_f that can be reached is ϕC_y.

Class 4 Sections

Sections used for columns, beams, or beam-columns may be composed of elements whose width-to-thickness ratios exceed those prescribed for Class 3 provided that the resistance equations are adjusted accordingly. These sections, called Class 4, are evaluated according to the rules given in Clause 13.3 or 13.5 as applicable.

12. GROSS AND NET AREAS

12.1 Application

The design and behaviour of tension members is integrally related to the proportioning and detailing of connections. Consequently, Clauses 12, 13.2 and 13.11 are related. Two possible overall failure modes exist: unrestricted plastic flow of the gross section and fracture of a net section. The second of these consists itself of three modes depending on the failure path and the degree of ductility available. The Commentary on Clause 13.2 discusses three specific tensile failure modes and that on Clause 13.11 treats combined tension and shear. Commentaries on failure areas are given here.

12.2 Gross Area

Yielding on the gross area from one end of the member to the other resulting in unrestricted plastic flow can occur before fracture on a net section. The gross area is obtained simply as the sum of the products of the thickness and gross widths of all cross-sectional elements.

12.3 Net Area

This clause defines areas used to determine tension member resistances. The requirements apply to both bolted and welded connections.

12.3.1 General

When each portion of the cross-section of a tension member is connected with sufficient fasteners to transmit the load attributable to that portion, the stress distribution at the connection is reasonably uniform, and the provisions of Clause 12.3.1 apply to the net area calculations. To establish the critical net area, all potential failure paths are examined. When the failure plane includes segments inclined to the applied force, an empirical term, $s^2t/4g$, is added to the net area to correct for the presence of each inclined segment.

In determining the net area by summing the net area of each segment along the critical path, it is assumed, as has been demonstrated (Birkemoe and Gilmor 1978; Ricles and Yura 1983; Hardash and Bjorhovde, 1985), that all segments reach their full capacity simultaneously.

12.3.2 Allowance for Bolt Holes

The 2 mm allowance for bolt holes accounts for distortion or local material damage that may occur in forming the hole by punching. If it is not known at the design stage that the holes will be drilled or sub-punched and reamed, then punched holes should be assumed. The 2 mm allowance also is used with oversize or slotted holes.

12.3.3 Effective Net Area - Shear Lag

When the critical net section fracture path crosses unconnected cross-sectional elements, the directly connected elements tend to reach their ultimate strength before the complete net section strength is reached due to shear lag. When all cross-sectional elements are directly connected, shear lag does not occur and the effective net area is the total net area.

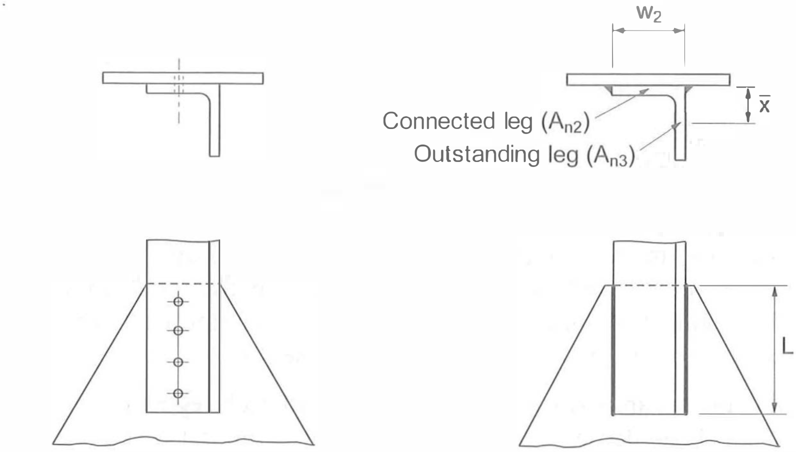
The loss in efficiency due to shear lag can be expressed as a reduction in the net area. Munse and Chesson (1963) suggested that this reduction could be taken as $1-\bar{x}/L$ where $\bar{x}$ is the distance from the shear plane to the centroid of that portion of the cross-section being developed and L is the connected length.

Because the connected length is usually not known at the time of tension member design, reduction factors have been derived for specific cases, as given in Clause 12.3.3.2, based on an extensive examination of the results of over 1000 tests (Kulak *et al.* 1987). The reduction factor depends on the cross-sectional shape and the number of bolts (2, 3 or more) in the direction of the tensile load.

More severe reductions for shear lag are provided for angles connected by one leg based on work by Wu and Kulak (1993), who examined many test results on angles in tension connected with mechanical fasteners.

When block tear-out occurs in those elements that are directly connected, shear lag is not a factor. Shear lag need only be considered when the potential failure path under consideration crosses unconnected elements.

12.3.3.3 Similar reductions due to shear lag have been observed in welded connections (Kulak *et al.* 1987) when only welds parallel to the tensile load in the member are used. If the elements of the cross-section are connected by welds transverse to the tensile load, no reduction due to shear lag is necessary. For welded connections with matching electrodes and material of G40.21-300W grade steel, shear lag will be critical for cases where $A_{ne} \leq 0.78\,A_g$. For angles, this generally occurs when the length of weld along the toe exceeds the length of weld along the heel.



Bolted angles with
4 or more transv. lines of bolts
$A_{ne} = 0.80A_n$

(a)

Welded angles
connected by one leg
$A_{ne} = A_{n2} + A_{n3}$

(b)

Figure 2-10
Dimensions Used for Shear Lag Calculations

When the weld length is less than the distance between welds, it is likely that the weld is critical.

Provisions for shear lag in bolted and welded angles are illustrated in Figure 2-10.

12.3.3.4 The shear lag expressions from past specifications have been expanded to specifically address slotted HSS brace end connections. This addition reflects the work of Martinez-Saucedo and Packer (2009) who demonstrated that a non-linear function best described the shear lag effect on a number of different slotted round and rectangular/square HSS connections. This function, designated U for the cross-sectional efficiency such that $A_{ne} = UA_n$, is plotted on Figure 2-11 and makes a smooth transition across the three limit states observed during testing: (1) yielding and necking, (2) net section fracture from shear lag effects, and (3) tube wall tear out from block shear. The new expression from clause 12.3.3.4 is seen to be a reasonably conservative linear approximation of the expression proposed by Martinez-Saucedo and Packer. The term was introduced to emphasize that the eccentricity that should be considered in the shear lag expressions should be measured from the face of the gusset.

12.4 Pin-Connected Members in Tension

The dimensional requirements presented in Figure 2-12 must be met to provide for the proper functioning of the pin.

The pin hole shall be located midway between the edges of the member in the direction normal to the applied force.

The plate shall be of uniform thickness. The width of the plate at the pin hole shall not be less than $2b_e + d$, and the clear end distance, a, beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than $1.33b_e$. The corners beyond the pin hole may be cut at 45° to the axis of the member, provided $c \geq a$ as shown in Figure 2-12.

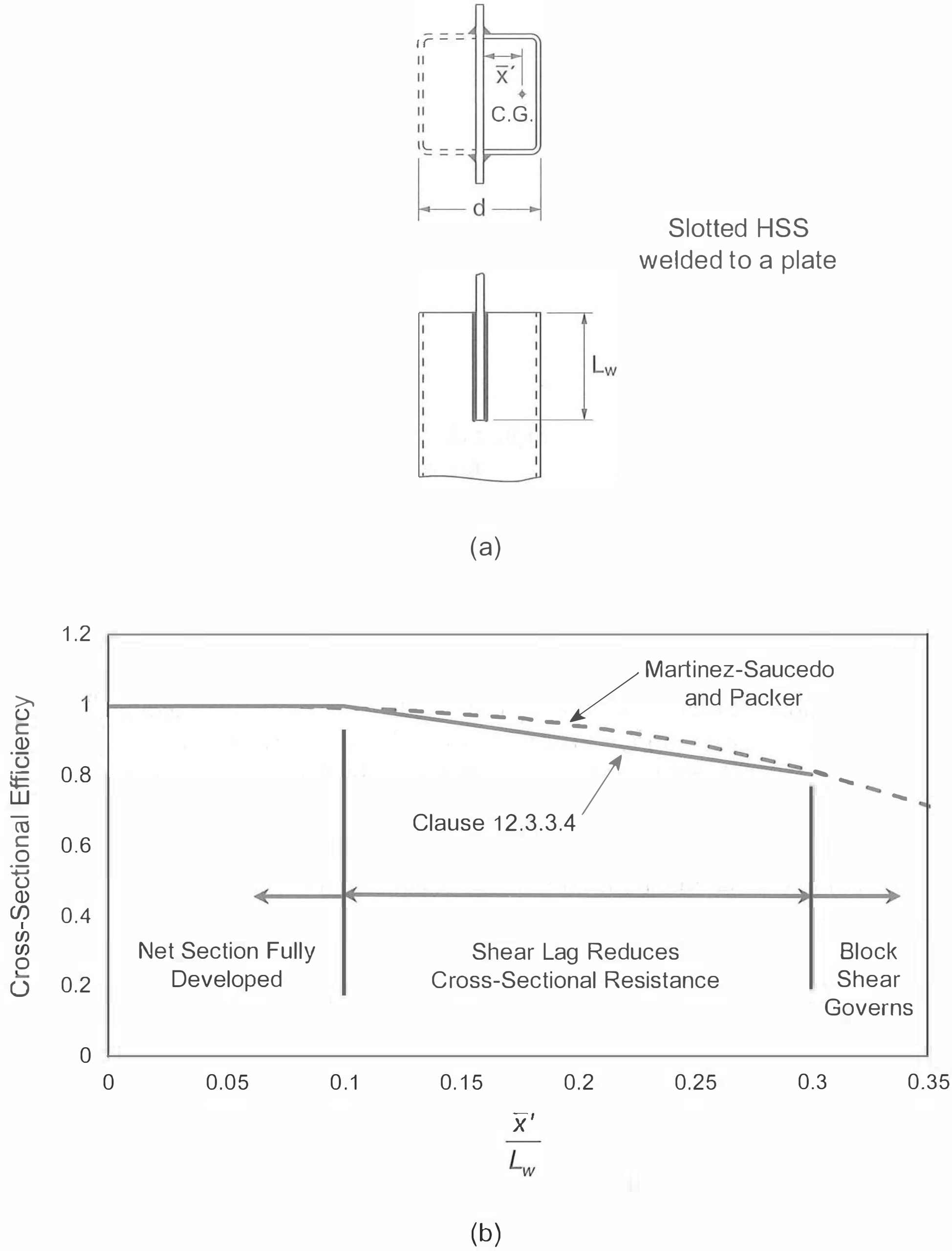
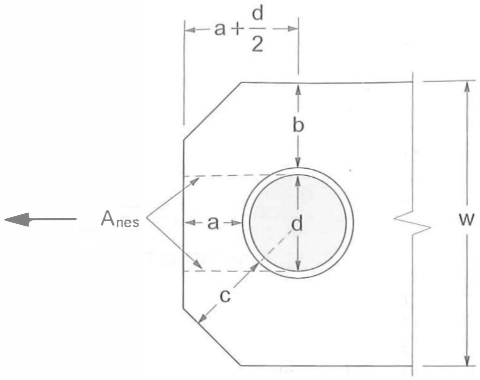


Figure 2-11
Shear Lag Effects on Slotted HSS Brace Ends

Pin-connected members are occasionally used as tension members with very large dead loads. Pin-connected members are not recommended when there is sufficient variation in live loading to cause wearing of the pins in the holes.



Dimensional Requirements

1. $a \geq 1.33 b_e$
2. $w \geq 2b_e + d$
3. $c \geq a$

where $b_e = 2t + 16$ mm $\leq b$

Figure 2-12
Dimensional Requirements for Pin-Connected Members

13. MEMBER AND CONNECTION RESISTANCE

13.1 Resistance Factors

For convenience, all resistance factors are listed in Clause 13.1. The long-used basic value of ϕ of 0.90 for most resistances continues to provide consistent and adequate values of the reliability index when used with the load factors of Clause 7.2. (Kennedy and Gad Aly 1980, Baker and Kennedy 1984, Schmidt and Bartlett 2002). In S16-14, no new resistance factors have been added to the group.

In S16-09, the resistance factor for bolt bearing on steel was increased from 0.67, adopted in a previous edition of the Standard, to 0.80. A recent reliability analysis demonstrated that a resistance factor of 0.80 provides an adequate margin of safety (Stankevicius *et al.* 2009). In earlier versions of S16, the resistance factor $\phi = 0.90$ was used in the equation for rupture of tension members at the net section. This resistance factor was multiplied by a factor of 0.85 to increase the safety index to about 4.0 to 4.5 for this ultimate limit state. Beginning in S16-09, 0.85ϕ has been replaced by $\phi_u = 0.75$, which is just slightly lower than 0.85ϕ. The resistance factor ϕ_c is 0.65, which is consistent with the reinforced concrete design standard A23.3.

13.2 Axial Tension

The two overall potential failure modes for tension members and their connections are yielding of the gross section and fracture of a net section. Fracture of a net section further consists of three possible modes depending on how the elements of the cross-section are connected and how the net sections are loaded. Thus all possible failure modes described must be examined to establish the value that governs the factored tensile resistance. The resistances of two of the fracture modes chiefly involving tension on the net section are presented in Clause 13.2. The failure mode involving a combination of tension and shear, in which a block of material tears out, is referred to as block shear failure and is discussed in the commentary to Clause 13.11.

The appropriate areas to be used in each of the three modes are described in Clause 12.

The first of the three failure modes involves unrestricted plastic flow of the gross section when the yield deformations over the length of the member are excessive. This represents a limit state for which the failure is gradual. A reliability index β of 3.0 is considered acceptable for the tension member and thus the tensile resistance is

$T_r = \phi A_g F_y$; (with $\phi = 0.90$)

The second failure mode, involving a combination of tension and shear, in which a block of material tears out – block shear failure – is discussed in the commentary on Clause 13.11.

The third failure mode involves fracture of the member at the net section. The net section area can either be fully effective if all parts of the cross-section are connected, or it can be only partially effective if shear lag is present. Because this fracture occurs with little deformation and no reserve of strength exists beyond rupture, an increased value of β is appropriate for cases of fracture at the net section. In S16-14 (since S16-09) the tensile resistance for this mode is written as:

$$T_r = \phi_u A_{ne} F_u$$

The resistance factor $\phi_u = 0.75$ used for this limit state results in an increased value of β of about 4.5. This philosophy is consistent with the reduced resistance factor used for connectors (bolts, welds, and shear connectors). The net effective area, A_{ne}, accounts for possible shear lag effect. If no shear lag is present, then $A_{ne} = A_n$.

Clause 13.2(b) applies to pin connections, except that more specific requirements apply to eyebars. The equation in (ii) gives the net section fracture resistance, whereas the equation in (iii) covers shear rupture or end tear-out.

13.2(b) Pin Connections

The tensile strength requirements for pin-connected members use the same resistance factor ϕ as elsewhere in this Standard for similar limit states. However, the definitions of effective net area for tension and shear, as given in Clause 12.4, are different. The requirements in this Clause have been adapted from ANSI/AISC 360-10. Design of eyebars requires more specific rules.

13.3 Axial Compression

Depending on the type of cross-section, the buckling load of an axially loaded compression member may be governed by flexural buckling, by torsional buckling or by flexural-torsional buckling.

13.3.1 Flexural Buckling of Doubly Symmetric Shapes

Axially loaded compression members with doubly-symmetric cross-sections, such as wide-flange shapes, I-shaped and HSS members that dominate in steel construction, normally reach their ultimate capacity either by yielding or by flexural buckling, the most common buckling mode.

Steel columns are conveniently classified as short, intermediate, or long members, and each category has an associated characteristic type of behaviour. A short column is one that can resist a load equal to the yield load ($C_y = A F_y$). A long column fails by elastic buckling. The maximum load depends only on the bending stiffness (EI) and length of the member. Columns in the intermediate range are most common in steel buildings. Failure is characterized by inelastic buckling and is greatly influenced by the magnitude and pattern of residual stresses that are present and the magnitude and shape of the initial imperfections or out-of-straightness. These effects are less severe for both shorter and longer columns. The expressions in this clause account for these effects that are dependent on the cross-section (Bjorhovde 1972).

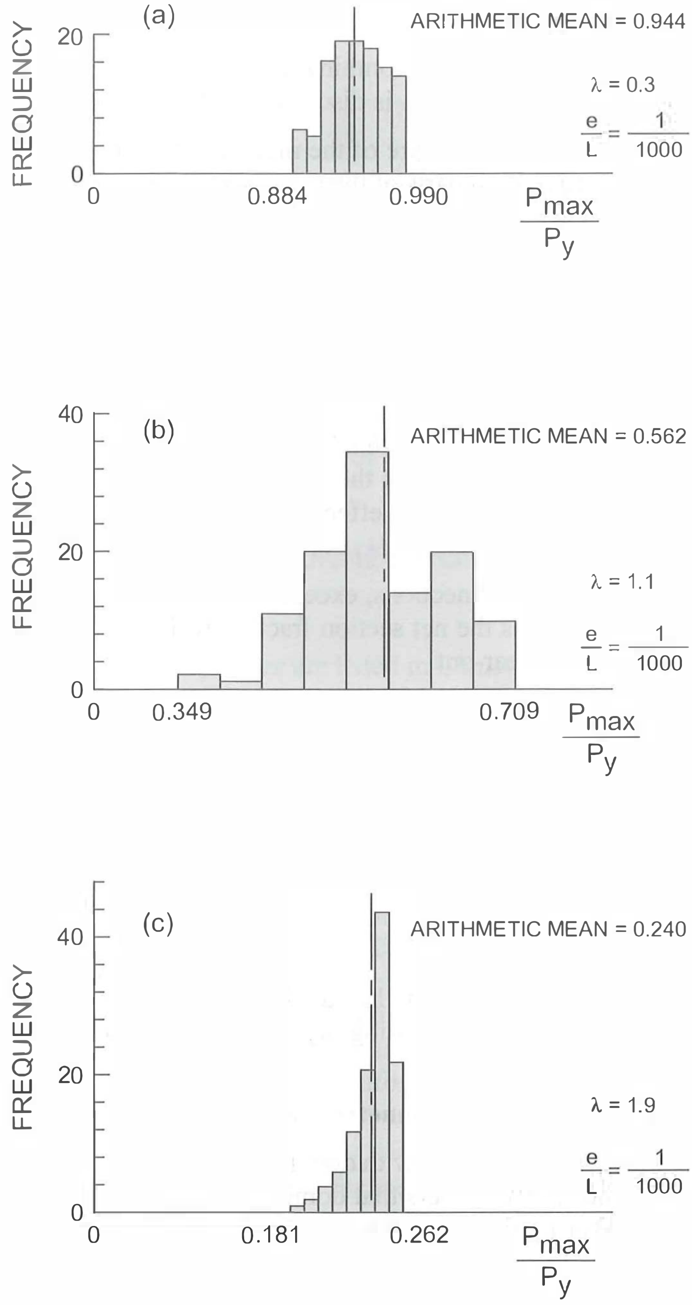


Figure 2-13
Typical Frequency Distribution Histograms for the Maximum Strength of 112 Column Curves (e/L = 1/1000)

Figure 2-13 indicates the variations in strengths for columns of three different values of the slenderness parameter, λ, and with the same out-of-straightness patterns and different residual stress patterns.

The compressive resistance expressions of Clause 13.3.1 are expressed in double exponential form (Loov 1996). With values of the parameter n of 1.34 and 2.24 for the cases shown in Clause 13.3.1, the expressions are always within 3% and generally within 1% of column Curves 2 and 1, respectively, of the Structural Stability Research Council (SSRC) (Ziemian 2010).

Steel shapes, unless explicitly stated, are assigned to SSRC Curve 2 ($n = 1.34$) which is used for hot-rolled, fabricated structural sections and for cold-formed, non-stress-relieved Class C hollow structural sections manufactured according to CSA Standard G40.20 (Bjorhovde and Birkemoe 1979). HSS produced to ASTM A500 grades B and C are cold-formed non-stress relieved, and the use of $n = 1.34$ is therefore appropriate.

Because of a more favourable residual stress pattern and out-of-straightness, hot-formed or cold-formed stress relieved (Class H) hollow structural sections (Kennedy and Gad Aly, 1980) are assigned to SSRC Curve 1 or its equivalent curve here with a value of $n = 2.24$. For the same reasons, doubly-symmetric three-plate members with flange edges oxy-flame-cut are also assigned to the curve with $n = 2.24$ (Chernenko and Kennedy, 1991).

For heavy sections (W310x313 and heavier and W360x347 and heavier, referred to as Groups 4 and 5 sections in earlier versions of CSA Standard G40.20) made of ASTM A7 or A36 steel and welded sections fabricated from universal mill plate, a resistance less than that corresponding to $n = 1.34$ (SSRC Curve 2) is appropriate, and it is recommended that a value of $n = 0.93$, corresponding to Column Curve 3 (Ziemian 2010), be used.

Because column strengths are influenced by the magnitude and distribution of residual stresses, care should be exercised in the use of the expressions in this Standard. For example, adding material such as welded cover plates increases the area and may reduce the slenderness ratio of an existing column, but it may also increase the compressive residual stresses in fibres remote from the centroid of the member, thus detracting from the strength.

13.3.2 Flexural, Torsional or Flexural-Torsional Buckling

Two other modes of buckling, which may occur prior to flexural buckling, are torsional or flexural-torsional buckling.

Torsional buckling with twisting about the shear centre is a possible failure mode for point-symmetric sections, e.g. a cruciform section, and in some circumstances, for doubly-symmetric sections. Flexural-torsional buckling, a combination of torsion and flexure is a possible failure mode for open sections that are singly-symmetric or asymmetric such as T's and angles. Thus, for sections with coincident shear centre and centroid, three potential compressive buckling modes exist (two flexural and one torsional), while for singly symmetric sections two potential compressive buckling modes (one flexural and one flexural-torsional) exist and, for a non-symmetric section, only one mode (flexural-torsional) exists. Closed sections, strong torsionally, also do not fail by flexural-torsional buckling (see Ziemian 2010). For the theory of elastic flexural-torsional buckling see Goodier (1942), Timoshenko and Gere (1961), Vlasov (1959) and Galambos (1968). The equations given here are developed in the latter among others.

As the problem of inelastic flexural-torsional buckling is quite complex and is amenable generally only to inelastic finite element analyses, the approach given here is to compute the elastic buckling stress, F_e, from the equations given for doubly symmetric, singly symmetric or asymmetric sections and then calculate an equivalent slenderness ratio $\lambda = \sqrt{F_y/F_e}$ to be used in the equations of Clause 13.3. This comes from the fact that an elastic buckling curve, when non-dimensionalized by dividing by F_y can be written as $F_e/F_y = 1/\lambda^2$. When the inelastic equations of 13.3 are entered with the equivalent slenderness ratio, an inelastic compressive resistance results.

The equations given here are equivalent to those in CSA Standard S136. There, however, for singly symmetric sections, the x-x axis is taken as the axis of symmetry, because cold-formed channel sections are frequently used.

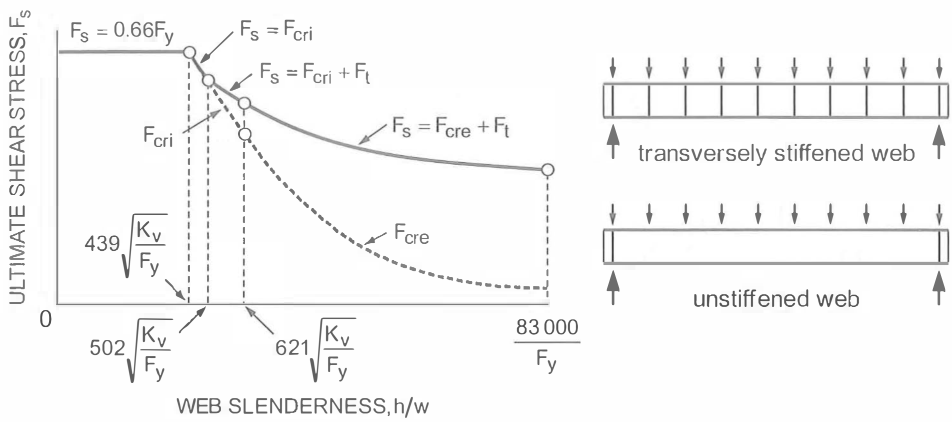
13.3.3 Single-Angle Members in Compression

The design of single angles subjected to axial compression is addressed. The angle is connected by a single leg, which is attached to a gusset plate or the projecting leg of another member by welding or by a bolted connection (at least two bolts), and is not subjected to any transverse loading. The effect of end eccentricity and rotational end restraint, hence, any resulting flexure of the angle, is indirectly accounted for by incorporating the equivalent slenderness expressions provided in this Clause. These expressions have also been adopted by the AISC Specification (2010b). They are essentially equivalent to those specified for equal-leg angles as web members in latticed transmission towers in ASCE 10-97 (ASCE 2000). The slenderness expressions are considered valid for equal-leg angles or unequal-leg angles connected by the longer leg (ratio of long leg / short leg < 1.7). It is assumed that significant restraint about the y-axis, which is perpendicular to the connected leg (note regarding convention: where the longer leg is connected, this axis is defined as the x-axis in the section properties tables for single angles in the CISC Handbook), exists due to the end connections. This causes the angle to flex and buckle primarily about the x-axis, hence, the use of the radius of gyration about the geometric axis parallel to the connected leg, r_x. The expressions for box trusses reflect greater rotational end restraint as compared to that provided by planar trusses. The slenderness expressions are not intended for use in the calculation of compression resistance of single angles used as diagonal braces in a braced frame. The procedure allows for the use of unequal-leg angles attached by the smaller leg provided that the equivalent slenderness is increased by an amount that is a function of the ratio of the longer to the shorter leg lengths. A minimum slenderness limit based on the slenderness about the minor principal axis must be met in all cases.

If the single-angle compression members cannot be evaluated using the equivalent slenderness expressions, then the provisions of Clause 13.3.2 shall be used for design accounting for the effect of end eccentricity and rotational end restraint. In evaluating C_r, the effective length due to end restraint should be considered. The procedure documented by Lutz (1992) to compute an effective radius of gyration for the angle can be implemented.

13.3.5 Members in Compression Subjected to Elastic Local Buckling

Two alternatives are available for approximating the factored compressive resistance of compression members that do not meet the local buckling requirements. The first is based on the notional removal of the width in excess of the limit for plate elements in axial compression to determine a reduced cross-sectional area. This area is used with the specified minimum yield strength and a slenderness based on the gross cross section to determine the factored compressive resistance by Clause 13.3.2 or 13.3.3. In the second alternative, the existing b/t ratio is used to establish the effective yield strength of a section just meeting the Class 3 limits. With this reduced yield strength and the gross cross section properties, Clause 13.3.2 or 13.3.3 establishes the factored resistance. Results by the two methods will not necessarily be the same. It is not necessary to refer to CSA S136 for members in axial compression that are subjected to elastic local buckling.



$$F_{cri} = \frac{290\sqrt{F_y k_v}}{h/w} \qquad k_v = 4 + \frac{5.34}{(a/h)^2} \qquad \text{when } \frac{a}{h} < 1$$

$$F_{cre} = \frac{180\,000 k_v}{(h/w)^2} \qquad k_v = 5.34 + \frac{4}{(a/h)^2} \qquad \text{when } \frac{a}{h} \geq 1$$

$$F_t = k_a\left(0.50F_y - 0.866F_{cri}\right) \text{ when } 502\sqrt{k_v/F_y} < h/w \leq 621\sqrt{k_v/F_y}$$

$$= k_a\left(0.50F_y - 0.866F_{cre}\right) \text{ when } 621\sqrt{k_v/F_y} < h/w$$

Figure 2-14
Ultimate Shear Stress – Webs of Flexural Members

13.4 Shear

13.4.1.1 Elastic Analysis

The expressions for shear strength are given for unstiffened and stiffened plate girders. Unstiffened plate girders and rolled beams are simply special cases for which the shear buckling coefficient, $k_v = 5.34$.

The four ranges of resistance based on Basler (1961) correspond to the following modes of behaviour and are illustrated in Figure 2-14 for stiffened webs:

(a) Full yielding followed by strain-hardening and large deformation. The limiting stress of $0.66F_y$ corresponds to shear deformation into the strain-hardening range and is higher than that derived from the von Mises criterion ($0.577\,F_y$), which forms the basis of Clause 13.4.2 for plastic analysis.

(b) A transition curve between strain-hardening and inelastic buckling at full shear yielding. ($F_s = 0.577F_y$);

(c) Inelastic buckling, F_{cri}, accompanied by post-buckling strength, F_t, due to tension field action, if the web is stiffened; and,

(d) Elastic buckling, F_{cre}, accompanied by post-buckling strength, F_t, due to tension field action, if the web is stiffened.

In computing the shear resistance, it is assumed that the shear stress is distributed uniformly over the depth of the web. The web area (A_w) is the product of web thickness (w) and

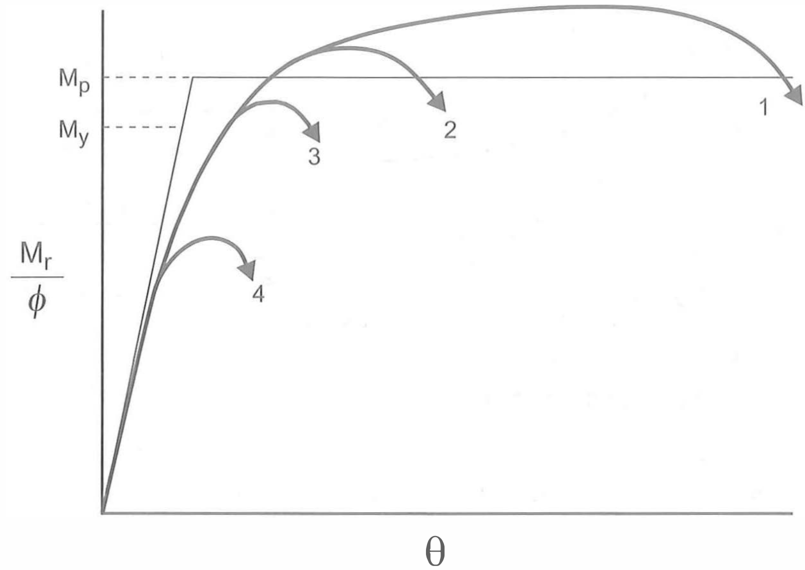


Figure 2-15
Moment-Rotation Curves

web depth (h) except for rolled shapes where it is customary to use the overall beam depth (d) in place of the web depth (h).

In panel zones and locations where strain-hardening develops quickly after the onset of shear yielding, the use of $0.66F_y$ is valid.

13.4.2 Plastic Analysis

For structures analyzed plastically, high shears and moments may occur simultaneously at a hinge location. Yang and Beedle (1951) have shown that, when the maximum shear stress is limited to the von Mises value, the flexural resistance can be maintained at M_p. Taking the effective section depth as 95% of the nominal depth, this Clause gives an approximate shear resistance limited to the von Mises stress. (See Commentary to Clause 8.3.2(d)).

13.4.3 Webs of Flexural Members Not Having Two Flanges

When cross-sections do not have two flanges, the shear stress distribution can no longer be assumed to be uniform. For W-shapes with one flange coped, the elastic shear stress distribution may be determined from $\tau = VQ/It$. Limiting the maximum value to $0.66F_y$ is conservative as it does not allow for any plastification as shear yielding spreads from the most heavily stressed region. For W-shapes with two flanges coped, a parabolic shear stress distribution results from this procedure with a maximum shear stress equal to 1.5 times the average. The maximum shear stress can be based on strain-hardening provided shear buckling does not occur.

13.4.4 Pins

Additional information for pins in combined shear and moment is given in the Canadian Highway Bridge Design Code, CSA S6-14.

13.5 Bending – Laterally Supported Members

The factored moment resistances are consistent with the classification of cross-sections given in Clause 11, as illustrated by moment–rotation curves given in Figure 2-15.

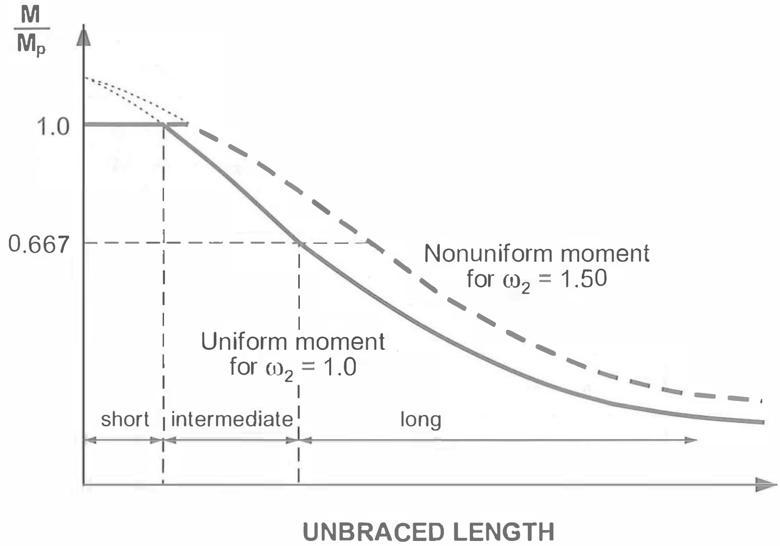


Figure 2-16
Variation of Uniform and Nonuniform Moment Resistances

The fully plastic moment, M_p, attained by Class 1 and 2 sections, implies that all fibres of the section are completely yielded. Any additional resistance that develops due to strain-hardening has been accounted for in the test/predicted ratio statistics used in developing resistance factors (Kennedy and Gad Aly 1980).

The stress distribution for Class 3 sections at the ultimate moment is assumed linear, with a maximum stress equal to the yield stress.

Class 4 sections reach their maximum moment resistance when a flange or web plate element buckles locally. Class 4 sections are divided into three categories.

The first consists of those sections with Class 4 flanges and webs. This type of section is designed to the requirements of CSA Standard S136 using the material properties appropriate to the structural steel specified.

The second category consists of those sections with Class 3 flanges and Class 4 webs. Clause 13.5(c)(ii) logically requires that these sections be designed in accordance with Clause 14.

For the third category with Class 4 flanges and Class 3 webs, a reduced, effective section modulus (Kalyanaraman *et al.* 1977) is used to compute the moment resistance. Alternatively, an effective yield stress established from Class 3 limits may be used to calculate the moment resistance.

13.6 Bending – Laterally Unsupported Members

Laterally unsupported beams may fail by lateral–torsional buckling at applied moments significantly less than the full cross-sectional strength (M_p or M_y). Even when the top flange is laterally supported, under some circumstances – for example, a roof beam subject to uplift – the laterally unsupported bottom flange may be in compression. General information on lateral–torsional buckling is summarized in Chen and Lui (1987).

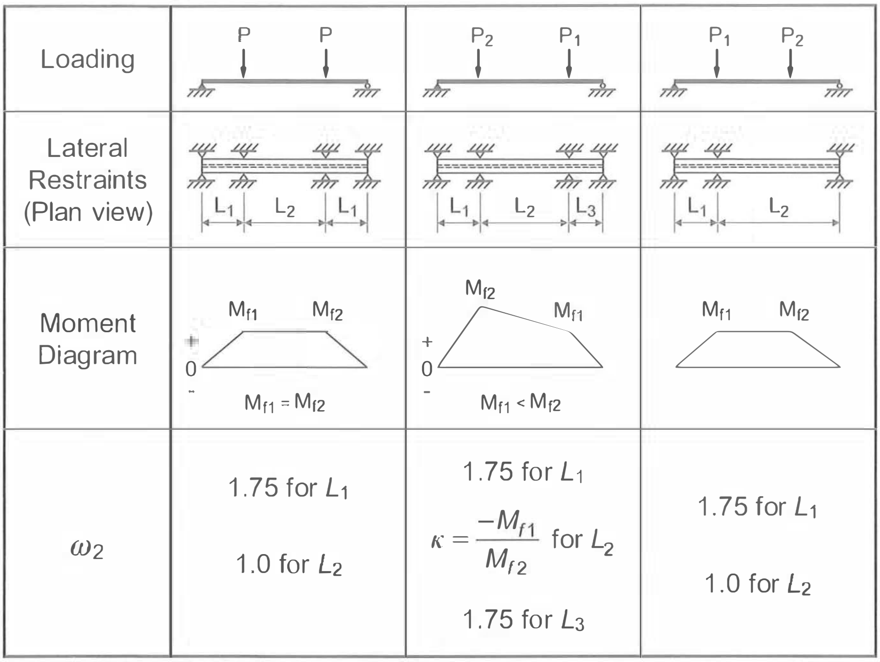


Figure 2-17
Various Cases of ω_2 for Linear Moment Gradients

Besides cross-sectional properties and aspects related to the loading itself, the lateral–torsional moment resistance depends on the unsupported (unbraced) length. Beams may be considered to be short, intermediate, or long depending on whether the moment resistance developed is the full cross-sectional strength, the inelastic lateral–torsional buckling strength, or the elastic lateral-torsional buckling strength, respectively, as shown in Figure 2-16 for Class 1 and 2 shapes capable of attaining M_p on the cross-section. The curve for Class 3 sections is similar, except that the maximum moment resistance is M_y, while for Class 4 sections, the maximum resistance is limited by local buckling.

The length, L, is generally taken as the distance between lateral supports. When beams are continuous through a series of lateral supports, interaction buckling (Trahair, 1968) occurs, and the segment that tends to buckle laterally first is restrained by the adjoining segments. Nethercot and Trahair (1976a, 1976b), Kirby and Nethercot (1978) and Schmitke and Kennedy (1985) give methods of computing effective lengths under these circumstances. Points of contraflexure for bending about the major axis are not related to lateral-torsional buckling and therefore cannot be considered as points of lateral support (Schmitke and Kennedy 1985).

Without the equivalent moment factor, ω_2, the expression given for M_u is that for a doubly-symmetric beam subject to uniform moment. The factor ω_2 ranges from 1.0 to 2.5 and takes into account the fact that for lateral–torsional buckling a varying moment is less severe than a uniform moment. Also plotted in Figure 2-16 is the moment resistance for a beam for which $\omega_2 = 1.5$. It is seen that in the elastic region ($M_r \leq ⅔\, M_p$) the full value of ω_2 is realized. In the inelastic region, however, the increase in M_r due to non-uniform moments gradually decreases to zero as the moment approaches M_p.

Wong and Driver (2010) and Driver and Wong (2007) demonstrate that the method for calculating ω_2 specified in 2001 and earlier editions of the Standard produces highly erroneous results in some common situations. To address this shortcoming, their general equation for determining ω_2 based on the moments at the quarter-points of the unbraced segment has been introduced into the Standard. This equation uses a similar method to that specified in the AISC Specification (AISC 2010b), except that it employs a square-root format that eliminates the non-conservative results that would otherwise arise in cases where the ends of the unbraced segment are close to rotationally fixed about the major axis. While the upper limit on the value of ω_2 of 2.5 that was associated with the previous method is theoretically no longer required, it was retained to acknowledge the fact that very high lateral-torsional buckling capacities attributable largely to the moment distribution can be highly sensitive to the assumptions about loading and end restraint, and they may not be achievable in practice. Wong and Driver (2010) provide a detailed discussion of aspects that affect the accuracy of equivalent moment factors and they compare numerous methods of determining this factor that have been proposed in the literature and that are being used in design standards around the world.

Due to its simplicity and familiarity to Canadian designers, the method from the 2001 edition of the Standard for determining ω_2 has been retained as an alternative approach for application only to cases where the moment gradient is linear between lateral supports, which is the scenario for which it was derived and therefore produces good results. Figure 2-17 illustrates several cases where this method may still be used. The quarter-point moment method in the Standard also gives excellent results for linear moment gradients.

The expression for M_u assumes that the beam is loaded at the elevation of the shear centre. A downward-acting load that is applied below the shear centre stabilizes the beam, whereas such a load applied above the shear centre destabilizes it. The Standard is now explicit that for the latter case, when the beam is laterally unbraced at the load point and the means of applying the load itself provides neither lateral nor rotational restraint, the reduction in moment capacity must be taken into account. For top-flange loading, a simple and conservative effective length approach (Wong *et al.* 2014) is provided as an alternative to more accurate methods. Since the effective length factor accounts for both the load height and moment distribution effects, ω_2 is set equal to unity. The two effective length factors specified in the Standard are distinguished by the in-plane rotational restraint at the ends of the unbraced beam segment: either simple or restrained. The method does not apply to cantilevers. Detailed discussions on this approach and a graphical method that gives more accurate effective length factors are presented by Wong *et al.* (2014). For other positions of the load, unusual loading cases and other support conditions, Ziemian (2010) may be consulted.

Because laterally unsupported, closed, square and circular sections with $I_x = I_y$ show no tendency to buckle laterally, their moment resistance is established using Clause 13.5 as emphasized in Clause 13.6(c).

For structural systems utilizing cantilever suspended-span construction (Gerber girders), see Albert *et al.* (1992), Essa and Kennedy (1994(a), 1994(b), 1995), and Ziemian (2010) for a rational method of determining the strength of cantilevered beams.

For members bent about both principal axes, it should be remembered that M_{ry} is either M_{yy} or M_{yp} as a function of the class of the section, because there is no reduction for lateral–torsional buckling for weak axis buckling.

The provisions in Clause 13.6(e) were introduced in the 2009 edition of the Standard and address beams that are generally I-shaped and are symmetric about the web's centreline, but which have flanges of unequal sizes, or only one flange (tee sections). In these sections, the shear centre is not coincident with the centroid of the section, and the smaller flange has higher stresses than the larger flange. The effects of these asymmetries are accounted for in the β_x term. The expression provided for β_x is an approximation of the complete expression,

$$\beta_x = \frac{1}{I_x} \int_A y\left(x^2 + y^2\right) dA - 2y_o$$

where x and y are coordinates on the cross-section based on an origin located at the geometric centroid, and y_o is the distance in the y-direction from the centroid to the shear centre. An approximate value for the warping torsional constant, C_w, is also provided in this clause. A more thorough treatment of monosymmetric beams can be found in Ziemian (2010).

Sections that have large differences in flange size may experience yielding of the smaller flange under service loads, if designed as Class 1 or Class 2 beams and the factored moment resistance is near ϕM_p. The maximum moment caused by the applied service loads must be less than the smaller M_y value to prevent permanent deformations from occurring during service conditions.

The method of strength determination generally follows the AISC (2010b) methodology, with the exception that the elastic buckling capacity is determined considering the distribution of moments. This approach requires finding two lengths, L_{yr} and L_u, for beams that are in the inelastic buckling regime. L_{yr} is the length at which the elastic buckling moment, M_u, reaches M_{yr} and causes the initiation of yielding; i.e., the extreme fibre reaches 0.70 F_y, and yielding would occur in regions where residual stresses reach 30% of yield. The smaller value of S_x is used to determine M_{yr} (corresponding to yielding of the smaller flange). L_{yr} can be determined by any method, such as iterative approximations, but can be found via a direct solution with the following equation:

$$L_{yr} = \sqrt{\frac{\left(2P\beta_x + Q\right) + \sqrt{\left(2P\beta_x + Q\right)^2 + 4RP^2}}{2P^2}}$$

where $P = \dfrac{1.4 F_y S_{x,min}}{\omega_3 \pi^2 E I_y}$, $Q = \dfrac{4GJ}{\pi^2 E I_y}$, and $R = \dfrac{4C_w}{I_y}$.

The other length, L_u, is the length at which the beam can carry its fully braced capacity, either M_p or M_y, depending on its local buckling classification. The value of:

$$1.1 r_t \sqrt{E/F_y}$$

is based on work by White and Jung (2004). The term r_t is the radius of gyration of the tee-shaped area formed by the compression flange, and one-third of the portion of the web in compression as defined by the elastic neutral axis. Inelastic buckling capacity is determined by linear interpolation between M_{yr} and M_p (or M_y), based on the unbraced length of beam, L.

The value of ω_3 for tee sections must be less than or equal to 1.0. This is because reverse curvature in these beams is a worse condition than a uniform moment (Attard and Lawther, 1989), which is different from the case for doubly-symmetric sections. The warping torsional constant for tee sections should be taken as zero.

For monosymmetric sections other than those described above, a rational method must be used.

13.7 Lateral Bracing for Members in Structures Analyzed Plastically

See the Commentary on Clause 8.3.2(c). This clause is consistent with seismic requirements.

13.8 Axial Compression and Bending

This Clause remains unchanged in the 2014 edition. The design for strength and stability of steel frames and beam-columns is based on "second-order analysis" and "notional lateral loads" (Clause 8.4), and "sway stiffness" (Clause 13.8).

(a) A distinction is made between braced and unbraced frames in that the design requirements for beam-columns are different for the two types of frames. The 5/1 stiffness ratio in Clause 13.8 originates from Eurocode 3 where it was stated that if bracing were added to a frame and it reduced the lateral sway deflection by 80% or more, then the bracing was sufficiently effective to consider the frame as being "braced". In the analysis, members such as columns with nominally pinned connections, which do not contribute to the lateral strength and stability of the frame/structure, may be considered to be braced by the frame. The notional loads attributed to such non-contributing columns must be included in the sway analysis of the frame

(b) Cross-sectional strength never governs for prismatic beam-columns in unbraced frames and need not be checked because it will never be smaller than the in-plane strength or the lateral-torsional buckling strength. Parenthetic statements in Clauses 13.8.2(a) and 13.8.3(a) waive this check.

(c) P-δ effects, related to the member deformation between the ends, have been found to be negligible for beam-columns in unbraced frames. This is because the maximum second-order elastic moment, including P-Δ (sway) effects, occurs at the ends of the beam-column. Therefore the factor U_1 is taken as 1.0 in the interaction equation for overall member strength of sway (unbraced) beam-columns in Clauses 13.8.2(b) and (c), and 13.8.3(b) and (c). (The P-δ effects continue to be considered for non-sway beam-columns.)

(d) For weak-axis bending, the in-plane strength interaction equation introduced in Clause 13.8.2 with a factor β accounts more accurately for the effect of distributed plasticity on stability, by fitting the plastic-zone strength curves for different values of λ_y more closely. β increases from 0.6 when $\lambda_y = 0$ to 0.85 for values of λ_y greater than 0.625 where the distributed plasticity has a greater effect on the overall weak-axis stiffness.

For a general discussion of all aspects of Clause 13.8 and worked examples, see Essa and Kennedy (2000).

The value each term in the interaction equation takes is prescribed in the three sub-clauses (a), (b), and (c) depending on the particular mode of failure: cross-sectional strength, overall member strength, and lateral-torsional buckling strength, respectively. Clause 13.8.2 is applicable to Class 1 and Class 2 sections of I-shaped members, while Clause 13.8.3 is applicable to all other classes of sections.

The interaction expressions account for the following:

- A laterally supported member fails when it reaches its in-plane moment capacity, reduced for the presence of axial load;
- A laterally unsupported member may fail by lateral-torsional buckling or a combination of weak-axis buckling and lateral buckling;
- A relatively short member can reach its full cross-sectional strength whether it is laterally supported or not;
- When subjected to axial load only, the axial compressive resistance, C_r, depends on the maximum slenderness ratio – below the yield load, the column fails by buckling.

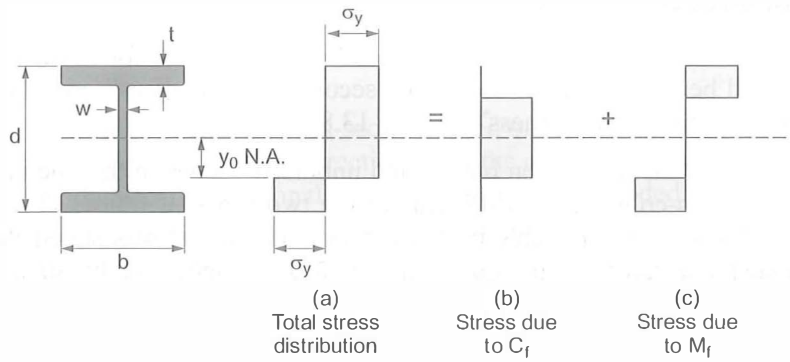


Figure 2-18

Idealized Stress Distribution in Plastified Section of Beam-Column

Column buckling is a bifurcation problem, not a bending strength problem;

- Members bent about the weak axis, or with the same strength about both axes, do not exhibit out-of-plane behaviour,
- A constant moment has the most severe effect on in-plane behaviour. Other moment diagrams can be replaced by equivalent moment diagrams of reduced but uniform intensity;
- A constant moment has the most severe effect on the lateral-torsional buckling behaviour. (See commentary on Clause 13.6). This effect disappears if the member is short enough, in which case, cross-sectional strength controls; and
- Moments may be amplified by axial loads increasing the deflections, the P-δ effect.

Four modes of failure, including local buckling of plate elements, are to be checked in design, as appropriate. They are addressed as follows:

1) Local buckling of an element

Before assessing the member failure modes, the element b/t ratios are checked to confirm the class of the section, the appropriate cross-sectional moment and axial compressive resistances, and to ensure local buckling does not occur prematurely.

2) Strength of the cross-section

The cross-sectional strength of a shape used as a beam-column is not to be exceeded. Clause 13.8.2(a) gives the cross-sectional strength requirements for Class 1 and Class 2 sections of I-shaped members and Clause 13.8.3(a) for all other classes of sections. The cross-sectional strength is also the limiting strength of short members. For prismatic beam-columns in unbraced frames, the cross-sectional strength never governs the design and need not be checked.

The cross-sectional strength of a Class 1 and Class 2 I-shaped section comprising relatively stocky plate elements is derived from the fully plastic stress distribution of the cross-section as shown in Figure 2-18. For uniaxial bending about the x-x axis and the y-y axis, expressions are respectively, using the limit states notation of this Standard:

$$M_{fx} = 1.18\phi\, M_{px}\left(1 - \frac{C_f}{\phi A F_y}\right) \leq \phi\, M_{px}$$

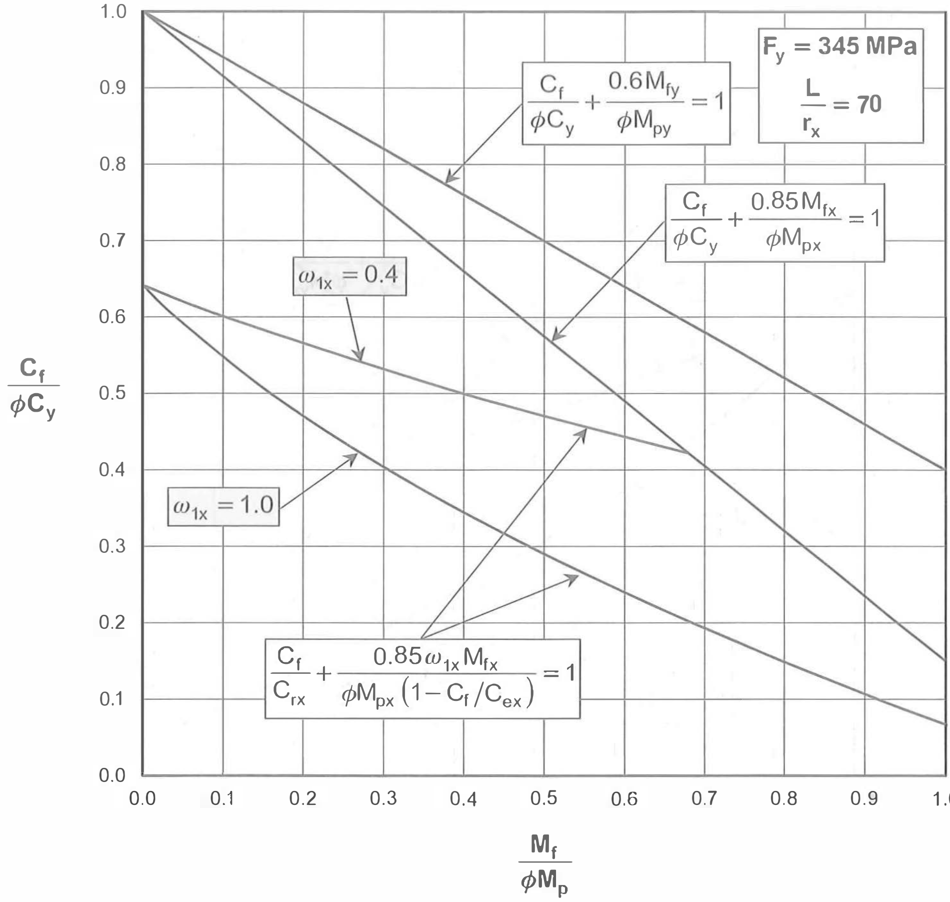


Figure 2-19
Interaction Expressions for Class 1 and Class 2 W-Shapes

$$M_{fy} = 1.67 \phi M_{py} \left(1 - \frac{C_f}{\phi A F_y}\right) \leq \phi M_{py}$$

Transposing the terms in the above expressions gives:

$$\frac{C_f}{\phi C_y} + 0.85 \frac{M_{fx}}{\phi M_{px}} \leq 1.0; \quad \frac{M_{fx}}{\phi M_{px}} \leq 1.0$$

$$\frac{C_f}{\phi C_y} + 0.6 \frac{M_{fy}}{\phi M_{py}} \leq 1.0; \quad \frac{M_{fy}}{\phi M_{py}} \leq 1.0$$

as shown in Figure 2-19. For biaxial bending it is conservative to combine these expressions linearly to give, using the limit states notation of this Standard:

$$\frac{C_f}{\phi C_y} + 0.85 \frac{M_{fx}}{\phi M_{px}} + 0.6 \frac{M_{fy}}{\phi M_{py}} \leq 1.0; \quad \frac{M_{fx}}{\phi M_{px}} + \frac{M_{fy}}{\phi M_{py}} \leq 1.0$$

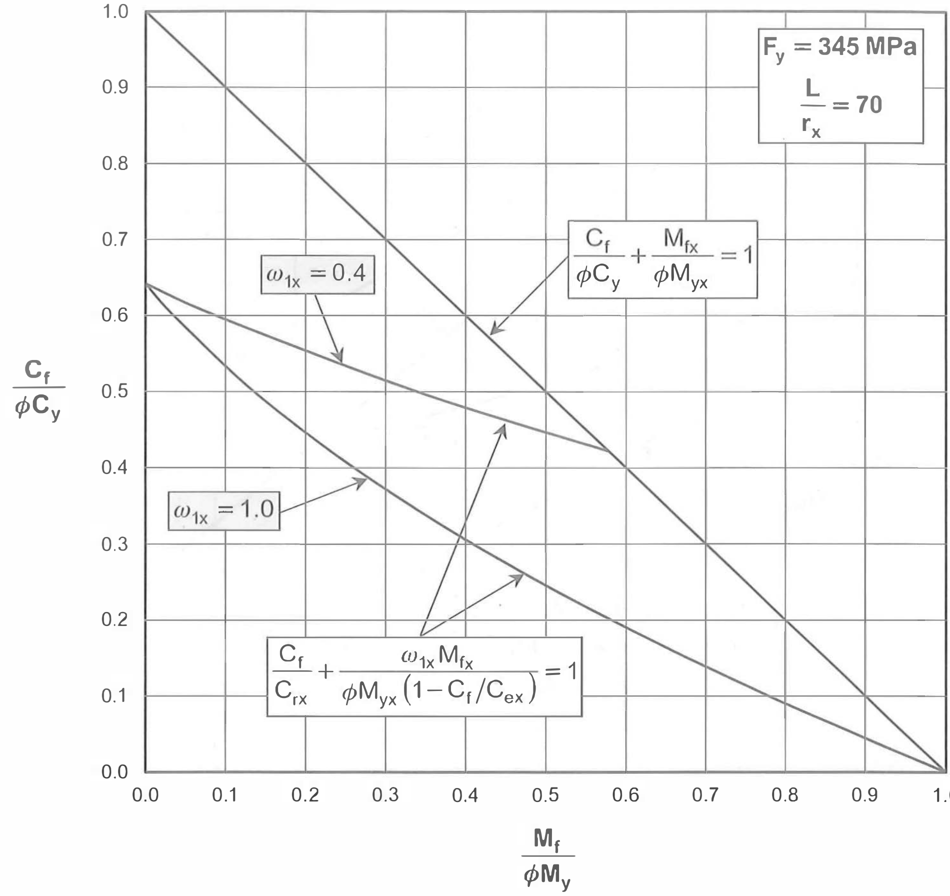


Figure 2-20
Interaction Expressions for Class 3 W-Shapes

This is identical to the two expressions in Clause 13.8.2 when, in the latter in accordance with Clause 13.8.2(a) for cross-sectional strength, U_{1x} and U_{1y} are set equal to 1.0, $C_r = \phi A F_y$ when $\lambda = 0$, $\beta = 0.6$ when $\lambda = 0$, and M_{rx} and M_{ry} are equal to ϕM_{px} and ϕM_{py}, respectively.

For uniaxial bending of sections other than Class 1 and Class 2 I-sections, the appropriate interaction expression is:

$$\frac{C_f}{\phi C_y} + \frac{M_{fx}}{M_{rx}} \leq 1.0$$

Extending this linear expression to biaxial bending gives:

$$\frac{C_f}{\phi C_y} + \frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \leq 1.0$$

This agrees with Clause 13.8.3(a) when the appropriate values of the factored cross-sectional resistance quantities are used. Thus, for Class 3 sections the factored moment resistances are limited to ϕM_y and for Class 4 sections the resistances, C_r, M_{rx}, and M_{ry}, are based on local buckling.

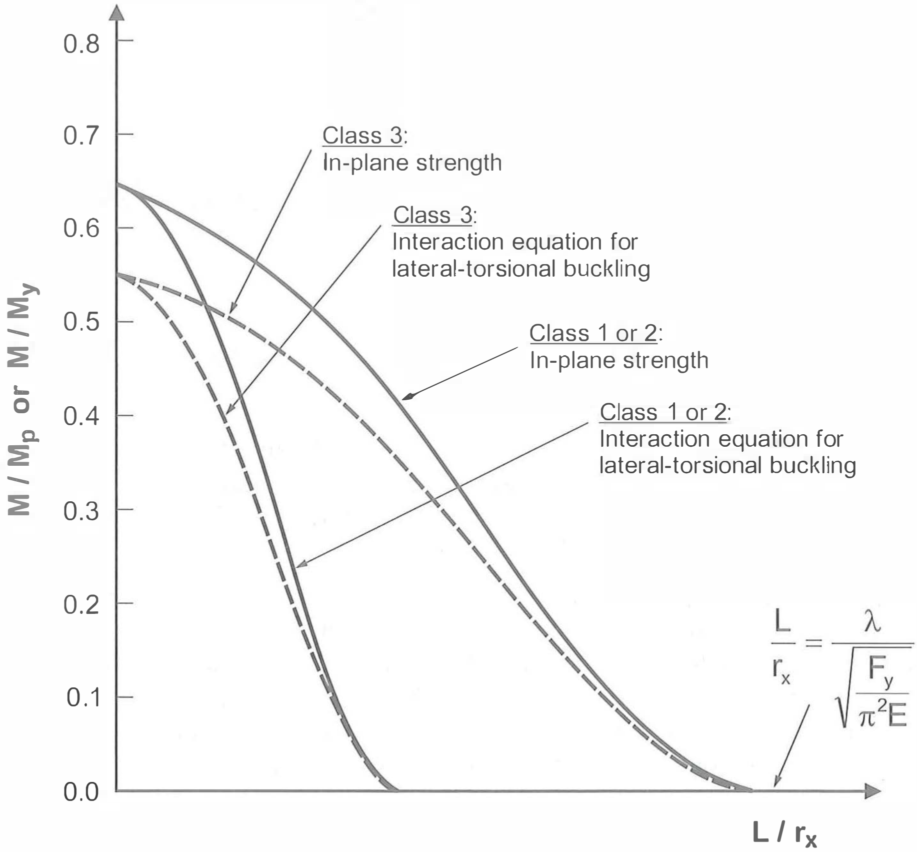


Figure 2-21
Variations of Moment Resistance with Slenderness Ratio

3) Overall member strength

The overall strength (in-plane bending strength) of a member depends on its slenderness. As an actual beam-column has length, the axial compressive resistance, C_r, depends on its slenderness ratio and will be less than or equal to the yield load. For any particular beam-column, this fraction of the yield load can be established and is illustrated in Figure 2-19 for Class 1 or 2 sections, and in Figure 2-20 for Class 3 sections.

In Figure 2-21 the variation in moment resistance in terms of M/M_p as a function of the slenderness L/r_x is plotted schematically as a solid line for a particular laterally supported Class 1 (or Class 2) section subject to a uniform moment about the x-axis and carrying an axial load of $0.35C_y$. An appropriate interaction expression for the in-plane strength of such a Class 1 (or Class 2) I-section is

$$\frac{C_f}{C_{rx}} + 0.85\frac{\omega_1 M_f}{\phi M_p\left(1 - C_f/C_e\right)} \le 1.0$$

which can be deduced from Clause 13.8.2(b) when the terms in that expression are appropriately defined. Note that if the member is short, the expression reduces to that for the cross-sectional strength. The compressive resistance, C_{rx}, is a function of the slenderness ratio L/r_x.

The term:

$\omega_1 = 0.6 - 0.4\,\kappa \geq 0.4$

multiplied by the maximum non-uniform moment, M_f, gives an equivalent uniform moment, $\omega_1 M_f$, having the same effect on the in-plane member strength as the non-uniform moment (Ketter 1961).

In order to account for the P-δ effects (the amplification of the moments caused by the axial loads acting on the deformed shape), the equivalent uniform moment, $\omega_1 M_f$, is amplified by the factor:

$$\frac{1}{1 - \dfrac{C_f}{C_e}} \quad \text{where: } C_e = \frac{\pi^2 E I}{L^2}$$

The in-plane strength of Class 1 or 2 sections is shown in Figure 2-19 for F_y = 345 MPa and L/r_x = 70. When L/r_x = 0 and ω_1 = 1, the in-plane strength expressions 13.8.2(b) and 13.8.3(b) become the cross-sectional strength expressions 13.8.2(a) and 13.8.3(a), respectively. The curve for Class 3 sections is given in Figure 2-20.

In Figure 2-21, the curve of moment resistance versus slenderness ratio for the in-plane strength of a Class 3 section of equivalent cross-sectional strength to the Class 1 or 2 section is also given. It is similar to that for a Class 1 or 2 section except that, because the cross-sectional strength expression for Class 3 sections does not have the 0.85 factor that is appropriate for Class 1 or 2 and because the Class 3 section can only attain M_y, the curve for Class 3 for zero slenderness ratio reaches only about $0.55 M_p$ and not $0.65 M_p$ as for the Class 1 or 2 sections.

For biaxial bending, C_r is based conservatively on the maximum slenderness ratio. It could be argued that for biaxial bending the value used for C_r be interpolated between C_{rx} and C_{ry} on the basis of the proportion of the interaction fractions for bending about two axes. In other words, if a beam-column carries only a small portion of bending about the y-axis, the decrease in C_r from C_{rx} toward C_{ry} should likewise be small.

In Figures 2-19 and 2-20, the in-plane strength interaction expressions are shown for ω_1 = 1. When $\omega_1 < 1$, the limiting strength for low ratios of axial load is the cross-sectional strength expression.

4) Lateral-torsional buckling strength

Building beam-columns are usually laterally unsupported for their full length and, even though they are subject to strong-axis bending moments, failure may occur when the column, after bending about the strong axis, buckles about the weak axis and twists simultaneously. Again this is a buckling or bifurcation problem. For such columns, the lateral-torsional buckling strength is likely to be less than both the cross-sectional strength and the overall member strength.

The curves in Figure 2-21 for a beam-column subject to uniform moment for Class 1 and 3 sections marked "*interaction equation for lateral-torsional buckling*", demonstrate this effect. They are much below those for in-plane strength and would only reach the full cross-sectional strength when the slenderness ratio is zero. The moment resistance is zero for laterally unsupported beam-columns when weak-axis buckling occurs. Thus, for these members the axial compressive resistance is based on L/r_y, and M_{rx} is based on the resistance of a laterally unsupported beam. When subjected to weak-axis bending, members do not exhibit out-of-plane buckling behaviour, and therefore the weak-axis moment resistance is based on the full cross-sectional strength, the plastic moment or yield moment capacity about the weak axis as appropriate for the Class of the section.

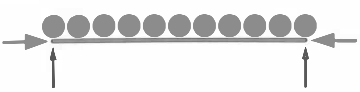
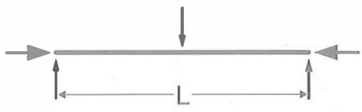
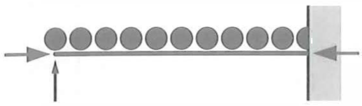
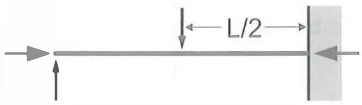
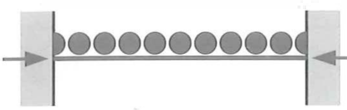
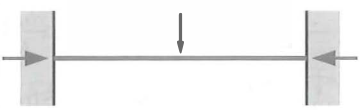
Case	ω_1	Case	ω_1
	1.0		$1-0.2\dfrac{C_f}{C_e}$
	$1-0.4\dfrac{C_f}{C_e}$		$1-0.3\dfrac{C_f}{C_e}$
	$1-0.4\dfrac{C_f}{C_e}$		$1-0.2\dfrac{C_f}{C_e}$

Figure 2-22
Values of ω_1 for Special Cases of Laterally Loaded Beam-Columns

In computing M_{rx} from Clause 13.6, the effect of non-uniform moments is included. Therefore, in the interaction expressions when lateral-torsional buckling is being investigated, the factored moment, M_{fx}, must also be a non-uniform moment, and not be replaced by an equivalent lesser moment. It is for this reason that the value of U_{1x} cannot be less than 1.0.

13.8.5 This clause gives generally conservative values of ω_1, the factor by which the maximum value of the non-uniform moment is multiplied to give an equivalent uniform moment having the same effect as the applied non-uniform moment on the overall strength of the member. For further discussion on ω_1, see Ziemian (2010) where it is called C_m. Figure 2-22 gives values of ω_1 for some special cases of transverse bending.

Figures 2-23 and 2-24 give additional guidance for the design of beam-columns subjected to various bending moment effects.

13.9 Axial Tension and Bending

The linear interaction expression of Clause 13.9.1 is a cross-sectional strength check. Conservatively, it does not take into account the fact that the bending resistance for Class 1 and 2 sections does not vary linearly with axial force, for which case a factor of 0.85 multiplying the moment term would appear to be appropriate (see Clause 13.8.2).

For members subjected predominantly to bending, i.e. when the tensile force is relatively small, failure may still occur by lateral-torsional buckling. The expressions of Clause 13.9.2 result from that of Clause 13.9.1 when a negative sign is assigned to the tension interaction component and when M_r is based on the overall member behaviour taking lateral-torsional buckling into account.

13.10 Load Bearing

The bearing resistance given for accurately cut or fitted parts in contact, Clause 13.10(a), reflects the fact that a triaxial compressive stress state, restricting yielding of the parts in contact,

<table>
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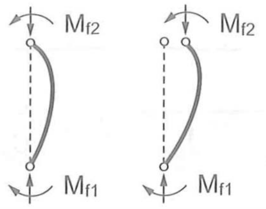

Single curvature bending

$M_{f2} \geq M_{f1}$

$\omega_1 = 0.6 + 0.4\dfrac{M_{f1}}{M_{f2}}$
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Beam-Colum Design Expressions

$P\Delta$ (frame sway effects), if any, are included in the analysis.

(1) Class 1 and 2 Sections of I-Shapes

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

$$\beta = 0.6 + 0.4\lambda_y \leq 0.85$$

$$\frac{M_{fx}}{M_{rx}{}^*} + \frac{M_{fy}}{M_{ry}{}^*} \leq 1.0$$

(2) All Classes Except Class 1 and 2 Sections of I-Shapes

$$\frac{C_f}{C_r} + \frac{U_{1x}M_{fx}}{M_{rx}} + \frac{U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

Member Strength Checks

(a) Cross-sectional strength (use actual M_f at each location)

$C_r = \phi A F_y$
$M_r = \phi Z F_y$ = (for Class 1 and 2 sections)
$= \phi S F_y$ = (for Class 3 sections)
= See S16-14 Clause 13.5(c) for Class 4 sections
$M_{rx}{}^*$ and $M_{ry}{}^*$ calculated according to Cl. 13.5 or 13.6 as appropriate
$U_1 = \omega_1/(1 - C_f/C_e) \geq 1.0$

(b) Overall member strength (use M_{f2} for M_f)

C_r = Factored compressive resistance (max. slenderness, K = 1), Cl. 13.3, except C_r based on axis of bending for uniaxial bending
M_r = as given for M_r in (a) above
$U_1 = \omega_1/(1 - C_f/C_e)$, except for unbraced frames $U_1 = 1.0$

(c) Lateral-torsional buckling strength (use M_{f2} for M_f)

C_r = Factored compressive resistance (max. slenderness), Cl. 13.3
M_{rx} = value given by Clause 13.6
M_{ry} = as given for M_r in (a) above

For braced frames:
$U_{1x} = \omega_{1x}/(1 - C_f/C_{ex}) \geq 1.0$
$U_{1y} = \omega_{1y}/(1 - C_f/C_{ey})$

For unbraced frames:
$U_{1x} = U_{1y} = 1.0$
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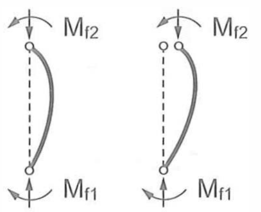

Single curvature bending

$M_{f1} = 0$

$\omega_1 = 0.6$
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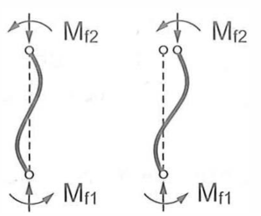


Double curvature bending

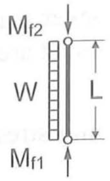
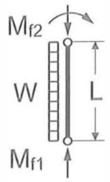
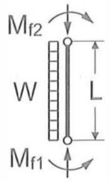
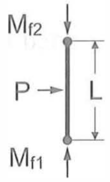
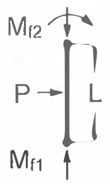
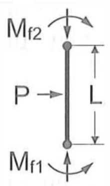
$M_{f2} \geq M_{f1}$

$\omega_1 = 0.6 - 0.4\dfrac{M_{f1}}{M_{f2}} \geq 0.4$
</td></tr>
<tr><td colspan="2">

C_f = Factored compressive load
C_r = Factored compressive resistance
M_f = Factored bending moment (x-x or y-y axis)
M_r or $M_r{}^*$ = Fact. moment resistance (x-x or y-y axis)
ω_1 = Coefficient used to determine equivalent uniform column bending effect (x-x or -y-y)
U_1 = Factor to account for moment gradient and member curvature second-order effects
</td></tr>
</table>

** Moments M_{f1} and M_{f2} may be applied about one or both axes.

Figure 2-23
Prismatic Beam-Columns – Moments at Ends – No Transverse Loads

Conditions **	
	Loaded with UDL $M_{f1} = M_{f2} = 0$ $M_{f3} = \frac{WL}{8}$ (max.)
	Loaded with UDL $M_{f1} = 0$ $M_{f2} = \frac{WL}{8}$ (max.) $M_{f3} = \frac{9WL}{128}$
	Loaded with UDL $M_{f1} = M_{f2} = \frac{WL}{12}$ (max.) $M_{f3} = \frac{WL}{24}$
	Loaded with PL $M_{f1} = M_{f2} = 0$ $M_{f3} = \frac{PL}{4}$ (max.)
	Loaded with PL $M_{f1} = 0$ $M_{f2} = \frac{3PL}{16}$ (max.) $M_{f3} = \frac{5PL}{32}$
	Loaded with PL $M_{f1} = M_{f2} = \frac{PL}{8}$ $M_{f3} = \frac{PL}{8}$ (max.)

Design Criteria

Beam-Colum Design Expressions

$P\Delta$ (frame sway effects), if any, are included in the analysis.

(1) Class 1 and 2 Sections of I-Shapes

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} + \frac{\beta U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

$$\beta = 0.6 + 0.4\lambda_y \leq 0.85$$

$$\frac{M_{fx}}{M_{rx}{}^*} + \frac{M_{fy}}{M_{ry}{}^*} \leq 1.0$$

(2) All Classes Except Class 1 and 2 Sections of I-Shapes

$$\frac{C_f}{C_r} + \frac{U_{1x}M_{fx}}{M_{rx}} + \frac{U_{1y}M_{fy}}{M_{ry}} \leq 1.0$$

Member Strength Checks

(a) Cross-sectional strength (use actual M_f at each location)

$C_r = \phi A F_y$

$M_r = \phi Z F_y$ = (for Class 1 and 2 sections)

$= \phi S F_y$ = (for Class 3 sections)

= See S16-14 Clause 13.5(c) for Class 4 sections

M_{rx}^* and M_{ry}^* calculated according to Cl. 13.5 or 13.6 as appropriate

$U_1 = \omega_1/(1 - C_f/C_e) \geq 1.0$

(b) Overall member strength (use M_{max} for M_f)

C_r = Factored compressive resistance (max. slenderness, K = 1)

Cl. 13.3, except C_r based on axis of bending for uniaxial bending

M_r = as given for M_r in (a) above

$U_1 = \omega_1/(1 - C_f/C_e)$, except for unbraced frames $U_1 = 1.0$

(c) Lateral-torsional buckling strength (use M_{max} for M_f)

C_r = Factored compressive resistance (max. slenderness), Cl. 13.3

M_{rx} = value given by Clause 13.6

M_{ry} = as given for M_r in (a) above

For braced frames:

$U_{1x} = \omega_{1x}/(1 - C_f/C_{ex}) \geq 1.0$

$U_{1y} = \omega_{1y}/(1 - C_f/C_{ey})$

For unbraced frames:

$U_{1x} = U_{1y} = 1.0$

C_f = Factored compressive load

C_r = Factored compressive resistance

M_f = Factored bending moment (x-x or y-y axis)

M_r or M_r^* = Fact. moment resistance (x-x or y-y axis)

ω_1 = Coefficient used to determine equivalent uniform column bending effect (x-x or -y-y)

U_1 = Factor to account for moment gradient and member curvature second-order effects

** Moments M_{f1} and M_{f2} may be applied about one or both axes.

Figure 2-24
Prismatic Beam-Columns with Transverse Loads

generally exists. The value given is based on earlier working stress design standards, which have given satisfactory results.

For a cylindrical roller or rocker, Clause 13.10(b) recognizes that the roller or rocker may rest in a cylindrical groove in the supporting plate. This results in a supporting or contact area larger than that for the case of a flat supporting plate.

In the case of a cylindrical groove in the supporting plate, the maximum shearing stress developed due to a line load of q N/mm, (Seeley and Smith, 1957) is,

$$\tau_{\max} = 0.27\sqrt{\frac{qE}{2\pi\left(1-\nu^2\right)}\left(\frac{R_2 - R_1}{R_2 R_1}\right)}$$

where ν is Poisson's ratio. From this, the unfactored bearing resistance, qL, is then

$$\frac{B_r}{\phi} = qL = \frac{2\pi L\left(1-\nu^2\right)\tau_{\max}^2}{0.27^2 E}\left(\frac{R_2 R_1}{R_2 - R_1}\right)$$

Calibrating this resistance to that given in S16-1969 at F_y = 300 MPa gives $\tau_{max} = 0.77F_y$, and

$$\frac{B_r}{\phi} = 0.000\,26\left(\frac{R_1}{1 - R_1/R_2}\right)LF_y^2$$

For a roller of radius R_1 on a flat plate with $R_2 = \infty$, the "Hertz" solution, as reported by Manniche and Ward-Hall (1975), gives the *allowable* load as

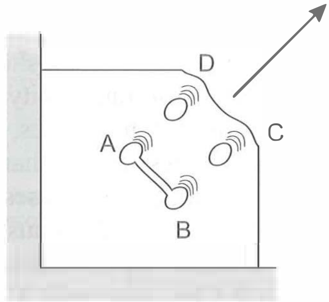
$$2.86DL\frac{\left(2.7F_y\right)^2}{E} = 0.000\,20R_1LF_y^2$$

where D is the roller diameter. The above expression indicates that the value of 0.000 26 R_1 obtained by calibration with the existing standard for a yield stress of about 300 MPa is somewhat non-conservative compared to the value of 0.000 21 R_1 proposed by Manniche and Ward-Hall (1975).

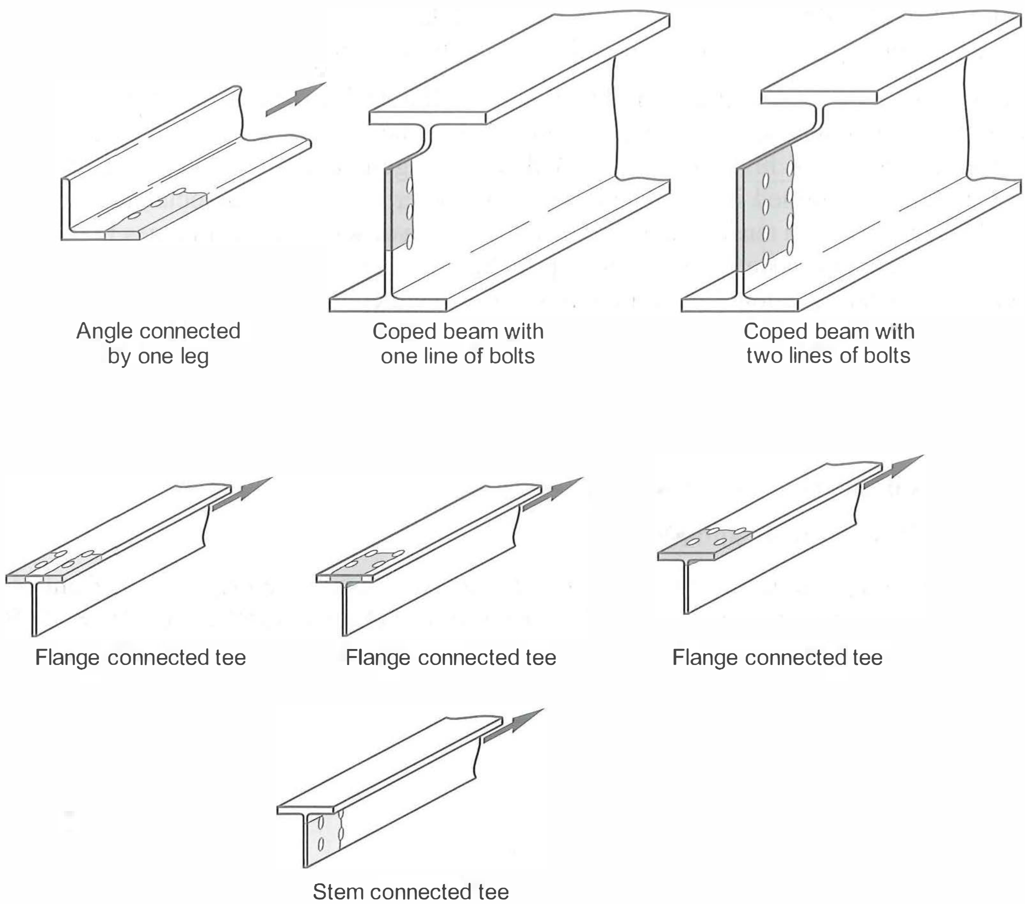
This is confirmed by Kennedy and Kennedy (1987) who reported that at this load no permanent deformation resulted and recommended that this value be used as a serviceability limit. They also reported that the rolling resistance of rollers varied as the fourth power of the unit normal load in kN/mm.

13.11 Block Shear – Tension Member, Beam, and Plate Connections

Tension rupture, which is discussed in Clause 13.2, can also take place in combination with shear through the failure of a block of material in a connection component. The provisions for block shear failure in Clause 13.11 reflect the findings of research by Driver *et al.* (2006), conducted to develop a single unified equation that can be adapted to any block configuration and, in the limit, is consistent with the provisions for pure tensile rupture. An examination of numerous test results on gusset plates, coped beams, angles, and tees indicated that rupture on the tension face occurs before rupture on the shear face of the block of material and, when rupture takes place on the tension face, the shear stress on the gross shear area exceeds the yield strength but is generally less than the ultimate strength. To reflect this limit state, the design equation uses a shear stress equal to the average of the yield and rupture shear strengths on the gross shear area, A_{gv}. The shear term alone also gives the end tear-out capacity for individual bolts or lines of bolts in the direction of the applied force (Cai and Driver, 2010). Due to



(a) Block Shear Failure of Gusset Plate



(b) Block Shear Failure of Angles, Coped Beams, and Tees

Figure 2-25

potentially reduced material ductility, the yield strength is used in the shear term of the design equation for higher strength steels.

The tension component is defined in the unified block shear equation as $U_t A_n F_u$, where U_t is an efficiency factor that accounts for the non-uniformity of the stress distribution on the tension face of the block of material at the limit state. Angles, tees connected by the stem, and coped beams have all shown lower block shear resistances than would be expected if the stress on the tension face were assumed to be uniform. In these cases only one shear face exists, thus resulting in eccentric loading on the block of material that causes the non-uniform tensile stress distribution. Values of U_t vary from 1.0 for cases where no load eccentricity exists on the block of material (e.g. typical gusset plates) to 0.3 for cases with a large eccentricity (coped beams with two lines of bolts). The low efficiency of the tension face in coped beams with two lines of bolts was noted in the work of Franchuk *et al.* (2003). Driver *et al.* (2006) recommended that U_t be taken as 0.9 for angles connected by one leg and stem-connected tees, in combination with an analogous coefficient of 0.9 on the shear term. However, since S16 adopted the unified block shear equation without the shear coefficient, the value of U_t was modified to 0.6 to maintain the same reliability index for the pool of test data available. The simplified approach in the standard could produce non-conservative results for long blocks with a small tension area.

As illustrated in Figure 2-25, the block shear failure of structural tees can take various forms (Epstein and Stamberg, 2002), depending on whether the tee section is flange-connected or stem-connected. The first mode associated with flange-connected tees consists of tension and shear failure confined in the flange only. The other two modes associated with flange-connected tees involve a tension plane in the flange (with or without shear planes in the flange) and a shear plane in the stem. The various possible modes should be investigated. Use of the unified block shear equation for common types of welded connections is discussed by Oosterhof and Driver (2011).

Recommended values of U_t for various connection details are given in Figure 2-26.

13.12 Bolts and Local Connection Resistance

13.12.1 Bolts in Bearing-Type Connections

13.12.1.2 Bolts in Bearing and Shear

In bearing-type connections (Clause 13.12.1.2(a)) excessive deformation in front of the loaded edge of the bolt hole may occur. Tests have shown (Munse 1959; Jones, 1958; de Back and de Jong 1968; Hirano 1970) that the ratio of the bearing stress (B_r/dt) to the ultimate tensile strength of the plate (F_u) is in the same ratio as the end distance of the bolt (e) to its diameter (d). Thus,

$$\frac{B_r}{\phi d t} = \frac{e}{d} F_u$$

or, for n fasteners, $B_r = \phi_{br} t n e F_u$

Because the test results do not provide data for e/d greater than 3, an upper limit of $e = 3d$ is imposed. That is,

$$B_r \leq 3 \phi_{br} t d n F_u$$

For the bearing of bolts on steel, the value of ϕ_{br} in Clause 13.12.1.2 is to be taken as 0.80. For the bearing resistance perpendicular to long slotted holes, see Clause 13.12.1.2(b).

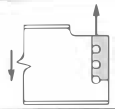
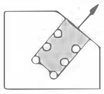
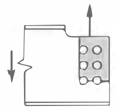
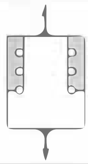
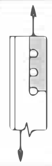
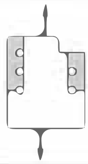
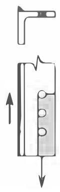
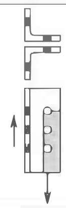
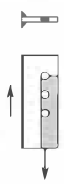
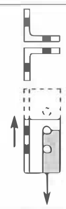
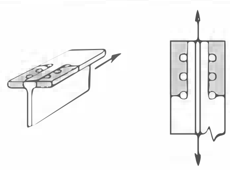
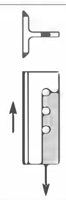
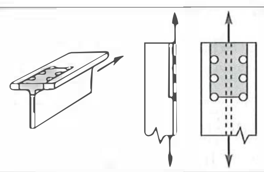
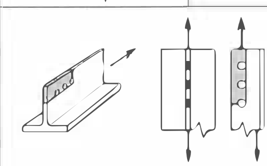
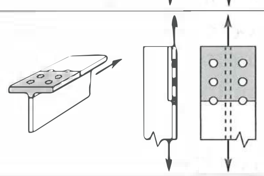
No.	Connections subject to block shear		U_t	No.	Connections subject to block shear		U_t
1		Coped beam with one row of bolts	0.9	7		Gusset plate, symmetrical block and uniform tensile stresses	1.0
2		Coped beam with two rows of bolts	0.3	8		End plate welded to supported beam, bolted to supporting member	0.9
3		Angle in tension connected to one leg	0.6	9		Similar to Case "8" above but with a clipped corner for erection safety	0.9
4		Angle in shear, one leg bolted to supported beam and other leg welded on 3 sides to supporting member	0.6	10		Double angles in shear, one leg bolted to supported beam and other leg bolted (or welded) to supporting member	0.6
5		Single plate (shear tab) bolted to supported beam, welded to supporting member	0.6	11		Similar to Case "10" above but with a clipped leg for erection safety (double-sided connection)	0.6
6a			0.9	12		Tee in shear, stem bolted to supported beam, flanges welded to supporting member	0.6
6b		Flange-connected Tee in tension	1.0	13		Stem-connected Tee in tension	0.6
6c			1.0	CSA S16-14 Clause 13.11: $T_r = \phi_u\,[U_t\,A_n\,F_u + 0.60\,A_{gv}\,(F_y + F_u)\,/\,2]$ $\phi_u = 0.75$			

Figure 2-26
Values of U_t for Block Shear

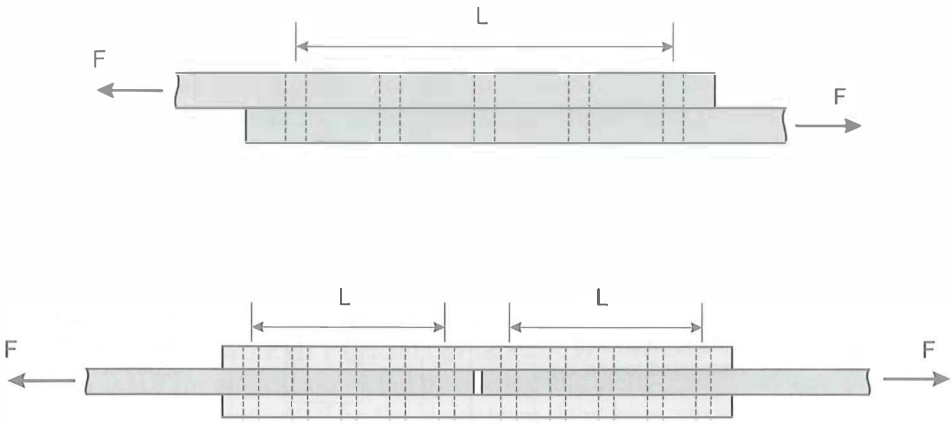


Figure 2-27
Lap Joint Length Definition for Lap and Butt Joints

The note directs designers to Clause 13.11, to investigate any potential for block tear-out when the end distance, e, is small and to Clause 22.3.4 for minimum end distances.

Based on extensive testing, it has been established that the shear strength of high-strength bolts is approximately 0.60 times the tensile strength of the bolt material. However, if threads are intercepted by a shear plane, there is less shear area available. The ratio of the area through the threads of a bolt to its shank area is about 0.70 for the usual structural sizes.

In the case of long joints, the load is not shared equally among the bolts with those fasteners towards the ends of the joint carrying the largest portion of the load. The linear reduction in S16-09 has been replaced by a step reduction in bolt capacity when the joint length, L, equals or exceeds 760 mm. This approach has also been adopted in CSA S6-14.

Note that the length L is that in which the load is transferred from one plate to another. For a lap joint with bolts in single shear, this is the total length between the centrelines of the end fasteners. For a butt joint with two lap plates and the bolts in double shear, it is the "half" length (see Figure 2-27).

In this context, "joint length" refers to an axially loaded connection, such as a lap splice, whose length is measured parallel to the direction of applied force. This clause does not apply to a shear connection at the end of a girder web where the load is distributed reasonably uniformly to the fasteners.

13.12.1.3 Bolts in Tension

The ultimate resistance of a single high-strength bolt loaded in tension is equal to the product of its tensile stress area (a value between the gross bolt area and the area at the root of the thread because the failure plane must intercept a thread) and the ultimate tensile strength of the bolt. The tensile stress area is very nearly equal to 0.75 of the gross area of the bolt.

In addition to the applied load, two other tensile forces – prying action and pretensioning – may act on the bolt, and their effects have to be examined. The Standard states, in fact, that the factored tensile force is independent of the pretension but that the tensile prying force shall be added to the external load.

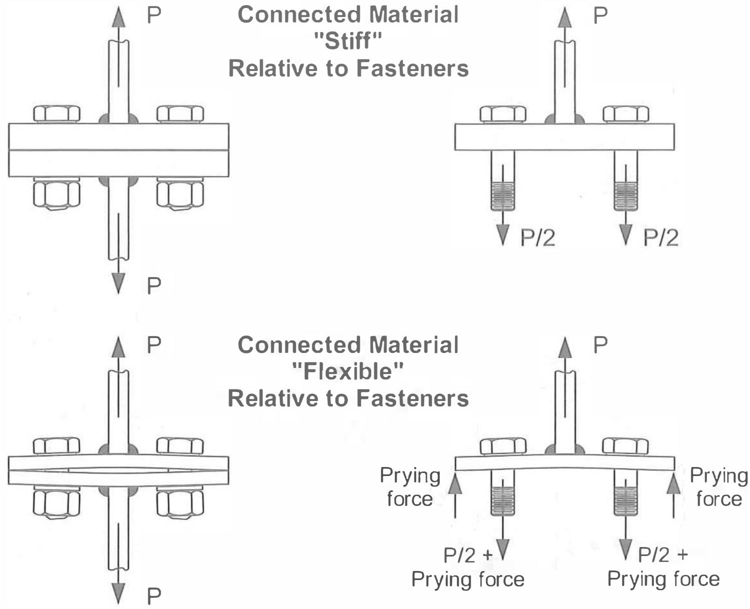


Figure 2-28
Effect of Prying Action on Bolt Tension

Figure 2-28 illustrates qualitatively that the amount of prying action depends on the flexibility of the connected material relative to the bolts. Kulak *et al.* (1987) present a procedure for calculating the prying force depending on the joint geometry, which is presented in Part 3 of this Handbook along with suggested detailing practices to minimize this force.

The statement that the factored tensile force is independent of the pretension derives from Figure 2-29 where, before any external load P is applied, the bolt pretension is balanced by the plate pre-compression. When the external load is applied without distorting the connected material as shown, or equivalently when the connected material is "stiff", as the external force is increased, the bolt force remains almost constant at the bolt pretension, while the contact pressure between the bolted plates decreases. Once the applied force is sufficiently large to separate the plates, the contact pressure goes to zero, and the sum of the bolt forces becomes equal to the applied external force. The level of bolt pretension therefore affects the force at which the bolted plates will separate, but it has no effect on the joint tension capacity.

On the other hand, when the external load is applied through some thickness of material causing it to compress, more bolt elongation is required and there is some increase in the bolt tension. Measurements of actual bolt forces in connections of practical sizes have shown that the increase in the bolt force due to the flexibility of the connection is usually only about 5 to 10%. The Standard neglects this. Figure 2-30 depicts possible variations of the tension on a pretensioned bolt as it is loaded with an external load, P, as pretensions, T_0, decrease and in the presence of a prying force, F.

This Standard requires that high-strength bolts subjected to tensile cyclic loading be fully pretensioned and that the prying force not exceed 30% of the externally applied load. Two options are given to calculate the tensile stress range to compare to the permissible values. The first and most difficult takes into account the prying action, the pretension with possible relaxation due to joint deformations and the applied load. The second assumes the range is that due

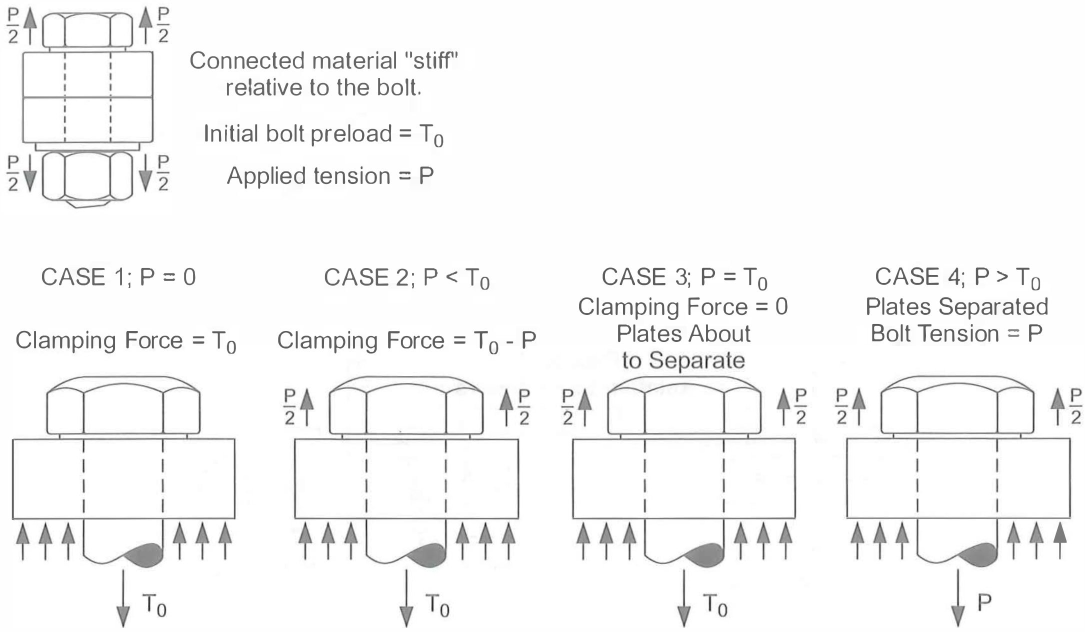


Figure 2-29
Effect of Applied Tension on Tightened High-Strengh Bolts

to the applied loads plus prying action. This is obviously conservative as the pretension reduces the applied load stress range.

13.12.1.4 Bolts in Combined Shear and Tension

The expression for the ultimate strength interaction between tension and shear applied to a fastener has been shown to model empirically the results of tests on single fasteners loaded simultaneously in shear and tension. The values of V_r and T_r are the full resistances in shear and tension, respectively, which would be used in the absence of the other loading. For small components of factored load relative to the resistance in one direction, the resistance in the other direction is reduced only a small amount; e.g. for a factored tension equal to 20 % of the full tensile resistance, the resistance available for shear is only reduced by 2 % of the full value that would be present in the absence of tension.

13.12.2 Bolts in Slip-Critical Connections

13.12.2.2 Shear Connections

Different installation procedures may result in different probabilities of slip; see Kulak *et al.* (1987).

Both the slip coefficient and the initial clamping force have considerable variation about their mean values. The coefficients of friction for coatings can vary as a function of the specific coating constituents and, therefore, values of the mean slip coefficient, k_s, may differ from one coating specification to another. The value of k_s intended for use on a project should be specified.

The clamping force is due to the pretensioning of the bolts to an initial tension, T_i, which is a minimum of 70% of the tensile strength ($0.70 A_s F_u$) where $A_s = 0.75 A_b$. Thus, the clamping force per bolt is

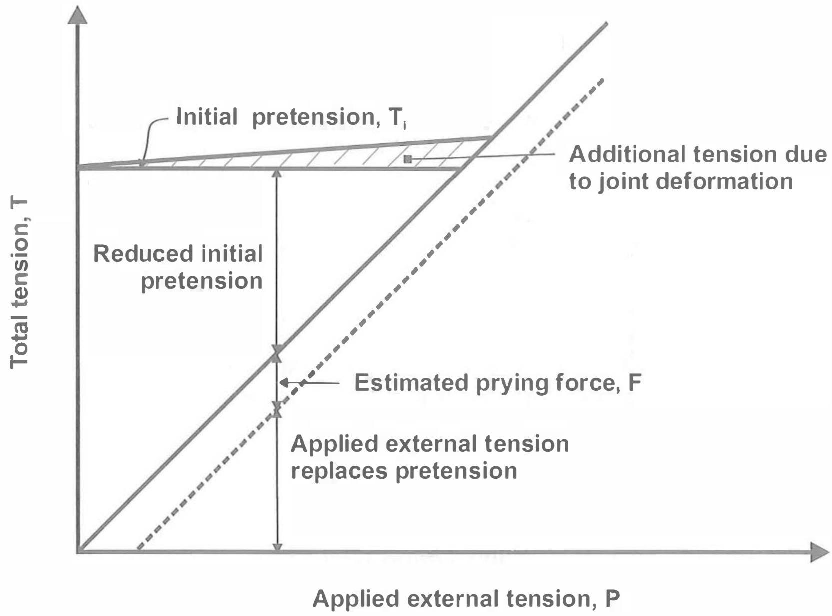


Figure 2-30
Total Tension vs. Applied Tension for a Pretensioned Bolt

$$0.70 \times 0.75 A_b F_u \quad \text{or} \quad 0.53 A_b F_u$$

The values of the resistance factor, c_s, establish a uniform probability level of slip for the bolt grades and the installation methods. Table 3 of S16-14 gives values of c_s for bolts installed by the three pretensioning methods permitted by the Standard: (a) turn-of-nut method for A325, A325M, A490 and A490M bolts, (b) F1852 and F2280 twist-off type of tension-control bolt assemblies, and (c) use of washer-type direct tension indicators (F959 washers with A325, A325M, A490 and A490M bolts). For methods (b) and (c), smaller c_s values are given as the clamping loads obtained are lower (but still above the minimum required by the Standard) than those obtained by the turn-of-nut procedure. Table 3 also provides the values of k_s for two classes of contact surface. In S16-14, hot-dip galvanized surfaces are grouped under Class A. Values of k_s for some other common surface conditions are given by Kulak *et al.* (1987).

The use of slip-critical connections should be the exception rather than the rule. They are the preferred solution only where cyclic loads or frequent load reversals are present, or where the use of the structure is such that the small one-time slips that may occur cannot be tolerated. See also the Commentary to Clause 22.2.2.

The slip resistance is reduced by a factor of 0.75 for slip-critical connections using long slotted holes to account for the reduced clamping force that otherwise would be present (Kulak *et al.* 1987).

13.12.2.3 The resistance to slip is reduced as tensile load is applied and reaches zero when the parts are on the verge of separation, as no clamping force then remains. The interaction relationship is linear.

The term $1.9/(n A_b F_u)$ is the reciprocal of the initial bolt tension, $0.53\, n A_b F_u$.

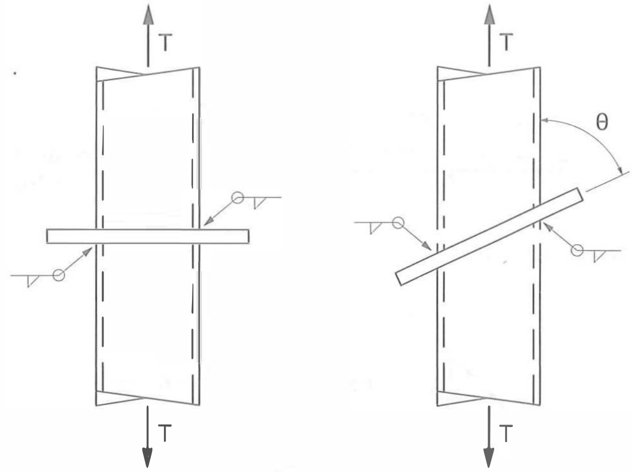


Figure 2-31
Fillet Welds to HSS

13.13 Welds

13.13.1 General

Clause 13.13 covers resistances for welded joints that satisfy matching conditions; provisions and restrictions for use of non-matching electrodes are included in Clause 24. Matching electrodes for various grades of base steel as specified in Table 4 of S16-14 and W59-13 typically are those with ultimate strengths similar to that of the base metal. When atmospheric corrosion-resistant steel grades are used in the uncoated condition, additional requirements for compatible corrosion resistance or colour are also required for matching electrodes.

A resistance factor of $\phi_w = 0.67$ is used universally in this section, recognizing that a larger value of the reliability index is used for connector resistances.

13.13.2 Shear

In general, the shear resistance of a weld is evaluated on the basis of both the resistance of the weld metal and of the base metal adjacent to the weld. Although the calculations indicate that the resistance of the base metal may govern the capacity of the welded joint, this is seldom the case. Thus, CJPG, PJPG, plug, and slot welds loaded in shear have resistances equal to the lesser of the weld throat or fusion face shear strength. Research on fillet-welded splices (Butler *et al.* 1972; Miazga and Kennedy, 1989; Ng *et al.*, 2004a; 2004b; Deng *et al.*, 2006; Callele *et al.* 2009) showed that even when fillet welds failed primarily in the fusion zones, the capacity of the weld calculated according to the weld metal capacity only provided a sufficient level of safety with a reliability index of about 4.5. Therefore, for fillet welds oriented at an angle greater than about 45°, where the calculation of the base metal strength indicates that the strength of the base metal would govern the capacity of the joint, the base metal check effectively prevents the designer from taking advantage of the full capacity of the weld. It was concluded by Callele *et al.* (2009) that the tensile strength of the base metal does not represent the actual tensile strength of the material at the fusion face, which is influenced by intermixing of the weld and base metals. According to Clause 13.13.2.2, when over-matched electrodes are used the base metal check is required for the design of fillet welds. However, W59-13 requires that the resistance be calculated using the tensile strength of the matching electrode even if electrode classification higher than matching is used.

Using the instantaneous centre of rotation concept, the resistance expression in 13.13.2.2 forms the basis of the eccentric load tables given in Part 3 of the CISC Handbook (Butler and Kulak 1971, Butler *et al.* 1972, Miazga and Kennedy 1989, Lesik and Kennedy 1990, Kennedy *et al.* 1990). This ultimate strength analysis, recognizing the true behaviour of the weldments, results in much more consistent strength predictions than the traditional approach (i.e., taking the quantity $1.00 + 0.50\sin^{1.5}\theta$ as 1.0).

In the expression for the shear strength of the weld, the factor 0.67 relates the shear strength of the weld to the weld metal tensile strength, as given by the rated electrode classification number. Lesik and Kennedy (1990) give 0.75 for this factor, based on 126 tests reported in the literature. The coefficient 0.50 in the quantity $1.00 + 0.50\sin^{1.5}\theta$ is for tension-induced shear and is slightly more liberal than the average value of tension- and compression-induced shear of 1.42 reported by Lesik and Kennedy. In addition, the factor 1.50 is the correct value for Clause 13.13.2.2 in which tension is the critical case. The value of 0.50 has also been adopted by AWS and AISC. However, recent experimental research on welded HSS joints (Packer *et al.* 2015) has shown that, in order to yield a reliability index, $\beta = 4.5$, the fillet weld "directional strength enhancement factor" ($1.00 + 0.50\sin^{1.5}\theta$) for fillet welds to HSS as shown in Figure 2-31 should be used with a ϕ_w value lower than 0.67. For this application, Packer *et al* (2015) recommend setting this factor to unity (i.e. $\theta = 0$) and keeping $\phi_w = 0.67$ for a conservative solution (i.e. $\beta > 4.5$). For further discussion on fillet welds to HSS when the "effective length concept" is used to proportion fillet welds, see the Commentary to Clause 13.13.4.3.

Callele *et al.* (2009) showed that when fillet welds with multiple orientations are contained within the same concentrically loaded joint, the lower ductility of the welds oriented closest to 90° prevents the more ductile welds from reaching their full capacity before failure of the joint takes place. The researchers proposed a simple means of accounting for this phenomenon by reducing the capacity of the more ductile weld segments by up to 15%. This method has been adopted into the Standard using the factor M_w.

Clause 13.13.2.3 provides users of the Standard with an expression to determine the factored resistance of flare bevel groove welds for open-web steel joists based on (a) observed data relating the face width to the effective throat thickness of flare bevel groove welds as reported by Skarborn and Daneff (1998), (b) other data on welds in general from Lesik and Kennedy (1990), and (c) the principles set forth in Galambos and Ravindra (1973). Thus, using $\phi_w = 0.67$ with the effective throat taken as 0.50 of the weld face as selected here leads to a reliability index of 4.25 as determined by Kennedy (2004).

13.13.3 Tension Normal to Axis of Weld

Gagnon and Kennedy (1989) established that the net area tensile resistance, i.e. on a unit area basis, transverse to the axis of a PJPG weld, is the same as for the base metal when matching electrodes are used. The previous conservative practice of assigning shear resistances to these welds was replaced in the 1989 edition with tensile resistances, consistent with the tensile resistance of complete penetration welds equalling the full tensile resistance of the member.

For T-type joints consisting of PJPG weld and a reinforcing fillet weld, Clause 13.13.3.3 provides a conservative estimate of the tensile resistance by taking the vector sum of the individual component resistances of the PJPG and fillet welds.

13.13.4.3 Welds for Hollow Structural Sections

There are two methods currently available for the design of welded connections between square and rectangular HSS (Packer *et al.*, 2010; McFadden *et al.*, 2013):

1) The welds may be designed as "fit-for-purpose" and proportioned to resist the applied forces in the branch. The non-uniform loading around the weld perimeter due to the relative

flexibility of the connecting RHS face requires the use of weld effective lengths. This approach may be appropriate when there is high confidence in the design forces or if the branch forces are particularly low relative to the branch member capacity. Where applicable, this approach may result in smaller weld sizes, providing a more economical design. Weld effective lengths, related to the type of HSS connection and type of loading, have been determined from research by Frater and Packer (1992a, 1992b), Packer and Cassidy (1995), McFadden and Packer (2014) and Tousignant and Packer (2015). An up-to-date summary of weld effective lengths (or weld effective properties) for HSS connections is given in Section K4 of the Specification AISC 360 (AISC 2010b).

However, the fillet weld "directional strength enhancement factor" $(1.00 + 0.50\sin^{1.5}\theta)M_w$, contained in Clause 13.13.2.2, should *not* be applied to fillet welds to HSS when the "effective length concept" is used to proportion fillet welds (McFadden and Packer, 2014; Tousignant and Packer, 2015).

2) The welds may be proportioned to develop the yield strength of the connected branch wall at all locations around the branch. This approach may be appropriate if there is low confidence in the design forces, uncertainty regarding Method (1) above, or if plastic stress redistribution is required in the connection. This method will produce an upper limit for the weld size required and may be excessively conservative in some situations.

13.14 Welds and High-Strength Bolts in Combination

This clause addresses the design of joints in which welds and high-strength bolts are placed in the same shear plane and are expected to share the applied shear force. The provisions are based on the work of Manuel and Kulak (1999) and Kulak and Grondin (2003). The capacity of each connector in this type of shear splice is reflected by its shear strength and shear deformation characteristics. When bolts and welds share the load, the fastener that possesses the least ductility (welds as opposed to bolts or transverse welds as opposed to longitudinal welds) is able to reach its full capacity before the full capacity of the more ductile fastener is fully developed. Therefore, the shear resistance of the joints consists of the full capacity of the least ductile fastener plus a fraction of the capacity of the more ductile fastener. The resistance of the joint is calculated based on the progression of failure from the least ductile fastener to the most ductile fastener. Consequently, the capacity of a typical joint that combines transverse and longitudinal welds and bolts could be limited by (i) the load at which the transverse weld fractures, (ii) the load at which the longitudinal welds fracture, or (iii) the load at which the bolts fracture.

When considering case (i), tests by Manuel and Kulak have shown that the ductility of transverse welds is insufficient to mobilize a significant portion of the bolt shear strength, but sufficient to mobilize about 85% of the strength of the longitudinal welds. Case (ii) considers that the transverse weld, if present, has already fractured. In this case, the longitudinal welds are sufficiently ductile to mobilize a significant portion of the bolt shear strength. The work of Manuel and Kulak showed that the portion of the bolt shear strength that is mobilized by the time the longitudinal welds have fractured depends on the bearing conditions of the bolts at the time that the welds are added to the joint. They made a distinction between the case where the bolts are in full bearing in the direction of the applied load (positive bearing) and the case where the bolts are in bearing in the direction opposite to the applied load (negative bearing).

The results of later tests presented by Kulak and Grondin showed that joints where the bearing conditions are varied randomly could develop at least 50% of the shear strength of the bolts by the time the longitudinal welds fracture. Case (iii) considers the situation where both the transverse and longitudinal welds have fractured. At this point, only the bolts are able to resist the applied load. It should be noted that in cases (i) and (ii) a contribution from the slip resistance can be accounted for when the bolts have been pretensioned in accordance with Clause

23.7. However, in case (iii) no slip resistance is accounted for since the shear deformation in the bolts at the time that their full strength has been mobilized is sufficient to have released their pretension.

Equation (a) of Clause 13.14 considers case (i) described above. For this case, only the welds contribute to the shear resistance plus 25% of the slip resistance if the bolts are pre-tensioned. The strength of the welds is calculated using Clause 13.13.2.2 with $\theta = 90°$ for the transverse weld segment and $\theta = 0°$ for the longitudinal weld segment. The factor 0.85 is the value of M_w when longitudinal and transverse welds are combined in the same shear plane. Equation (b) considers case (ii) where the transverse weld has already fractured, and only the longitudinal welds and the bolts are left to carry the load. By the time the ductility of the longitudinal welds has been exhausted, 50% of the shear capacity of the bolts would be mobilized. Equation (c) considers case (iii) where only the bolts are left in the joint. At this stage the limit state is fracture of the bolts, and the strength of the joint is limited to the shear resistance of the bolts or the bearing resistance of the plates against the bolts.

It should be noted that in all the cases tested experimentally, the bolt resistance was always governed by bolt shear rather than plate bearing. Since the bearing resistance usually requires more deformation to develop than the shear resistance, it is possible that the contribution from the bolts may be less than 50% when plate bearing governs the bolt resistance. When bearing governs, the designer may want to use less than 50% of the bolt shear resistance. For typical examples of joint strength calculations, see Kulak and Grondin (2003).

14. BEAMS AND GIRDERS

14.1 Proportioning

Lilley and Carpenter (1940) have shown that reductions of flange area up to 15% can be disregarded in determining the effective moment of inertia, due to the limited inelastic behaviour near the holes.

14.2 Flanges

The theoretical cut-off point is the location where the moment resistance of the beam without cover plates equals the factored moment (Figure 2-32). The distance a' increases as shear lag becomes more significant, as is the case when the weld size is smaller, or when there is no weld across the end of the plate. Theoretical and experimental studies of girders with welded cover plates (ASCE 1967) show that the cover plate load can be developed within length a'. Clause 14.2.4 limits the length of a' for welded cover plates and may therefore necessitate an increase in weld size or an extension of the cover plate so that the force at a distance a' from its end equals that which the terminal welds will support.

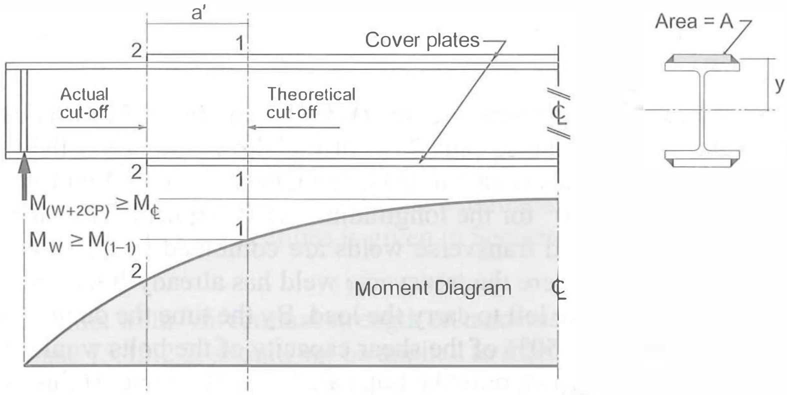
14.3 Webs

14.3.1 Maximum Slenderness

This limit prevents the web from buckling under the action of the vertical components of the flange force arising as a result of the curvature of the girder (Kulak and Grondin 2014).

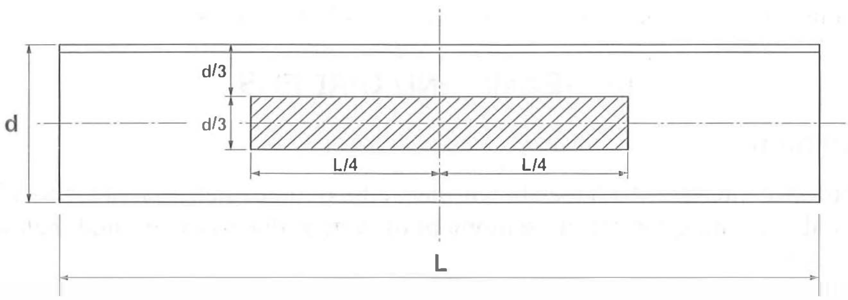
14.3.2 Web Crippling and Yielding

Loads and reactions acting perpendicular to a flange and over a short length along the flange will cause in-plane compressive stresses in the web. The ultimate strength of the unstiffened web subjected to such edge loading may be governed by either yielding of the web or crippling of the web (a localized out-of-plane buckling of the web adjacent to the loaded flange).



Force to be developed in cover plate within distance **a′** is: $\dfrac{AM_{(1-1)}y}{I_{(W+2CP)}}$

Figure 2-32
Cover Plate Development



Unreinforced circular holes may be placed anywhere within the hatched zone without affecting the strength of the beam for design purposes, provided:

1. Beam supports uniformly distributed load.
2. Beam section has an axis of symmetry in plane of bending.
3. Spacing of holes meets the requirements shown below.

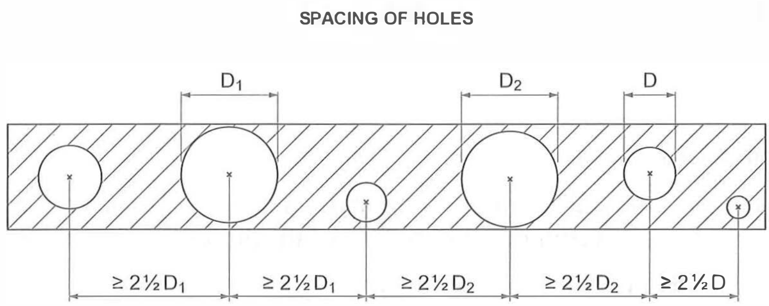


Figure 2-33
Unreinforced Circular Web Openings in Beams

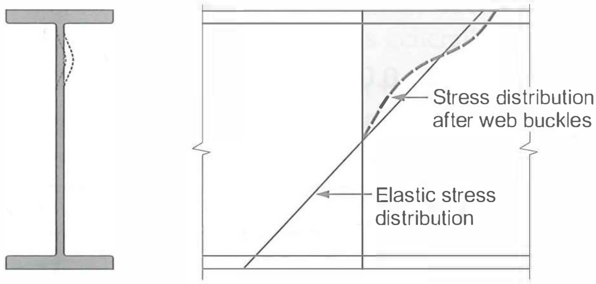


Figure 2-34
Approximate Stress Distribution in Girders with Buckled Web

If the web is relatively stocky, yielding will occur prior to crippling, and expressions 14.3.2(a)(i) and 14.3.2(b)(i) govern web resistances for interior loads and end reactions, respectively.

Relatively thin webs cripple before yielding, and the strength of the web is governed by expressions 14.3.2(a)(ii) and 14.3.2(b)(ii) for interior loads and end reactions, respectively.

The equations presented in the Standard are based on the work of Kennedy *et al.* (1998). These equations are much simplified relative to the 1994 Standard and correlate well with a set of 31 full-scale tests by Benichou (1994) and others at Carleton University. In the expression for web crippling, the contribution of the flange is neglected. It is argued that, at interior load points, the normal stress in the flanges of efficiently designed girders would approach the yield stress at factored loads. Consequently the flanges would not have significant plastic hinge capacity in developing a plastic hinge mechanism in the resistance of transverse loads.

For unstiffened portions of webs, when concentrated compressive loads are applied opposite one another to both flanges, the compressive resistance of the web acting as a column should also be investigated. (See also Clause 21.3.)

Care should be taken in assessing the bearing length under yielding or deforming supports such as girders cantilevering over columns.

14.3.3 Openings

The conditions under which unreinforced circular openings may be used are based on Redwood and McCutcheon (1968) and are illustrated in Figure 2-33.

Elastic and plastic analyses to determine the effect of openings in a member are given in Bower *et al.* (1971) and Redwood (1971, 1972, 1973), respectively. See Part 5 of the Handbook for worked examples.

A combination of vertical and horizontal intersecting stiffeners (particularly on both sides of a web) is seldom justified and quite expensive to fabricate. Generally, horizontal stiffeners alone are adequate. When both vertical and horizontal stiffeners are necessary, the horizontal stiffeners should be on one side of the web, and vertical stiffeners on the other, in order to achieve economy.

14.3.4 Effect of Thin Webs on Moment Resistance

A plate girder with Class 3 flanges and Class 4 webs has a maximum moment resistance less than ϕM_y because the Class 4 web may buckle before extreme fibre yielding due to the compressive bending stresses. The reduction in moment resistance is based on Basler and

$$\frac{M'_r}{M_r} = 1.0 - 0.000\,5\frac{A_w}{A_f}\left[\frac{h}{w} - \frac{1900}{\sqrt{M_f/(\phi S)}}\right]$$

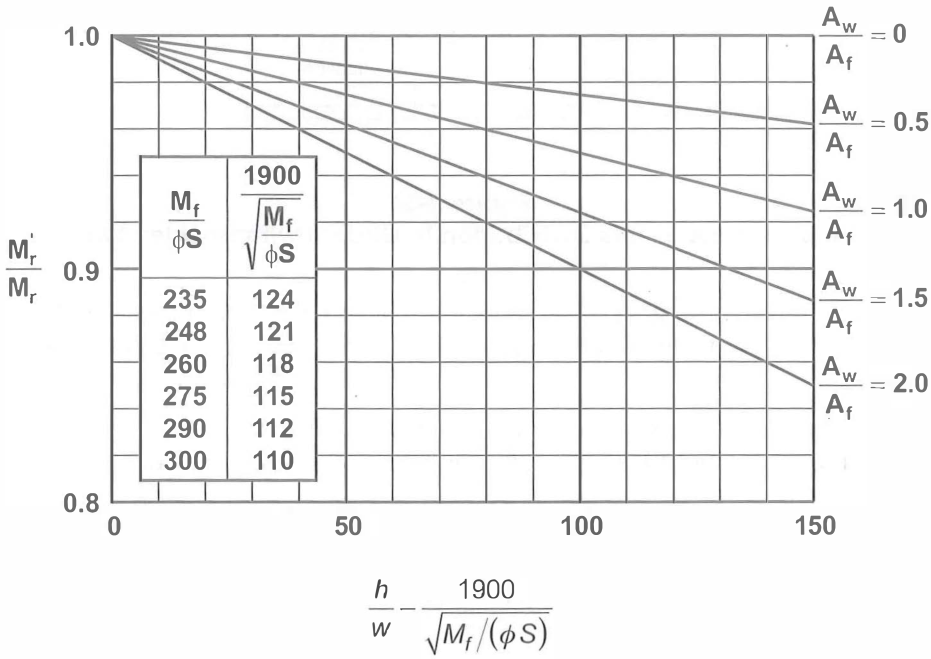


Figure 2-35
Reduced Moment Resistance in Girders with Thin Webs

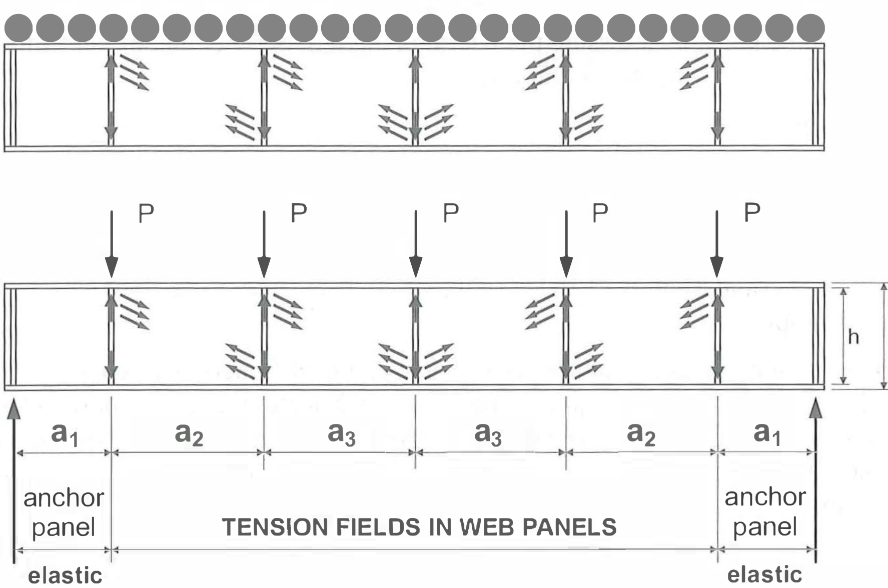


Figure 2-36
Action of a Thin-Web Plate Girder Under Load

Thurlimann (1961). Figure 2-34 shows an approximate stress distribution in a girder with a buckled web. The reduction in moment resistance is generally small, as shown in Figure 2-35.

The limit of $1900/\sqrt{F_y}$ for the slenderness of a Class 3 web is replaced in this clause by $1900/\sqrt{M_f/(\phi S)}$ to account for the possibility that the factored moment may be less than $M_r = \phi S F_y$, thereby reducing the propensity for web buckling.

In some circumstances, a plate girder may be subjected to an axial compressive force in addition to the bending moment (e.g. rafters in a heavy industrial gable frame, beams in a braced frame). The constant 1900 is then multiplied by the factor $(1.0 - 0.65\, C_f/\phi C_y)$ to account for the increased tendency for the web to buckle. The compressive stresses due to the axial load add to the compressive stress due to bending, thus increasing the depth of web in compression (see also commentary to Clause 11).

14.4 Bearing Stiffeners

The inclusion of a portion of the web in the column section resisting the direct load, and the assumption of an effective length of 0.75 times the stiffener length, are approximations to the behaviour of the web under edge loading that have proved satisfactory in many years of use.

14.4.2 In S16-14, the limit for width-to-thickness ratio for bearing stiffeners is explicitly stated. This limit corresponds to the Class 3 limit for plate elements supported along one edge and therefore applies to plate stiffeners for single web girders. For other types of stiffeners and stiffeners with other edge support conditions, Class 3 limits appropriate for each respective stiffener type and support condition may apply.

Where the Class 3 limit is exceeded because the plate width exceeds what is needed to satisfy other requirements in this Clause, the effective area method in accordance with Clause 13.3.5(a) may be used.

14.5 Intermediate Transverse Stiffeners

14.5.1 Figure 2-36 illustrates the action of a thin girder web under load. Tension fields are developed in the interior panels but cannot develop in the unanchored end panels, for which the maximum shear stress is, therefore, either the elastic or inelastic critical plate buckling stress in shear.

14.5.2 The limits on stiffener spacing are based on practical considerations. When $a/h > 3$, the tension field contribution is reduced. For slender webs ($h/w > 150$) the maximum stiffener spacing is reduced for ease in fabrication and handling.

14.5.3 Clause 14.5.3 requires that intermediate transverse stiffeners have both a minimum moment of inertia and a minimum area. The former provides the required stiffness when web panels are behaving in an elastic manner; the latter ensures that the stiffener can sustain the compression, to which it is subjected, when the web panel develops a tension field. Because stiffeners subject to compression act as columns, stiffeners placed only on one side of the web are loaded eccentrically and are less efficient. The stiffener factor (D) in the formula for stiffener area accounts for the lowered efficiency of stiffeners furnished singly, rather than in pairs.

14.5.4 The minimum shear to be transferred between the stiffener and the web is based on Basler (1961c).

14.5.5 The requirement of attaching single intermediate stiffeners to the compression flange is to prevent tipping of the flange under loading.

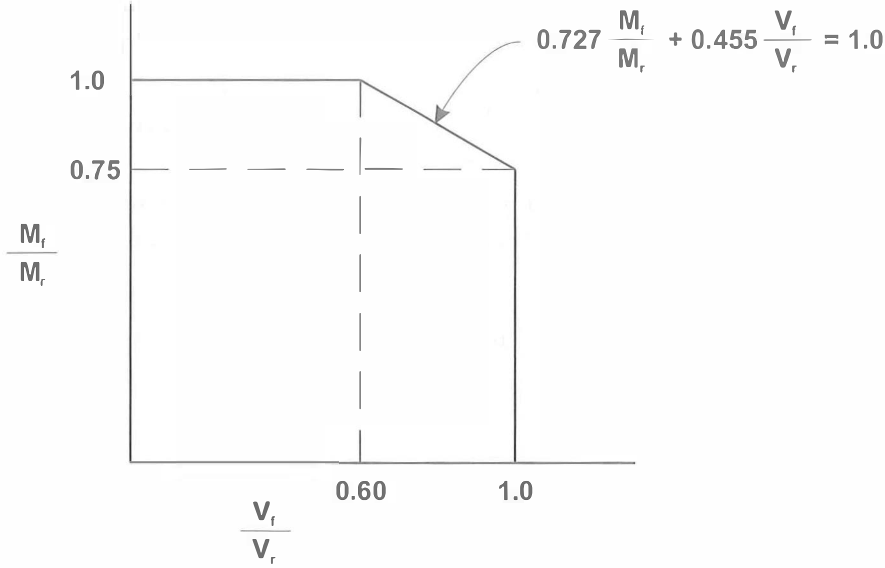


Figure 2-37
Combined Shear and Moment Interaction Expression

14.6 Combined Shear and Moment

This requirement recognizes the limit state of the web yielding by the combined action of flexural stress and the post-buckling components of the tension field development in the web near the flange (Basler, 1961b).

Figure 2-37 illustrates the interaction expression provided in Clause 14.6. When Clause 14.3.4 applies, M_r' replaces M_r in the interaction expression.

14.7 Rotational Restraint at Points of Support

A severe stability problem may exist when a beam or girder is continuous over the top of a column. The compression flange of the beam tends to buckle sideways and simultaneously, the beam-column junction tends to buckle sideways because of the compression in the column. Three mechanisms exist for providing lateral restraint: direct-acting bracing, such as provided by bottom chord extensions of joists, beam web stiffeners welded to the bottom flange, or the distortional stiffness of the web. In the latter two cases, the connection of the beam flange to the column cap plate must have strength and stiffness (Chien, 1989). The restraint offered by the distortion of the web requires very careful assessment. See also the commentaries on Clause 13.6 and Clause 9.2.

14.8 Copes

Flanges are coped to permit beams to be connected to girder webs with simple connections while maintaining the tops of the flanges at the same elevation. Long copes may seriously affect the lateral-torsional buckling resistance of a beam (Cheng and Yura, 1986). The reduced shear and moment resistance at the coped cross-section should be examined. See the Commentary on Clause 9.2.4.

14.10 Torsion

In many cases, beams are not subject to torsion because of the restraint provided by slabs, bracing or other framing members. The torsional resistance of open sections having two flanges consists of the St. Venant torsional resistance and the warping torsional resistance.

Information on moment–torque interaction diagrams for I-shaped members for use in design is given in Driver and Kennedy (1989), Bremault *et al.* (2008) and Estabrooks and Grondin (2008). Serviceability criteria will often govern the design of a beam subject to torsion. The maximum stress due to bending and warping at the specified load level shall be limited to the yield strength to guard against inelastic deformation. For inelastic torsion of steel I-beams, see Pi and Trahair (1995). For elastic analyses, see Seaburg and Carter (1997), and Brockenbrough and Johnston (1974). For methods of predicting the angle of twist in a wide-flange shape beam, see Englekirk (1994).

15. TRUSSES

15.1 Analysis

A "pure" truss is a triangulated system with pinned joints and with loads applied only at the joints. This being the case, the members of the truss are axially loaded "two-force" members acting either in tension or compression. Such trusses are now seldom made, and the members meeting at a joint are likely welded or bolted together, and not infrequently the chords are continuous through several joints. Under these circumstances, when the truss is loaded and the members change length, the geometry of the triangles (including the angles) changes, resulting in rotations of the joints, and end moments develop in the members, causing single or double in-plane curvatures. These deformation moments are called secondary moments, as they are not due to the primary loading but solely due to the deformation of the truss with rigid joints. Moreover, because the truss members are much stiffer axially than they are flexurally, several researchers (Parcel and Murer 1934, Aziz 1972) have shown that, for steel trusses with rigid welded or bolted joints, after initial elastic behaviour the extreme fibres of the members begin to yield under the axial and bending strains. With further axial straining, the moment that can coexist decreases and approaches zero, as shown schematically in Figure 2-38, when all the strains in a member (though not uniform) are either in compression or tension. Thus the truss with sufficient ductility, even with rigid joints, behaves as though its members were pin-ended.

Primary moments are moments that can be induced in truss members due to loadings or due to connection geometry. Sometimes, for example, a top chord is used to support a roof deck directly and the transverse loads between joints bend the chord and induce end moments at the panel points, which are distributed among the members meeting at a joint with some moments carried over to other joints. Thus, there are primary moments distributed throughout the truss. A common procedure is to analyze such a truss as a pin-jointed assemblage and to add to the forces so found the moments due to the transverse loadings.

Primary moments are also induced when the centroidal axes of the members meeting at a joint do not intersect at a common point, causing a rotation of the joint. These can be analyzed as for the other primary moments, taking the truss members as axially loaded members with the bending moments added. If the trusses with primary moments are analyzed using, say, an elastic plane frame analysis, then the stress resultants found will include the axial forces in the members and both the primary and secondary moments. Because the secondary moments for ductile trusses are of little or no consequence, trusses proportioned on this basis will be stronger than they need be.

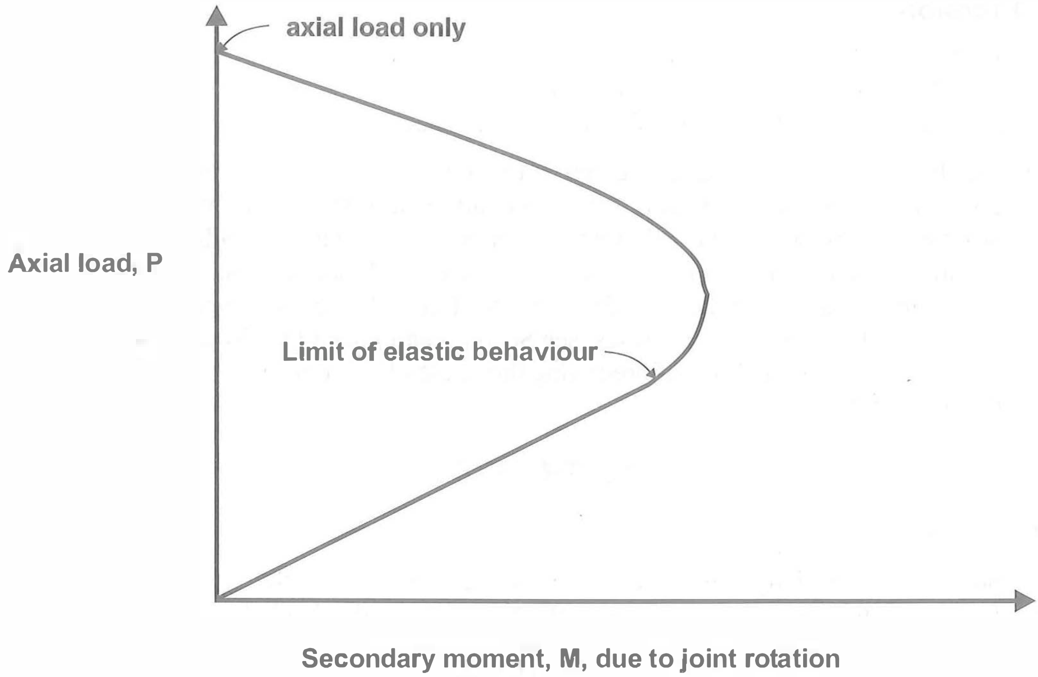


Figure 2-38
Axial Load – Secondary Moment Interaction Diagram
for a Rigid-Jointed Ductile Steel Truss

15.1.1 Simplified Method

The "Simplified Method" of analysis based on pin-connected truss members predicts closely the failure load of the tests, even with large rigid connections, provided there is sufficient ductility at the connections, so that redistribution of forces and moments may take place at the joints as the failure load is approached. Thus the sections must be at least Class 3. Bending effects of transverse loads applied between joints are simply treated as additional load actions to be carried. Out-of-plane buckling of compression members is conservatively not allowed. Alternatively, though not stated, the reduced strength of the truss because of this failure mode could be taken into account.

15.1.2 Detailed Method

This Clause lists the type of trusses for which the assumption of pin connections is not considered valid. Joint fixity must be considered, and the members must be designed for the combination of axial load and bending.

15.2.1 Effective Lengths of Compression Members

The potential failure modes of compression members in trusses are either in-plane bending or buckling modes. The effective length factors are, therefore, either taken to be equal to one or are based on the restraint at the ends. Thus, the following situations arise for in-plane and out-of-plane behaviour.

(a) In-plane behaviour

A compression member with bolted or welded end connections and with in-plane joint eccentricities acts in-plane as a beam-column with axial forces and end moments that can be established. It can be isolated from the structure and is designed as a beam-column based on its actual length, that is, with an effective length factor of 1.0.

A compression member with bolted or welded end connections and without in-plane joint eccentricities, designed as an axially loaded member, has end restraints, provided that all members meeting at the two end joints do not reach their ultimate loads (yielding in tension or buckling in compression) simultaneously. The effective length factor depends on the degree of restraint. This typically occurs for trusses in which some members are oversize, for example, trusses with constant size chords. All members do not fail simultaneously, and the effective length factors may be less than one.

If, however, all members reach their ultimate loads simultaneously and none restrain others, the effective length factor should be taken as 1.0.

(b) Out-of-plane behaviour

Unless members out-of-plane of the truss exist at the end joints under consideration, the restraint to out-of-plane buckling is small and should be neglected. Provided no out-of-plane displacement of the members' ends occurs, an effective length factor of 1.0 is therefore appropriate. It should be noted that Clause 13.3.3 provides a modified slenderness-ratio method, which accounts for the end eccentricity and fixity, for single-angle members that comply with the conditions stated in that Clause.

15.2.2 Joint Eccentricities

When the centroidal axes of the truss members do not intersect at a common point, the Standard requires that the bending moment due to the joint eccentricities be considered in the design.

15.2.3 Stability

Lateral bracing, which provides stability to the compression chords of trusses, must have stiffness and strength to satisfy the requirements of Clause 9.2. Braces must be properly attached to the member being braced, and their ends must be fastened to rigid supports.

15.2.5 Web Members

It has been observed, on occasion, in tests of standardized trusses and joists that the first compression web member fails first, even though the truss deformations may be quite significant. In these cases, certain chords and webs had been designed to S16 requirements to reach their factored loads more or less simultaneously. Because the tension chord, after yielding in the panel where the bending moment is a maximum, continues to carry load into the strain-hardening range, it overloads itself and the truss. The first compression web member with no such reserve then fails by buckling. By reducing the resistance factors for this member and its connections to 85%, more ductile modes of failure are encouraged at little extra cost. This requirement is also applied to joists in Clause 16.5.7.

In tests of trusses where the bottom chord bears on a reaction, severe bending deformations have been observed near the connections of the end compression diagonal because of the geometric distortion of the truss as deflections increase. The Standard requires that the stresses arising from these bending moments be included in the design of the end diagonal. Thus, the analysis of trusses with the bottom chord bearing must be carried out using the Detailed Method.

15.2.6 Compression Chord Supports

A frequently used rule to provide full support (Winter 1960) is for a brace to have a capacity in the order of 2% of the force in the main compression member.

15.2.7 Maximum Slenderness Ratio of Tension Chords

The slenderness ratio of tension chords is limited to 240 simply to facilitate handling during erection. The exceptions to this are noted in the clause.

16. OPEN-WEB STEEL JOISTS

16.1 Scope

Open-web steel joists (OWSJ or joists), as described in Clause 16.2, are generally proprietary products whose design, manufacture, transport, and erection are covered by the requirements of Clause 16. The Standard clarifies the information to be provided by the building designer (user-purchaser) and the joist manufacturer (joist designer-fabricator).

16.2 General

The distinction between a standard and a non-standard OWSJ no longer exists, as OWSJs are designed specifically for each situation by the joist manufacturer.

This clause lists functions that joists may fulfil other than the simple support systems for floors or roofs. These include continuous joists, cantilever joists, joists in lateral-load-resisting systems and support for bracing members.

16.3 Materials

The use of yield strength levels reported on mill test certificates for the purposes of design is prohibited here as throughout the Standard. This practice could significantly lower the margin of safety because any deviation from the specified value has already been accounted for statistically in the bias value – the ratio of the mean strength to the specified minimum value. Thus, all design rules have been, and are, based on the use of the specified minimum yield point or yield strength. For structural members cold-formed to shape, the increase in yield strength due to cold forming, as given in CSA Standard S136, may be taken into account provided that the increase is based on the specified minimum values in the relevant structural steel material standard.

16.4 Design Documents

16.4.1 Building Structural Design Documents

The Standard recognizes that the building designer may not be the joist designer; therefore, the building structural design documents are required to provide specific information for the design of the joists. The information to be supplied includes a note that any drilling, cutting or welding has to be approved by the building designer.

Uplift and downward wind effects, as well as balanced, unbalanced, non-uniform and concentrated loads, are to be shown by the building designer. Figure 2-39 shows a sample joist schedule that could be used to record all gravity loads on joists and any in-plane wind load acting normal to the top chord. Prior to the introduction of National Building Code of Canada 2005, the significance of downward wind effects on roof members depended primarily on the wind-to-snow load ratio. The adoption of load combinations in companion action format in NBCC 2005 eliminated the application of the combination (reduction) factor when wind acts in combination with variable gravity loads. This change resulted in the addition of downward

Mark	Depth (mm)	Spacing (mm)	Specified Dead Load	Specified Live Load	Specified Snow Load	Specified Wind Load	Remarks
J1	600	1 300	4.0 kPa	2.4 kPa			$\Delta_{live} \le \frac{span}{320}$ Suggested I_{eff} for vibration = ______
J2	700	2 000	8.9 kN 1.5 kN/m 3m 12 000		4.38 kN/m 10.2 kN/m 3m 12 000	0.6 kN/m 12 000 -1.5 kN/m (uplift)	$\Delta \le \frac{span}{240}$

Figure 2-39
Joist Schedule

wind effects to snow or live load regardless of wind-to-snow load ratios. The NBCC (2015) requires the internal suction in combination with any external downward wind pressure to be included in the total downward wind effect.

All heavy concentrated loads such as those resulting from partitions, large pipes, mechanical, and other equipment to be supported by OWSJs, should be shown on the structural design documents. Small concentrated loads may be allowed for in the uniform dead load.

The building designer should specify the building Importance Category as defined in the NBCC (2015). Alternatively, the NBCC Importance Factors, I_S, I_W and I_E, as appropriate, and the importance factor for live load (see Clause 6.2.2) when not equal to 1.0, should be specified.

Options, such as attachments for deck when used as a diaphragm, special camber and any other special requirements should also be provided. Where vibration of a floor system is a consideration, it is recommended that the building designer give a suggested effective composite moment of inertia, I_{eff} (Murray *et al.* 1997). Because the depth of joists supplied among different joist manufacturers may vary slightly from nominal values, the depth, when it is critical, should be specified.

When sprayed fire protection is contemplated, reduce clearance by thickness of sprayed fire protection material.

Although steel joist manufacturers may indicate the maximum clear openings for ducts, etc. which can be accommodated through the web openings of each depth of their OWSJs, building designers should, in general, show on the building design drawings the size, location and elevation of openings required through the OWSJs (Figure 2-40). Large ducts may be accommodated by special design. Ducts which require open panels and corresponding reinforcement of the joist should, where possible, be located within the middle half of the joist to minimize shear effects. This information is required prior to the time of tendering to permit appropriate costing.

Specific joist designations from a manufacturer's catalog or from the AISC and Steel Joist Institute of the U.S.A. are not appropriate and should not be specified.

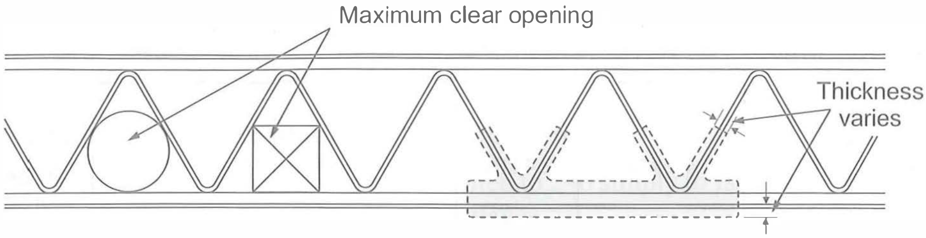


Figure 2-40
Sizes of Openings for Electrical and Mechanical Equipment

16.4.2 Joist Design Documents

The design information of a joist manufacturer may come in varying forms such as: design sheets, computer printout, and tables. Not all joist manufacturers make "traditional" detail drawings.

16.5.1 Loading for Open-Web Steel Joists

Maximum factored moments and shears are established either from the loading conditions in the design documents or from the loading conditions listed in Clause 16.5.1.

These loading conditions are consistent with Section 4.1 and Table 4.1.3.2.A of the National Building Code of Canada (2015). In particular, as required by the National Building Code of Canada, roofs and the joists supporting them may be subject to uplift loads due to wind.

16.5.2 Design Assumptions

The loads may be replaced by statically equivalent loads applied at the panel points for the purpose of determining axial forces in all members. It is assumed that any moments induced in the joist chord by direct loading do not influence the magnitude of the axial forces in the members. Tests on trusses (Aziz 1972) have shown that the secondary moments induced at rigid joints due to joint rotations do not affect the ultimate axial forces determined by a pin-jointed truss analysis.

16.5.5 Bottom Chord

A minimum radius of gyration is specified for bottom chord members, when in tension, to provide a minimum stiffness for handling and erection.

Under certain loading conditions, net compression forces may occur in segments of bottom chords and must be considered. Bracing of the chord, for compression, may be provided by regular bridging only if the bridging meets requirements of Clause 9.2. As a minimum, lines of bracing are specifically required near the ends of bottom chords in tension in order to enhance stability when the wind causes a net uplift.

Bottom chord bracing may be required for continuous and cantilever joists as shown in Figure 2-41.

In those cases, where the bottom chord has little or no net compression, bracing is not required for cantilever joists. However, it is generally considered good practice to install a line of bridging at the first bottom chord panel point as shown in Figure 2-41.

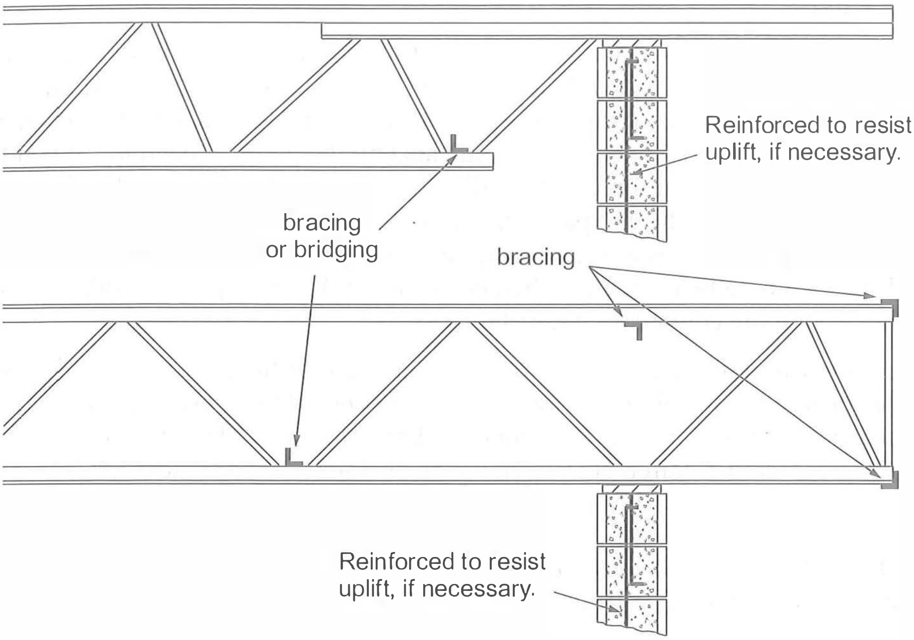


Figure 2-41
Bracing and Bridging of Cantilever Joists

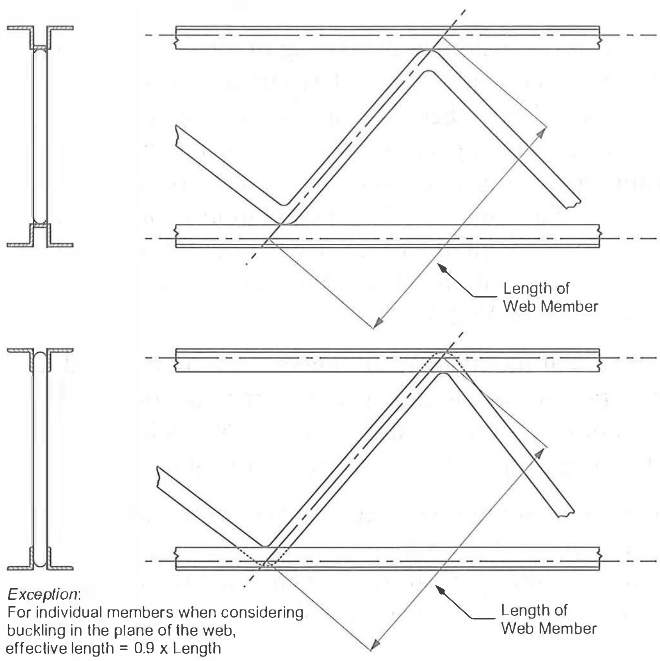


Figure 2-42
Length of Joist Web Members

16.5.6 Top Chord

When the conditions set out in Clause 16.5.6.1 are fulfilled, only axial force need be considered when the panel length is less than 610 mm (Kennedy and Rowan 1964). In these cases, the stiffness of the floor or roof structure tends to help transfer loads to the panel points of the joist, thus offsetting the reduction in chord capacity due to local bending. When the panel length exceeds 610 mm, axial force and bending moment need to be considered. When calculating bending moments in the end panel, it is customary to assume the end of the chord to be pinned, even though the joist bearing is welded to its support. The stiffening effect of supported deck or of the web is to be neglected when determining the appropriate width-thickness ratio (Clause 16.5.4.1) of the compression top chord.

The requirement in Clause 16.5.6.5, that the flat width of the chord component be at least 5 mm larger than the nominal dimension of the weld, should be considered an absolute minimum. Increasing the dimension may improve workmanship. See Clauses 16.8.5.1 and 16.8.5.2 regarding workmanship requirements when laying and attaching deck to joists.

16.5.6.6

S16-14 stipulates this minimum thickness of joist top chord when the deck is connected to it by mechanical fasteners. Joist top chords that are too thin do not work well with pins or screws.

16.5.7 Webs

The length of web members for purposes of design are shown in Figure 2-42. With the exception of web members made of individual members, the effective length factor is always taken as 1.0. For individual members this factor is 0.9 for buckling in the plane of the web (see Clause G7 of Annex G), but is 1.0 for buckling perpendicular to the plane of the web.

It has been observed, on occasion, in the testing of joists that with critical chords and webs designed to reach their factored loads more or less simultaneously using the S16 requirements, that the first compression web member fails first, even though the joist deformations may be quite significant. This appears to happen because the tension chord, after yielding in the panel where the joist bending moment is a maximum, continues to carry load into the strain-hardening range. It overloads itself and the joist. The first compression web member with no such reserve fails by buckling. By reducing the resistance factors for this member and its connections to 85%, more ductile modes of failure are encouraged at little extra cost. This requirement is also applied to trusses in Clause 15.2.5.

Vertical web members of modified Warren geometry are required to resist load applied at the panel point plus a bracing force to preclude in-plane buckling of the compression chord. A frequently used rule to provide full support (Winter 1960) is for a brace to have a capacity in the order of 2% of the force in the main compression member.

Web members in tension are not required to meet a limiting slenderness ratio. This is significant when flats are used as tension members. However, attention should be paid to those loading cases where the possibility of shear reversal along the length of the joist exists. Under these circumstances, it is likely that some diagonals generally near mid-span may have to resist compression forces.

16.5.8 Spacers and Battens

Spacers and battens must be an integral part of the joist, and the steel deck is not to be considered to act as spacers or battens (see Clause 16.5.6.2(c)).

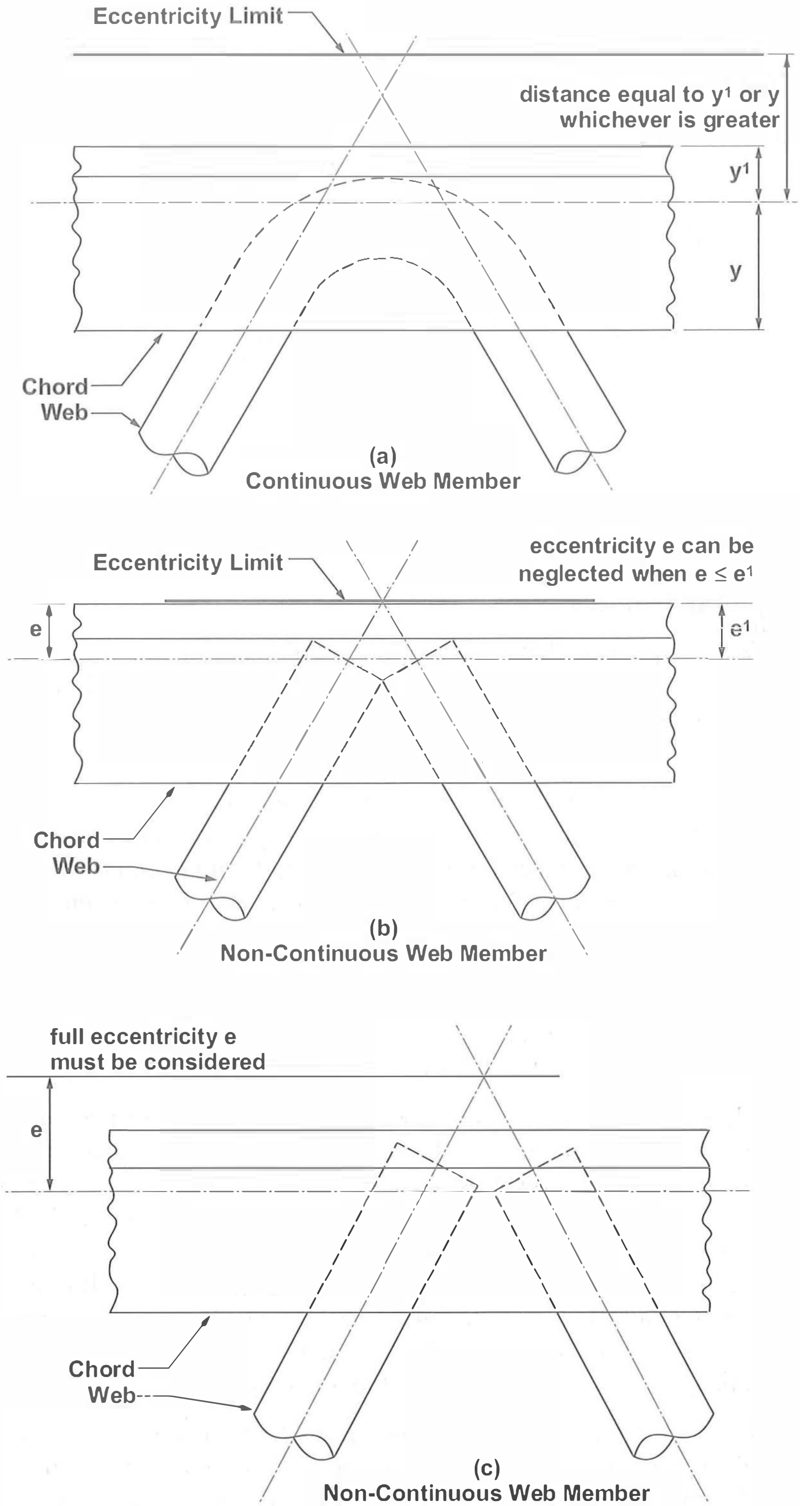


Figure 2-43
Eccentricity Limits at Panel Points of Joists

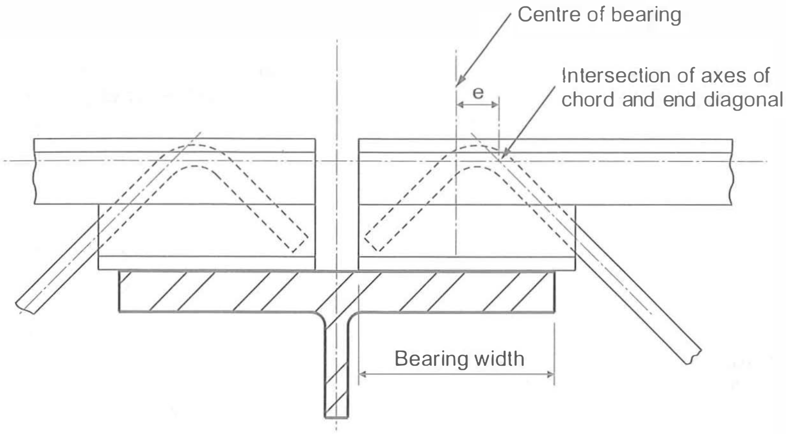


Figure 2-44
Joist End Bearing Eccentricity

16.5.9 Connections and Splices

Although splices are permitted at any point in chord or web members, the splices must be capable of carrying the factored loads without exceeding the factored resistances of the members. Butt-welded splices are permitted, provided they develop the factored tensile resistance of the member.

As a general rule, the gravity axes of members should meet at a common point within a joint. However, when this is not practical, eccentricities may be neglected if they do not exceed those described in Clause 16.5.9.4; see Figure 2-43. Kaliandasani *et al.* (1977) have shown that the effect of small eccentricities is of minor consequence, except for eccentricities at the end bearing and the intersection of the end diagonal and bottom chord. (See also Clause 16.5.10.4.)

16.5.10 Bearings

16.5.10.1 As required by Clause 16.4.1(c), the factored bearing resistance of the supporting material or the size of the bearing plates must be given on the building design drawings.

16.5.10.2 It is likely that the centre of bearing will be eccentric with respect to the intersection of the axes of the chord and the end diagonal, as shown in Figure 2-44. Because the location of the centre of bearing is dependent on the field support conditions and their construction tolerances, it may be wise to assume a maximum eccentricity when designing the bearing detail. In lieu of specific information, a reasonable assumption is to use a minimum eccentricity of one half the minimum bearing on a steel support of 65 mm. When detailing joists, care must be taken to provide clearance between the end diagonal and the supporting member or wall. See Figure 2-45. A maximum clearance of 25 mm is suggested to minimize eccentricities. One solution, to obtain proper bearing, is to increase the depth of the bearing shoe.

For spandrel beams and other beams on which joists frame from one side only, good practice suggests that the centre of the bearing shoe be located within the middle third of the flange of the supporting beam (Figure 2-46(a)). As the depth of bearing shoes vary, the building designer should check with the joist manufacturer in setting "top of steel" elevations. By using a deep shoe, interference between the support and the end diagonal will be avoided, as shown in Figure 2-46(b).

If the support is found to be improperly located, such that the span of the joist is increased, the resulting eccentricity may be greater than that assumed. Increasing the length of the bearing

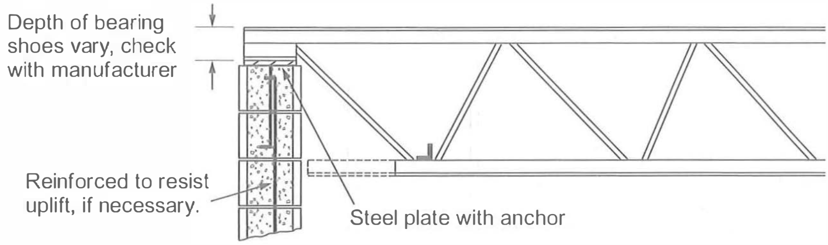
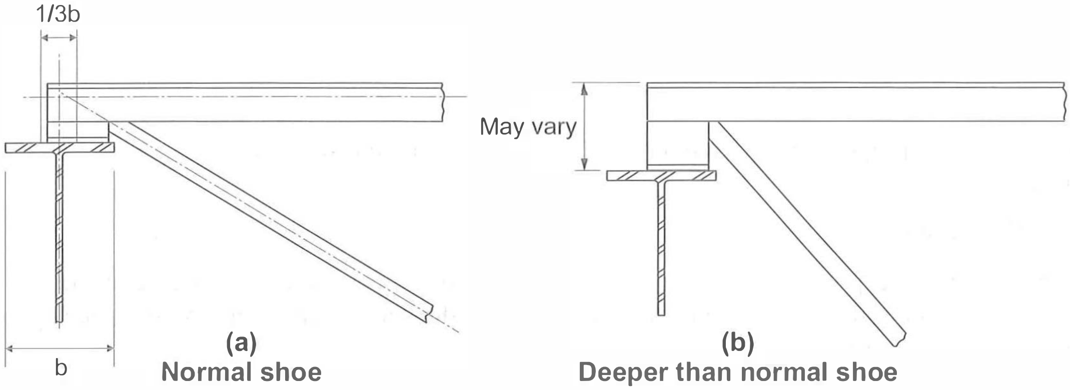
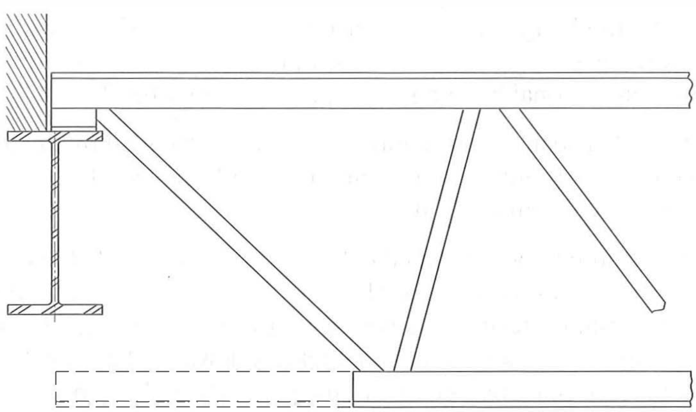


Figure 2-45
Joists Bearing on Steel Plate Anchored to Concrete and Masonry



(a)
Normal shoe

(b)
Deeper than normal shoe



(c)
See Clause 16.5.10.3 when bearing is less than 65 mm

Figure 2-46
Joists Bearing on Steel

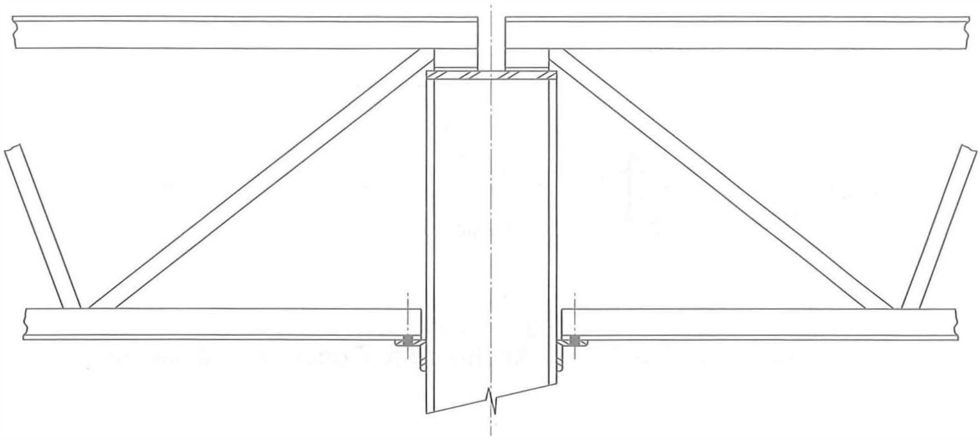


Figure 2-47
Tie Joists

shoe to obtain proper bearing may create the more serious problem of increasing the amount of eccentricity.

16.5.11 Anchorage

16.5.11.1 When a joist is subject to net uplift, not only must the anchorage be sufficient to transmit the net uplift to the supporting structure, but the supporting structure must be capable of resisting that force.

The anchorage of joist ends to supporting steel beams provide both lateral restraint and torsional restraint to the top flange of the supporting steel beam (Albert *et al.* 1992). When the supporting beam is simply supported, the restraint provided to the compression flange likely means that the full cross-sectional bending resistance can be realized.

In cantilever-suspended span construction, the restraint provided by the joists is applied to the tension flange in negative moment regions and is, therefore, less effective in restraining the bottom (compression) flange from buckling.

Albert *et al.* (1992) and Essa and Kennedy (1993) show that, while the increase in moment resistance due to lateral restraint is substantial, in cantilever-suspended span construction, the further increase when torsional restraint is considered is even greater. The torsional restraint develops when the compression flange tends to buckle sideways, distorting the web and twisting the top flange that is restrained by bending of the joists about the strong axis. The anchorage must therefore be capable of transmitting the moment that develops. For welds, a pair of 5 mm fillet welds 50 mm long coupled with the bearing of the joist seat would develop a factored moment resistance of about 1.8 kN·m

16.5.11.2 The function of tie joists is to assist in the erection and plumbing of the steel frame. Either the top or bottom chord is connected by bolting and, after plumbing the columns, the other chord is usually welded (Figure 2-47). In most buildings, tie joists remain as installed with both top and bottom chords connected; however, current practices vary throughout Canada with, in some cases, the bottom chord connections to the columns being made with slotted holes. Shrivastava *et al.* (1979) studied the behaviour of tie joist connections and concluded that they may be insufficient to carry lateral loads which could result from rigid bolting.

The designation tie joist is not intended to be used for joists participating in frame action.

Table 2-1
Camber for Joists

	Camber (mm)		
Span	**Nominal Camber**	**Minimum Camber**	**Maximum Camber**
Up to 6 000	12 +	4	20
7 000	14	6	22
8 000	16	8	24
9 000	18	10	26
10 000	20	11	29
11 000	22	13	31
12 000	24	15	33
13 000	26	17	35
14 000	28	18	38
15 000	30	20	40
16 000	32	22	42

16.5.11.3 When joists are used as part of a frame to brace columns, or to resist lateral forces on the finished structure, the appropriate moments and forces are to be shown on the building design drawings to enable the joists and the joist-to-column connections to be designed by the joist manufacturer.

In cantilever-suspended span roof framing, joists may also be used to provide stability for girders passing over columns. See also the commentary on Clauses 16.5.11.1 and 13.6.

16.5.12 Deflection

The method of computing deflections is based on truss action, taking into account the axial deformation of all components rather than the former approximate method of using a moment of inertia equal to that of the truss chords and adding an allowance for the "shear" deformation of the web members.

16.5.13 Camber

The nominal camber based on Clause 16.5.13 is taken to vary linearly with the span and is tabulated in Table 2-1, rounded to the nearest millimetre. Manufacturing tolerances are covered in Clause 16.10.9. The maximum difference in camber of 20 mm for joists of the same span, set to limit the difference between two adjacent joists, is reached at a span of 16 000 mm.

16.5.14 Vibration

Annex E of S16-14, Guide for Floor Vibrations, contains recommendations for floors supported on steel joists. By increasing the floor thickness (mass), both the frequency and the peak acceleration are reduced, thus reducing the annoyance more efficiently than by increasing the moment of inertia (I_x) of the joists. For this reason, the building designer should weigh, at the building design stage, the options in the Guide for Floor Vibrations to achieve the best performance.

16.5.15 Welding

This clause makes reference to Clause 24, which requires that open-web steel joist fabricators be certified by the Canadian Welding Bureau to CSA W47.1 for arc-welded joists, to CSA W55.3 for resistance welded joists, or to both.

This clause further requires that fabricators have welding procedures specific to the fabrication of joists in place; this may include items such as weld sequence, length and profile unique to the joist fabrication. The development and qualification of welding procedures is a mandatory requirement of all fabricators who are certified to the requirements of CSA W47.1 or CSA W55.3.

16.6 Stability During Construction

A distinction is made between bridging, put in to meet the slenderness ratio requirements for top and bottom chords, and the temporary support required by Clause 16.6 to hold joists against movement during construction. Permanent bridging, of course, can be used for both purposes.

16.7 Bridging

Figures 2-48, 2-49 and 2-50 provide illustrations of bridging and details of bridging connections.

16.7.7 Anchorage of Bridging

Ends of bridging lines may be anchored to the adjacent steel frame, or adjacent concrete or masonry walls, as shown in Figure 2-51.

Where attachment to the adjacent steel frame or walls is not practicable, diagonal and horizontal bridging shall be provided in combination between adjacent joists near the ends of bridging lines as shown in Figure 2-52. Joists bearing on the bottom chord will require bridging at the ends of the top chord.

16.7.9 Spacing of Bridging

Either horizontal or diagonal bridging is acceptable, although horizontal bridging is generally recommended for shorter spans, up to about 15 m, and is usually attached by welding. Diagonal bridging is recommended for longer spans and is usually attached by bolting. Bridging need not be attached at panel points and may be fastened at any point along the length of the joists. When horizontal bridging is used, bridging lines will not necessarily appear in pairs as the requirements for support of tension chords are not the same as those for compression chords. Because the ends of joists are anchored, the supports may be assumed to be equivalent to bridging lines.

16.8 Decking

16.8.1 Decking to Provide Lateral Support

When the decking complies with Clause 16.8 and is sufficiently rigid to provide lateral support to the top (compression) chord, the top chord bridging may be removed when it is no longer required. Bottom (tension) chord bridging is permanently required to limit the unsupported length of the chord to $240r$, as defined in Clause 16.7.9.

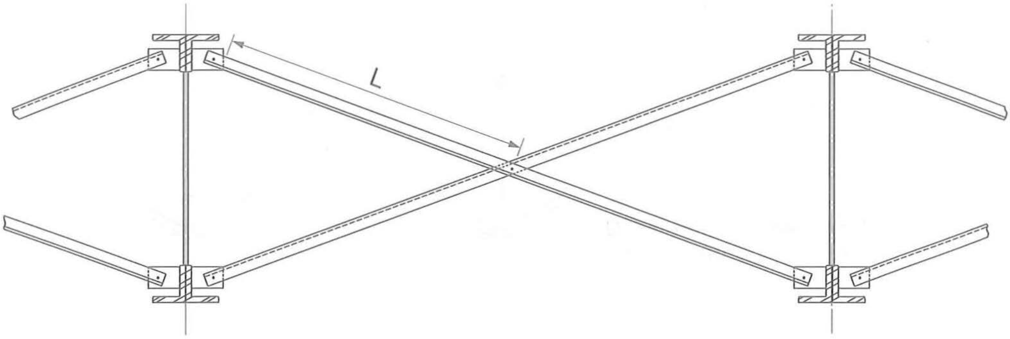


Figure 2-48
Diagonal Bridging of Joists

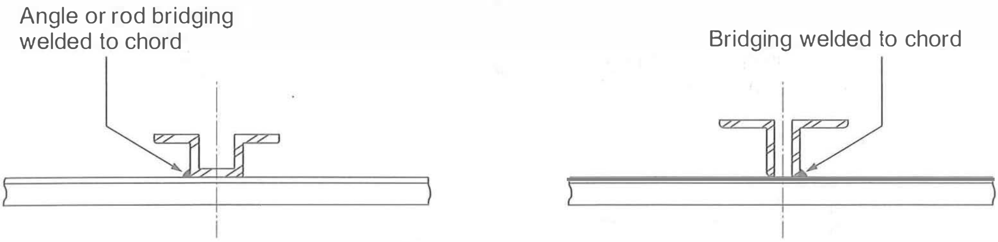


Figure 2-49
Horizontal Bridging Connections to the Joist's Top Chord

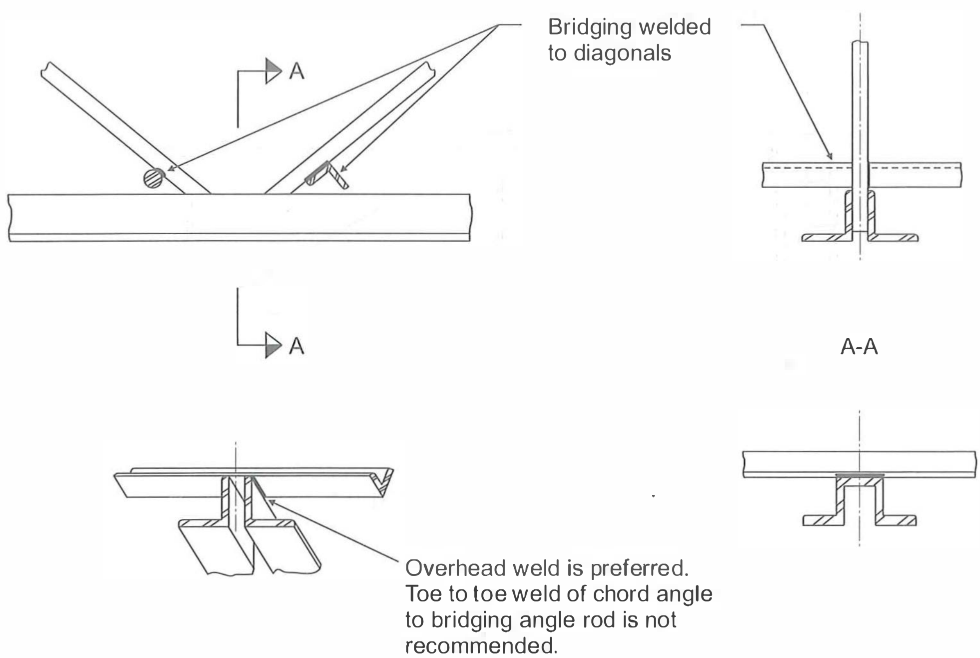


Figure 2-50
Horizontal Bridging Connections to the Joist's Bottom Chord

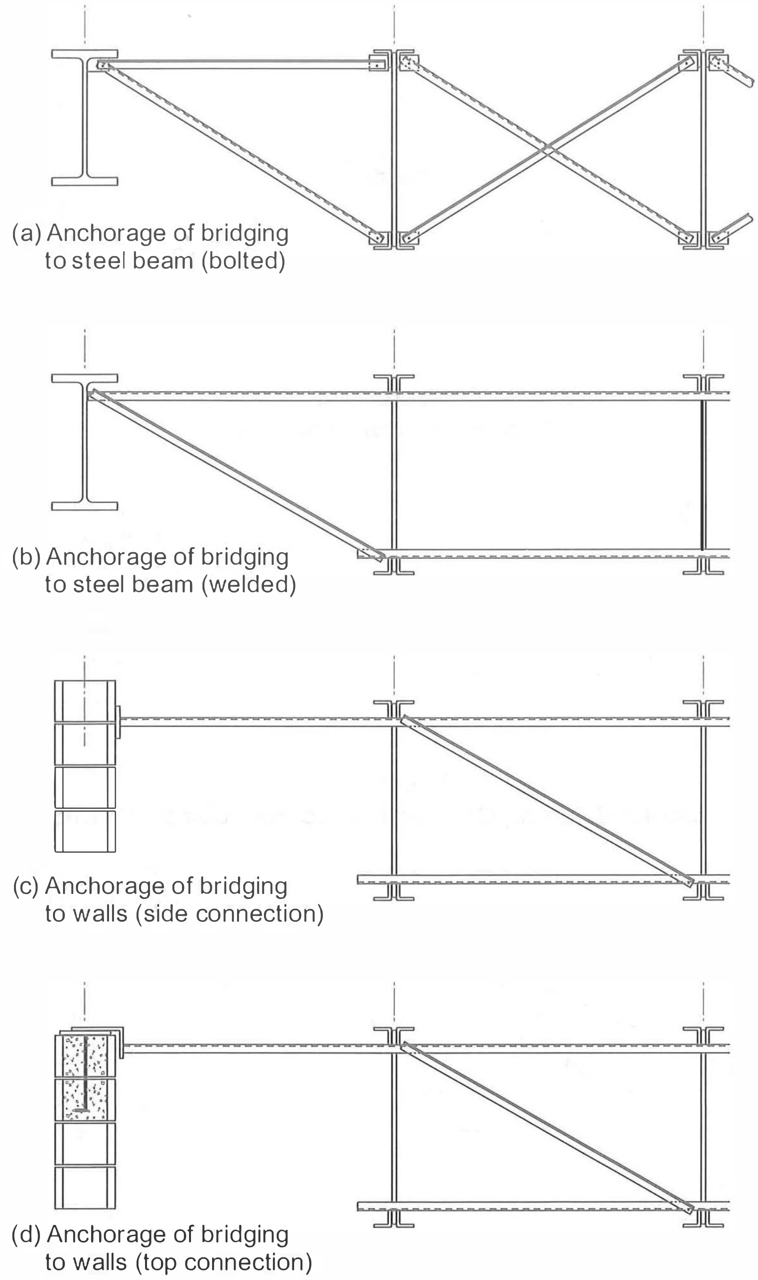
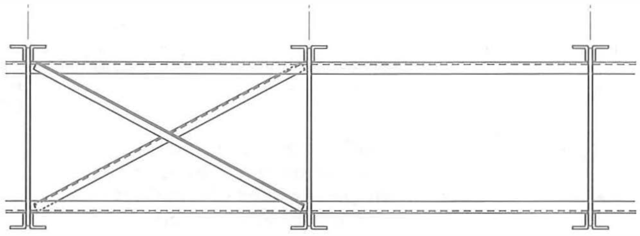


Figure 2-51
Anchorage of Joist Bridging

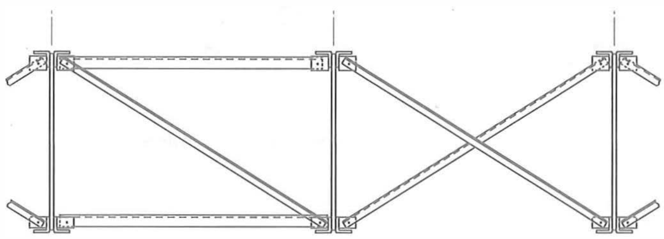
16.8.5 Installation of Steel Deck

16.8.5.1 Workmanship is of concern when decking is to be attached by arc-spot welding to top chords of joists. When the joist location is marked on the deck as the deck is positioned, the welders will be more likely to position the arc-spot welds correctly.

16.8.5.2 Arc-spot welds for attaching the deck to joists are structural welds and require proper welding procedures.



(a) diagonal bridging with horizontal bridging



(b) horizontal bridging with diagonal bridging

Figure 2-52
Bracing of Joist Bridging

16.9 Shop Coating

Interiors of buildings conditioned for human comfort are generally assumed to be of a non-corrosive environment and therefore do not require corrosion protection.

Joists normally receive one coat of paint suitable for a production line application, usually by dipping a bundle of joists into a tank. This paint is generally adequate for three months of exposure, which should be ample time to enclose or paint the joists.

Special coatings and paints that require special surface preparations are expensive, because these have to be applied individually to each joist by spraying or other means. For joists comprised of cold-formed members, surface preparations that were meant to remove mill scale from hot-rolled members are not appropriate.

16.10 Manufacturing Tolerances

Figure 2-53 illustrates many of the manufacturing tolerance requirements.

16.11 Inspection and Quality Control

16.11.3 Quality Control

When testing forms part of the manufacturer's normal quality control program, the test may follow steps 1 to 4 of the loading procedure given in Part 5 of Steel Joist Facts (CISC 1980).

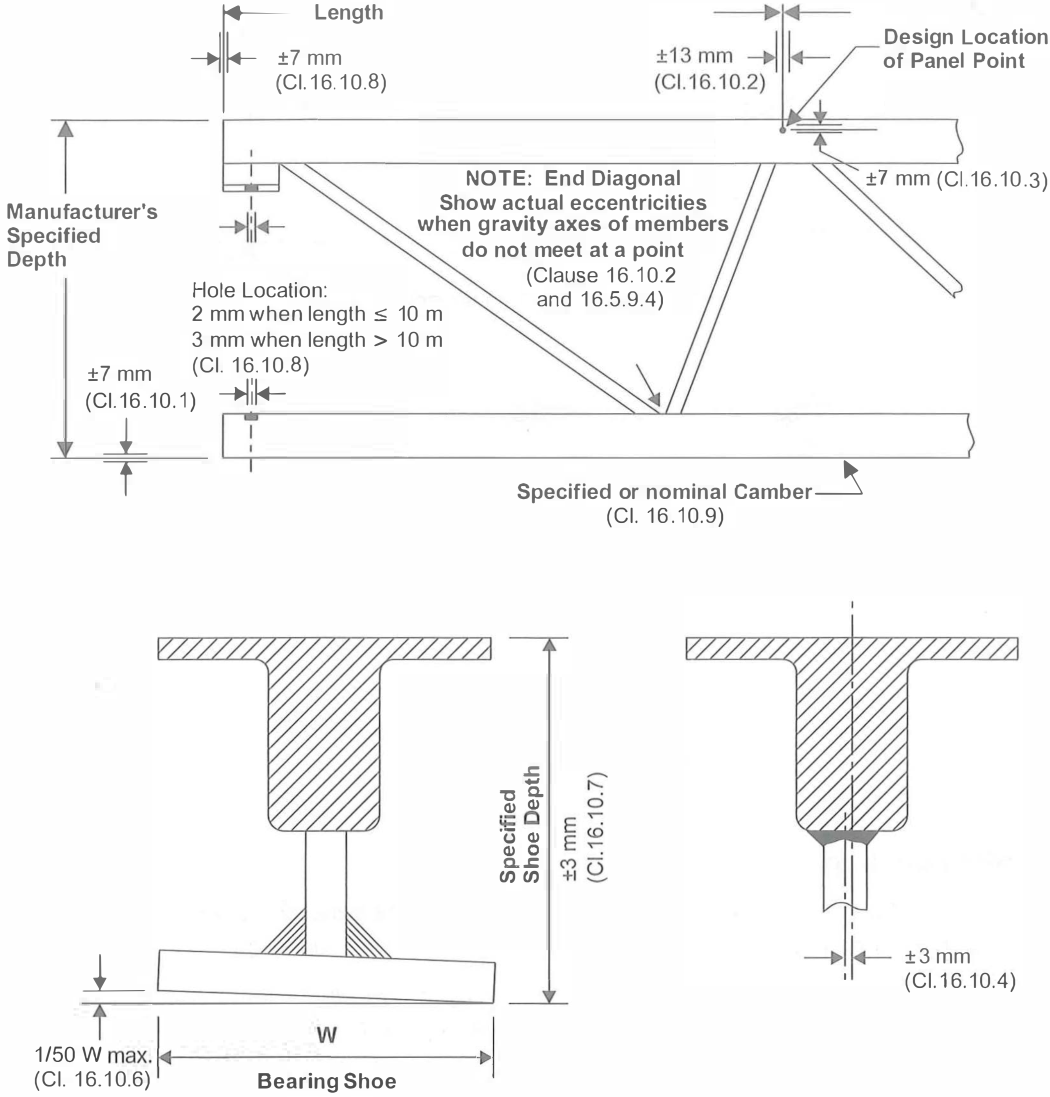


Figure 2-53
Joist Manufacturing Tolerances

16.12 Handling and Erection

16.12.2 Erection Tolerances

Figure 2-54 illustrates many of the erection tolerance requirements. The provisions of Clause 16.12.2.5 aim to control the differential deflection between any three adjacent joists to smooth the supported deck's profile.

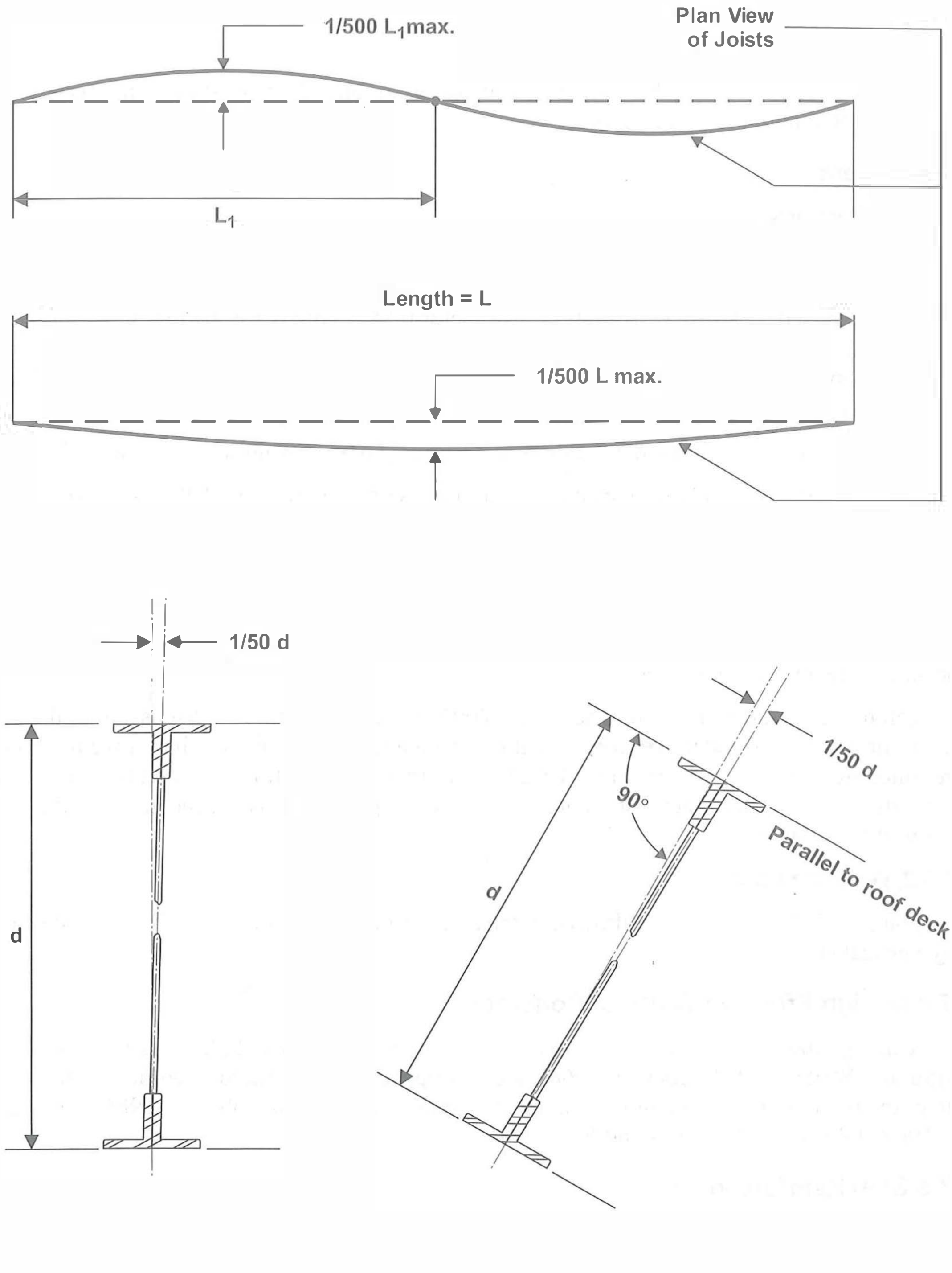


Figure 2-54
Joist Erection Tolerances

17.2 Definitions

Definitions particular to this Clause are given here. Figure 2-55 illustrates various cases of effective slab and cover slab thickness.

17.3 General

17.3.1 Deflections

The moment of inertia is reduced from the transformed value to account for the increased flexibility resulting from partial shear connection, p, and for interfacial slip, similar to that coefficient proposed by Grant *et al.* (1977). The factor 0.85 accounts for the loss in stiffness due to interfacial slip, even with full shear connection. To include the effect of shear deformation of the web systems of joists and trusses, the moment of inertia I_s is reduced by 15% unless a detailed analysis is used.

The increase of the elastic deflection of 15% for creep is an arbitrary but reasonable value.

Annex H of the Standard gives a detailed discussion of shrinkage deflections. There it is emphasized that appropriate values of the shrinkage strain and age-adjusted effective modulus of concrete, which in turn depends on the aging and creep coefficients, should be used in calculating these deflections. Values of shrinkage strain, and aging and creep coefficients, for conditions not anticipated in the annex may be obtained from Ghali *et al* (2002). CSA S6 Canadian Highway Bridge Design Code (2014) also contains detailed procedures for evaluating the shrinkage and creep of concrete.

Reference should be made to Ghali *et al* (2002) for a more complete discussion of the procedure proposed in the standard for evaluating shrinkage deflections. An alternative method is presented by Kennedy and Brattland (1992), with further information provided by Maurer and Kennedy (1994) on interfacial slip and, for composite joists or trusses, increases in the flexibility of the system.

17.3.2 Vertical Shear

Clauses 17.3.2 and 17.3.3 follow from the assumption that the concrete slab does not carry any vertical shear.

17.4 Design Effective Width of Concrete

Although the effective width rules were formulated on the basis of elastic conditions (Robinson and Wallace 1973, Adekola 1968), the differences at ultimate load do not significantly affect the moment resistance of the composite beam (Elkelish and Robinson 1986, Hagood *et al.* 1968, Johnson 1975, Heins and Fan 1976).

17.5 Slab Reinforcement

17.5.2 The effectiveness of the minimum requirement of two 15M bars at the ends of beams supporting ribbed slabs perpendicular to the beam proposed by Ritchie and Chien (1980) has been verified experimentally by Jent (1989).

17.5.3 The longitudinal shear forces generated by interconnecting concrete slabs to steel sections, trusses, or joists by means of shear connectors may cause longitudinal cracking of the slab directly over the steel. This effect is independent of any flexural cracking that may occur due to the slab spanning continuously over supports, although the two effects may combine. Longitudinal shear cracking is more apt to start from the underside of the solid slab, whereas flexural cracking is more apt to start at the top surface of the slab. Investigations by Johnson

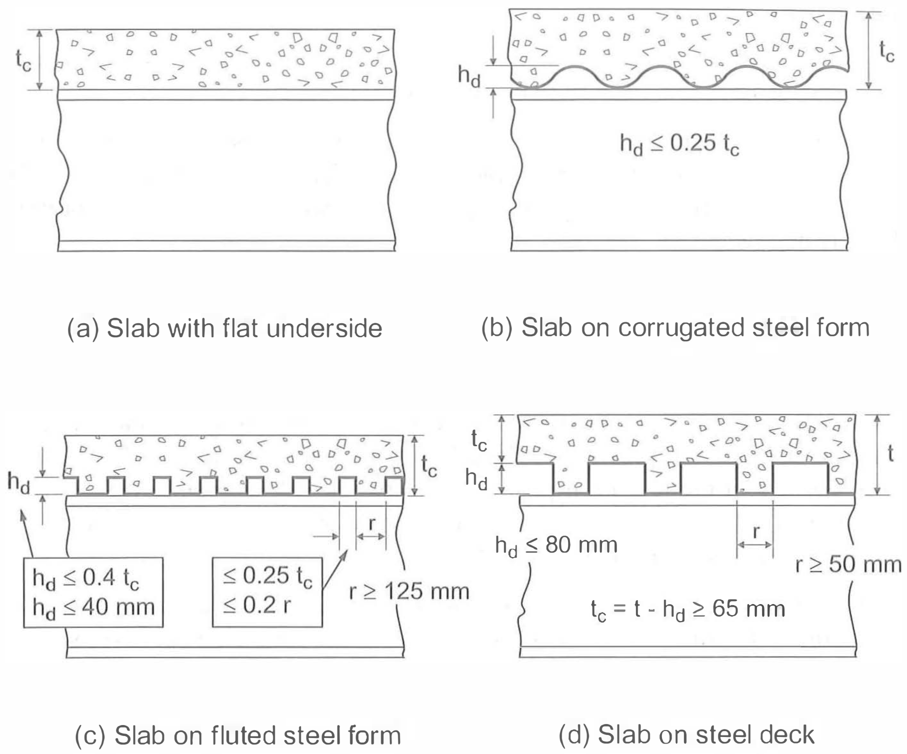


Figure 2-55
Effective Slab Thickness for Composite Beams

(1970), El-Ghazzi *et al.* (1976), and Davies (1969) have shown that a minimum area of transverse reinforcing steel is required to improve the longitudinal shear capacity of a composite beam slab. The minimum reinforcement ratio is the same as that specified in CSA Standard A23.3 (CSA 2014) for temperature and shrinkage reinforcement in reinforced concrete slabs.

17.5.4 For the reasons given in Clause 17.5.3, a minimum transverse reinforcement ratio of 0.002 is also specified for composite beams with ribbed slab when the ribs are parallel to the beam span. This ratio is reduced to 0.001 when the ribs are perpendicular to the beam span, because the steel deck provides a measure of transverse reinforcement. Reinforcement of the cover slab may also be necessary for flexure, fire resistance, shrinkage, or temperature effects.

17.6 Interconnection

When unpainted sections, trusses, or joists are totally encased in concrete as specified, effective interconnection is obtained, and no shear connectors are required.

The total sheet thickness and the total amount of zinc coating are limited in order to achieve sound welds.

Tests have shown that a shear connector is not fully effective if welded to a support which is too thin or flexible (Gobel 1968). For this reason, the stud diameter is limited to 2.5 times the thickness of the part to which it is welded.

17.7 Shear Connectors

The factored resistance of end-welded studs in a solid slab is different from that in a ribbed slab, which depends upon the deck ribs' orientation and size.

For end-welded studs in a solid slab, the values given in Clause 17.7.2.2 are based on work by Olgaard *et al.* (1971) in both normal and light-density solid concrete slabs. The limiting value of $\phi_{sc} A_{sc} F_u$ represents the tensile strength of the stud, as the stud eventually bends over and finally fails in tension.

In previous editions of the Standard, Clause 17.7.2.3(a) gave the same factored shear resistance for studs in ribbed slabs, with ribs parallel to the beam, as in solid slabs, provided that the rib flute is wide enough (Johnson 1975). Hosain and Wu (2002) have shown that this may not always be the case. In S16-09, equation (a) was revised to account for the lower capacity observed through push-out and full-size beam tests. When the flutes are narrow, however, the factored shear resistance of the stud is reduced. The equation in Clause 17.7.2.3(b) of S16-09 gives a more consistent prediction of push-out test results (Hosain and Pashan 2002). A limit is placed on equation (b), such that the shear resistance for $w_d / h_d < 1.5$ does not exceed that obtained with the revised equation (a) at $w_d / h_d = 1.5$.

The provisions for ribbed slabs with ribs perpendicular to the beam are based on work by Jayas and Hosain (1988 and 1989). Push-out tests, as well as full-size beam tests, indicated that failure in this type of composite beam would likely occur due to concrete pull-out. The equations of Clause 17.7.2.4, similar to those suggested by Hawkins and Mitchell (1984), provide better correlation with test results than those using the reduction factor method adopted by AISC (2010b). Figure 2-56 gives diagrams of the pullout surface area. Pullout areas for specific deck profiles and studs are given in Part 5 of the Handbook.

In order to minimize localized stresses in concrete, the lateral spacing centre-to-centre of studs used in pairs should be not less than four stud diameters. The minimum longitudinal spacing of connectors, in both solid slabs and ribbed slabs with ribs parallel to the beam, is based on Olgaard *et al.* (1971). The maximum spacing limits specified for mechanical ties in Clause 17.8 is applicable to headed studs, as they function in this capacity.

Further information on end-welded studs is found in Johnson (1970), Chien and Ritchie (1984), and Robinson (1988).

17.7.3 The shear value of channel connectors is based on Slutter and Driscoll (1965).

17.9 Design of Composite Beams with Shear Connectors

In order to minimize eccentricities before and after composite action, in composite joists and trusses, the web members should be positioned such that the lines of action intersect at a point halfway between the mid-depth of the cover slab and the centroid of the steel top chord.

17.9.1 A minimum flat width for the top chord of $1.4d + 20$ mm is stipulated to facilitate placement of the shear studs.

17.9.3 The factored moment resistance of a composite flexural member is based on the ultimate capacity of the cross-section (Robinson 1969, Vincent 1969, Hansell and Viest 1971, Robinson and Wallace 1973, Tall *et al.* 1974) where the following assumptions are made:

- Concrete in tension is neglected;
- Only the lower chord of a steel joist or truss is considered effective when computing the moment resistance;
- The internal couple consists of equal tension and compression forces;

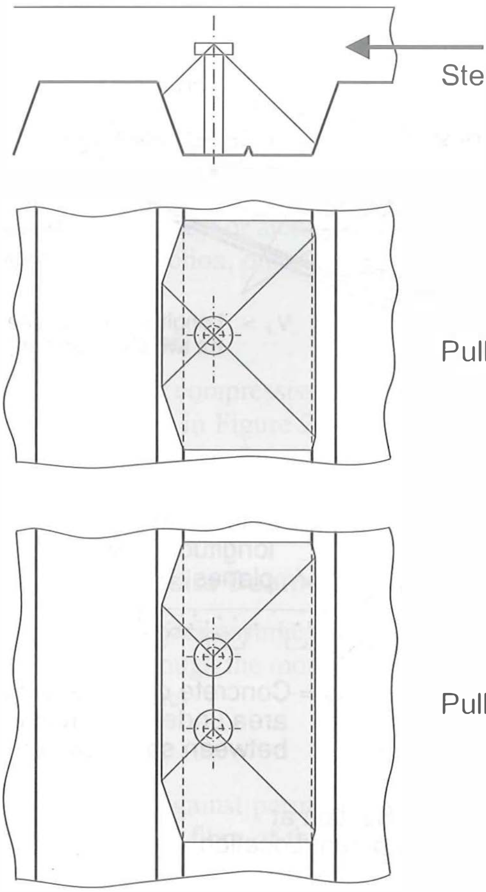


Figure 2-56
Pullout Surface Area with Ribbed Metal Deck

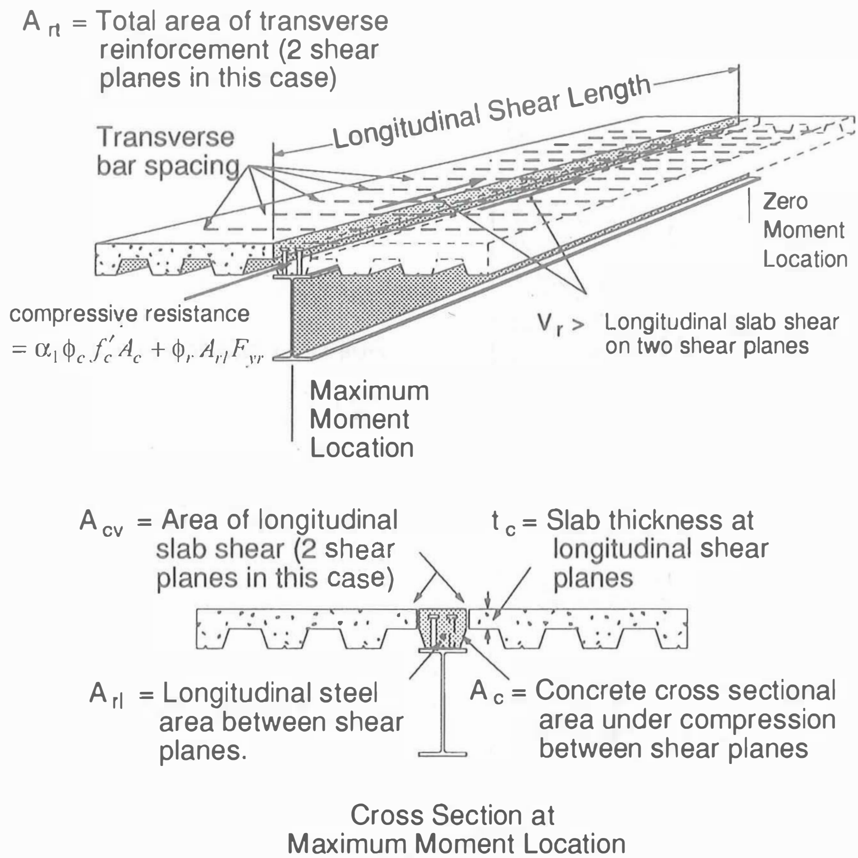
- The forces are obtained as the product of a limit states stress (ϕF_y for steel and $\alpha_1 \phi_c f'_c$ for concrete) times the respective effective areas; and,
- To take into account the greater variability of concrete elements strengths, the resistance factor is taken as 0.65 for concrete as compared to 0.90 for steel.

Three design cases are considered:

- Case 1 representing full shear connection with the plastic neutral axis in the slab;
- Case 2 representing full shear connection with the plastic neutral axis in the steel section; and,
- Case 3 representing partial shear connection for which the plastic neutral axis is always in the steel section.

Only Case 1 is permitted when joists or trusses are used to prevent buckling of the top chord and overloading of the shear connectors. For Case 3, the depth of the concrete in compression is determined by the expression for "a" (Robinson 1969).

Since the release of S16-09, these assumptions have been modified somewhat such that the resistance factor for concrete and the ratio of average stress in the rectangular compression block to the specified concrete strength are consistent with the CSA Standard A23.3-04 (and



$$V_u = \sum q_r - \alpha_1 \phi_c f'_c A_c - \phi_r A_{rl} F_{yr}$$

$$V_r = (0.80 \phi_r A_{rt} F_{yr} + 2.76 \phi_c A_{cv}) \leq 0.50 \phi_c f'_c A_{cv}$$

Figure 2-57
Potential Longitudinal Shear Planes

A23.3-14). Accordingly, $\phi_c = 0.65$ is used in place of 0.60, and $\alpha_1 \phi_c f'_c$ is given as the concrete strength in place of $0.85\, \phi_c f'_c$. Reasoning for these changes to the concrete resistance factor and strength is described by Bartlett (2007).

17.9.4 Robinson (1988) and Jayas and Hosain (1989) show that a lower limit of 40% of full shear connection is acceptable for strength calculations. Below this value, the interfacial slip is such that integral composite action cannot be assured. A lower limit of 25% of full shear connection is used for deflection designs, as deflections are computed at specified load levels. This latter provision is used where the flexural strength is based on the bare steel beam, but the increased stiffness due to the concrete is considered for deflection calculations.

17.9.5 Between the point of zero and maximum moment, a horizontal force associated with the internal resisting couple must be transmitted across the steel-concrete interface.

17.9.8 Uniform spacing of shear connectors is generally satisfactory because the flexibility of the connectors provides a redistribution of the interface shear among them. However, to ensure that sufficient moment capacity is achieved at points of concentrated load, the second provision of this clause is invoked. As the moment capacity of the steel section does not depend on shear connectors, this capacity is subtracted from both M_f and M_{f1}.

17.9.9 To justify composite action in the end panel of joists and trusses, sufficient shear studs must be provided above the seat or along a top chord extension, to transfer the horizontal shear from the slab to the steel section, otherwise the steel top chord acting alone must resist all the forces.

17.9.10 Longitudinal Shear

In order to develop the compressive force in the portion of the concrete slab outside the potential shear plane shown in Figure 2-57, net shear forces, totalling V_u, must be developed on these planes. The expressions for shear resistance are based on Mattock (1974). Values for semi-low-density and low-density concrete are given by Mattock *et al.* (1976) and Chien and Ritchie (1984).

17.10 Design of Composite Beams Without Shear Connectors

This conservative approach assumes that the composite section is about 10% stronger than the bare steel member, although the moment resistance computed according to Clause 17.10.2 typically gives a larger value.

17.11 Unshored Beams

This provision guards against permanent deformations under specified loads by limiting the total stress in the bottom fibre of the steel section. This limit has been shown (Kemp and Trinchero 1992) to be conservative. The ultimate strength of the composite beam, which exhibits ductile behaviour, is not affected by the stress state at the specified load level.

18. COMPOSITE COLUMNS

This clause includes, in addition to the concrete-filled hollow structural sections, partially encased composite columns acting in compression in Clause 18.3, and rolled steel shapes encased in concrete in Clause 18.4. The latter parallels the requirements of CSA Standard A23.3, but using a column curve consistent with those used throughout this Standard. Thus the designer has in this Standard three types of steel-concrete columns from which to choose.

The design rules apply to specific research which should be consulted in conjunction with the requirements of this Clause.

18.1 Resistance Prior to Composite Action

For some of the systems described here, the designer should be aware that the steel component may be designed to carry some of the loads before the concrete has gained strength.

18.2 Concrete-Filled Hollow Structural Sections

18.2.1 General

18.2.1.2 Axial Load on Concrete

Kennedy and MacGregor (1984) showed that direct bearing of the load on the concrete was not necessary for either axially loaded columns or beam-columns. When loads are applied to the steel shell, pinching between the steel and concrete quickly transfers loads to the concrete core. The Standard conservatively retains the requirement of direct bearing for the uppermost level but not for intermediate levels of multi-storey columns.

18.2.1.3 Composite Action in Bending

CIDECT (1970), Knowles and Park (1970), Wakabayashi (1977), Stelco (1981), Budijgnto (1983) and Bergmann *et al.* (1995) have demonstrated that the compression resistance of composite columns, consisting of hollow structural sections (HSS) completely filled with concrete, arises from both the steel and the concrete core. Obviously the full composite bending resistance at the ends of such members can only be realized when the connections are able to transfer the loads to the composite beam-column.

18.2.2 Compressive Resistance

The expressions for compressive resistance introduced in S16-01 give a better fit to test results than those found in the preceding standard. The contributions of the concrete core and the hollow steel section are simply superposed. Both the steel and concrete contributions to the compressive resistances are decreased as a function of the slenderness parameter, λ, of the composite section that is considered in turn to depend on the elastic flexural stiffness of the steel section and a flexural stiffness of the concrete that is modified to account for creep under sustained loads. The same double exponential form of column curve, as used for other compressive resistances in the standard, is used for consistency. The value of the exponent "n" in the expression is taken as 1.80 to get the best fit with experimental results.

The triaxial load effect on the concrete due to the confining effect of the walls of circular HSS is based on work by Virdi and Dowling (1976). The triaxial effects increase the failure load of the concrete ($\tau' > 1.0$) and decrease the capacity of the steel section ($\tau < 1.0$), because the steel is in a biaxial stress state.

In S16-09, the standard introduced a value of $\phi_c = 0.65$ in place of the earlier 0.60, and $\alpha_1 \phi_c f'_c$ in place of $0.85 \phi_c f'_c$. Reasoning for these changes to the concrete resistance factor and strength is described by Bartlett (2007). In S16-14 the lower limit for α_1 is changed to 0.73, which is the lowest value possible, considering that the scope only covers concrete strengths up to 80 MPa for axially loaded columns.

18.2.3 Bending Resistance

Lu and Kennedy (1994) show, for rectangular hollow sections with measured flange b/t ratios up to $700/\sqrt{F_y}$, that fully plastic stress blocks are developed in the steel and in the concrete. Their proposed model, based on such stress blocks with the steel stress level taken equal to the yield value, F_y, and the concrete stress level taken equal to the concrete strength, f'_c, at the time of testing, agreed excellently with test results. The two components support each other. The steel restrains or confines the concrete, increasing its compressive resistance to the full value rather than 0.85 of it, as used in reinforced concrete theory, while the concrete prevents inward buckling of the steel wall, thus increasing the steel strain at which local buckling occurs. Therefore, sections not even meeting the requirements of Class 3 sections in bending develop fully plastic stress blocks.

Geometric expressions are given to determine the factored compressive forces in the steel and concrete with rectangular stress blocks when in equilibrium, for both rectangular and circular hollow structural sections.

In S16-09, the standard introduced a value of $\phi_c = 0.65$ in place of the earlier 0.60, and $\alpha_1 \phi_c f'_c$ in place of $0.85 \phi_c f'_c$. Reasoning for these changes to the concrete resistance factor and strength is described by Bartlett (2007).

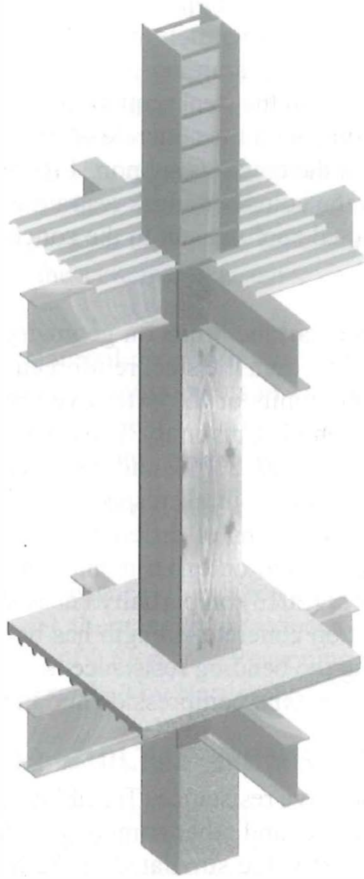


Figure 2-58
Partially Encased Composite Columns

18.2.4 Axial Compression and Bending

This clause is analogous to the expression in Clause 13.8.3 for I-shaped beam-columns. Extending the analogy, the cross-sectional resistance and in-plane strength should be checked and, if applicable, the lateral-torsional buckling strength should be checked for rectangular sections bent about their strong axis. Because of the very large torsional resistance of closed shapes, the latter is very unlikely to be a factor. With expressions introduced in S16-01 for circular hollow sections filled with concrete, the lower bound solution for such sections given in former Standards is no longer required.

In S16-09, the standard introduced a value of $\phi_c = 0.65$ in place of the earlier 0.60, and $\alpha_1 \phi_c f'_c$ in place of $0.85\, \phi_c f'_c$. Reasoning for these changes to the concrete resistance factor and strength is described by Bartlett (2007).

18.3 Partially Encased Composite Columns

As stated in the note to this Clause, these columns are a patented structural component. By CSA regulations, in the interests of promoting new technology, they are referenced in this Standard on the understanding with the patent holder that any patent rights will be made available either as a free license or on reasonable terms and conditions.

The basic concept is to provide a steel H-shape of relatively thin plates but with sufficient strength to carry gravity loads during construction until the concrete cast around the shape reaches sufficient strength to carry the remaining dead loads and all live and environmental loads, while working compositely with the steel section. It is envisaged that the columns could be used in multi-storey applications, with the concrete of the about-to-be encased steel shapes cast with the next higher floor that the columns support. Figure 2-58 shows an elevation of the column. The steel links between the column flanges restrain the flanges from buckling locally and at the same time provide limited confinement to the concrete.

18.3.1 General

The scope Clause lays out in detail the limits on geometry and strength of the component elements and materials – the steel section, the steel reinforcement and the concrete – that must be satisfied. These derive from the limits of the extensive series of tests, including full-scale tests, which were carried out at Lehigh University, University of Toronto, McGill University and École Polytechnique (Tremblay *et al.* 2000), and the University of Alberta to confirm and quantify the performance of the columns in all respects. While extensive, the limits are sufficiently broad in scope to design columns of different cross-sections and slenderness limits to carry a wide range of loadings. Based on experimental and numerical research by Prickett and Driver (2006) and Begum *et al.*(2013) on partially encased composite columns with high-strength concrete, the upper limit on concrete strength has been increased from 40 MPa to 70 MPa. The method for determining the bending resistance is provided in Clause 18.3.3, and the interaction expression for combined axial compression and bending in Clause 18.3.4.

18.3.2 Compressive Resistance

The expression for the compressive resistance (Tremblay *et al.* 2000) is of the same double exponential format used for both steel and other composite columns throughout this Standard. The exponent "n", 1.34, is the least value stipulated in the Standard. For both the steel section and the steel reinforcement, specified minimum yield strengths are used as the reference strengths and, for the concrete, 0.95 α_1 of the specified 28-day strength is used, as this value gave a better fit to the test data than the 0.85 factor commonly used. The resistance factors for the three components are consistent with the remainder of the Standard. In S16-09, the standard introduced a value of $\phi_c = 0.65$ in place of 0.60 (Bartlett 2007).

18.3.5 Special Reinforcement for Seismic Zones

Details are provided for longitudinal and transverse bars to be used where the specified one-second spectral acceleration ratio, $I_E F_v S_a(1.0)$, is greater than 0.30, in order to provide satisfactory performance compatible with that of reinforced concrete buildings designed for such seismic categories.

18.4 Encased Composite Columns

18.4.1 General

This Clause is provided because such columns may be found in a steel building structure. This Clause provides the designer with all the information needed to design this composite component as well as all other components in the building. The scope limits the doubly symmetric steel columns encased in concrete to which this clause applies to those given in CSA Standard A23.3.

18.4.2 Compressive Resistance

The factored compressive resistance is of the exact same form as that given in Clause 18.3 for concrete-filled hollow structural sections. In this regard, it differs in form from the resistance given in CSA Standard A23.3 but matches the factored compressive resistance of the latter closely for all slenderness ratios. The contributions of the concrete, structural steel shape and reinforcing steel to the strength are simply superposed. Both the steel and concrete contributions to the compressive resistances are decreased as a function of the slenderness parameter, λ, of the composite section that is considered in turn to depend on the elastic flexural stiffness of the steel section and a flexural stiffness of the concrete that is modified to account for creep under sustained loads. The same double exponential form of column curve, as used for other compressive resistances in the Standard, is used for consistency. In S16-09, the standard introduced a value of $\phi_c = 0.65$ in place of the former 0.60, and $\alpha_1 \phi_c f'_c$ in place of $0.85 \phi_c f'_c$. Reasoning for these changes to the concrete resistance factor and strength is described by Bartlett (2007).

18.4.4, 18.4.5 and 18.4.6 In the unusual case with multiple steel shapes enclosed in the concrete, the steel shapes are to meet the requirements of Clause 19 for built-up shapes until the concrete reaches $0.75 f'_c$. Alternatively, the load on the steel shapes could be limited to the sum of their independent resistances, having due regard as to how the loads are applied.

18.4.5 This clause emphasizes that there must be direct transfer of any load considered to be carried by the concrete.

18.4.6 To determine the bending resistance of encased composite columns, the designer is referred to Ziemian (2010). In S16-09, the standard introduced a value of $\phi_c = 0.65$ in place of the former 0.60, and $\alpha_1 \phi_c f'_c$ in place of $0.85 \phi_c f'_c$. Reasoning for these changes to the concrete resistance factor and strength is described by Bartlett (2007).

19. BUILT-UP MEMBERS

The term built-up member refers to any structural member assembled from two or more components. Such members may be used to resist compression, tension or bending, and the requirements for fastening together the various components vary accordingly.

The diagrams of Figures 2-59 and 2-60 illustrate the main provisions of Clause 19.

Many of the provisions are based on long-established practice and have proven satisfactory. In Clause 19.2.3, it is emphasized that the buckling could occur for outside components. In Clause 19.2.10, because it has been established that the tension diagonal of a crossed tension-compression pair supports the latter (see Commentary to Clause 27.5.3.1), the effective buckling length of the compression lacing can be taken as 0.50 of its total length.

Tension members are stitched together sufficiently to work in unison and to minimize vibration. For exposed members, components in contact should be fitted tightly together to minimize corrosion problems (Brockenbrough 1983).

When a built-up column buckles, shear is introduced in lacing bars (Clause 19.2.9) and battens and their connections (Clause 19.2.17), in addition to any transverse shears (Bleich, 1952).

Further discussion on columns with lacing and battens is given in Ziemian (2010).

For compression members composed of two or more rolled shapes connected at intervals, Clause 19.2.4 requires the use of an equivalent slenderness ratio, increased to take into account the flexibility of the interconnector. This increase is applied to the axis of buckling where the buckling mode of the member involves relative deformation that produces shear forces (see Clause 19.2.6) in the interconnectors between the individual shapes (Duan and Chen, 1988).

The requirements for starred angles are based on work by Temple *et al.* (1986), who showed that with fewer interconnectors the buckling strength was reduced.

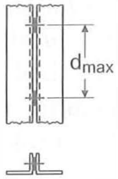
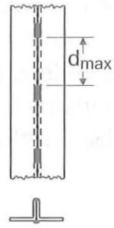
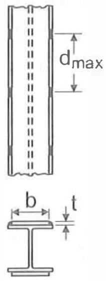
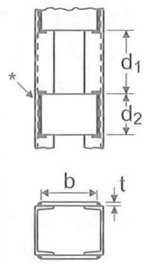
Tension Members	Requirements	Tension Members	Requirements
	TWO ROLLED SHAPES NOT IN CONTACT d_{max} = 300 × Least radius of gyration of one component		TWO ROLLED SHAPES IN CONTACT d_{max} = 600 mm d_{max} may be increased when justified
	SHAPE AND PLATE IN CONTACT d_{max} = 36t or 450 mm whichever is lesser		BATTENS $b \leq 60t \quad d_2 \geq \frac{2b}{3}$ d_{max} = 300 × Least radius of gyration of one component * For intermittent welds or fasteners, max. longitudinal pitch = 150 mm

Figure 2-59
Built-up Tension Member Details

20. PLATE WALLS

20.1 General

Early research at the University of Alberta (Kulak 1991, Driver *et al.* 1997, 1998(a), and 1998(b)) demonstrated that the plate wall system is an attractive alternative for resisting lateral wind and seismic loads. The system has the advantage that it is stiff enough to minimize displacements under extreme loading conditions and has a high degree of redundancy. The system can be used for both new construction and the upgrading of existing structures.

Figure 2-61 shows a typical plate wall. The walls considered by Clause 20 imply thin, unstiffened infill plates. Under lateral loads, it is assumed that the buckling strength of the infill plate is negligible, but tension field action develops to resist lateral shears.

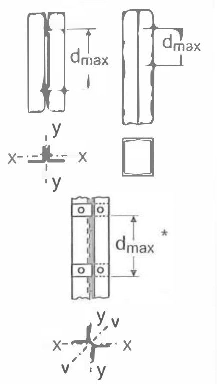
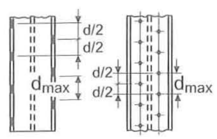
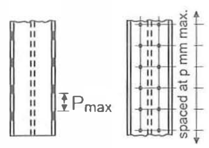
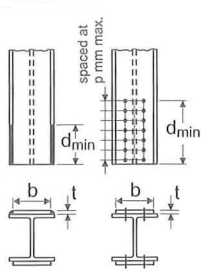
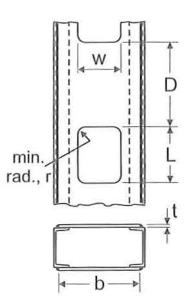
A brief overview of steel-plate shear wall research, along with a comprehensive list of relevant references, can be found in Chapter 6 of Ziemian (2010). Moghimi and Driver (2013) have provided recommendations specifically for designing plate walls economically when a high degree of ductility is not required (e.g. low seismic zones) by using modular construction and shear connections in the boundary frame.

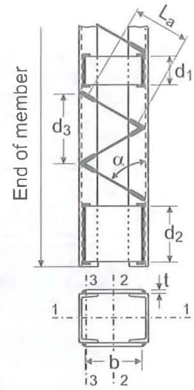
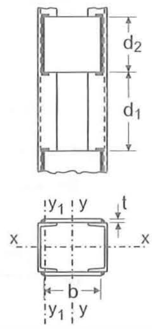
20.2 Seismic Applications

The provisions of Clause 20 must be met for all plate walls. Additional requirements specifically for seismic applications are laid out in Clause 27.

20.3 Analysis

Thorburn *et al.* (1983) demonstrated that the strip model shown in Figure 2-63 predicts the development of tension field action in plate walls subjected to lateral loads. The forces and moments in a plate wall may be estimated by extending the strip model over all storeys using

Compression Members	Requirements
	ROLLED SHAPES $d_{max} = \left(\frac{KL}{r}\right) r_{min}$ KL/r = slenderness of member as a whole r_{min} = least radius of gyration of one component * For starred angles, 1/3 points Cl. 19.2.5. See Clause 19.2.4 for design slenderness and effective slenderness ratios 19.2.4
	STAGGERED FASTENERS OR WELDS $d_{max} = \frac{525t}{\sqrt{F_y}}$ or 450 mm t = outside plate thickness 19.2.3
	FASTENERS OR WELDS NOT STAGGERED $P_{max} = \frac{330t}{\sqrt{F_y}}$ or 300 mm t = outside plate thickness 19.2.3
	ENDS OF BUILT-UP COLUMNS Welded connection: $d_{min} = b$ Bolted connection: $d_{min} = 1.5b$ P_{max} = 4 × diameter of fastener 19.2.2
	PERFORATED COVER PLATES $b \le \frac{840t}{\sqrt{F_y}}$ $L \le 2W$ $D \ge b$ $r \ge 40$ mm Note: For bolted fabrication, $b \ge 400$ mm is preferred. 19.2.15

Compression Members	Requirements
	LACING AND TIE PLATES $b \le 60t$ $d_1 \ge b/2$ $d_2 \ge b$ $d_3 \le \left(\frac{KL}{r_1}\right) r_3 \le \left(\frac{KL}{r_2}\right) r_3$ $L_a \le 140 \times$ radius of gyration of lacing member $\alpha \ge 45°$ KL = Effective length of member with respect to appropriate axis 19.2.9 – 13
	BATTENS $b \le 60\,t$ $d_2 \ge b$ $\left(\frac{KL}{r}\right)_e = \sqrt{\left(\frac{KL}{r}\right)_y^2 + \left(\frac{KL}{r}\right)_{y1}^2}$ 19.2.4 and 19.2.16

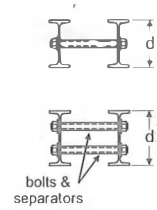
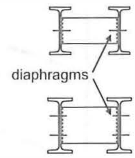
Beams and Grillages	Requirements
	NON-LOAD-SHARING BEAMS Not less than one bolt: $d < 300$ mm Two or more bolts: $d \ge 300$ mm Centres of separator groups ≤ 1500 mm 19.4
	LOAD-SHARING BEAMS Diaphragm shall have sufficient stiffness to distribute required loads. Centres of diaphragms ≤ 1500 mm 19.4

Figure 2-60
Built-up Compression Member Details

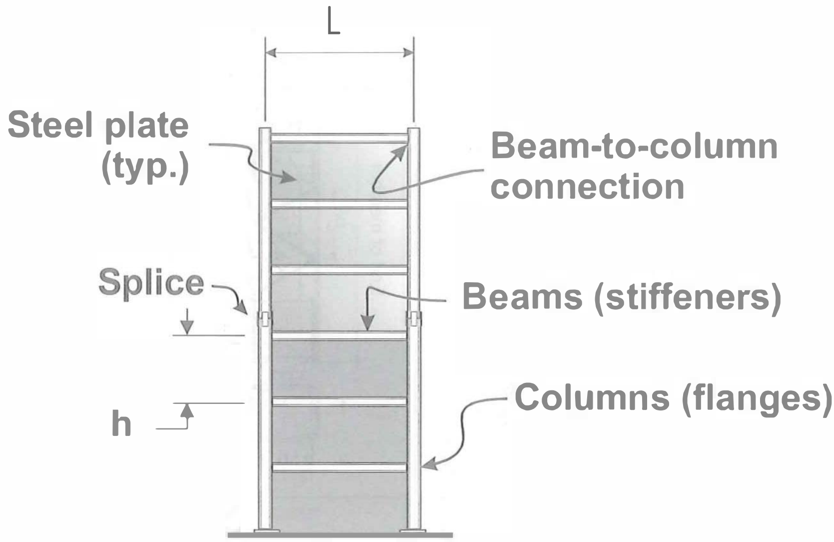


Figure 2-61
Typical Plate Wall

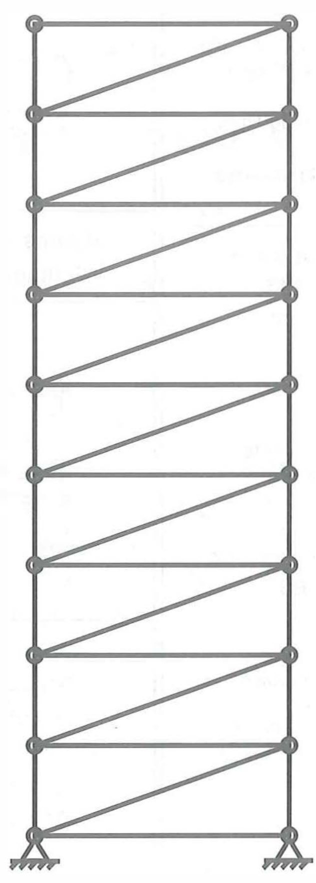


Figure 2-62
Plate Wall Diagonal Tension Brace Model

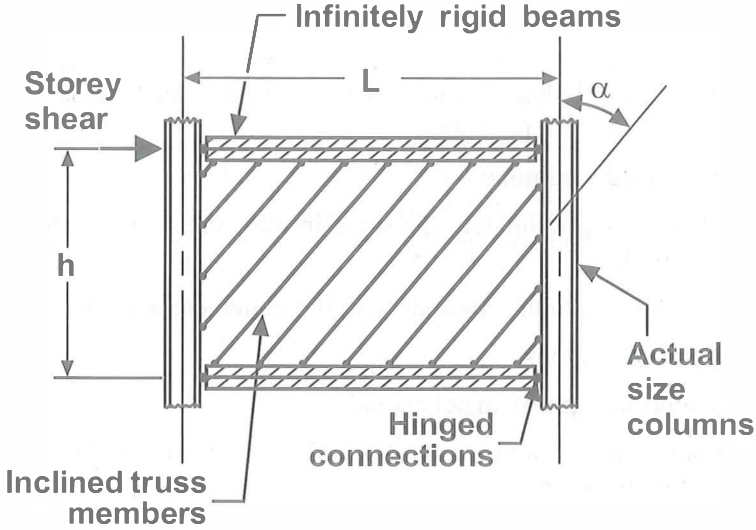


Figure 2-63
Strip Model for a Plate Wall

a plane frame structural analysis program. Ten strips per panel have been found to be sufficient in most cases. The continuity of connections between beams and columns, and the actual sizes of the beams, are accounted for in the analysis. When the entire plate wall is modelled, the average angle of inclination may be used for the complete wall, as stated in Clause 20.4.2. The analysis determines the tensile forces in the infill plates from tension field action, the forces imposed by the infill plate on the boundary beams and columns, and the forces and moments in the boundary beams and columns. Shishkin *et al.* (2009) discuss means of optimizing the strip model in terms of both accuracy and modelling efficiency.

For preliminary design, the overall behaviour of a plate wall can be approximated in a plane frame analysis as a vertical truss by representing each infill panel by a single diagonal tension brace (see Figure 2-62). Thorburn *et al.* (1983) express the equivalent area, A, of the diagonal tension brace as

$$A = \frac{wL\sin^2 2\alpha}{2\sin\theta\sin 2\theta}$$

The beams and columns are taken to have their actual cross-sectional properties in the analysis. When plate walls with moment frames are used, this model also determines the beam and column moments that develop as a result of frame action.

20.4 Angle of Inclination

Shishkin *et al.* (2009) demonstrate that, when using the strip model to analyze plate walls of typical proportions, the overall behaviour of the walls is relatively insensitive to the angle of inclination of the strips. They showed that selecting an angle of 40° from the vertical provides accurate, yet conservative, results over a wide range of wall configurations.

For cases that fall outside of the limits investigated by Shishkin *et al.* (2009), an expression developed by Timler and Kulak (1983) for the angle of inclination of the tension field strips is provided. This expression was determined by minimizing the work in one panel owing to the tension field action in the infill plate, flexure and axial forces in the boundary columns, and the axial force in one boundary beam per panel.

The expression was derived assuming:

- The storey shear is approximately the same in the panels above and below the storey under consideration.
- The beams are attached to the columns with pin-ended connections.
- The columns are continuous.
- The storey heights are approximately equal.

When these assumptions are not met, see Appendix A of Timler and Kulak (1983) to apply the least work derivation to other cases.

20.5 Limits on Column and Beam Flexibilities

In order for the tension field to develop relatively uniformly in the infill plate at each storey, the columns of the plate wall must be sufficiently stiff. Based on the work of Kuhn *et al.* (1952), the column flexibility parameter, ω_h, as given in Clause 20.5.1, shall not exceed 2.5.

The uniformity of the tension fields in the top and bottom panels of the plate wall depend on the stiffnesses of both the adjacent columns and the top or bottom beam, as appropriate. Dastfan and Driver (2008) developed a boundary member flexibility parameter, ω_L, to characterize the boundary stiffness for these extreme panels. The value of ω_L shall not exceed 2.5 at the top of the wall and 2.0 at the bottom, reflecting the relative importance of the behaviour of the bottom panel on the overall performance of the plate wall. The lower limit on ω_L of $0.84\,\omega_h$ is to prevent obtaining a negative beam stiffness. The derivation and application of both ω_L and ω_h are discussed by Dastfan and Driver (2008, 2009).

20.7 Beams and 20.8 Columns

Under high lateral loads, plastic hinges tend to develop in the beams and columns of plate walls. To avoid premature failure, beams shall be Class 1 or Class 2 sections, and columns Class 1 sections.

20.9 Anchorage of Infill Plates

These requirements ensure that the top and bottom infill plates are anchored to members that are sufficiently stiff to develop relatively uniform tension fields, and that the forces developed at the base of the wall are transferred properly into the substructure.

20.10 Infill Plate Connections

The infill plate is to be connected to the surrounding frame – and spliced, if required – to resist the factored ultimate tensile strength of the plate in order to ensure a ductile failure mode. These connections may be either welded or bolted.

21. CONNECTIONS

21.3 Restrained Members

When the compressive or tensile force transmitted by a beam flange to a column (approximated by the factored moment divided by the depth of the beam) exceeds the factored web bearing or flange tensile resistance of the column, stiffeners are required to develop the load in excess of the bearing or tensile resistance.

Taking the length of the column web resisting the compressive force as the thickness of the beam flange plus ten times the thickness of the column flange as in Clause 14.3.2(a)(i) results in the first equation given in Clause 21.3 for the bearing resistance of columns with Class 1 and 2 webs. For members with Class 3 and 4 webs, the bearing resistance of the web is limited by its buckling strength. The expression for the factored bearing resistance is conservatively based on the critical buckling stress of a plate with simply-supported edges:

$$\sigma_{cr} = k \frac{\pi^2 E}{12\left(1-\nu^2\right)\left(h_c/w_c\right)^2} = \frac{723\,000}{\left(h_c/w_c\right)^2} \quad \text{when } k_{min} = 4$$

The number 640 000, given in Clause 21.3(a), reflects a further reduction for the effect of possible residual stresses.

Although not stated, the bearing resistance computed from the second equation should not exceed the first. In both expressions, if the compression flange is applied at the end of a column, the loaded length should be reduced to $t_b + 4t_c$, and the resistance factor should be reduced to ϕ_{be}.

Graham *et al.* (1959) also show, based on a yield line analysis, that the column flange bending resistance, when subject to a tensile load from the beam flange, can be taken conservatively to be $7t_c^2F_{yc}$. Tests have shown that connections proportioned in accordance with this equation have carried the plastic moment of the beam satisfactorily.

When moment connections are made between beams and columns with relatively thick flanges (greater than about 50 mm), prudent fabrication practice suggests that the column flanges be inspected (such as radiographically) in the region surrounding the proposed weld locations to detect and thereby avoid any possible laminations that might be detrimental to the through-thickness behaviour of the column flange. Dexter and Melendrez (2000) reported on the results of recent studies on this topic.

Huang *et al.* (1973) demonstrated that beam-column connections designed such that the web was connected only for the shear force were capable of reaching the plastic capacity of the beam even though in some tests the webs were connected with bolts based on bearing-type connections in round or slotted holes. The slips that occurred were not detrimental to the static ultimate load capacity. For joints in zones of high seismicity, see Commentary on Clause 27.

Bolted extended end-plate-type connections are also commonly used for beam-to-column moment connections. Murray (2003) presents equations for the bearing and tensile resistances of the column flange opposite the flanges of the beam, for use with extended end-plate-type connections. AISC (2013) and Carter (1999) have adopted the design equations presented by Murray (2003). Note that the equation used in calculating the tensile resistance of the column flange is based on research using only ASTM A36 material. For this reason, if columns with higher yield strengths are used, it is recommended (conservatively) that the column yield strength be limited to 250 MPa (36 ksi) for calculating the tensile resistance of the column flange. Detailed design procedures for other limit states (other possible failure modes) for this type of connection are presented in Murray (1990) and AISC (2013). Prying action should also be checked on the end plate connection and the column flange opposite the tension flange of the beam. Clause 22.2.2(e) requires that bolts subject to tensile forces be pretensioned.

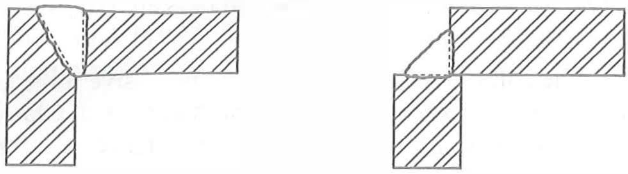
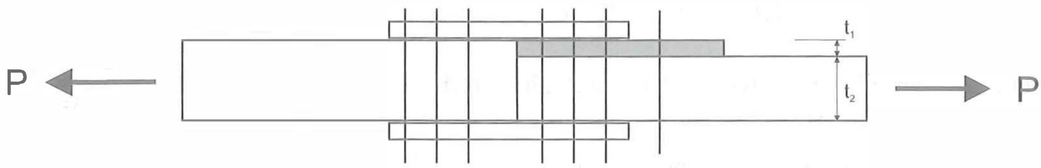


Figure 2-64
Details to Minimize Lamellar Tearing



$$\text{Load on Filler} = P\frac{w_1 t_1}{w_1 t_1 + w_2 t_2}$$

Figure 2-65
Load on Filler Plate

21.4 Connections of Tension or Compression Members

Obviously, the end connections must transmit the factored loads. In order to guard against providing a connection inconsistent with the member it connects, when the member size has been selected for some criterion other than strength, the designer may choose to provide a minimum connection with a capacity higher than the design load.

The requirement for a connection at least equal to 50% of the member's capacity was withdrawn in S16-01 as it was often misapplied, resulting in grossly oversized connections.

21.5 Bearing Joints In Compression Members

When determining the requirement for fasteners or welds to hold all parts securely in place, the stability of the structure shall be considered for all possible load conditions in accordance with the requirements of Clause 6.1.

21.6 Lamellar Tearing

In cases where shrinkage results as a consequence of welding under highly restrained conditions, very large tensile strains may be set up. If these are transferred across the through-thickness direction of rolled structural members or plates, lamellar tearing may result. Thornton (1973) and AISC (1973) give methods of minimizing lamellar tearing. Figure 2-64 illustrates one such case.

21.7 Placement of Fasteners and Welds

Gibson and Wake (1942) have shown that, except for cases of repeated loads, end welds on tension angles and other similar members need not be placed so as to balance the forces about the neutral axis of the member.

21.8 Fillers

The intent of this clause is to ensure that the total load transferred through a connection will be transferred uniformly over the combined cross section of the filler plate and the connected material, in order to avoid bending in the bolt shank. In order to do this, the filler plate should be connected for a load equivalent to the total load multiplied by the ratio of the filler plate thickness to the combined thickness of the filler plate and the connected material (see Figure 2-65). However, in slip-critical joints, tests with fillers up to 25 mm (1 inch) in thickness and with surface conditions comparable to other joint components show that the fillers act integrally with the remainder of the joint, and they need not be developed before the splice material (Kulak *et al.* 2001).

21.10 Fasteners and Welds in Combination

21.10.1 Requirements for the design of joints that combine welds and high-strength bolts placed in the same shear plane are covered in Clause 13.14.

21.11 High-Strength Bolts (in Slip-Critical Joints) and Rivets in Combination

Hot-driven rivets have a clamping force comparable to that of the pretensioned bolts, albeit somewhat more variable.

21.12 Connected Elements Under Combined Tension and Shear Stresses

The new clause 21.12 in the 2014 edition of the standard addresses the state of combined shear and tensile normal stresses in a plane of a connected element. This combined stress state occurs in many common connecting elements, including gusset plates, shear tabs, and beam webs welded to end plates. The research reported by Guravich and Dawe (2006) suggests that the presence of the state of combined stresses does impact the connecting element's strength, but this only occurs after a certain threshold has been reached, i.e. for certain combinations, normal and shear stresses may be considered in isolation. The new clause recognizes this fact and suggests that the tensile stress at full yield can occur simultaneously with a shear stress within 75% of the ultimate shear stress capacity ($0.75 \times 0.66 \approx 0.5$). This clause covers the strength limit state only, and stability of the connecting elements under combined stresses should also be considered.

22. DESIGN AND DETAILING OF BOLTED CONNECTIONS

Note: This Clause primarily applies to high-strength bolts. Although the Standard permits the use of A307 bolts in certain applications, some of the requirements in this Clause do not apply to them.

22.1 General

The behaviour of a joint depends both on how the bolts are loaded and installed. In the 1984 edition, for the first time, the use of snug-tightened high-strength bolts was permitted. Their use has proved successful. As there are four basic types of connections, three with bolts in shear and one with bolts in tension, it is absolutely essential that the design documents specify the type of connections used.

Kulak *et al.* (2001) showed that the ultimate shear and bearing resistances of a bolted connection are not dependent on the pretension in the bolt. As the number of situations (Clause 22.2.2) where pretensioning is required is limited, the norm for building construction is to use snug-tightened bearing-type connections. Departures from the norm are only to be made with due consideration. Few joints in building construction are subject to frequent load reversal nor are there many situations where a one-time slip into bearing cannot be tolerated.

High-strength bolts must be pretensioned when they are subject to shear in slip-critical connections, tension, seismic forces in applications as required in Clause 27.1, or any combination thereof. High-strength bolts subject to shear in bearing-type connections may either be pretensioned or snug-tightened. Only A325 and A490 bolts, the tension-control bolt assemblies (F1852 and F2280), and the metric series (A325M and A490M) may be used in joints requiring pretensioned high-strength bolts.

As a result of normal fabrication practice, minor misalignment of bolt holes may occur in connections with two or more bolts. Such misalignment, if anything, has a beneficial effect (Kulak *et al.* 2001) resulting in a stiffer joint, improved slip resistance and decreased rigid body motion.

A comprehensive summary of bolt requirements is given by Kulak *et al.* (2001).

22.2.1 Use of Snug-Tightened High-Strength Bolts

Snug-tightened bolts may be used, except for the specific cases given in Clause 22.2.2 where the use of pretensioned high-strength bolts is required. Bolts that are not pretensioned must be installed to a snug-tightened condition. These may be A307, A325 or A490 bolts. Because the ultimate limit states of shear through the bolt and bearing on the plate material are not significantly affected by the level of pretension (Kulak *et al.* 2001), it is only logical to permit bolts of higher strength than the A307 bolt to also be installed snug-tight in similar connections. This was recognized, in part, as early as the 1984 edition.

22.2.2 Use of Pretensioned High-Strength Bolts

(a) Pretensioning of the bolts provides the clamping force in slip-critical connections and hence the slip resistance at the specified load level appropriate to the condition of the faying surfaces.

(b) Pretensioning of the bolts provides energy dissipation under cyclic earthquake loading in connections proportioned for seismic applications that trigger the requirements in Clause 27.1, although these connections are proportioned as bearing-type connections for the ultimate limit state. The contact surfaces should be Class A or better for such joints.

(c) See (d).

(d) Pretensioning in both these connections ensures that the bolts don't work loose and, if necessary, ensures adequate fatigue behaviour.

(e) An example of such a connection is a tee-hanger connection. Pretensioning reduces the prying action and the stress range.

(f) In connections with oversize or slotted holes, pretensioning prevents gross movement within the joint. See also Clause 22.3.5.2 to determine for which cases slip-critical connections are required.

For the usual building structure, full wind loads and earthquake loads are too infrequent to warrant design for fatigue, as the number of stress cycles are less than the lower limits given in Clause 26.3.5. Therefore, slip-critical connections are not normally required in buildings for wind or seismic load combinations. However, connections of a member subject to flutter, where the number of cycles is likely higher, is an exception. Popov and Stephen (1972) observed that the bolted web connections of welded-bolted moment connections slipped early in the cyclic process.

Slip-critical connections are required in connections involving oversized holes, certain slotted holes, fatigue loading, or crane runways and bridges. In assessing whether or not the joint slip is detrimental at service level loads, Popov and Stephen (1972) and Kulak *et al.* (2001) have shown that, in joints with standard holes, the average slip is much less than a millimetre. Bolts of joints in statically loaded structures are most likely in direct bearing after removal of the drift pins, due to the self weight of the member, and are thus incapable of further slip (RCSC 2014).

22.2.5 Fastener Components

A325M, A490M, A325 and A490 bolts are produced by quenching and tempering (ASTM 2013, 2012, 2010, 2012, respectively). A325 bolts are not as strong as A490 bolts but have greater ductility. For this reason and reasons of availability, the use of A490 bolts is subject to restrictions as discussed subsequently. ASTM F1852 and F2280 bolts, commonly known as "tension control" or "twist-off" bolts, have mechanical and chemical properties equivalent to A325 and A490 bolts, respectively (ASTM 2011, 2012).

The normal bolt assembly consists of an A325, A490, F1852 or F2280 bolt, with a heavy hex head, restricted thread length, coarse threads, and a heavy hex nut. F1852 and F2280 bolt assemblies consist of a bolt with a splined end which typically has a button head. Alternatives to the normal bolt assemblies are available which differ in various aspects and, in some cases, may offer one or more advantages. Their use is permissible under the conditions set forth in Clause 22.2.5.4.

At the time of preparation of this Commentary, availability of A325M and A490M bolts requires an order of unusually large quantity and a long lead time.

Galvanized A325 bolts are permitted; however, metallic coated A490 bolts are not permitted, as they are especially susceptible to stress corrosion and hydrogen stress cracking (Kulak *et al.* 2001). The rotation requirement of this clause provides a means of testing the galvanized assembly for proper fit and for proper thread lubrication. Installation of F1852 and F2280 assemblies is dependent upon consistent friction properties of the bolt threads and the nut. Therefore, these assemblies should not be hot-dip galvanized. F2280 assemblies should not be electroplated.

22.3.5 Bolt Holes

Details on the sizes and types of holes (standard, oversize, or slotted) permitted for bearing-type and slip-critical connections are given. While the Standard permits several hole-making methods, punching and drilling are the most common. Incremental punching is sometimes used in fabricating slotted holes – especially long slots. Thermal cutting of holes, such as cutting the edges of a slot between two punched holes, is acceptable within the requirements of Clause 28.4.3.

Clause 22.3.5.1 allows selected Imperial bolts in metric holes without restriction.

A hardened washer, when required in Clause 22.3.5.2, is intended to cover the hole (or bridge the slot) if it occurs in an outer ply.

The object for the requirement of Clause 23.4.2(d) for an 8-mm hardened washer for large-diameter A490 bolts is to distribute the high clamping forces of these bolts. It is an acceptable alternative to cover the hole with a 10 mm mild steel plate washer with a standard hardened washer under the head or the nut. The contract documents should specify any specific requirements.

23. INSTALLATION AND INSPECTION OF BOLTED JOINTS

Note: This Clause primarily applies to high-strength bolts. Although the Standard permits the use of A307 bolts in certain applications, some of the requirements in this Clause do not apply to them.

Bolts required to be pretensioned must be tightened to tensions of at least 70% of their specified minimum tensile strength. All other bolts need only be snug-tightened.

Except when galvanized, A325 bolts may be reused once or twice, providing that proper control on the number of reuses can be established (Kulak *et al.* 2001; RCSC 2014). A490, F1852 and F2280 bolts should not be reused. The level of pretension attained in bolts of Grade A490 decreases significantly when the bolts are re-used.

23.1 Connection Fit-up

The simple phrase "connections in firm contact when assembled" describes the snug-tightened condition.

23.2 Surface Conditions for Slip-Critical Connections

The treatment of the faying surfaces within the plies of slip-critical joints is to be consistent with the mean slip coefficient chosen for design (Clause 13.12). For clean mill scale, the surfaces must be free of substances which would reduce the slip coefficient. For other coatings, the surface preparation, coating application, and curing should be similar to those used in the tests to obtain the slip coefficient. The Society for Protective Coatings (SSPC) provides specifications for cleaning and coating of steel structures. Kulak *et al.* (2001) provide information on slip for various surface conditions and coating types.

23.4 Use of Washers

Clauses 23.4.1 and 23.4.2 list the circumstances when ASTM F436 hardened washers are required under the turned element and with pretensioned bolts. It follows that these washers are not required in A325 bolt installations except for oversized or slotted holes in pretensioned connections. An alternative to the use of an 8-mm F436 washer for pretensioned large A490 and F2280 bolts in oversized and slotted holes is provided in Clause 22.3.5.2(d).

The requirements for bevelled washers with ASTM A490 bolts are more stringent than for A325 bolts because of the somewhat reduced ductility of the former.

23.5 Storage of Fastener Components for Pretensioned Bolt Assemblies

This clause emphasizes that proper storage of fastener components is particularly critical for ASTM F1852 and F2280 assemblies, because the torque at which the splined end is sheared off the bolt shank is dependent on the friction characteristics between the bolt threads and the nut, which therefore must be maintained at the as-manufactured condition, so that the relationship between the twist-off torque and bolt pretension is what was intended and is expected to be.

23.7 Pretensioned High-Strength Bolt Assemblies

For all pretensioned high-strength bolt installations, it is critical that inspection for bolt pretension be done while the bolt tightening is in progress. Verification that the installation techniques described in this clause have been followed will provide adequate assurance that the required bolt tensions are being attained.

23.7.1 Installation Procedure

The pretensioning procedures included in this Standard have been proven (Kulak and Birkemoe 1993, Kulak and Undershute 1998) to provide bolt tensions required by this clause. Torque-tension relationships are highly variable and dependent upon many factors including installation procedures, bolt finish, and bolt and nut thread conditions. For this reason, it is not possible to establish a standard bolt torque value that corresponds to the required bolt pretension values. Clause 23.8 describes the proper, simple inspection procedures for bolted connections.

23.7.2 Turn-of-Nut Method

Any installation procedure used for pretensioning high-strength bolts involves elongating the bolt to produce the desired tension. Although the shank of the bolt probably remains elastic, the threaded portion behaves plastically. Because the bolt as a whole is tightened into the inelastic range (the flat portion of the load-deformation curve), the exact location of "snug-tight" is not critical (Kulak *et al.* 2001). The turn-of-nut method is a strain or deformation control method, and even a considerable change in deformation results in little change in load. Thus, application of the specified amount of nut rotation results in pretensions that are not greatly variable. They are also greater than those prescribed in Table 7, which occur about where inelastic action begins. Although there is a reasonable margin against twist-off, the tolerance of $\pm 30^{\circ}$ or $\pm 1/12$ of a turn on nut rotation prescribed in the footnote to Table 8 is good practice, particularly when galvanized A325 bolts or black A490 bolts are used.

23.7.3 and 23.7.4 Use of ASTM F959, F1852 and F2280 Bolting Systems

The Standard permits the use of F1852, F2280 and F959 bolting systems. These systems are proprietary in nature, relying on a discernible physical change in a part of the bolt system indicating that the minimum bolt tension has been achieved. Systems that rely on irreversible deformations or fracture of a part serve only to indicate that, during installation, a force or torque sufficient to deform or fracture the part had been reached. Even with such a system, reliable results are dependent on strict adherence to the installation procedures for snugging of the joint and patterned tightening operations as given in Clause 23.7.1 and to the storage fastener requirements of Clause 23.5 for F1852 and F2280 bolts.

23.8 Inspection Procedures

Bolts, nuts, and washers are normally received with a light residual coating of oil. This coating is not detrimental; in fact it is desirable and should not be removed. This is especially important for F1852 and F2280 bolts, since these bolts depend on the lubricant to achieve the desired level of pretension. Galvanized bolts and/or nuts may be coated with a special lubricant to facilitate tightening. Obviously, this should not be removed.

The inspection procedures used depend on whether the bolts are specified to be snug-tightened or pretensioned. In all cases, by Clause 23.8.1, the inspector shall observe that the procedure for the installation of the bolts conforms with the requirements of this Standard.

When snug-tightening is specified, the tightening is deemed satisfactory when all of the connected elements are in full contact. Galling of the turned element may be evident. Inadvertent pretensioning of snug-tightened bolts is normally not a cause for concern.

When pretensioning is specified, the tightening is deemed satisfactory when all of the elements are in full contact, and observation of the sides of the turned elements shows that they have been slightly galled by the wrench. This is all that is required.

When bolts are tightened by the turn-of-nut method and when there is rotation of the part not turned by the wrench, the outer face of the nut may be match-marked with the bolt point before final tightening, thus affording the inspector visual means of noting nut rotation. Such marks may be made with crayon or paint by the wrench operator after the bolts have been snugged.

Should disagreement arise concerning the results of inspection of bolt tension of bolts specified to be pretensioned, arbitration procedures as given in Annex I are to be followed. The use of inspection torque values other than those established according to the requirements of Annex I is invalid because of the variability of the torque-tension relationship. The inspection procedure given in Annex I is the same as that recommended by the Research Council on Structural Connections (RCSC 2014) and places its emphasis on the need to observe the installation for the proper tightening procedures, rather than using the arbitration procedures which in fact are less reliable.

Regardless of the installation procedure or the type of bolt-washer-nut assembly used, it is important to have all of the plies drawn up tight before starting the specific tightening procedure. This is particularly so for stiff joints that require pattern tightening.

24. WELDING

24.1 and 24.2 Arc and Resistance Welding

Consistent with CSA policy that the requirements of one standard are not repeated in another, the user of this Standard is referred to CSA Standards W59 and W55.3 for the requirements for arc and resistance welding (e.g. weld quality, welding procedure and practice, etc.), respectively, but with two distinct exceptions. The distinction is made that, for arc welds with matching electrodes, the factored resistances for static loadings and the fatigue resistance for fatigue loadings are obtained from Clauses 13.13 and 26 of this Standard, respectively. (Much of the research into weld strengths and formulation of weld resistances has been done by members of this Standard committee.)

Designers' attention is drawn to the fact that, in the U.S.A., cracking has been noted after welding of column web stiffener or of doubler plates on heavily rotarized W-shapes in the fillet regions. This is attributed to the loss of ductility due to cold working.

W59 permits the use of intermittent fillet welds in the compression zone, irrespective of whether fatigue is a consideration.

24.3 Fabricator and Erector Qualification

The intent of Clause 24.3 is simply that the responsibility for structural welding shall lie with the fabricators and erectors certified by the Canadian Welding Bureau to the requirements of CSA W47.1 and/or CSA W55.3, as stated specifically in the clause. Such certification should ensure that the fabricators and erectors have the capability to make structural welds of the quality assumed by S16-14.

There is a specific requirement that fabricators and erectors meet the requirements of CSA W47.1 in Division 1 or Division 2 for arc welding. This will ensure that the fabricator or erector has a suitably qualified welding engineer either on staff or on retainer. However, the clause does permit work to be sublet to a Division 3 fabricator or erector (i.e. organizations without a welding engineer on staff or on retainer), provided the Division 1 or 2 fabricator or erector retains responsibility for the work.

25. COLUMN BASES AND ANCHOR RODS

The clauses on column bases and anchor rods have been combined, as the two are likely found together as components of the same foundation unit. The designer is referred to appropriate clauses of CSA A23.3 (CSA 2014) for the various resistances of the reinforced concrete elements.

In general, the use of base plates bearing directly on grout is preferred to the use of levelling plates interposed between the base plate and the grout. The latter condition may lead to uneven bearing.

Typically, anchor rods – formerly referred to as anchor bolts – are threaded rods that are either supplied in accordance with ASTM F1554 or fabricated from a steel bar of A36 or G40.21-300W steel. The expressions for the tensile, shear, and combined shear and tensile resistance of anchor rods are similar to those for high-strength bolts. The basic elliptical interaction diagram is used for combined shear and tension. For tension and bending, the factored moment resistance is limited to the factored yield moment, because the ductility of the steel used may be limited. For anchor rods in tension, the designer should specify a material with fracture toughness appropriate for the minimum service temperature. Pretensioning of anchor rods is usually not recommended, as there is a tendency for relaxation and a possibility of stress corrosion. Pretensioning requires special attention.

All anchor rod resistances, whether for tension, shear, bearing or moment, or for use in interaction equations, are those given in this Clause.

26. FATIGUE

Clause 26 provides the requirements for the design of members and connections subjected to cyclic loading and susceptible to the formation and growth of cracks during the design life of the structure. The phenomenon of formation and growth of cracks under cyclic loading is called fatigue. The fatigue limit state, which is the limiting case of the slow propagation of a crack within a structural element, can result from either live load effects directly or as the consequence of local distortion within the structure due indirectly to live load effects. These two cases are referred to as live-load-induced fatigue and distortion-induced fatigue, respectively. The limit state of fatigue is checked at load levels expected to occur many times during the life of the structure – loads that are considered to be repetitive. In the event that more than 20 000 stress cycles take place, the loaded members, connections, and fastening elements shall be proportioned so that the probability of fatigue failure is acceptably small. In such cases, the design shall be based on the best available information on the fatigue characteristics of the materials and components to be used. In the absence of more specific information, which is subject to the approval of the owner, the requirements of Clause 26 in its entirety provide guidance in proportioning members and parts. The fatigue design loads are taken to be the specified loads. In addition, Clause 26.1 requires that all members and connections in the structure meet the ultimate limit state requirements, i.e. that factored resistances be at least equal to the effect of factored static loads – load levels that occur very seldom, perhaps only a few times in the life of the structure, but which the structure must nevertheless be able to withstand in order to achieve the required level of safety.

A substantial amount of experimental data, developed on steel beams since 1967 under the sponsorship of the National Co-operative Highway Research Program (NCHRP 1970, 1974; Fisher 1974) of the U.S.A., has shown that the most important factors governing fatigue resistance are the stress range, the type of detail, and the number of cycles. Steel grade and fracture toughness do not have a significant effect on the fatigue resistance.

The provisions of this clause are those commonly used in North American design standards, except for the long-life region of behaviour. The North American fatigue design approach for most civil engineering structures is to base the fatigue life calculation on a nominal stress range (a stress range calculated using basic strength of materials approach, which does not account for stress concentration) and to account for stress concentration in the detail of interest by selecting the appropriate fatigue category varying from Category A, the most favorable detail with no stress concentration, to Category E1, the least desirable detail. Experience has shown that fatigue considerations for details of Category A through B1 rarely govern. Nevertheless, these are included for completeness.

While fatigue is generally not a design consideration for buildings such as those for commercial or residential occupancies, industrial buildings may have many members, such as crane girders, for which fatigue is a concern. Other instances where fatigue is likely a consideration are amusement rides, wave guides, sign support structures, and beams supporting reciprocating machinery. When members and connections are subjected to fatigue loading, Clause 26 requires that they be designed, detailed, and fabricated to minimize stress concentrations and abrupt changes in cross-section. Consideration should also be given to the service conditions, which may change the condition of stress concentration, namely, the fatigue category after the structure has been placed in service. For example, a detail with no significant stress concentration can become one with high stress concentration if the member is exposed to a corrosive environment. Therefore, the designer must consider the possibility of changing stress conditions during the service life of the structure.

Fatigue crack growth is referred to either as load-induced or as distortion-induced. Load-induced stresses are those corresponding to the design loads normally considered by a first-order analysis where the effect of deformations on force effects are not considered. Distortion-induced stresses are those resulting from the relative movement of connected parts of an assemblage in such a way that large localized strains are produced. Because this phenomenon is difficult to include with any level of accuracy in the design calculations, distortion-induced fatigue is best avoided by using recognized details to obviate potential problems. An accurate assessment of distortion-induced stresses requires detailed modeling of the interactions between all structural and non-structural elements of the structure.

26.3 Live-Load-Induced Fatigue

26.3.1 Calculation of Stress Range

The stress range is the algebraic difference between the maximum stress and the minimum stress at a given location due to the passage of the live load. When calculating the applied stress range, the effect of any load eccentricity must be accounted for. Although minor eccentricities are usually ignored at the ultimate limit state because it is expected that yielding of the member at the ultimate limit state will reduce this effect, the member remains elastic at the fatigue limit state. Since the effect of stress concentration is not included in the stress range calculation, its effect must be incorporated by selecting the appropriate fatigue category as described for usual structural details illustrated in Figure 2 and described in Table 9 of the Standard.

Because fatigue cracks grow only if there is a net tensile stress from the live load, it is not necessary to investigate fatigue at locations where the applied stresses are always in compression and at locations where the maximum tensile live load stress is less than the compressive dead load stress.

26.3.2 Design Criteria

The criterion expressed by the relationship $F_{sr} \geq f_{sr}$ simply states that the fatigue resistance (or allowable stress range) of a given detail for the design number of load cycles, nN, over the design life of the structure must be equal to or exceed the calculated stress range. The allowable stress range may be calculated from the equation $F_{sr} = (\gamma/nN)^{1/3} \geq F_{srt}$ when the fatigue resistance is greater than the constant amplitude threshold stress range, or from $F_{sr} = (\gamma'/nN)^{1/5} \leq F_{srt}$ when the fatigue resistance is less than the constant amplitude threshold stress range. Each fatigue curve represents the mean fatigue life minus two standard deviations from a series of constant-amplitude fatigue tests on details representative of the category. The fatigue life constants, γ and γ', for the appropriate detail are obtained from Table 10 of the Standard. Also shown in Table 10 is the constant-amplitude threshold stress range F_{srt}, represented in Figure 1 of the Standard by the horizontal dashed lines. For constant-amplitude stress ranges below F_{srt}, crack growth does not occur, i.e. the fatigue life of the detail is infinite. For variable-amplitude fatigue loading, the slope of the fatigue curves below F_{srt} is reduced to 1/5 because it is expected that some of the applied stress ranges will still lie above F_{srt}, even if the average stress range is smaller than F_{srt}. The number of cycles at which the slope of the fatigue curves changes from 1/3 to 1/5 is designated as nN' and can be either calculated from $(\gamma/nN')^{1/3} = (\gamma'/nN')^{1/5}$ or obtained from Table 10.

26.3.3 Cumulative Fatigue Damage

In reality, fatigue loading is rarely at constant amplitude; it is usually of variable stress amplitude, which results in variable numbers of stress ranges of different magnitudes. The cumulative fatigue damage that results from variable-amplitude loading can be evaluated using the linear damage theory known as the Palmgren-Miner rule. Over the design life of the structure, the number of cycles for each identified stress range is estimated, and the fraction of the fatigue life expended by these cycles of loading is obtained by dividing the number of cycles at a given stress range by the fatigue life for that stress range as found from Table 10 or Figure 1 of the Standard. The sum of these fractions so determined, including those for the long-life region of behaviour where the slope of the S–N curve is 1/5, shall not exceed unity as given in this Clause. Chapter 11 of Kulak and Grondin (2014) provides more detailed information on fatigue.

26.3.4 Fatigue Constants and Detail Categories

The fatigue constants defining the eight fatigue curves illustrated in Figure 1 are given in Table 10. Selection of the appropriate fatigue category is carried out with the assistance of Figure 2 and Table 9. Also added to Table 9 are high-strength bolts under tensile cyclic loading.

26.3.5 Limited Number of Cycles

This clause gives a limit on the number of cycles below which no special consideration other than good detailing is necessary for fatigue. The limit is the greater of 20 000 cycles and the fatigue life of the detail.

26.4 Distortion-Induced Fatigue

Secondary stresses due to deformations and out-of-plane movements are not normally calculated in the design process but can be a source of fatigue failures when proper detailing practices are not followed (Fisher 1978 and 1984). Crane girders, their attachments, and supports require careful design and attention to details to minimize fatigue cracks (Griggs 1976).

If the web of a plate girder without longitudinal stiffeners is sufficiently slender, fatigue cracks may develop at the web-to-flange juncture due to a lateral bending of the web. Tests on girders with a web of h/w ratio greater than $3150/\sqrt{F_y}$ have shown a significant reduction in fatigue resistance to the out-of-plane movement of the web when subjected to in-plane bending (Toprac and Natarajan 1971).

26.5 High-Strength Bolts

High-strength bolts loaded in shear are not susceptible to fatigue failure. However, this is not the case when bolts are placed in direct tension. Pretensioned high-strength bolts in joints that are nominally loaded in tension experience little, if any, increase in axial stress under service loads (Kulak *et al.* 1987). For this reason, bolts that are subjected to cyclic tension shall be pretensioned using the procedure outlined in Clause 23.7 of the Standard. In addition, the prying action shall be kept at a relatively small fraction of the total bolt force. The Research Council on Structural Connections (2014) limits the prying action in joints subjected to cyclic tension to a maximum of 30 percent of the externally applied force.

27. SEISMIC DESIGN

Specific seismic design requirements are given in this clause. While the requirements represent the best available knowledge, designers should be alert to new information leading to improved design procedures.

The NBCC assigns ductility-related force modification factors, R_d, and overstrength-related force modification factors, R_o, (i.e. load reduction factors) to various structural systems in relation to their capacity to dissipate energy by undergoing inelastic deformations and to the minimum level of overstrength which can be counted on for each particular seismic-force-resisting system. The greater the ability of the structure to dissipate energy, the higher is the assigned value of R_d. Values of R_d greater than 1.0 can be justified only if the structure has the ability to undergo inelastic deformations without loss of resistance. The product of R_d and R_o is used as a divisor to reduce the magnitude of the design seismic force.

The objective of Clause 27 is to provide details that will exhibit ductility consistent with the values of R_d and R_o assumed in the analysis. The Clause applies to all steel structures in Canada for which seismic energy dissipation capability is required through ductile inelastic response, i.e. all structures for which $R_d \geq 2.0$. Clause 27 defines the requirements for nine classes of structures with $R_d \geq 2.0$:

- Ductile moment-resisting frames (Type D, with $R_d = 5.0$ and $R_o = 1.5$)
- Moderately ductile moment-resisting frames (Type MD, with $R_d = 3.5$ and $R_o = 1.5$)
- Limited-ductility moment-resisting frames (Type LD with $R_d = 2.0$ and $R_o = 1.3$)
- Moderately ductile concentrically braced frames (Type MD, with $R_d = 3.0$ and $R_o = 1.3$)
- Limited-ductility concentrically braced frames (Type LD, with $R_d = 2.0$ and $R_o = 1.3$)
- Ductile eccentrically braced frames (Type D, with $R_d = 4.0$ and $R_o = 1.5$)
- Ductile buckling-restrained braced frames (Type D, with $R_d = 4.0$ and $R_o = 1.2$)
- Ductile plate walls (Type D, with $R_d = 5.0$ and $R_o = 1.6$)
- Limited-ductility plate walls (Type LD, with $R_d = 2.0$ and $R_o = 1.5$).

In addition, other special framing systems are permitted under Clause 27.12.

In each structural system, certain structural elements are designed to dissipate energy by inelastic straining; other members and connections in the frame must be designed to respond elastically to the loads induced by the yielding elements. Generally, the dissipating elements in moment frames are the beams, in concentrically braced frames the braces, in eccentrically braced frames the links, and in plate walls the wall infill plates. Other elements may also contribute, but to a much lesser extent, for example the connection panel zone in moment-resisting frames, the gusset plates in concentrically braced frames, the outer beam segments in eccentrically braced frames, and beams and columns in steel plate walls.

Properly detailed moment-resisting frames can exhibit very ductile behaviour. Three categories of moment-resisting frames are recognized: first, ductile moment-resisting, or Type D frames, in which members and connections are selected and braced to ensure that severe inelastic straining can take place; second, moderately ductile moment-resisting frames, or Type MD, in which the member details can satisfy the lower inelastic straining demand in structures proportioned to resist the greater design loads, while at the same time, connections are adequate to accommodate the associated forces and deformations. For both systems, beam-to-column connections are required to be designed and detailed in accordance with the CISC Moment Connections for Seismic Applications (CISC 2014), or their performance has to be demonstrated, by means of physical testing, as satisfying minimum criteria under the action of cyclic load as described in Annex J. The third system, Type LD for limited ductility, undergoes still less inelastic demand consistent with the higher design loads and can in general make use of traditional connection detailing, combined with special requirements associated with welding, etc.

Concentrically braced frames are those in which the centre-lines of diagonal braces, beams, and columns are approximately concurrent with little or no joint eccentricity. Inelastic straining must take place in bracing members subjected principally to axial load. Compression members dissipate energy by inelastic bending after buckling, and in subsequent straightening after load reversal. Cyclic local buckling can lead to early fracture, and consequently width-to-thickness limits are restricted for braces. These frames usually have limited redundancy and are prone to concentration of inelastic response in one or a few storeys where energy dissipation is localized. Emphasis in these categories is placed on the presence of braces with similar tensile strength in opposite directions, such that the reduction in storey shear resistance is minimized in the event of brace buckling in a storey.

Two categories of concentrically braced frames are considered, those with moderate ductility (MD) and limited ductility (LD). Both permit several bracing configurations. Compared with past editions of the Standard, the provisions maintain strict limits on width-thickness ratios; overall slenderness limits of braces are relaxed, and changes have been made to the requirements for connection design forces. However, height limitations apply. Since S16-09, bracing configurations with braces intersecting columns at one or more elevations between horizontal diaphragms have been permitted for Type LD braced frames, provided that the columns can accommodate the bending demand due to buckling and yielding of braces within the storey and that horizontal struts are introduced to ensure a continuous load path between tension-acting braces. In S16-2014, the use of this framing configuration has been extended to include Type MD braced frames, and height limits have been extended so that multi-tiered solutions can be obtained from a wider choice.

Ductile eccentrically braced frames are those in which diagonal braces, at least at one end, intersect the beam instead of the beam and column intersection or, in the case of chevron bracing, the two braces do not intersect the beam at a common point. These configurations create eccentric beam links that are designed to dissipate energy. The Standard gives provisions for frames with links in the beams. Beams can be W-shapes or built-up rectangular tubular sections. Lateral bracing at the link ends can be omitted when the latter is used. Provisions for modular links that can be replaced after a severe earthquake have been introduced in CSA S16-14.

The ductile buckling-restrained braced frame system was introduced in CSA S16-09. The braces include a core element with reduced cross-section segment where yielding is expected to develop in both compression and tension. The core is prevented from buckling by means of a lateral restraining mechanism. The system is expected to offer a higher ductility (R_d = 4.0) compared to Type MD and Type LD concentrically braced frames. Typically, brace details vary depending on the suppliers, but the inelastic cyclic performance must be demonstrated by means of sub-assemblage and individual qualification cyclic physical testing. The brace compressive and tensile resistances established in these tests must be used in the capacity design process.

Plate walls are formed by thin infill wall plates framed by beams and columns. These highly redundant and stiff systems dissipate energy by yielding of the infill plate and, often, yielding of the framing members. The good seismic performance anticipated is reflected in their respective applicable values of R_d and R_o. Two categories are defined, Types D and LD. In S16-09, new design requirements were included for beams and columns of Type D plate walls. The Standard also permits the introduction of uniformly distributed circular perforations in the infill plates, to avoid excessive lateral overstrength without resorting to using plates that are too thin for practical construction. Corner openings can also be introduced in the wall plates to facilitate the passage of electrical and mechanical equipment. In general, the design and detailing requirements specified for Type D walls also apply to Type LD walls, except that beams and columns need not be rigidly connected for Type LD walls.

In all systems, because the behaviour of connections will often be critical for good performance under severe earthquake loading, the engineer's responsibility for a seismically critical structure includes not only the provision of connection design loads but also the specification of connection type and details.

Structures for which $R_d = 1.5$ have been assumed in the past to have sufficient inherent energy dissipation capacity arising from traditional design and fabrication practices, so that no additional requirements were necessary. However, since energy dissipation properties can only be mobilized if brittle failure is avoided, minimum requirements are prescribed in Clause 27.11 to achieve this for structures subjected to higher seismic demand. The NBCC (since its 2010 edition), permits the use of structures with $R_d = 1.5$ for buildings taller than 15 m when used for occupancies other than assembly occupancy. Special requirements are given in Clause 27.11 for these taller structures. In addition, other special framing systems are permitted under Clause 27.12.

27.1 General

The expression "$I_E F_a S_a(0.2)$", adopted in the NBCC 2010, is referred to as the "specified short-period spectral acceleration ratio" in Clause 27, whereas the expression "$I_E F_v S_a(1.0)$" is referred to as the "specified one-second spectral acceleration ratio".

27.1.1 A distinction is made between the "seismic-force-resisting system" (SFRS) and the "vertical seismic-force-resisting system". The latter corresponds essentially to the vertical bracing, wall or frame system that takes the form of one or more of the systems described in Clause 27. The SFRS is the whole structural system resisting lateral loads, including the foundations, anchorage to foundations, the vertical seismic-force-resisting system, collector elements, and roof and floor diaphragms. In some cases, members specifically designed for gravity loading only may be relied upon for a contribution to a reserve lateral resistance following storey yielding, and in this case some provisions of Clause 27 apply also to these members (see Clause 27.5.5.2).

27.1.2 This Clause sets out the principles of capacity design and states that the ductile energy-dissipating elements must be clearly identified and detailed along the lateral load path, and that a proper strength hierarchy must be provided in the seismic-force-resisting system to constrain inelastic response to these ductile elements. The energy-dissipating elements must be designed to sustain several reversed cycles of inelastic loading with minimum strength and stiffness deterioration. Other elements must be designed to remain essentially elastic for the duration of the seismic ground motion. Anchor rods must transfer the loads to the foundation.

The maximum anticipated seismic loads imposed on the non-dissipating elements can be determined by hand calculations, static incremental (push-over) analysis or nonlinear dynamic time-history analysis. The inelastic behaviour under cyclic loading of the dissipating elements, including yielding, strain hardening or strength degradation, must be accounted for in the calculations and numerical models. Non-dissipative elements can be assumed to behave elastically in numerical models. A number of site-representative ground motions are necessary in nonlinear dynamic analysis, and maximum response loads in the members are to be determined. Such analyses may be of particular value for tall buildings, especially those beyond the height limits imposed by some other provisions of Clause 27. Other applications may be justified in cases where the requirements of capacity design are known to lead to conservative design loads (e.g. moment-resisting frames proportioned for stiffness and wind effects, plate walls or eccentrically braced frames with long links (Han 1998)).

In cases where the energy-dissipating elements have been oversized, a limit has been placed on the maximum forces that the non-dissipating elements must resist by setting the maximum anticipated seismic load equal to that corresponding to $R_d\, R_o = 1.3$. This maximum load corresponds to the elastic seismic load level determined using $R_d = 1.0$, while it is also recognized that the non-dissipating elements generally possess an overstrength level that justifies $R_o = 1.3$. Connections designed for seismic loads corresponding to $R_d\, R_o = 1.3$ must exhibit a ductile governing failure mode, such as yielding in tension or bolt bearing (Tremblay *et al.* 2009). Otherwise, the limit on seismic loads must be increased to loads corresponding to $R_d\, R_o = 1.0$. In computing the forces on the structure corresponding to $R_d R_o = 1.3$, the upper limit of $V = (2/3)S(0.2)I_E\, W/(R_d\, R_o)$ given in the NBCC applies, provided that the seismic-force-resisting system has an R_d equal to or greater than 1.5. In this case, the upper limit becomes $(2/3)S(0.2)I_E W/1.3$. Also, where foundation "rocking" is accounted for in accordance with NBCC, design forces for the SFRS may be limited to values associated with maximum forces that can develop with foundation rocking. Foundation rocking, however, induces larger storey drifts that must be accounted for in the design.

27.1.3 The vertical seismic force-resisting systems described in Clause 27 are expected to exhibit proper performance when non-structural elements such as walls or interior partitions are separated from the structural elements under earthquake deflections. If this cannot be achieved, the effects of the interaction must be accounted for in the analysis and the design.

27.1.4 Gravity load-carrying elements such as columns and beam-to-column connection elements must be able to support the companion gravity loads while undergoing the large deformations expected during earthquakes. For example, a simple beam end connection in the displaced configuration should resist shear due to the companion gravity loads.

Under a severe ground motion, columns in multi-storey structures will be subjected to shear forces and bending moments due to variations in storey drifts that will develop along the structure height. Splices in the columns that are not part of the seismic force-resisting systems must be designed to resist shear forces associated with this response. This provision applies in both orthogonal directions. Requirements for splices in columns that are part of the seismic-resisting systems are given in Clauses 27.2 to 27.11.

27.1.5 This Clause applies principally to the materials used in the yielding elements and connections of the seismic force-resisting system. Limits on the yield stress and the provisions of Clause 8.3.2(a) ensure adequate post-yield behaviour of the material. Use of other materials would require demonstration that the energy-dissipating elements can sustain the very high post-yield strains needed to achieve the performance assumed in design. Because of the dynamic loading, toughness requirements are specified for buildings with specified short-period spectral acceleration ratios, $I_E\, F_a S_a(0.2)$, greater than 0.55 for thick plates and shapes in energy-dissipating elements, and in welded members anywhere in the seismic force-resisting system. Weld metal in primary connections is also subject to toughness requirements when $I_E\, F_a S_a(0.2)$ is greater than 0.35. Temperatures for Charpy V-notch testing are specified in the Standard.

In S16-14, welds that are expected to sustain high demand under seismic loading are designated as demand-critical welds, and additional notch-toughness requirements are specified for them. The requirements adopted in S16-14 are consistent with those in ANSI/AISC 341-10 (2010) and AWS D1.8 (2009). Weld metals used for demand-critical welds in clad and heated structures, where the service temperatures seldom drop below +10°C, are required to meet a minimum average Charpy V-notch impact test value of 54 J at +20°C. The 10-degree temperature difference between test and service temperatures accounts for the severity in strain rate of the impact test, etc. For structures exposed to lower service temperatures, the point-in-time service temperature is taken to be 10°C above the 2.5% January design temperature specified in the NBCC. The Standard also permits the test temperature to be 10 degrees warmer to account for the strain rate difference, etc. More stringent test conditions must be considered when more critical service temperatures are expected. For example, for a cold storage structure whose service temperature is lower than the above-mentioned point-in-time temperature, the minimum test temperature should be lower, i.e. 10 degrees above its service temperature.

Lamellar tearing represents a brittle and undesirable failure mode. Welded T-joints and corner-joints must be designed and detailed to minimize the probability of this failure mode in accordance with CSA W59.

27.1.6 The requirements for bolted connections ensure that friction plays a role in load transfer and that too rapid a slip into bearing is avoided.

If beam-to-column connections are demonstrated by means of physical testing to meet the various deformation requirements for different categories of moment-resisting frames and eccentrically braced frames, then the requirements of this clause can be waived.

27.1.7 In order to ensure the desired hierarchy of yielding, the relative strengths of dissipating and non-dissipating structural elements must be known. This requires knowledge of the actual, or probable, yield stresses. The specified minimum yield stress must be used when computing the resistance of the non-dissipating elements, whereas the probable yield stress is used in estimating the loads arising from yielding elements. The probable yield stress may be obtained from coupon tests on the same heats of the materials used in the construction or, since the material will not in general be available at the time of design, may be estimated by use of the factor R_y given in this clause. The imposed minimum value of 385 MPa implies a high R_y value for lower-yield steels in common use until quite recently and is due in part to the use of multi-grade material in recent years, and also to the uncertainty of the actual yields achieved in earlier grades.

For W-shapes, similar ratios between expected and nominal yield strengths are observed for the flanges and the web and, hence, the same R_y value can be used for the entire cross section. Surveys by Schmidt and Bartlett (2002) and by Liu *et al.* (2007) showed that HSS exhibit higher characteristic-to-nominal yield strength ratios compared to W-shapes. Furthermore, the ratio for HSS generally increases when the perimeter-to-wall thickness ratio is decreased, i.e. larger ratios for more compact sections such as those required for the energy-dissipating elements. A higher R_y value elevating the product $R_y F_y$ to 460 MPa is therefore specified for HSS in CSA S16. This value corresponds to the mean yield strength value of the data collected by Schmidt and Bartlett (2002). CSA S16 does not provide any requirements for ASTM A53 pipes used as energy-dissipating elements such as bracing members. If this material is used, appropriate R_y values should be considered (see AISC 2010a).

The error in using the minimum specified value rather than the probable value when calculating width-thickness limits is acceptably small. However, a minimum value of F_y is set at 350 MPa for use in this calculation due to the common use of multi-grade steels in recent years.

A reduced value of 300 MPa is permitted to be used to verify the width-to-thickness ratios of angles when the specified yield strength is equal to or less than 300 MPa.

27.1.8 In the computation of second-order effects, a linear amplification is given following the procedure outlined in the Structural Commentaries to the National Building Code of Canada. This method differs from that given in Clause 8.4.2 since the displacements, under which this provision ensures that the prescribed lateral resistance can be developed, result from the anticipated inelastic seismic deformations. Notional loads and P-Δ effects must be considered for the design of the energy-dissipating elements. They need not be considered for the design of the non-dissipating elements (e.g. beams and columns in concentrically braced steel frames) as the lateral load effects on these elements are limited by the capacity of the dissipating elements. In case the dissipating elements are overstrong and the seismic loads corresponding to $R_d R_o =$ 1.3 (or 1.0, as applicable) are used to size the non-dissipating elements, notional loads and P-Δ effects must be included in the analysis.

27.1.9 Regions where large inelastic strains are expected to occur in the SFRS are designated as protected zones. Protected zones include plastic hinging regions in moment frames, links of ductile eccentrically braced frames, braces in concentrically braced steel frames, etc. They are defined in the Clauses applicable to the designated system. Within these zones, discontinuity, rapid change in cross-section or material embrittlement caused by welding, cutting or penetration at the fabrication plant or the construction site may lead to premature fracture under cyclic inelastic response. Hence, unless engineered or part of test assemblies satisfying the specified performance, welded, bolted, screwed or shot-in attachments for perimeter edge angles, exterior facades, partitions, ductwork, piping or other construction shall not be placed within protected zones. For instance, welded shear studs and decking attachments that penetrate the beam flange shall not be placed on the beam flanges within the protected zone, unless approved by the Designer. Decking arc-spot welds required to secure decking are, however, permitted. Fabrication or erection operations that cause discontinuities are also prohibited in protected zones. Discontinuities accidentally created within protected zones, such as tack welds, erection aids, air-arc gouging and thermal cutting shall be repaired as required by the Designer. Guidance on acceptable repair methods can be found in CSA-W59.

The extent of the protected zones must be identified on the design documents. The information can be conveyed to the construction site by means of coating and labels on both faces with large lettering pertaining to the restriction on attachments and penetrations. Where the protected zones are subsequently covered by fire protection material, provision for visible labels after the application of fire protection should be considered.

27.2 Type D (Ductile) Moment-Resisting Frames, R_d = 5.0, R_o = 1.5

27.2.1 General

27.2.1.1 Type D moment-resisting steel frames have traditionally been designed to develop inelastic deformations at beam-to-column joints, either by plastic hinging in the beams or columns, or by inelastic shear deformations in the panel zone of H-shaped columns (bent about the strong axis). However, numerous welded moment frames have suffered connection fractures as a result of the 1994 Northridge and 1995 Kobe earthquakes, calling for a comprehensive review of that design practice. Extensive revisions to Clause 27.2 were introduced in the 2001 edition of S16 based on the research findings and engineering consensus reached following these two earthquakes (FEMA 1995, 1997, 2000).

The current design philosophy requires that plastic hinges develop at predetermined locations within the frame, such as in beams away from the face of the columns. This is possible either by locally strengthening the beams near the columns (by haunches, cover plates or other

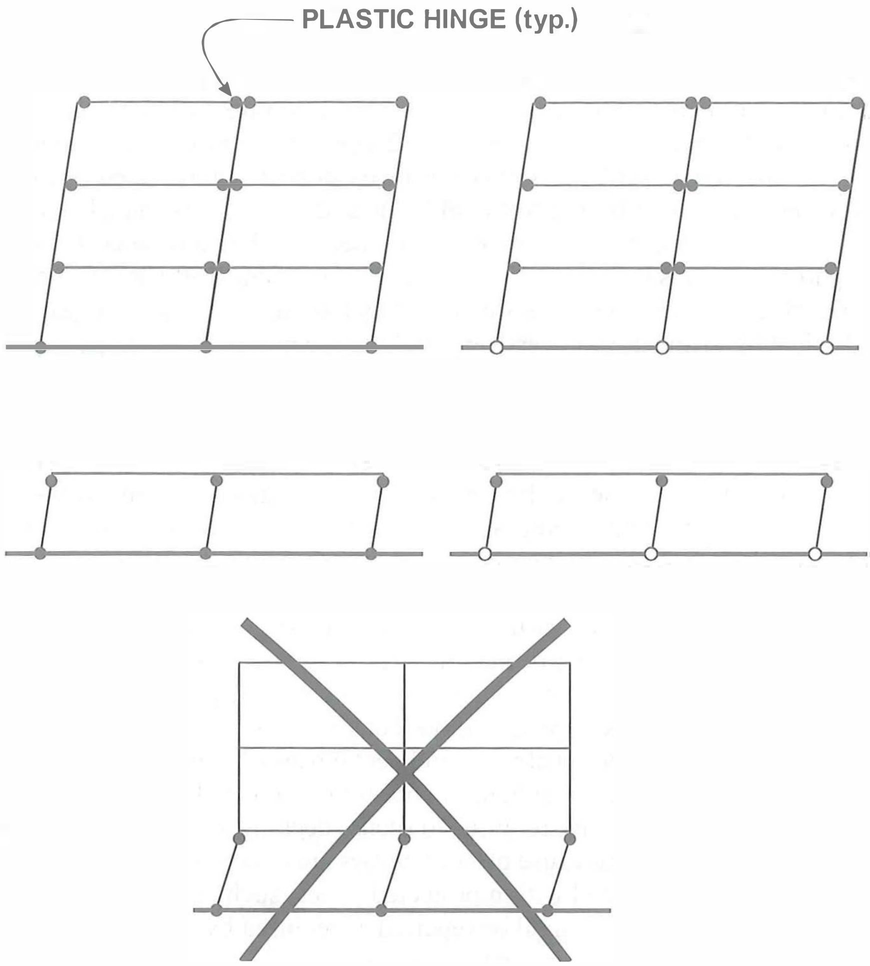


Figure 2-66
Desirable Beam-Sway Collapse Mechanism and Undesirable Column-Sway Mechanism

methods), by locally weakening the beams at selected plastic hinge locations some distance from the columns, or by using special detailing that ensures ductile response. Annex J references documents giving specific details that will achieve the necessary ductility. Other systems are permissible if demonstrated by physical tests to be capable of providing the performance specified in later Clauses.

Whenever column bases are designed to have a flexural resistance, plastic hinges are necessary to permit development of the preferred plastic collapse mechanism (see Figure 2-66 – Desirable beam-sway collapse mechanism, and undesirable column-sway mechanism). In multi-storey applications, column plastic hinging is otherwise undesirable, as it may lead to formation of a storey plastic mechanism with undue ductility demands compared to other storeys. Column plastic hinging is however permitted at the top of columns terminating under a roof beam, as this behaviour is not expected to result in excessive localized damage. This hinging scenario can represent an appropriate solution when deep beams or trusses are used for the roof. Special requirements (see 27.2.3) must be satisfied when column hinging is expected.

Although the panel zone provides excellent ability to absorb energy by means of cyclic plastic shearing deformations (Popov *et al.* 1986), large inelastic deformations there result in large curvatures in the column flanges. For joints in which the beams are welded to the columns, these curvatures may precipitate cracking of the beam weld at that location. Panel zone yielding without considerable concurrent beam yielding is generally not desirable for these connections, and the current provisions limit this behaviour except when using a connection detail for which panel zone yielding has been found appropriate by testing. Note that optimization of panel zone and beam yielding is difficult, given the inherent statistical variability in the steel strength of beams and columns.

27.2.1.3 In evaluating the relative strengths of the structural components at the joint, an estimate should be made of the contribution of the slab. Clause 27.2.8 specifies that studs are not permitted in beam plastic hinge regions. Thus the contribution of the slab can be neglected if specific construction details are provided that prevent the slab bearing on the columns. In the absence of such details, under positive bending moment, the ultimate compressive resistance of the concrete can reach values of $1.3f'_c$.

27.2.2 Beams

In moment frames, beams are nearly always bent in reverse curvature between columns unless one end is pinned. The lateral bracing requirements here assume that the seismic moment at one end of the beam is M_p, and that zero seismic moment exists at the other end; to these the gravity load moments must be added.

Lateral bracing of beams near the plastic hinge location should be provided according to the configuration, strength, and stiffness considered in the tests referenced in the commentary on Clause 27.2.5. Attachments in the area of anticipated plastic behaviour are in general proscribed (see Clause 27.2.8).

27.2.3 Columns (Including Beam-Columns)

27.2.3.1 The width-thickness requirements for columns that develop plastic hinging follow from Clause 27.2.1.2. The axial load in the column is also restricted because the rapid deterioration of beam-column flexural strength (when high axial loads are acting) limits the ductility.

When columns are expected to develop plastic hinging, structural elements adjacent to the column plastic hinges must be able to resist the full plastic moment of the columns. For example, at the base of a column, the intended performance would not be achieved if anchor rods yield instead of the column itself. Due to anchor rod elongation, column base fixity would be lost after a few cycles, resulting in a considerable reduction in base shear resistance and storey stiffness, and the ensuing risk of an undesirable localized storey-collapse mechanism at the first level.

27.2.3.2 Columns may accumulate forces from several yielding elements, and these must be considered.

The equation presented in this clause is intended to minimize plastic hinging in columns and promote plastic hinging of beams. Hence, it does not apply to columns in cases where plastic hinging is expected near the top of the columns. This equation cannot ensure that individual columns will not yield at some time during earthquake response, because of the shifting of column inflection points during dynamic response (Bondy 1996), but the extent of this yielding should not be detrimental. This requirement is in addition to the requirements of Clause 13.8.

For the equation presented to be statically correct, equilibrium requires that the moment at the intersection of the beam and column centrelines should be determined by projecting the sum of the nominal column plastic moment from the top and bottom of the beam moment connection (Figure 2-67 – Free-body diagrams to calculate V_h at the plastic hinge location, and moment at face and centre of column). However, this may be conservative for connections

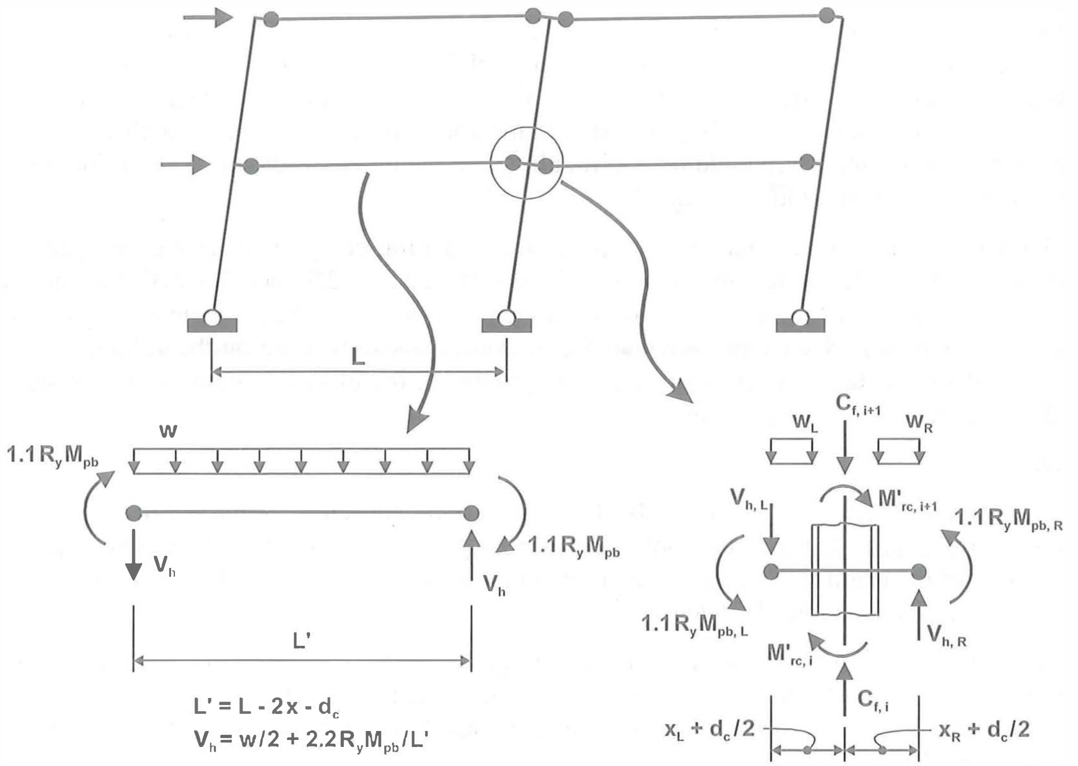


Figure 2-67
Type D Moment-Resisting Frame – Free-Body Diagram

having deep panel zones and/or haunches, and current North American practice permits that the sum of moments at the top and bottom of the panel zone be substituted for the statically correct value.

In addition to this requirement, columns that are expected not to develop plastic hinging must satisfy the requirements of Clause 13.8 under the forces induced by plastic hinging of the beams. In this case, the sum of the beam moments at the centreline of the joints, as defined in Clause 27.2.3.2, can be distributed above and below the joints in the same proportion as the moments obtained from an elastic analysis under the factored seismic loads plus gravity loads. Meeting the requirements of Clause 13.8 may be more critical than Clause 27.2.3.2, particularly for slender columns.

27.2.3.3 The moments in a column when the structure is responding inelastically will not, in general, be known. Conservative estimates of the moment at a splice should be made, based on the possible bending strengths at each end of the column. Because partial-joint-penetration groove welded splices are not ductile under tensile loading (Popov and Steven 1977; Bruneau *et al.*1987), splices are designed more conservatively, and half-penetration welds on flanges are required as a minimum.

27.2.4 Column Joint Panel Zone

27.2.4.1 Shear force demands on joint panel zones are determined from beam forces acting at the column faces and column forces acting at the levels of the top and bottom beam flanges.

27.2.4.2 The column panel zone has a shear strength greater than the von Mises shear yield value on the web due to: (i) considerable strain-hardening in shear, and (ii) flexural resistance of the column flanges during panel yielding in shear (Krawinkler and Popov 1982). This strength is assumed to be attained at a shear distortion equal to four times the yield shear distortion. This amount of panel zone yielding may be tolerable, provided that plastic hinging first develops in the beams.

Yielding in the panel zone is perceived by some as beneficial, since it reduces the inelastic demand on the beams and provides sharing of energy dissipation. However, some concerns remain for beam welded connections because of the impact of plastic shear distortions and localized column flange bending on the integrity of the beam flange welds. A consensus opinion has not yet been reached. An upper limit of 0.2 is therefore placed on the term $3b_c t_c{}^2/d_c d_b w'$ to ensure that the panel zone strength is not reached prior to development of the plastic moment enhanced by strain-hardening in the adjacent beams.

The stronger panel zone option is usually more economical. In this case, the von Mises yield criterion ($0.58F_y$ on the entire web of the column) is adopted. The panel zone remains elastic, and special detailing of the panel zone described in the first part of this clause is not warranted. Note that the 0.55 in the shear strength equations is obtained by taking the depth of column web equal to $0.95d_c$.

27.2.4.3 Requirements are provided to ensure stable response of the panel zone. Doubler plates can be used to achieve the required shear strength for the panel zone. Special detailing requirements must then be satisfied to ensure that the shear capacity of the doubler plates can be mobilized and proper load paths exist between the beams, columns, doubler plates and continuity plates, when present.

27.2.5 Beam-to-Column Joints and Connections

27.2.5.1 Extensive research was initiated following the Northridge earthquake to identify the reasons that led to the numerous observed beam-to-column connection fractures and to formulate new connection design requirements. The result of this large research endeavour is a database of connection types that have been experimentally proven able to provide satisfactory seismic performance, with specific information regarding configurations, details, quality control, and other requirements. Minimum performance criteria under reversed cyclic loading are specified in the Standard. The designer must either:

(a) Use connections designed and detailed in accordance with the CISC Moment Connections for Seismic Applications (CISC 2014), or

(b) Use test results in compliance with this Clause. A protocol for such testing is referenced in Annex J.

The 2014 edition of the CISC guide for moment connections provides design and detailing provisions for four different connections: reduced beam section (RBS), bolted unstiffened end-plate (BUEP), bolted stiffened end-plate (BSEP), and bolted flange plate (BFP) connections. The latter has been added to the previous edition of the guide. Provisions are now also given for built-up column shapes, and the ranges of acceptable beam and column shapes have been adjusted to reflect new available data. Additional information on pre-qualified beam-to-column connections can be found in AISC (2011).

27.2.5.2 The beam web connection shall have a resistance adequate to carry shears induced by yielding at the beam-to-column joint.

27.2.6 Bracing

Bracing of both top and bottom beam flanges as well as column flanges shall be considered. If no transverse beams exist at a level, the column must be designed to provide restraint to yielding beam flanges in the manner indicated in (d).

27.2.7 Fasteners

Consideration should be given to the fact that plastic hinge locations will not be predicted by an elastic analysis of the frame.

27.2.8 Protected Zones

Clause 27.2.8 describes the zones that must be protected in ductile moment-resisting frames. This clause should be applied in conjunction with Clause 27.1.9 where limitations applicable to protected zones are defined. In moment-resisting frames, protected zones include segments along the beams and columns where plastic hinges are expected to occur. Limitations on cross-section changes in beam plastic hinges are also specified.

27.3 Type MD (Moderately Ductile) Moment-Resisting Frames, R_d = 3.5, R_o = 1.5

The ductility-related force modification factor of 3.5 is sufficiently large for Type MD moment-resisting frames to develop large cyclic inelastic deformations during earthquakes. For that reason, and because larger structural members will result from the larger design forces considered, most requirements of Clause 27.2 are applicable. However, beam-to-column joints need only be able to develop a minimum drift angle rotation of 0.03 radians (compared with 0.04 for comparably designed Type D moment-resisting frames). This reduced deformation requirement may be useful when a tested connection fails to reach the 0.04 requirement. A greater advantage of this clause may, however, consist in adopting the relaxed provisions of 27.3(a) and (b) in combination with the higher design load. Practical applications of this system include moment frames in moderate seismicity regions where added frame stiffness is required to satisfy $U_2 \leq 1.4$ (Clause 27.1.8) or wind effects.

27.4 Type LD (Limited-Ductility) Moment-Resisting Frames, R_d = 2.0, R_o = 1.3

This system can accept limited yielding in beams, columns or joints. Panel zone design follows Clause 27.2.4.2, and thus only limited yielding is expected. These frames are subject to restrictions on height and seismic demand level. They are restricted to 60 metres and 30 metres in height for regions of moderate and high seismicity, respectively. In addition, the strong-column/weak-beam design concept applies to buildings with specified short-period spectral acceleration ratios ($I_E F_a S_a(0.2)$) greater than 0.55 and buildings taller than 60 metres (permitted in low seismicity regions only). However, probable plastic beam moments without strain hardening effects ($R_y M_{pb}$) may be considered in the strong-column design, because of limited beam yielding in this system as compared to that in the more ductile categories. To accommodate yielding, sections must be Class 2 or better.

It is anticipated that in many cases joint details will conform to traditional forms of construction used for moment-resisting frames. Clause 27.4.4.2 provides design and detailing requirements for joints with beams welded directly to the flanges of I-shaped columns. Connections that either can accommodate an interstorey drift angle of 0.02 radians, following tests as discussed in Annex J, or are in compliance with CISC (2014) may also be used.

Practical applications of this system include moment-resisting frames in moderate seismicity regions where added frame stiffness is required to satisfy drift limits, $U_2 \leq 1.4$ (Clause 27.1.8) or wind effects, and certain low-rise buildings in higher seismicity areas.

27.5 Type MD (Moderately Ductile) Concentrically Braced Frames, $R_d = 3.0$, $R_o = 1.3$

27.5.1 General

Type MD concentrically braced frames are designed to dissipate energy essentially by yielding of the bracing members. Energy dissipation occurs under brace elongation, inelastic buckling of the braces, and inelastic bending when the braces are subsequently straightened. In low-rise V-brace or chevron-brace frames, energy can also be dissipated through limited bending of the beams at the brace intersection point.

27.5.2 Bracing Systems

27.5.2.1 General

Three bracing configurations are explicitly provided for in the braced frame category, and a maximum building height is specified for each. Multi-storey concentrically braced frames have limited capability of distributing vertically the inelastic demand after buckling and yielding of the braces have developed at a given level. Lateral overstrength resulting from the inherent difference in capacity between tension and compression braces acting in pairs serves to prevent the concentration of inelastic demand (Lacerte and Tremblay 2006). The continuity of the columns which, when provided as specified in Clause 27.5.5.2, provides sufficient reserve strength and stiffness, also helps mitigate the formation of a weak storey response and dynamic instability under severe earthquakes (MacRae *et al.* 2004, Chen *et al.* 2008).

The tendency to instability is more pronounced in tall frames in which the inelastic demand tends to concentrate in the bottom floors, which are the first affected by the ground motion, or in the upper levels due to higher mode effects. Thus, a maximum height is specified for each of the three concentric bracing configurations explicitly provided for in Clauses 27.5 and 27.6.

The provisions of Clauses 27.5 and 27.6 are based on the results of frame behavioural studies using inelastic time-history analysis (Tremblay 2000, Tremblay and Robert 2001, Marino and Nakashima 2006). The buildings studied were regular in form with uniform storey height varying between 3.5 and 4 m. Frames with heights up to 80% of the height limits as specified in NBCC can be expected to perform satisfactorily with no further inelastic analysis needed. Those within the height range of 80% to 100% of the NBCC limits are required to be designed for additional seismic forces as stipulated in Clauses 27.5.2 and 27.6.2 for each respective braced frame configuration. The additional seismic forces need not be considered for determining deflections. Taller buildings, notably those with significantly greater storey heights or other systems (e.g. bracing combined with moment-resisting beam-to-column connections), may require further study, and such systems can be investigated using inelastic time-history analysis. Alternatively, it would be necessary to demonstrate that each storey possesses a reserve of strength and stiffness at the drifts expected under the inelastic response, to prevent a concentration of inelastic actions.

Judgement must also be exercised when the geometry of the frame deviates significantly from the uniform configuration considered in the referenced studies. For instance, industrial buildings or hangars in which the bracing system in any one level includes a stack of two or more bracing panels may be prone to concentration of the inelastic demand in a few bracing members. Such configuration is only permitted for Type LD braced steel frames, and special requirements apply, as described in Clause 27.6.6.

Knee bracing and K-bracing are excluded from the Type MD braced frame category, because plastic hinging that will develop within the clear length of the columns may lead to their instability. Braced frames consisting of more than one X-bracing panel are permitted for Type LD braced frames, as described in the subsequent section.

27.5.2.2 Proportioning

In order to achieve symmetric inelastic response, the storey shear resistance in opposite directions should remain equal or nearly the same under the design earthquake. Because the capacity of a concentrically braced frame after buckling of the braces is mainly governed by its tension braces, the requirement is based on the storey shear resistance provided by the tension-acting braces in each direction. In order to avoid excessive torsional response in the inelastic response, this requirement must be met in each vertical plane of braces and in both orthogonal directions.

27.5.2.3 Tension-Compression Bracing

In tension-compression bracing systems, braces in each vertical plane are designed to resist their share of factored tensile and compressive forces based on the analysis. These braces typically act in pairs as is the case of single-storey X-bracing, two-storey (split) X-bracing, chevron bracing, or V-bracing configuration. Tension-compression bracing also includes configurations consisting of an odd number of braces, provided that they satisfy Clause 27.5.2.2 in every plane of bracing at every level. Compared with the tension-only system, the stockier braces in this system provide greater post-buckling capacity and stiffness. This, combined with the stiffness provided by continuous columns, has been shown to provide stability in frames up to about 32 metres in height (Tremblay 2000, Tremblay and Poncet 2007, Izvernari *et al.* 2007). Therefore, Moderately Ductile tension-compression frames that are within 40 metres in height, as permitted in NBCC, but exceed 32 metres, should be designed for higher forces as required in this clause.

27.5.2.4 Chevron Bracing

The commentary to Clause 27.5.2.3 also applies to this Clause. Chevron bracing, in which the braces (which may be either both above the beam or both below it) meet within the central region of the beam, is permitted in the Type MD concentrically braced frame category, provided that the beams in the bracing bents remain essentially elastic after buckling of the bracing members has occurred. Braces in frames with such strong beams can develop their full yield capacity in tension, and the structure exhibits a more stable hysteretic response than when weaker beams are employed. Frames with weaker beams typically experience rapid and significant deterioration of their storey shear resistance and stiffness after buckling of the braces (Remennikov and Walpole 1998a; Tremblay and Robert 2000, 2001). When the tension brace yields in tension, the compression brace at the same level only develops its post-buckling resistance, C'_u, as defined in Clause 27.5.3.4. This case is illustrated in Figure 2-69. When braces are connected to the beam from above, the expected brace compression resistance of the brace, C_u, must also be considered; this condition may be more critical when there is an extremely high gravity load and the beam plastic bending produces a downward displacement at the plastic hinge. For both cases, the beams must be checked as beam-columns resisting the bending moments and axial forces due to gravity loading and these brace loads without the vertical support provided by the braces. Beam-to-column connections must be sized for the same loading conditions.

Limited yielding in the beams does not adversely affect the response of low-rise chevron braced frames, and the brace tension load to be used in the design of the beams in frames up to 4 storeys has been reduced for such frames (Tremblay and Robert 2000, 2001). In such a case, plastic hinging will likely develop in the beams, and the beam connections should then be designed for shear forces associated with the probable bending resistance of the beams.

In both designs, the beams must be adequately laterally restrained at the brace connection point to resist out-of-plane components of the axial load acting in the beams and the braces.

27.5.2.5 Tension-Only Bracing

Designing the braces to resist, in tension, 100% of the lateral loads acting in each direction can lead to a more economical design when lateral loads are low or moderate, or when long braces are used. Tension-only bracing is not permitted in V-or chevron bracing. Although the contribution of these braces when acting in compression is ignored in resisting design lateral loads, the braces must meet the slenderness limit and detailing requirements in Clause 27.5.3, and the compression loads they can deliver must be accounted for in the design of connections, beams, and columns (see Clauses 27.5.4 and 27.5.5). Because the braces are generally less stocky as compared to tension-compression braces, this system exhibits less energy dissipation capacity, and larger inelastic deformations are therefore expected. Every column in the building is required to be fully continuous in order to resist in bending the concentration of inelastic demand in a single storey. It has been shown that frames up to about 16 metres in height perform satisfactorily (Tremblay 2000). Moderately Ductile tension-only braced frames that are within 20 metres in height, as permitted in NBCC, but exceed 16 metres, should be designed for higher forces as required in this clause. However, other bracing systems may prove to be more economical for frames taller than 3 storeys in height, because erection safety usually dictates field splices for column tiers spanning more than 3 storeys.

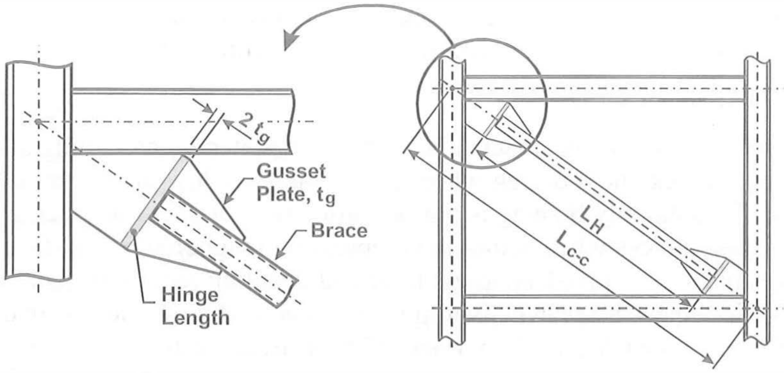
27.5.3 Diagonal Bracing Members

In X-bracing, one of the braces is usually built from two segments and inter-connected at the brace intersection. By selecting both brace segments from the same heat of steel, concentration of yielding in the weaker segment and the potential for premature brace fracture can be avoided.

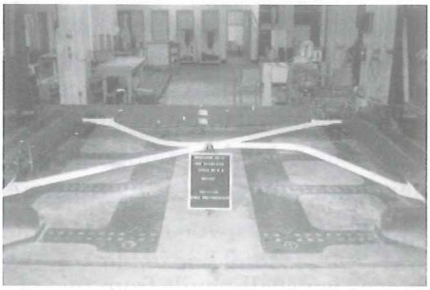
In most cases, including tension-only systems, the post-buckling capacity of braces is necessary to ensure stability, and therefore, in all these systems, the slenderness limits specified in this clause apply to braces in all Type MD concentrically braced frames, including tension-only systems.

27.5.3.1 The energy dissipation capacity of bracing members under cyclic inelastic loading increases when the effective slenderness ratio, KL/r, is decreased (Jain *et al.* 1980, Popov and Black 1980, Tremblay *et al.* 2003, Lee and Bruneau 2005), and maximum brace slenderness has traditionally been specified to control the dynamic response of braced frames. Bracing systems with slender braces designed to act both in tension and compression have, however, significant lateral overstrength due to the difference that exists between the compressive and tensile capacities of the braces. This overstrength permits the maintenance of a stable inelastic response under severe earthquakes, and for this reason it is possible to allow a brace slenderness limit of 200 for Type MD frames. This limit still provides a minimum energy dissipation capacity that allows the use of tension-only braces in low-rise structures. Past test programs showed that rectangular and circular HSS bracing members with low slenderness ratios can develop premature fracture at the plastic hinge region (Fell *et al* 2009, Tang and Goel 1989, Tremblay *et al.* 2002, Tremblay *et al.* 2008); a minimum effective slenderness ratio is specified to preclude this undesirable failure mode.

When determining the brace slenderness, the actual support conditions of the braces must be accounted for in determining KL. As discussed later (see Clause 27.5.4.3), the brace end connection detail with a single gusset plate and free hinge zone in the gusset, shown in Figure 2-68(a), has gained wide acceptance in practice. When using this detail, the brace effective length KL for out-of-plane buckling can be taken equal to the length between the hinge locations, L_H. Tests on double-angle braces using that detail have shown that a K factor of 0.5 can be



(a) Single Brace

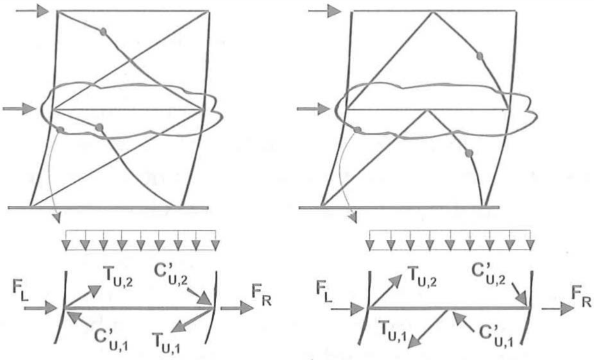


(b) X-bracing

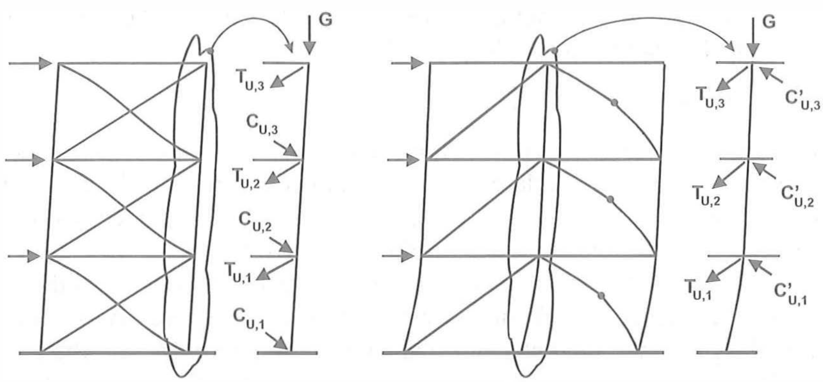
Figure 2-68
Out-of-Plane Buckling of a Brace with Gusset Plates Detailed to Accommodate End Inelastic Rotation

applied to evaluate the brace slenderness for in-plane buckling (Astaneh-Asl and Goel 1984). For X-bracing, when the brace end connections are detailed with single vertical gussets, K can be taken equal to 0.4 and 0.5 for in-plane and out-of-plane buckling, respectively, with L taken as the length between the anticipated plastic hinge locations at the ends of the bracing members (El-Tayem and Goel 1986, Sabelli and Hohbach 1999, Tremblay *et al.* 2003). Caution must be exercised when one of the braces is interrupted at the brace connection point of X-bracing, as this can reduce the stiffness of the tension brace supporting the compression brace and/or lead to local instability of the connecting elements (Kim and Goel 1996, Davaran 2001, Doravan and Hoveidae 2009). These effects can be minimized by reducing the length of the connection or by ensuring minimum continuity at the brace intersection. Additional information on brace effective length can be found in Ziemian (2010).

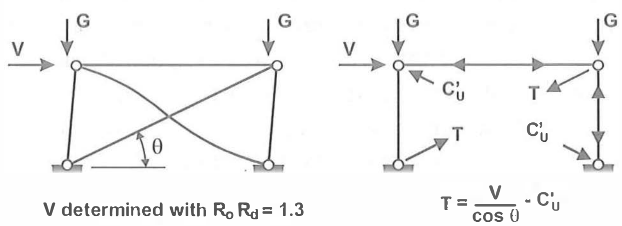
27.5.3.2 Several cycles of inelastic bending are anticipated at hinge location(s) along the bracing members, and limits are imposed on the width-to-thickness ratios of the braces to prevent premature fracture of these members. Physical testing has shown that HSS bracing members exhibit limited fracture life, and relatively more stringent limits are specified for these sections (Fell *et al.* 1989, Lee and Goel 1987, Liu 1987, Sherman 1996, Tang and Goel 1989). Relaxation of width-to-thickness limits is permitted when lower inelastic demand is expected in the braces, such as when slender bracing members are used (buckling becomes essentially elastic) or when the structure is located in a region of low seismicity (Tremblay 2001). The inelastic demand is also less critical in the vertical legs of double-angle bracing members buckling about their plane of symmetry, and less stringent requirements are specified for this case.



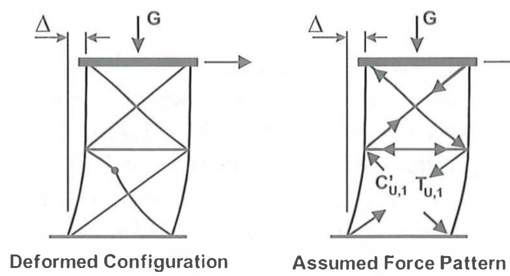
(a) Beams in X-Bracing and Chevron Bracing



(b) Exterior and Interior Columns



(c) Tension Brace Connection – Beam and Column Forces at $R_d R_o$ = 1.3



(d) Columns and Struts When Braces Meet Columns Between Floors

Figure 2-69
Brace Axial Loads for the Design of Members and Connections

27.5.3.3 Buckling of the individual elements of built-up bracing members under earthquake loading may result in high localized inelastic deformations which can lead to premature fracture of the braces (Aslani and Goel 1991). Individual buckling is therefore precluded by limiting the slenderness of the individual components. When buckling of the braces induces shear in the stitch fasteners, these fasteners are expected to transfer in shear the full yield capacity of the smaller brace component upon subsequent straightening of the braces, and the stitch connections must be designed accordingly (Astaneh-Asl and Goel 1985).

Braces with bolt holes at the location of the plastic hinges have exhibited early fracture at the net section, and bolted stitches must be avoided in these regions (Astaneh-Asl and Goel 1984). In determining the governing overall slenderness of the bracing members and the location of plastic hinges, attention must be paid to the actual end fixity and support conditions of the bracing members (see also Clause 27.5.3.1). Plastic hinges in the bracing members will develop approximately at half the distance between supports, i.e., at one quarter and three quarters of the brace length in X-bracing, as well as near the brace end connections if such connections do not permit rotation to develop upon buckling.

27.5.3.4 Probable Brace Resistances

In previous editions of CSA S16, brace expected strength values to be used in capacity design were specified in Clauses related to brace connection design. Recognizing that brace capacities are also used for the design of beams, columns, and connections other than brace connections, etc., a separate clause was introduced in S16-09 to define clearly the expected strength values of braces in tension and compression. A realistic estimate of the expected compressive strength of a brace, C_u, is obtained by multiplying its compressive resistance by 1.2. In this calculation, the probable yield stress of the steel should be used, and the resistance factor does not apply. In tension, the maximum anticipated brace force T_u corresponds to the probable yield tensile strength. The compressive resistance of a brace reduces when the brace is subjected to cyclic inelastic axial loading (Lee and Bruneau 2005), and this post-buckling brace compression resistance can lead to more critical loading conditions for members or connections, such as beams of chevron bracing or interior columns. Figure 2-69(a) shows examples where compression-acting braces in the buckled state (C'_u) produce maximum axial compression in the beam of an X-bracing and maximum bending moment in the beam of a chevron bracing. For the frame in Figure 2-69(b), the exterior columns should be designed for the condition at the brace's probable compressive resistance (C_u), whereas the buckled brace condition (C'_u) should be assumed for the interior. In S16, C'_u is taken as $0.2\,A\,R_y\,F_y$, which corresponds to the value observed in tests at a ductility of 3.0. Tests suggest that higher values can be used for bracing members with very low slenderness, i.e. with λ less than 0.4 (Remennikov and Walpole 1998b, Tremblay *et al.* 2002).

In some cases, braces can be oversized to meet other design criteria such as drift, width-to-thickness ratio, or slenderness limits. For such cases, the brace loads need not exceed the forces induced by a storey shear calculated with $R_d R_o = 1.3$, as specified in Clause 27.1.2. The possibility of brace buckling under that storey shear must be considered in the calculations; the forces in the compression braces are then limited to the probable buckling or post-buckling strength, whichever is more critical, and the load redistribution from the compression braces to the tension braces due to brace buckling must be accounted for when evaluating the forces acting in the tension braces. This is illustrated in Figure 2-69(c) where maximum tension in the tension brace, maximum compression in the beam, and maximum compression in the right-hand-side columns are obtained when the compression brace carries a load C'_u.

27.5.4 Brace Connections

27.5.4.1 Eccentricities in brace connections can lead to damage under cyclic loading and should therefore be kept to a minimum in ductile braced frames.

27.5.4.2 Brace connections must be designed to resist brace axial loads that correspond to the probable buckling strength and tensile yielding strength of the braces. Actual brace end restraint conditions and the presence of intermediate supports must also be taken into account when evaluating the buckling strength of the braces (see Clause 27.5.3.1).

In view of the uncertainty associated with the amplitude of the seismic ground motions and their effects on building structures, connections designed for the upper brace force limit corresponding to $R_d R_o = 1.3$ must be detailed for a ductile mode of behaviour. Details that may be considered to achieve ductile failure modes include gusset plates proportioned for ductility (Cheng and Grondin, 1999), connections that rely on yielding of elements or in which bearing failure of bolts governs (Tremblay *et al.* 2009) in preference to net section fracture or bolt shear failure. Otherwise, the limit on seismic loads must be increased to loads corresponding to $R_d R_o = 1.0$.

The brace tension load can be limited by beam yielding in chevron bracing in which the beams are not designed to carry the full tensile yield load of the braces. In such a case, the brace tension connection load at any level is determined assuming the beam yields while the compression brace still carries 1.2 times its probable nominal compressive strength.

The net section resistance of braces may be based on the probable tensile strength of the brace material, since the load level corresponds to the probable yield stress of the brace. Furthermore, since the principal geometrical parameter of the net and gross sections is identical, the resistance factor may be taken as 1.0. Based on coupon test data assembled by Schmidt (2000), this can be achieved by multiplying the factored net section resistance of the brace by R_y/ϕ, with R_y not exceeding 1.2 for HSS and 1.1 for other shapes. This factor cannot be applied to the factored resistance of other components of the connections such as net section reinforcement plates, gusset plates, bolts, or welds. Information on net section reinforcement for slotted HSS members can be found in Yang and Mahin (2005), and Haddad and Tremblay (2006). Alternative solutions have recently been proposed for HSS brace connections including the modified hidden gap connection by Martinez-Saucedo *et al.* (2008) and structural cast connectors (de Oliveira *et al.* 2008).

27.5.4.3 Buckling of the braces will induce a rotational demand at the brace ends, and the connections must be detailed to avoid any premature fracture at this location. Proper detailing must be provided to allow this rotation to develop in the brace connections or through controlled plastic hinging in the bracing members away from the connections. Note that this ductile rotational behaviour must be allowed for, either in or out of the plane of the frame, depending on the governing effective brace slenderness. If a single gusset plate connection is used, the latter case can be achieved by leaving a clear distance equal to two times the thickness of the gusset at the end of the bracing member (or the connecting elements), as illustrated in Figure 2-68, in order to allow the formation of a hinge in the gusset plate along a line perpendicular to the brace member's longitudinal axis (Astaneh-Asl and Goel 1985). Tearing of the gusset plate will rapidly develop if this geometry is not carefully met. If a plastic hinge is to develop in the bracing member, the connection must have a factored flexural resistance about the anticipated buckling axis equal to $1.1R_y M_p$ of the bracing member. The Commentary to Clause 27.5.4.2 concerning the factor R_y/ϕ applies also here, except that R_y is not limited to 1.1 when both load and resistance are directly related to the yield stress.

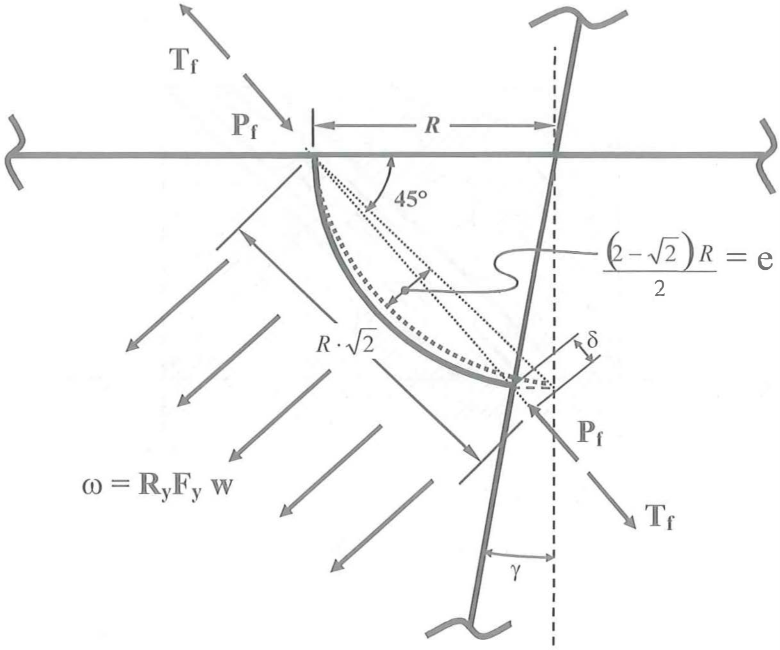


Figure 2-79
Infill Plate with Reinforced Cut-Out Corner –
Arch End Reactions Due to Frame Deformations
and Tension Field Forces on the Arches

where the radius R and the distance e are defined in Figure 2-79. For thrusting action due to frame deformation, the arch must resist the combined effect of a factored axial load $P_{f'}$ (P_{frame} in Figure 2-78(b)) and a bending moment $M_f = P_{f'}e$, where $P_{f'}$ is given by:

$$P_{f'} = \frac{15EI_y}{16e^2}\frac{\Delta}{h_s}$$

In the expression for $P_{f'}$, I_y is the moment of inertia of the reinforcement, Δ is the design storey drift, and h_s is the storey height. It is noted that the arch plate width is irrelevant in that calculation, and it is instead conservatively obtained by considering the strength required to resist the axial component of force in the arch due to the panel forces at the closing corner. The design for the two loading cases, i.e., $T_{f'}$, and $P_{f'}$ and $M_{f'}$, can be done independently, because the components of arch forces due to tension field action ($T_{f'}$) forces are opposing those due to frame corner opening (P_f) (see Figures 2-78(b) and 2-79). Beams and columns must resist the tension and compression forces acting at the ends of the arching reinforcement. Further details are given in Vian *et al.* (2009) and Purba and Bruneau (2007, 2009).

27.9.3 Beams

Beams are expected to develop plastic hinges at their ends. Past tests have shown that lateral resistance and energy dissipation capacity under cyclic loading is essentially supplied by moment-resisting frame response once the infill plate has been stretched in larger cycles. To achieve minimum frame response, CSA S16 requires the boundary moment-resisting frame to be designed for a factored storey shear resistance, $V_{r,\,MRF}$ = 25% of the design seismic storey shear. This factored resistance is taken as $V_{r,MRF} = 2\,M_{rb}/h_s$, where M_{rb} is the beam factored resistance in bending in the absence of axial loads, and h_s is the storey height (Berman and Bruneau 2003, Qu and Bruneau 2009).

Beams must also be designed to resist the combined effects of axial loads, shear forces, and bending moments due to gravity loads together with infill plate yielding in tension and plastic hinging at the ends of the beams, as depicted in Figure 2-77. Beam axial loads are due to the horizontal components of the plate yield loads acting both along the beams and the columns (Berman and Bruneau 2008b). When calculating the plastic hinge resistance of the beams, the axial load effects should be taken into account. Beam and column forces can be determined by manual calculations. Alternatively, a static incremental (push-over) analysis can be performed using the infill plate strip model by Thorburn *et al.* (1983) with inelastic response assigned to the strips, beams, and columns (Berman and Bruneau 2003, 2008b).

In Clause 27.9.2, the shear resistance of the infill plates is based on the assumption that full tension field response can eventually develop in the plates. For this to occur, the horizontal boundary members at the base and top of plate walls should meet the minimum flexural stiffness requirement, as specified in Clause 20.5.2. Alternatively, the plate panel at the wall base can be attached to a steel member embedded in the foundations. For long walls, Sabelli and Bruneau (2007) suggest that vertical struts could be added at the center of the wall to provide a vertical support to the top beam.

27.9.4 Columns

Columns must be designed to remain essentially elastic once yielding develops in the infill plates and beams. Axial loads, shear forces, and bending moments arising from yielding of the infill plate and beams, as illustrated in Figure 2-77, must therefore be added to the effects of gravity loads on the columns (Sabelli and Bruneau 2007, Berman and Bruneau 2008b). Plastic hinges in columns are permitted only at the column bases.

Bending moments and shear forces induced by infill plate forces can be significant. Column shear yielding must be considered. Li *et al.* (2009) proposed and verified through testing the use of struts between floors to reduce shear and bending moment demands on columns. Composite columns inherently possess high axial and flexural strength and stiffness, and can therefore represent an effective design solution (Astaneh-Asl 2001, Deng *et al.* 2008).

Columns must satisfy the minimum flexural stiffness requirement of Clause 20.5.1, to ensure adequate infill plate tension field response.

When plastic hinging is expected at the column bases, the columns must be detailed so that plastic rotation develops above the base plate or the foundation beam. Premature local buckling in the plastic hinge region must also be prevented (Driver *et al.* 1997).

27.9.5 Minimum Stiffness for Beams and Columns

Minimum stiffness is required to develop uniform yielding of the infill plate.

27.9.6 Column Joint Panel Zones

Joint panel zones in plate walls must satisfy the design and detailing requirements specified for panel zones used in ductile moment-resisting frames.

27.9.7 Beam-to-Column Joints and Connections

Beam-to-column connections must be designed to resist forces anticipated in the infill plates, beams, and columns. Plate walls inherently possess high lateral stiffness, and the anticipated storey drifts are less than anticipated in ductile moment-resisting frames; this is reflected in the limited rotation capacity requirement for the beam-to-column joint (0.02 rad). Maximum moments imposed by beam hinging must however be taken equal to $1.1 R_y$ times the plastic moment of the beam. Effects of axial loads acting in the beams may be accounted for when determining the moments imposed by beam hinging.

Reduced beam section (RBS) beam-to-column connections have been used in past cyclic test programs (e.g. Vian *et al.* 2009, Qu *et al.* 2008). Well proportioned RBS connections exhibit good plastic rotation capacity and help to minimize shear forces in beams, and flexural and axial load demands on columns.

27.9.8 Protected Zones

Components of the plate walls that are expected to develop large inelastic deformations, such as infill plates, hinges in beams and columns, and their connections, are designated as protected zones and must satisfy the requirements of Clause 27.1.9.

27.10 Type LD (Limited-Ductility) Plate Walls, R_d = 2.0, R_o = 1.5

For plate walls with limited ductility, seismic energy input is expected to be dissipated primarily by yielding of the infill plate panels. Rigid frame connections are not necessary. However, capacity design requirements for beams, columns, and connections apply.

Commencing in the 2014 edition of the Standard, all requirements for Type LD plate walls have been incorporated in Clause 27.10 to form a stand-alone set of provisions for user friendliness and clarity.

Type LD plate walls are expected to sustain lower inelastic deformation demands compared to Type D plate walls, and several relaxations are permitted. For the beams, Class 1 and Class 2 sections are permitted, and lateral bracing requirements are reduced. When rigid beam-to-column connections are used, the moment demand imposed by beam plastic hinging used for the design of the columns and beam-to-column joints can be based on $R_y M_{pb}$ rather than $1.1 R_y M_{pb}$. Design forces for column splices in structures located in low and moderate seismic regions need not satisfy the requirement of Clause 27.1.4.

27.11 Conventional Construction, R_d = 1.5, R_o = 1.3

In its 2001 edition, the standard introduced provisions for structures of Conventional Construction. The provisions were considered necessary because it was recognized that Conventional Construction would be used for many low-rise structures subjected to considerable seismic hazard, and that most steel structure failures in seismic events are associated with brittle connection details. Provisions related to connections and diaphragms were introduced to prevent brittle failure either by providing ductile connection details, or increasing the design loads. These provisions still apply for seismic-force-resisting systems with specified short-period spectral acceleration ratios ($I_E F_a S_a(0.2)$) greater than 0.45.

Connections of primary framing members forming the seismic-force-resisting system are typically beam-to-column connections in the moment-resisting frame or braced frame, including member splices subjected to seismic forces in tension or shear, or both, and connections to the foundations. In braced frames, they also include brace-to-beam, brace-to-column, and brace-to-brace connections. Beams acting as collectors, chords, and struts in diaphragms are also primary framing members.

Connections that may be considered ductile if appropriately proportioned include extended-end-plate moment connections, flange-plate moment connections, gusset plates proportioned for ductility (Cheng and Grondin 1999), and bolted connections in which the governing failure mode corresponds to bolt bearing failure. Tests (Tremblay *et al.* 2009) showed that welded connections comprising fillet welds may not possess sufficient ductility to prevent fracture, regardless of load direction. They should also be designed for the amplified loads.

The failure of steel deck diaphragms is typically controlled by failure of the connections between the individual deck sheets and between the deck sheets and the supporting structure. Diaphragms designed and constructed using connections that have been shown by testing to be ductile can be designed using the factored forces calculated for Conventional Construction, while those diaphragms with connections that have not been shown to be ductile should be designed using forces calculated using $R_d R_o = 1.3$. Button-punched side lap connections or arc-spot welded connections commonly used for steel decks have not shown adequate ductile behaviour under cyclic loading. Research investigation into diaphragm designs for more ductile response is underway. Test results reported by Essa *et al.* (2003), Tremblay *et al.* (2004) and Hilti (2007) suggest that diaphragms made of thin steel deck sheets (0.76 mm and 0.91 mm) with power-actuated frame fasteners and screwed sidelaps can accommodate some inelastic deformations through screw tilting and bearing, and tearing of the steel deck sheets at frame fasteners. Welded connections with washers, when properly fabricated, can also sustain inelastic deformation demand (Peuler *et al.* 2002), although this approach is generally less appealing from a practical standpoint.

Cantilever column structures composed of single or multiple beam-columns fixed at the base and pin-connected or free at their upper ends can be designated as Conventional Construction, provided that they are proportioned to satisfy the specific requirements in this clause.

In NBCC 2005, the use of Conventional Construction for steel buildings subject to moderate and high seismicities was restricted to buildings not exceeding 15 metres in height. This restriction was intended to retain the traditional 3-storey height limit stipulated in previous editions of the NBCC. In NBCC 2010, the height limit for steel seismic force-resisting systems of the Conventional Construction category was extended to 60 m in moderate seismic regions and 40 m when subjected to higher seismicities. This relaxation applies to all building occupancies except assembly occupancy. Structures such as stadia, large exhibition halls, arenas, convention centres, and other similar structures must comply with the 15 m height restrictions.

Conventional Construction was permitted for certain buildings in the User's Guide to NBCC 2005, and special requirements and height restrictions were introduced in S16-09 to ensure proper response and to prevent premature failure and non-ductile behaviour for these taller structures. The additional requirements are maintained in CSA S16-14. Amplified design seismic loads are specified to compensate for the greater uncertainty in the prediction of the force demand in taller structures. Response spectrum or time-history dynamic analysis must be used to determine forces and deformations. Minimum ductility requirements for steel material and notch-toughness for thick plates, heavy shapes, and weld metal apply to these structures, and more stringent cross-section stockiness requirements are prescribed to delay local buckling. Amplified design forces are specified for columns, in view of the consequences of column buckling. Higher design loads for columns should encourage yielding in adjacent members such as beams, braces, etc. A special requirement is given to prevent overloading of columns that serve as part of two or more systems intersecting in plan. To avoid premature connection failure, a member's end connections should resist the lesser of its gross cross-sectional probable capacity and the amplified connection design forces given in this Clause. In addition, unless yielding is expected in the adjoining members, connections must also be designed and detailed for a minimum inelastic deformation capacity. This could be achieved through plate yielding or bolt bearing. Higher seismic design loads are also specified for diaphragms so that they remain essentially elastic and can maintain their capacity to distribute seismic forces among the vertical elements of the seismic force-resisting system. Lastly, a minimum out-of-plane force is specified at unbraced member intersections to prevent excessive out-of-plane deformations and/or instability.

27.12 Special Seismic Construction

Many different types of alternative structural systems have been developed to dissipate seismic energy in a ductile and stable manner. One such system, the Special Truss Moment Frames (Goel and Itani 1994, Goel *et al.* 1998), can sustain significant inelastic deformations within a specially designed and detailed segment of the truss. The AISC Seismic Provisions (AISC 2010a) provide design and detailing guidance for this system. Design provisions for seismically isolated structures are available (BSSC 2003). In these cases the provisions could be modified as appropriate to provide a level safety and seismic performance comparable to that implied by the S16 requirements.

28. SHOP AND FIELD FABRICATION AND COATING

This clause and the clauses on erection and inspection serve to show that design cannot be considered in isolation but is part of the design and construction sequence. The resistance factors used in this Standard and the methods of analysis are related to tolerances and good practices in fabrication, erection, and inspection procedures.

CISC Quality Certification for plant fabrication of structural steel is an option for project teams and owners requiring a proven level of fabrication quality and control over processes. This quality management system written specifically for the Canadian structural steel industry is third-party audited by independent auditors. CISC Quality Certification is globally recognized and available to steel fabricators within and outside of Canada. A list of CISC Certified companies is available at *www.cisc-icca.ca*.

28.1 Cambering, Curving, and Straightening

CSA Standard W59 specifies that the temperature of the heated areas shall not exceed 650°C in general and not more than 590°C for QT plate.

28.3 Sheared or Thermally Cut Edge Finish

28.3.2 The use of sheared edges is restricted because the micro-cracking induced may reduce the ductility.

28.4 Fastener Holes

28.4.1 The thickness of 700Q steels that can be punched is restricted because of the excessive damage that occurs at the edge of the hole. The maximum plate thickness for thermally cut holes, as allowed in Clause 28.4.3, is dependent upon the thermal cutting process and equipment used.

28.4.2 The restriction of this clause is similar to that of Clause 28.3.2.

28.4.3 Thermally cut holes are allowed for static load applications when subject to the restrictions of this Clause. Iwankiw and Schlafly (1982) found no significant difference in the connection strength of double lap joints with holes made by punching, drilling, and flame cutting.

28.5 Joints in Contact Bearing

Milling techniques will realistically result in some measurable deviation. Tests by Popov and Stephen (1977a) on columns with intentionally introduced gaps at milled splice joints indicated that the compressive resistance of spliced columns is similar to that of unspliced columns. Local yielding reduces the gap. While in these tests column splice gaps of 1.6 mm were left unshimmed, the Standard is more restrictive and defines full contact as a separation not exceeding 0.5 mm. Because shims will be subjected to either biaxial or triaxial stress fields, mild steel shims may be used regardless of the grade of the main material.

28.6 Member Tolerances

The resistance factors given in this Standard, particularly for compression members, are consistent with the distribution of out-of-straightness of members produced to the straightness tolerances given here (Kennedy and Gad Aly 1980, Chernenko and Kennedy 1991).

28.7 Cleaning, Surface Preparation, and Shop Coating

Throughout this section, the word "painting" has been replaced by "coating" to accommodate coating systems other than paint.

There are five instances where steelwork need not be or should not be coated:

- steelwork concealed by an interior building finish or in a limited corrosive environment;
- steelwork encased in concrete;
- faying surfaces of slip-critical joints, except as permitted by Clause 23;
- surfaces finished to bear unless otherwise specified;
- steelwork where any coating could be detrimental to achieving a sound weldment; and,
- surfaces in an enclosed space entirely sealed off from an external source of oxygen.

Specific requirements are provided in Clause 28.7.4.3 for a limited number of applications where welding over coating is permitted.

28.7.5 Metallic Zinc Coatings

These represent coatings other than paint and include hot-dip galvanizing and zinc metallized coatings, both of which are to comply with the relevant CSA Standards.

29. ERECTION

29.3 Erection Tolerances

This entire clause provides helpful definitions of tolerances for the location of the ends of members with respect to their theoretical locations. Tolerances are given for column base plates and for the alignment and elevations of horizontal or sloping members. For column splice tolerances, also see the Commentary on Clause 28.5.

Clauses 29.3.4, 29.3.5, 29.3.6, and 29.3.8 are written in a parallel manner, in that the offset of one end relative to the other, or the elevation of one end relative to the other, both with respect to their theoretical locations shown on the drawings (e.g. the member is not plumb or not level), is expressed as a function of the length but with upper and lower limits. The lower limit represents a realistic assessment of adequate positioning, and the upper limit is a maximum not to be exceeded by the largest members, as illustrated in Figure 2-80 for horizontal alignment of spandrel beams.

29.3.7 Alignment of Braced Members

This clause is an outgrowth of the extensive work on restructuring Clause 9 on Stability of Structures and Members during the preparation of S16-01. Clause 9.2.1 requires the structure to be brought into line so that the initial misalignment of members at any brace point when the brace is installed does not exceed the limits of Clause 29.3. Thus, the initial misalignment at the

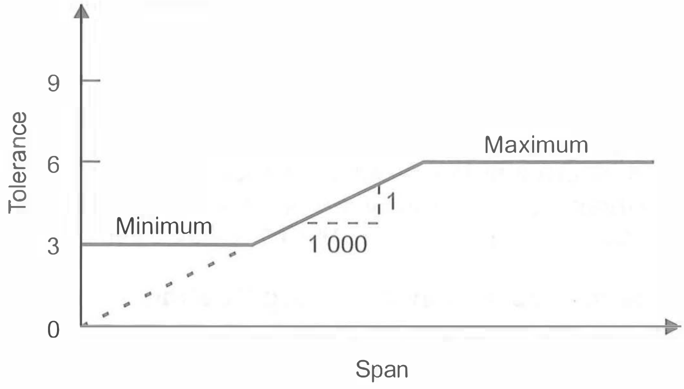


Figure 2-80
Horizontal Alignment Tolerances of Spandrel Beams

particular brace point, Δ_0, relative to the adjacent ones is established, and the analyses given in Clause 9 can follow confidently. This Clause again emphasizes that design, fabrication, and erection are inextricably linked.

30. INSPECTION

This clause outlines quality assurance practices with the objective of ensuring that all shop work and field erection work are in essential compliance with this Standard, in order to provide a structure that is fit for purpose with the requisite strength and stiffness.

30.5 Third-Party Welding Inspection

Third-party welding inspection, just as for any other inspection procedures, is to ensure, insofar as possible by visual inspection, that the welds are fit for purpose, that is, they have the requisite strength and stiffness. The first distinction the inspector should make is to determine from the design engineer whether any components of the structure are subject to fatigue loading, in which case visible weld defects can have a significant effect on the fatigue life.

In cases where an engineering assessment of welds in existing structures is required due to defects (such as porosity, poor profile, lack of fusion, undercut, blow holes, lack of penetration, and craters) that exceed the limits given in CSA W59, the underlying question is: for statically loaded structures, how much have the defects reduced the weld strength? Kennedy (1967, 1968) suggests for statically loaded welds that weld strengths are reduced in the proportion of the defect area to the effective throat area.

REFERENCES

AASHTO. 2007 (R2009). LRFD bridge design specifications, SI Units, 4th Edition. American Association of State Highway and Transportation Officials, Washington, DC.

Adams, P. F. 1974. The design of steel beam-columns. Canadian Steel Industries Construction Council, Willowdale, Ont.

Adams, P. F., and Galambos, T. V. 1969. Material considerations in plastic design. International Association for Bridge and Structural Engineering, 29-II.

Adekola, A. O. 1968. Effective widths of composite beams of steel and concrete. The Structural Engineer, 46(9): 285-289.

Ahmad, M., Chien, E. Y. L., and Hosain, M. U. 1990. Modified stub-girder floor system: full-scale tests. ASCE Structures Congress. Baltimore, MA.

AISC. 1973. Commentary on highly restrained welded connections. Engineering Journal, American Institute of Steel Construction, Third Quarter.

— 2005. Seismic design manual. American Institute of Steel Construction, Chicago, IL.

— 2010a. Seismic provisions for structural steel buildings, ANSI/AISC 341-10. American Institute of Steel Construction (AISC), Chicago, IL.

— 2010b. Specification for structural steel buildings, ANSI/AISC 360-10. American Institute of Steel Construction (AISC), Chicago, IL.

— 2011. Prequalified connections for special and intermediate steel moment frames for seismic applications, including Supplement No. 1, ANSI/AISC 358-10 and ANSI/AISC 358s1-11. American Institute of Steel Construction (AISC), Chicago, IL.

— 2013. Steel construction manual. 14th Edition. American Institute of Steel Construction. Chicago, IL.

AISI. 1968. Plastic design of braced multi-storey steel frames. American Iron and Steel Institute, Washington, DC.

Albert, C., Essa, H. S., and Kennedy, D.J.L. 1992. Distortional buckling of steel beams in cantilever-suspended span construction. Canadian Journal of Civil Engineering, 19(5): 767-780.

Alinia, M.M., and Dastfan, M. 2007. Cyclic behaviour, deformability and rigidity of stiffened steel shear panels. Journal of Constructional Steel Research, 63, 554–563

Allen, D. L. 1974. Vibrational behaviour of long-span floor slabs. Canadian Journal of Civil Engineering, 1(1).

— 1975. Limit states design – A probabilistic study. Canadian Journal of Civil Engineering, 2(1).

Allen, D. E., and Murray, T. M. 1993. Design criterion for vibrations due to walking. Engineering Journal, American Institute of Steel Construction, 30(4): 117-129.

Allen, D. E., and Rainer, J. H. 1976. Vibration criteria for long-span floors. Canadian Journal of Civil Engineering, 3(2).

Allen, D. E., Rainer, J. H., and Pernica, G. 1985. Vibration criteria for assembly occupancies. Canadian Journal of Civil Engineering, 12(3).

ASCE. 1967. Commentary on welded cover-plated beams. Subcommittee on Cover Plates, Task Committee on Flexural Members, ASCE, J. of the Structural Division, 93(ST4).

— 1971. Commentary on plastic design in steel. Manual of Engineering Practice, No. 41, American Society of Civil Engineers.

— 2000. Design of latticed steel transmission structures, ASCE 10-97. American Society of Civil Engineers, Reston, VA.

Aslani, F., and Goel, S.C. 1991. Stitch spacing and local buckling in seismic-resistant double-angle bracing, J. of Struct. Eng., ASCE, 2442-2463.

Astaneh-Asl, A. 2001. Seismic behavior and design of steel shear walls. Steel Tips, Structural Steel Educational Council, Moraga, CA.

Astaneh-Asl, A., and Goel, S.C. 1984. Cyclic in-plane buckling of double-angle bracing. J. of Struct. Eng., ASCE, 110(ST9): 2036-2055.

— 1985. Cyclic out-of-plane buckling of double-angle bracing. J. of Struct. Eng., ASCE, 111(ST5): 1135-1153.

Astaneh, A., Goel, S.C., and Hanson, R.D. 1986. Earthquake-resistant design of double-angle bracings. Engineering Journal, AISC, 23(4): 133-147.

ASTM. 2010. Standard specification for structural bolts, steel, heat-treated, 120/105 ksi minimum tensile strength. Standard A325-10e1, American Society for Testing and Material.

— 2011. Standard specification for "twist off" type tension control structural bolt/nut/washer assemblies, steel, heat treated, 120/105 ksi minimum tensile strength. Standard F1852-11, American Society for Testing and Material.

— 2012a. Standard specification for high-strength steel bolts, Class 10.9 and 10.9.3, for structural steel joints (metric). Standard A490M-12, American Society for Testing and Material.

— 2012b. Standard specification for structural bolts, alloy steel, heat-treated, 150 ksi minimum tensile strength. Standard A490-12, American Society for Testing and Material.

— 2012c. Standard specification for "twist off" type tension control structural bolt/nut/washer assemblies, steel, heat treated, 150 ksi minimum tensile strength. Standard F2280-12, American Society for Testing and Material.

— 2013. Standard specification for high-strength bolts for structural steel joints (metric). Standard A325M-13. American Society for Testing and Material.

Attard, M.M., and Lawther, R. 1989. Effect of secondary warping on lateral buckling, Engineering Structures, Vol 11, April 1989, pp 112-118.

Aziz, T.S.A. 1972. Inelastic nonlinear behaviour of steel triangulated planar frames. M. Eng. Thesis, Carleton University, Ottawa, Ont.

Baker, K. A., and Kennedy, D.J.L. 1984. Resistance factors for laterally unsupported steel beams and biaxially loaded steel beam-columns. Canadian Journal of Civil Engineering, 11(4): 1008-1019.

Bansal, J.P. 1971. The lateral instability of continuous steel beams. Ph.D. Department, University of Texas, Austin.

Bartlett, F.M. 2007. Canadian Standards Association standard A23.3-04 resistance factor for concrete in compression, Canadian Journal of Civil Engineering, 34, 1029-1037.

Basler, K. 1961a. New provisions for plate girder design. Proceedings, AISC National Engineering Conference.

— 1961b. Strength of plate girders under combined bending and shear. ASCE Journal of the Structural Division, 87(ST7).

— 1961c. Strength of plate girders in shear. ASCE Journal of the Structural Division, 87(ST7).

Basler, K., and Thurlimann, B. 1961. Strength of plate girders in bending. ASCE Journal of the Structural Division, 87(ST6).

Beaulieu, D., and Adams, P. F. 1980. Significance of structural out-of-plumb forces and recommendations for design. Canadian Journal of Civil Engineering, 7(1).

Beedle, L. S., Lu, L. W., and Lim, L. C. 1969. Recent developments in plastic design practice. ASCE Journal of the Structural Division, 95(ST9).

Begum, M., Driver, R.G., and Elwi, A.E. 2007. Numerical simulations of the behaviour of partially encased composite columns. Structural Engineering Report No. 269, July, Department of Civil and Environmental Engineering, Univ. of Alberta, Edmonton, Canada. 251 pp.

Begum, M., Driver, R.G., and Elwi, A.E. 2013. Behaviour of partially encased composite columns with high strength concrete. Engineering Structures, Vol. 56, November, pp. 1718-1727.

Benichou, N. 1994. Behaviour of webs of rolled steel beams subjected to concentrated loads. Ph.D. Thesis, Dept. of Civ. and Env. Eng., Carleton University, Ottawa, Sept, 219 pages.

Bergmann, R., Matsui, C., Meinsma, C. and Dutta, D. 1995. Design guide for concrete filled hollow section columns under static and seismic loading. CIDECT Design Guide No. 5, International Committee for the Study and Development of Tubular Structures, CIDECT and Verlag TÜV Rheinland GmbH, Köln, Germany.

Berman, J.W., and Bruneau, M. 2003. Plastic analysis and design of steel plate shear walls. J. Struct. Eng., ASCE, 129, 11, 1448-1456.

— 2005a. Approaches for the seismic retrofit of braced steel bridge piers and proof-of-concept testing of a laterally stable eccentrically braced frame. Technical Report MCEER-05-0004, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, NY.

— 2005b. Experimental investigation of light-gauge steel plate shear walls. J. Struct. Eng., ASCE, 131, 2, 259–267.

— 2008a. Tubular links for eccentrically braced frames I: Finite element parametric study & II: Experimental verification. J. Struct. Eng., ASCE, 134, 5, 692-712.

— 2008b. Capacity design of vertical boundary elements in steel plate shear walls. Eng. J., AISC, 45, 3, 57-71.

— 2009. Cyclic testing of a buckling restrained braced frame with unconstrained gusset connections. J. Struct. Eng., ASCE, 135, 12, 1499-1510.

Birkemoe, P. C., and Gilmor, M. I. 1978. Behaviour of bearing critical, double-angle beam connections. Engineering Journal, AISC, Fourth Quarter.

Bjorhovde, R. A. 1972. A probabilistic approach to maximum column strength. Proceedings, ASCE Conference on Safety and Reliability of Metal Structures.

Bjorhovde, R., and Birkemoe, P. C. 1979. Limit states design of HSS columns. Canadian Journal of Civil Engineering, 6(2).

Bjorhovde, R., and Zimmerman, T. J. 1980. Some aspects of stub-girder design. Proceedings of the Canadian Structural Engineering Conference. Canadian Steel Construction Council. Willowdale, Ont.

Bleich, F. 1952. Buckling strength of metal structures. McGraw-Hill. New York., NY.

Bondy, K.D. 1996. A more rational approach to capacity design of seismic moment frame columns. Earthquake Spectra. Vol.12, No.3: 395-406.

Bower, J. E., *et al*. 1971. Suggested design guide for beams with web holes. Journal of the Structural Division, American Society of Civil Engineers, 97(ST11).

Bremault, D., Driver, R., and Grondin, G. 2008. Limit states design approach for rolled wide flange beams subject to combined torsion and flexure. Structural engineering report 279, Department of Civil and Environmental Engineering, University of Alberta, Edmonton, Canada.

Bridge, R.O., Clarke, M.J., Leon, R.T., Lui, E.M., Sheikh, T.M., White, D.W. and Hajjar, J.F. 1997. Effective length and notional load approaches for assessing frame stability: Implications for American steel design. American Society of Civil Engineers, New York, N.Y.

Brockenbrough, R. L. 1983. Considerations in the design of bolted steel joints for weathering steel. Engineering Journal, American Institute of Steel Construction 20(1).

Brockenbrough, R. L., and Johnston, B. G. 1974. Steel design manual. United States Steel Corp., Pittsburgh, PA.

Bruneau, M., Mahin, S., and Popov, E.P. 1987. Ultimate behavior of butt welded splices in heavy rolled steel sections. Report No. UCB/EERC-87/10 Earthquake Engineering Research Center, Berkeley, CA.

Bruneau, M., Uang, C.M. and Whittaker, A. 1998. Ductile Design of Steel Structures. McGraw-Hill, New York, NY. 485 pp.

BSSC. 2003. NEHRP Recommended provisions for seismic regulations for new buildings and other structures. FEMA 450, Building Seismic Safety Council, Washington, D.C.

Budijgnto, P. 1983. Design methods for composite columns. Project Report G83-6, Department of Civil Engineering, McGill University.

Butler, L.J., and Kulak, G.L. 1971. Strength of fillet welds as a function of direction of load. Welding Research Supplement, Welding Journal. Welding Research Council, 36(5): 2315-2345.

Butler, L.J., Pal, S. and Kulak, G.L. 1972. Eccentrically loaded welded connections. Journal of the Structural Division, American Society of Civil Engineers 98(ST5): 989-1005.

Cai, Q. and Driver, R.G. 2010. Prediction of bolted connection capacity for block shear failures along atypical paths. Engineering Journal, American Institute of Steel Construction, vol. 47, 4th Quarter.

Callele, L.J., Driver, R.G., and Grondin, G.Y. 2009. Design and behavior of multi-orientation fillet weld connections. Engineering Journal, American Institute of Steel Construction, vol. 46, no. 4, pp. 257-272.

Carter, C. J. 1999. Stiffening of wide flange columns at moment connections: wind and seismic applications. American Institute of Steel Construction, Chicago IL

Chen, C.-H., Lai, J.-W., and Mahin, S. 2008. Numerical modelling and performance assessment of concentrically braced steel frames. Proc. ASCE Struct. Congress Crossing Borders, Vancouver, BC, 314, 260.

Chen, W.-F., Kishi, N. and Komuro, M., Editors. 2011. Semi-rigid connections handbook, J. Ross Publishing.

Chen, W. F., and Lui, E.M. 1987. Structural stability, theory and implementation. Elsevier. New York, NY.

Chen, Z., Ge, H., and Usami, T. 2006. Hysteretic model of stiffened shear panel dampers. J. Struct. Eng., ASCE, 132, 3, 478-483.

Cheng, J.J.R. and Grondin, G.Y. 1999. Recent developments in the behavior of cyclically loaded gusset plate connections. Proc. North American Steel Construction Conference, AISC. pp. 8-1 to 8-22.

Cheng, J.J.R., and Yura, J. A. 1986. Local web buckling of coped beams. Journal of Structural Engineering, American Society of Civil Engineers, 112(10).

— 1988. Lateral buckling tests on coped steel beams. Journal of Structural Engineering, American Society of Civil Engineers, 114(1): 16-30.

Cheng, J.J.R., Yura, J.A., and Johnson, C.P. 1988. Lateral buckling of coped steel beams. Journal of Structural Engineering, American Society of Civil Engineers, 114(1): 1-15.

Chernenko, D.E. and Kennedy, D.J.L. 1991. An analysis of the performance of welded wide flange columns. Canadian Journal of Civil Engineering, 18(4).

Chien, E.Y.L. 1989. Roof framing with cantilever (Gerber) girders & open-web steel joists. Canadian Institute of Steel Construction, Willowdale, Ont.

Chien, E.Y.L., and Ritchie, J. K. 1984. Design and construction of composite floor systems. Canadian Institute of Steel Construction, Willowdale, Ont.

Chusilp, P., and Usami, T. 2002. New elastic stability formulas for multiple-stiffened shear panels. J. Struct. Eng., ASCE, 128, 6, 833-836.

CIDECT. 1970. Concrete filled hollow section steel columns design manual. Monograph No. 1, International Committee for the Study and Development of Tubular Structures. Whitefriars Press Ltd., London.

CISC. 1980. Steel Joist Facts, 2nd Edition. Canadian Institute of Steel Construction, Willowdale, Ont.

— 2002. Steel Fabrication Quality Systems Guideline. Canadian Institute of Steel Construction, Willowdale, Ont.

— 2014. Moment connections for seismic applications. 2nd Edition. Canadian Institute of Steel Construction, Markham, Ontario.

Clarke, M.J. and Bridge, R.Q. 1992. The inclusion of imperfections in the design of beam-columns, Proc. 1992 Annual Tech. Session, Structural Stability Research Council, Bethlehem: 327-346.

— 1995. The notional load approach to the design of frames, Res. Rep. No. R718, School of Civil and Mining Eng., University of Sydney, Australia, Dec.

CSA. 1969. Steel structures for buildings. CSA Standard S16-1969, Canadian Standards Association, Toronto, Ont.

— 2007. North American specification for the design of cold-formed steel structural members. CSA Standard S136-07. Canadian Standards Association, Mississauga, Ont.

— 2010. Certification of manufacturers of steel building systems, CSA Standard A660-10, Canadian Standards Association, Mississauga, Ont.

— 2014. Canadian highway bridge design code. CSA Standard S6-14. Canadian Standards Association, Mississauga, Ont.

— 2014. Design of concrete structures. CSA Standard A23.3-14, Canadian Standards Association, Mississauga, Ont.

Dastfan, M. and Driver, R.G. 2008. Flexural stiffness limits for frame members of steel plate shear wall systems. Proc., Annual Stability Conference, Structural Stability Research Council, April 2-5, Nashville, TN, USA.

— 2009. Investigations on the effect of frame member connection rigidity on the behavior of steel plate shear wall systems. Proc., STESSA 2009 Behaviour of Steel Structures in Seismic Areas, August, Philadelphia, PA, USA.

Davaran, A. 2001. Effective length factor for discontinuous X-bracing systems. J. Struct. Eng., ASCE, 127, 2, 106-112.

Davaran, A., and Hoveidae, N. 2009. Effect of mid-connection detail on the behavior of X-bracing systems. J. Constr. Steel Research, 65, 2, 290-298.

Davies, C. 1969. Tests on half-scale steel concrete composite beams with welded stud connectors. The Structural Engineer, 47(1): 29-40.

Dawe, J. L., and Kulak, G. L. 1984. Local buckling of W shape columns and beams. Journal of Structural Engineering, American Society of Civil Engineers, 110(6).

— 1986. Local buckling behavior of beam-columns. Journal of Structural Engineering, American Society of Civil Engineers, 112(11).

de Back, J., and de Jong, A. 1968. Measurement on connections with high strength bolts, particularly in view of the permissible arithmetical bearing stress. Report 6-68-3, Stevin Laboratory, Delft University of Technology, Netherlands.

de Oliveira, J.C., Packer, J.A., Christopoulos, C. 2008. Cast steel connectors for circular hollow section braces under inelastic cyclic loading. J. Struct. Eng., ASCE, 134, 3, 374-383.

Deng, K., Grondin, G.Y., and Driver, R.G. 2006. Effect of loading angle on the behavior of fillet welds. Engineering Journal, American Institute of Steel Construction, vol. 43, no. 1, pp. 9-23.

Deng, X., Dastfan, M., and Driver, R.G. 2008. Behaviour of steel plate shear walls with composite columns. Proc., Structures Congress 2008 Crossing Borders, ASCE, 314, Vancouver, BC, Canada.

Dexter, R.J., Altstadt, S.A., and Gardner, C.A. 2002. Strength and ductility of HPS70W tension members and tension flanges with holes, Final Report to American Iron and Steel Institute, University of Minnesota, March.

Dexter, R.J., and Gentilcore, M.L. 1997. Evaluation of ductile fracture models for ship structural details, Report SSC-393, Ship Structure Committee, Washington, DC.

Dexter, R.J., and Melendrez, M.I. 2000. Through-thickness properties of column flanges in welded moment connections. Journal of Structural Engineering, American Society of Civil Engineers, 126(1): 24-31.

Disque, R. O. 1964. Wind connections with simple framing. Engineering Journal, AISC, July.

Driver, R. G., Grondin, G. Y., and Kulak, G. L. 2006. Unified block shear equation for achieving consistent reliability. Journal of Constructional Steel Research, Vol. 62, no. 3, pp. 210-222.

Driver, R. G., and Kennedy, D. J. L. 1989. Combined flexure and torsion of I-shaped steel beams. Canadian Journal of Civil Engineering, 16(2): 124-139.

Driver, R.G., *et al*. 1997. Seismic behaviour of steel plate shear walls, Structural Engineering Report No. 215, Department of Civil and Environmental Engineering, University of Alberta.

— 1998(a). FE and simplified modelling of a steel plate shear wall, Journal of Structural Engineering, American Society of Civil Engineers, 124(2): 121-130.

— 1998(b). Cyclic test of a four storey steel plate shear wall, Journal of Structural Engineering, American Society of Civil Engineers, 124(2): 112-120.

Driver, R.G., and Wong, E. 2007. Critical evaluation of the CSA S16-01 equivalent moment factor for laterally unsupported beams. Proc., Canadian Society for Civil Engineering Annual Conference, June 6–9, Yellowknife, Canada.

Duan, L., and Chen, W-F. 1988. Design rules of built-up members in load and resistance factor design. Journal of Structural Engineering, American Society of Civil Engineers, 114(11): 2544-2554.

El-Ghazzi, M. N., Robinson, H., and Elkholy, I.A.S. 1976. Longitudinal shear capacity of slabs of composite beams. Canadian Journal of Civil Engineering, 3(4).

Elkelish, S., and Robinson, H. 1986. Effective widths of composite beams with ribbed metal deck. Canadian Journal of Civil Engineering, 13(5): 575-582.

El-Tayem, A.A., and Goel, S.C. 1986. Effective length factor for the design of X-bracing systems. Engineering Journal, AISC, 24: 41-45.

Englehardt, M.D., and Popov, E.P. 1989. Behaviour of long links in eccentrically braced frames. EERC Report 89-01, University of California, Berkeley, CA.

— 1992. Experimental performance of long links in eccentrically braced frames. ASCE Journal of Structural Engineering, 118(11): 3067-3088.

Englekirk, R. 1994. Steel structures – Controlling behavior through design. John Wiley & Sons. New York, NY.

Epstein, H. I., and Stamberg, H. 2002. Block shear and net section capacities of structural Tees in tension: test results and code implications. AISC Engineering Journal, Vol. 39, Fourth quarter, pp. 228-239.

Essa, H.S. and Kennedy, D.J.L. 1994a, Station Square revisited: distortional buckling collapse. Canadian Journal of Civil Engineering, 21(3): 377-381.

— 1994b, Design of cantilever beams: a refined approach. Journal of Structural Engineering, American Society of Civil Engineers. 120 (ST9): 2623-2636.

— 1995, Design of steel beams in cantilever-suspended-span construction. Journal of Structural Engineering, American Society of Civil Engineers. 121(ST11): 1667-1673.

— 2000. Proposed provisions for the design of steel beam-columns in S16-2001. Canadian Journal of Civil Engineering, 27(4): 610-619.

Essa, H.S., Tremblay, R., and Rogers, C. 2003. Behavior of roof deck diaphragms under quasi-static cyclic loading. Journal of Structural Engineering, ASCE, 129(12): 1658-1666.

Estabrooks, B., and Grondin, G. 2008. Combined bending and torsion of steel I-shaped beams. Structural Engineering report 276, Department of Civil and Environmental Engineering, University of Alberta, *www.engineering.ualberta.ca/structures/reports.cfm.*

EUROCODE No. 4. 1987. Common unified rules for composite steel and concrete structures. Report EUR 9886 EN, Commission of the European Communities, Luxembourg.

Faella, C., Piluso, V., and Rizzano, G. 2000. Structural steel semirigid connections – theory, design and software. CRC Press LLC, Boca Raton, Florida.

Fahnestock, L. A., Ricles, J.M., and Sause, R. 2007. Experimental evaluation of a large-scale buckling-restrained braced frame, J. Struct. Eng., ASCE, 133, 9, 1205–1214.

Fell, B.V., Kanwinde, A.M., Deierlein, G.G., and Myers, A.T. 2009. Experimental investigation of inelastic cyclic buckling and fracture of steel braces, J. Struct. Eng., ASCE, 135, 1, 1-19.

FEMA. 1995. FEMA 267 (SAC 95-02) Interim guidelines: Evaluation, repair, modification and design of steel moment frames. Federal Emergency Management Agency. Washington, DC.

— 1997. FEMA 267A (SAC 96-03) Interim guidelines advisory no. 1 – Supplement to FEMA 267. Federal Emergency Management Agency. Washington, DC.

— 2000. Recommended seismic design criteria for new steel moment-frame buildings. Report FEMA 350. Federal Emergency Management Agency, Washington, D.C.

Fisher, J. W. 1974. Guide to 1974 AASHTO fatigue specifications. AISC, Chicago, IL.

— 1978. Fatigue cracking in bridges from out-of-plane displacements. Canadian Journal of Civil Engineering, 5(4).

— 1984. Fatigue and fracture in steel bridges – Case studies. John Wiley & Sons.

Fisher, J, W., Lee, G. C, Yura, J. A., and Driscoll, G. C. 1963. Plastic analysis and tests of haunched corner connections. Bulletin No. 91, Welding Research Council.

Franchuk, C.R., Driver, R.G., and Grondin, G.Y· 2003. Experimental investigation of block shear failure in coped steel beams. Canadian Journal of Civil Engineering, 30: 871-881.

Frank, K. H. 1980. Fatigue strength of anchor bolts. Journal of the Structural Division, American Society of Civil Engineers, 106(ST6).

Gagnon, D. G., and Kennedy, D. J. L. 1989. Behaviour and ultimate tensile strength of partial joint penetration groove welds. Canadian Journal of Civil Engineering, 16(3).

Galambos, T. V. 1968. Structural members and frames. Prentice-Hall Inc., Englewood Cliffs, NJ.

Galambos, T.V., and Ravindra, M.K. 1973. Tentative load and resistance factor design criteria for steel buildings. Res. Rep. 18, Civil Engineering Department, Washington University, St. Louis, MO.

Ghali, A., Favre, R., and Elbadry, M. 2002. Concrete structures: Stresses and deformations, 3rd ed. London: Spon Press.

Ghobarah, A., Korol, R.M., and Osman, A. 1992. Cyclic behaviour of extended end plate joints. ASCE. Journal of Structural Engineering, 118(ST5)

Gibson, G. T., and Wake, B. T. 1942. An investigation of welded connections for angle tension members. Welding Journal, American Welding Society.

Gobel, G. 1968. Shear strength of thin flange composite specimens. Engineering Journal, AISC, April.

Goel, S.C., and Itani, A. 1994. Seismic resistant special truss moment frames. ASCE Journal of Structural Engineering. 120(6).

Goel, S.C., Rai, D.C., and Hisham, S. 1998. Special truss moment resisting frames – Design guide. Research Report UMCEE 98-44. Dept. of Civ. Eng., Univ. of Michigan, Ann Arbor, MI.

Goodier, J.N. 1942, Flexural-torsional buckling of bars of open section. Cornell University Engineering Experiment Station Bulletin No. 28, January.

Graham, J. D., Sherbourne, A. N., Khabbaz, R. N., and Jensen, C. D. 1959. Welded interior beam-to-column connections. American Institute of Steel Construction.

Grant, J. A., Fisher, J. W., and Slutter, R. G. 1977. Composite beams with formed steel deck. Engineering Journal, AISC, First Quarter.

Griggs, P. H. 1976. Mill building structures. Proceedings, Canadian Structural Engineering Conference, Canadian Steel Construction Council, Willowdale, Ont.

Guravich, S.J., and Dawe, J.L. 2006. Simple beam connections in combined shear and tension. Canadian Journal of Civil Engineering, 33: 357-372.

Haaijer, G., and Thurlimann, B. 1958. On inelastic buckling in steel. ASCE Journal of the Engineering Mechanics Division, April.

Haddad, M., and Tremblay, R. 2006. Influence of connection design on the inelastic seismic response of HSS steel bracing members. In Tubular Structures XI: Proc. 11th Int. Symp. and IIW Int. Conf. on Tubular Structures, Québec, QC, 639-646.

Hagood, T. A., Jr., Guthrie, L., and Hoadley, G. 1968. An investigation of the effective concrete slab width for composite construction. Engineering Journal, AISC, 5(1): 20-25.

Han, X.M., 1998. Design and behaviour of eccentrically braced frames in moderate seismic zones, M.Eng. thesis, Department of Civil Engineering and Applied Mechanics, McGill University, Montreal.

Hansell, W. C., and Viest, I. M. 1971. Load factor design for steel highway bridges. Engineering Journal, AISC, 8(4).

Hardash, S., and Bjorhovde, R. 1985. New design criteria for gusset plates in tension. Engineering Journal, AISC, 21(2): 77-94.

Hart, W. H., and Milek, W. A. 1965. Splices in plastically designed continuous structures. Engineering Journal, AISC, April.

Hawkins, N. M., and Mitchell, D. 1984. Seismic response of composite shear connections. ASCE Journal of Structural Engineering, 110(9): 2120-2136.

Heins, C. P., and Fan, H. M. 1976. Effective composite beam width at ultimate load. ASCE Journal of the Structural Division, 102(ST11).

Hirano, N. 1970. Bearing stresses in bolted joints. Society of Steel Construction of Japan, Tokyo, 6(58).

Holtz, N. M., and Kulak, G. L. 1970. High strength bolts and welds in load-sharing systems. Studies in Structural Engineering, No. 8, Technical University of Nova Scotia.

— 1973. Web slenderness limits for compact beams. SER 43, Department of Civil Engineering, University of Alberta.

— 1975. Web slenderness limits for non-compact beams. SER 51, Department of Civil Engineering, University of Alberta.

Hosain, M.U., and Pashan, A. 2002. Research in Canada on steel-concrete composite floor systems: an update. Proceedings of ICASS'02 -Third International Conference on Advances in Steel Structures, Hong Kong, 9-11 December, Volume 1: 527-534, Elsevier, Oxford, UK.

Hosain, M.U., and Wu. H. 2002. Composite beams with parallel wide ribbed metal deck: push-out and full size beam tests, Composite Construction in Steel and Concrete IV, Special Monograph of the Structural Engineering Institute, ASCE, 415-425.

Huang, J.S., Chen, W.F., and Beedle, L.S. 1973. Behaviour and design of steel beam-to-column moment connections. Welding Research Council Bulletin, 188: 1-23.

Imanpour, A., Auger, K., and Tremblay, R. 2014. Seismic design of multi-tiered steel braced frames including the contribution from gravity columns. Proceedings 12th International Conference on Computational Structures Technology CST, Naples, Italy.

Imanpour, A., Stoakes, C., Tremblay, R., Fahnestock, L., and Davaran, A. 2013. Seismic stability response of columns in multi-tiered braced steel frames for industrial applications. Proceedings ASCE Structures Congress 2013, Pittsburgh, PA, 2650-2661.

Imanpour, A., Tremblay, R., and Davaran, A. 2012a. Seismic performance of steel concentrically braced frames with bracing members intersecting columns between floors. Proceedings 7th STESSA Conference, Santiago, Chile, 447-453.

Imanpour, A., Tremblay, R., and Davaran, A. 2012b. Seismic Evaluation of Multi-Panel Steel Concentrically Braced Frames. Proceedings 15th World Conf. on Earthquake Engineering, Lisbon, Portugal, Paper No. 2996.

Imanpour, A., and Tremblay, R. 2012. Analytical assessment of stability of unbraced column in two panel concentrically braced frames. Proceedings 3rd International Structural Specialty Conference, Alberta, Canada, Paper No. 1218.

Imanpour, A., and Tremblay, R. 2014a. Seismic performance evaluation and design of multi-tiered steel concentrically braced frames. Proceedings 10th U.S. National Conference on Earthquake Engineering, Anchorage, AK.

Imanpour, A., and Tremblay, R. 2014b. Seismic design of steel multi-tiered braced frames: Application of incremental static analysis for design of steel multi-tiered braced frames. Proceedings 2014 Eurosteel Conference, Naples, Italy, Paper no. 688.

Iwankiw, N., and Schlafly, T. 1982. Effect of hole-making on the strength of double lap joints, Engineering Journal, AISC, 19(3): 170-178.

Izvernari, C., Lacerte, M., and Tremblay, R. 2007. Seismic performance of multi-storey concentrically braced steel frames designed according to the 2005 Canadian seismic provisions. Proc. 9th Canadian Conference on Earthquake Engineering, Ottawa, ON. Paper No. 1419.

Jain, A.K., Goel, S.C., and Hanson, R.D. 1980. Hysteretic cycles of axially loaded steel members. J. of the Struct. Div., ASCE, 106: 1777-1795.

Jayas, B. S., and Hosain, M. U. 1988. Behaviour of headed studs in composite beams: push-out tests. Canadian Journal of Civil Engineering, 15(2), 240-253.

— 1989. Behaviour of headed studs in composite beams: full-size tests. Canadian Journal of Civil Engineering, 16(5), 712-724.

Jent, K. A. 1989. Effects of shrinkage, creep and applied loads on continuous deck-slab composite beams. M.Sc. thesis, Queen's University, Kingston, Ont.

Ji, X., Kato, M., Wang, M., Hitaka, T., and Nakashima, M. 2009. Effect of gravity columns on mitigation of drift concentration for braced frames. Journal of Constructional Steel Research, 65, 12, 2148-2156.

Johnson, R. P. 1970. Longitudinal shear strength of composite beams. ACI Journal Proceedings, 67.

— 1975. Composite structures of steel and concrete. Volume 1: beams, columns, frames and applications in buildings. Crosby Lookwood Staples, London, England : 210.

Johnston, B. G. 1939. Pin connected plate links. Transactions, American Society of Civil Engineers.

Jones, J. 1958. Bearing-ratio effect on strength of riveted joints. Transactions, American Society of Civil Engineers, 123: 964-972.

Kaliandasani, R. A., Simmonds, S. H., and Murray, D. W. 1977. Behaviour of open web steel joists. Report No. 62, Department of Civil Engineering, University of Alberta.

Kalyanaraman, V., Pekoz, T., and Winter, G. 1977. Unstiffened compression elements. ASCE. Journal of the Structural Division, 103(ST9).

Kanchanalai, T. 1977. The design and behavior of beam-columns in unbraced steel frames. CESRL Report No. 77-2, AISI Project No. 189 – Column design in unbraced frames, Report No. 2, Structures Research Laboratory, The University of Texas at Austin.

Kasai, K., and Han, X. 1997. New EBF design method and application: Redesign and analysis of US-Japan EBF. STESSA Conference, Tokyo.

Kasai, K., and Popov, E.P. 1986. General behavior of WF steel shear link beams. ASCE Journal of Structural Engineering, 112(ST2): 362-382.

Kemp, A.R., and Trinchero, P. 1992. Serviceability stress limits for composite beams. Composite Construction and Engineering Foundation Conference, June 15-19, Potosi, MO.

Kennedy, D.J.L. 1967. Evaluation of structural weld defects. Canadian Welding Society Annual Meeting, Montreal, May.

— 1968. Evaluation of structural weld defects. Canadian Welder and Fabricator: 10-14.

— 1974. Limit states design – An innovation in design standards for steel structures. Canadian Journal of Civil Engineering, 1(1).

— 1995. Limit states design of beam-columns in CSA S16.1-94. Proc. Int. Conf. on Struct. Stability and Des., G.J. Hancock and M.A.Bradford, eds., Oct 30-Nov 1, Balkerna, Rotterdam, 461-465.

— 2004. Analysis of strength of flare bevel groove welds in open web steel joists. Committee communication.

Kennedy, D.J.L., Allen, D. E., Adams, P. F., Kulak, G. L., Turner, D. K., and Tarlton, D. L. 1976. Limit states design. Proceedings of the Canadian Structural Engineering Conference, Canadian Steel Industries Construction Council, Willowdale, Ont.

Kennedy, D.J.L., and Brattland, A. 1992. Shrinkage tests of two full-scale composite trusses. Canadian Journal of Civil Engineering, 19(2).

Kennedy, D.J.L., and Gad Aly, M. 1980. Limit states design of steel structures – Performance factors. Canadian Journal of Civil Engineering, 7(1).

Kennedy, D.J.L., Kulak, G.L., and Driver, R.G. 1994. Discussion to postbuckling behavior of steel-plate shear walls, by Elgaaly, M., Caccese, V., and Du., C. Journal of Structural Engineering, ASCE, 120(7): 2250-2251.

Kennedy, D.J.L., Miazga, G.S., and Lesik, D.F. 1990. Discussion of evaluation of fillet weld shear strength of FCAW electrodes by McClellan, R.W. Welding Journal, August 1989, Welding Journal Reference, 44-46.

Kennedy, D.J.L., Picard, A., and Beaulieu, D. 1990. New Canadian provisions for the design of steel beam-columns. Canadian Journal of Civil Engineering, 17(6).

Kennedy, D.J.L., and Rowan, W.H.D. 1964. Behaviour of compression chords of open web steel joists. Canadian Institute of Steel Construction, Willowdale, ON.

Kennedy, J.B., and Neville, A.M. 1986. Basic statistical methods for engineers and scientists. Third edition, Harper and Row, New York, NY.

Kennedy, S. J., and Kennedy, D. J. L. 1987. The performance and strength of hardened steel test roller assemblies. Proceedings, CSCE Centennial Conference, May 19-22. Montreal, QC: 513-531.

Kennedy, S. J. Kennedy, D. J. L., and Medhekar, M.S. 1998. The bearing resistance of webs: further studies of the post buckling strength. Proc. Annual Meeting, Structural Stability Research Council, Atlanta, Sept. 21 -23

Kennedy, S.J., and MacGregor, J.G. 1984. End connection effects on the strength of concrete filled HSS beam-columns. Structural Engineering Report 115, Department of Civil Engineering, University of Alberta, Edmonton, Alberta.

Ketter, R. L. 1961. Further studies of the strength of beam-columns. ASCE, Journal of the Structural Division, 87(ST6): 135-152.

Kirby, P. A., and Nethercot, D. A. 1978. Design for structural stability. Granada. London, England.

Knowles, R. B., and Park, R. 1970. Axial load design for concrete filled steel tubes. ASCE, Journal of the Structural Division, 96(ST10).

Koboevic, S. 2000. An approach to seismic design of eccentrically braced frames. Ph.D. Thesis, Department of Civil Engineering and Applied Mechanics, McGill University, Montreal.

Krawinkler, H., and Popov, E.P. 1982. Seismic behaviour of moment connections and joints. ASCE, Journal of the Structural Division, 108(ST2), 373-391.

Kuhn, P., Peterson, J. P., and Levin, L. R. 1952, A summary of diagonal tension, Part I – Methods of analysis, Technical Note 2661, National Advisory Committee for Aeronautics, Langley Aeronautical Laboratory, Langely Field, VA.

Kulak, G.L. 1985. Behaviour of steel plate shear walls, Proceedings, The 1985 International Engineering Symposium on Structural Steel, Chicago.

— 1986. Unstiffened steel plate shear walls: Static and seismic behaviour, in steel structures, Recent Research Advances and Their Applications to Design, Edited by M.N. Pavlovic, Elsevier Applied Science Publishers, London.

— 1991. Unstiffened steel plate shear walls. Structures subjected to repeated loading: Stability and strength, R. Narayanan and T.M. Roberts, eds., Elsevier Applied Science, New York, N.Y.

Kulak, G.L., and Birkemoe, P.C. 1993. Field studies of bolt pretension. Journal of Constructional Steel Research, No. 25: 95-106.

Kulak, G.L., and Dawe, J. L. 1991. Discussion of design interaction equations for steel members. Sohal, I. S., Duan, L., and Chen W-F. ASCE, Journal of Structural Engineering, 117(ST7): 2191-2193.

Kulak, G.L., Fisher, J. W., and Struik, J.H.A. 2001. Guide to design criteria for bolted and riveted joints, Second edition. Research Council on Structural Connections. (Original copyright 1987 by John Wiley & Sons, Inc. transferred to RCSC). *www.boltcouncil.org/files/2ndEditionGuide.pdf*

Kulak, G.L., and Grondin, G.Y., 2000. Block shear failure in steel members – A review of design practice. Proceedings of the Fourth International Workshop on Connections in Steel Structures IV: Steel Connections in the New Millennium. Roanoke, VA., AISC, Chicago.

— 2003. Strength of joints that combine bolts and welds. AISC Engineering Journal, Second quarter, pp. 89-98.

— 2014. Limit states design in structural steel, 9th Edition. Canadian Institute of Steel Construction, Markham, Ontario.

Kulak, G.L., Kennedy, D.J.L., and Driver, R.G. 1994. Discussion to experimental study of thin steel-plate shear walls under cyclic load, by Caccese, V., Elgaaly, M., and Chen, R. Journal of Structural Engineering, ASCE, Vol 120, No.10, pp. 3072-3073.

Kulak, G.L., Kennedy, D.J.L., Driver, R.G., and Medhekar, M. 1999. Behavior of steel plate shear walls. Proceedings, 1999 North American Steel Construction Conference, Toronto.

Kulak, G.L., and Smith, I.F.C. 1993. Analysis and design of fabricated steel structures for fatigue: A primer for civil engineers. Dept. of Civil Eng. SER 190. University of Alberta, Edmonton. AB.

Kulak, G.L., and Undershute, S.T. 1998. Tension control bolts: Strength and installation. Journal of Bridge Engineering, ASCE, 3(1).

Kullman R. B., and Hosain, M. U. 1985. Shear capacity of stub-girders: full-scale tests. ASCE, Journal of Structural Engineering, 111(ST1): 56-75.

Lacerte, M., and Tremblay, R. 2006. Making use of brace overstrength to improve the seismic response of multi-storey split-X concentrically braced steel frames. Can. J. of Civ. Eng., 33, 8, 1005-1021.

Lay, M. G. 1965. Flange local buckling in wide-flange shapes. ASCE Journal of the Structural Division, 91(ST6).

Lay, M. G., and Galambos, T. V. 1966. Bracing requirements for inelastic steel beams. ASCE Journal of the Structural Division, 92(ST2).

— 1967. Inelastic beams under moment gradient. ASCE Journal of the Structural Division, 93(ST1).

Lee, K., and Bruneau, M. 2005. Energy dissipation of compression members in concentrically braced frames: Review of experimental data. J. Struct. Eng., ASCE, 131, 4, 552–559.

Lee, S., and Goel, S.C. 1987. Seismic behavior of hollow and concrete-filled square tubular bracing members. Research Report UMCE 87-11, Dept. of Civ. Eng., Univ. of Michigan, Ann Arbor, Mich.

Lesik, D.F., and Kennedy, D.J.L. 1990. Ultimate strength of fillet welded connections loaded in plane. Canadian Journal of Civil Engineering 17(1), 55-67.

Li, C.-H., Tsai, K.-C., Lin, C.-H., and Chen, P.-C. 2009. Cyclic tests of four two-storey narrow steel plate shear walls. Part 1: Analytical studies and specimen design & Part 2: Experimental results and design implications. Earthquake Eng. and Struct. Dyn. Published Online: 26 Nov 2009.

Lilley, S. B., and Carpenter, S. T. 1940. Effective moment of inertia of a riveted plate girder. Transactions, American Society of Civil Engineers.

Liu, J., Sabelli, R., Brockenbrough, R.L., and Fraser, T.P. 2007. Expected yield stress and tensile strength ratios for determination of expected member capacity in the 2005 AISC seismic provisions. AISC Eng. J., 44, 1, 15-25.

Liu, Z. 1987. Investigation of concrete-filled steel tubes under cyclic bending and buckling. Ph.D. Thesis, Dept. of Civ. Eng., Univ. of Michigan, Ann Arbor, Michigan.

Liu, Z., and Goel, S.C. 1988. Cyclic load behavior of concrete-filled tubular braces. ASCE Journal of Structural Engineering, 114(ST7):1488-1506.

Loov, R. 1996. A simple equation for axially loaded steel column design curves. Canadian Journal of Civil Engineering. 23(1): 272-276.

López, W.A., and Sabelli, R. 2004. Seismic design of buckling-restrained braced frames, Steel Tips, Structural Steel Education Council, Moraga, CA.

Lu, Y.Q., and Kennedy, D.J.L. 1994. The flexural behaviour of concrete-filled hollow structural sections. Canadian Journal of Civil Engineering, 21(1), 111-130.

Lubell, A.S., Prion, H.G.L., Ventura, C.E., and Rezai, M. 2000. Unstiffened steel plate shear wall performance under cyclic loading. J. Struct. Eng., ASCE, 126, 4, 453-460.

Lukey, A. F., and Adams, P. F. 1969. Rotation capacity of beams under moment gradient. ASCE Journal of the Structural Division, 95(ST6).

Lutz, L.A. 1992. Critical slenderness of compression members with effective lengths about non-principal axes. Proceedings of the Annual Technical Session and Meeting of the Structural Stability Research Council, Bethlehem, PA, 107-116.

MacPhedran, I.J., and Grondin, G.Y. 2006. A brief history of beam-column design. 1st International Structural Specialty Conference, Calgary, Alberta, May 23-26.

MacRae, G.A., Kimura, Y., and Roeder, C. 2004. Effect of column stiffness on braced frame seismic behavior, ASCE J. Struct. Eng., Vol. 130, No. 3, 381–391.

Mahin, S., Uriz, P., Aiken, I., Field, C., and Ko, E. 2004. Seismic performance of buckling restrained braced frame systems. Proc. 13th World Conf. on Earthquake Eng., Vancouver, B.C., Canada, Paper No. 1681.

Malley, J.O., and Popov. E.P. 1983. Design considerations for shear links in eccentrically braced steel frames. EERC Report No. UCB/EERC-83/24, Univ. of California, Berkeley, CA.

Manniche, K., and Ward-Hall, G. 1975. Mission bridge – Design and construction of the steel box girder. Canadian Journal of Civil Engineering, 2(2).

Mansour, N. 2010. Development of the design of eccentrically braced frames with replaceable shear links. Ph.D. Thesis, Dept. of Civil Engineering, University of Toronto. Toronto, ON.

Mansour, N., Christopoulos, C., and Tremblay, R. 2011. Experimental validation of replaceable shear links for eccentrically braced frame. Journal of Structural Engineering, ASCE, 137, 10, 1141-1152.

Manuel, T.J., and Kulak, G.L. 1998. Strength of joints that combine bolts and welds. Dept. of Civil Eng. SER 222. University of Alberta, Edmonton, AB.

— 2000. Strength of joints that combine bolts and welds. ASCE Journal of Structural Engineering, 126(3): 279-287.

Marino, E.M., and Nakashima, M. 2006. Seismic performance and new design procedure for chevron-braced frames. Earthquake Eng. and Struct. Dyn., Vol. 35, No. 4, 433-452.

Martinez-Saucedo, G., Packer, J.A., and Christopoulos, C. 2008. Gusset plate connections to circular hollow section braces under inelastic cyclic loading. J. Struct. Eng. 134, 7, 1252-1258.

Martinez-Saucedo, G., and Packer, J.A. 2009. Static design recommendations for slotted end HSS connections in tension. ASCE Journal of Structural Engineering, 135(7): 797-805.

Massey, C. 1962. Lateral bracing forces of steel I-beams. ASCE Engineering Mechanics Division, 88(EM6).

Mattock, A. H. 1974. Shear transfer on concrete having reinforcement at an angle to the shear plane. Special Publication 42, Shear in Reinforced Concrete, American Concrete Institute: 17-42.

Mattock, A. H., Li, W. K., and Wang, T.C. 1976. Shear transfer in lightweight reinforced concrete. PCI Journal, 21(1): 20-39.

Maurer, M.B., and Kennedy D.J.L. 1994. Shrinkage and flexural tests of a full-scale composite truss. Structural Engineering Report 206, Department of Civil Engineering, University of Alberta.

McFadden, M.R. and Packer, J.A. 2014. Effective weld properties for hollow structural section T-connections under branch in-plane bending. Engineering Journal, American Institute of Steel Construction, 51(4), pp. 247-266.

McFadden, M.R., Sun, M. and Packer, J.A. 2013. Weld design and fabrication for RHS connections. Steel Construction, 6(1), pp. 5-10.

Merzouq, S., and Tremblay, R. 2006. Seismic design of dual concentrically braced steel frames for stable seismic performance for multi-storey buildings. Proc. 8th U.S. National Conference on Earthquake Eng., San Francisco, CA, Paper 1909.

Miazga, G.S., and Kennedy, D.J.L. 1989. Behaviour of fillet welds as a function of the angle of loading. Canadian Journal of Civil Engineering 16: 583-599.

Moghimi, H., and Driver, R.G. 2013. Economical steel plate shear walls for low-seismic regions. Journal of Structural Engineering, American Society of Civil Engineers, Vol. 139, no. 3, pp. 379-388.

Munse, W. H. 1959. The effect of bearing pressure on the static strength of riveted connections. Bulletin No. 454, Engineering Experimental Station, University of Illinois, Urbana, IL.

Munse, W. H., and Chesson, E. 1963. Riveted and bolted joints: net section design. ASCE. Journal of the Structural Division, 89(ST1), Part 1.

Murray, T. M. 1975. Design to prevent floor vibrations. Engineering Journal, AISC, third quarter, pp. 83-87.

— 2003. Extended end plate connections – Seismic and wind applications. Steel Design Guide 4, American Institute of Steel Construction, Chicago.

Murray, T.M., Allen, D.E., and Ungar, E.E. 1997. Floor vibrations due to human activity. Steel Design Guide Series 11. American Institute of Steel Construction, Chicago; Canadian Institute of Steel Construction, Toronto.

Nash, D. S., and Kulak, G. L. 1976. Web slenderness limits for non-compact beam-columns. Report No. 53, Department of Civil Engineering, University of Alberta.

NBCC. 2005. National Building Code of Canada. National Research Council of Canada, Ottawa, Ont.

— 2010. National Building Code of Canada. National Research Council of Canada, Ottawa, Ont.

NCHRP. 1970. Effect of weldments on the fatigue strength of steel beams. Report 102, National Cooperative Highway Research Program, Transportation Research Board, National Academy of Sciences, Washington, DC.

— 1974. Fatigue strength of steel beams with welded stiffeners and attachments. Report 147, National Cooperative Highway Research Program, Transportation Research Board, National Academy of Sciences, Washington, DC.

Nethercot, D. A., and Trahair, N. S. 1976a. Inelastic lateral buckling of determinate beams. ASCE Journal of the Structural Division, 102(ST4):701-717.

— 1976b. Lateral buckling approximations for elastic beams. ISE. The Structural Engineer. 54(6): 197-204.

Ng, A.K.F., Deng, K., Grondin, G.Y., and Driver, R.G. 2004a. Behavior of transverse fillet welds: experimental program. Engineering Journal, American Institute of Steel Construction, vol. 41, no. 2, pp. 39-54.

Ng, A.K.F., Driver, R.G., and Grondin, G.Y. 2004b. Behavior of transverse fillet welds: parametric and reliability analyses. Engineering Journal, American Institute of Steel Construction, vol. 41, no. 2, pp. 55-67.

Nixon, D. 1981. The use of frame action to resist lateral loads in simple construction. Canadian Journal of Civil Engineering, 8(4).

Okazaki, T., Arce, G., Ryu, H.-C., and Engelhardt, M.D. 2005. Experimental study of local buckling, overstrength, and fracture of links in eccentrically braced frames. J. Struct. Eng., ASCE, 131, 10, 1526-1535.

Okazaki,T., Engelhardt, M.D., Nakashima, M., and Suita, K. 2006. Experimental performance of link-to-column connections in eccentrically braced frames. Journal of Structural Engineering, ASCE, 132, 8, 1201–1211.

Okazaki, T., and Engelhardt, M.D. 2007. Cyclic loading behavior of EBF links constructed of ASTM A992 steel. Journal of Constructional Steel Research, 63, 751-765.

Olgaard, J. G., Slutter, R. G., and Fisher, J. W. 1971. Shear strength of stud connectors in light-weight and normal-weight concrete. Engineering Journal, American Institute of Steel Construction (AISC), April.

Oosterhof, S.A., and Driver, R.G. 2011. Performance of the unified block shear equation for common types of welded steel connections. Engineering Journal, American Institute of Steel Construction (AISC), Vol. 48, 2nd Quarter, pp. 77-92.

Packer, J.A., Sherman, D. and Lecce, M. 2010. Hollow structural section connections. Steel design Guide 24, American Institute of Steel Construction, Chicago, Illinois.

Packer, J.A., Sun, M., Oatway, P. and Frater, G.S. 2015. Experimental evaluation of the directional strength increase for fillet welds to rectangular hollow sections. Proceedings of the 15th. International Symposium on Tubular Structures, Rio de Janeiro, Brazil.

Parcel, J.I., and Murer, E.B. 1934. The effect of secondary stresses upon ultimate strength, Proceedings (with discussions in Transactions) American Society of Civil Engineers, Reston, Virginia, Vol. 60: 55p.

Perlynn, M. J., and Kulak, G. L. 1974. Web slenderness limits for compact beam-columns. Structural Engineering Report 50, Department of Civil Engineering, University of Alberta.

Popov, E.P., Amin, N.R., Louie, J.J.C., and Stephen, R.M. 1986. Cyclic behaviour of large beam-column assemblies. Engineering Journal, AISC, 23(1): 9-23.

Popov, E.P., and Black, R.G. 1980. Steel struts under severe cyclic loading. J. of the Struct. Div., ASCE, 107(ST9): 1587-1881.

Popov, E.P., Englehardt, M.D., and Ricles, J.M. 1989. Eccentrically braced frames: U.S. practice. AISC Engineering Journal, 36(2): 66-80.

Popov, E. P., and Pinkney, R. B. 1969. Cyclic yield reversal in steel building connections. ASCE Journal of the Structural Division, 95(ST3).

Popov, E. P., and Stephen, R. M. 1972. Cyclic loading of full-size steel connections. Steel Research for Construction, Bulletin No.21, AISI.

— 1977a. Capacity of columns with splice imperfections. Engineering Journal, AISC, 14(1).

— 1977b. Tensile capacity of partial penetration groove welds. ASCE. Journal of the Structural Division, 103(ST9).

Prickett, B.S., and Driver, R.G. 2006. Behaviour of partially encased composite columns made with high performance concrete. Structural Engineering Report No. 262, January, Department of Civil and Environmental Engineering, University of Alberta, Edmonton, Canada.

Purba, R., Bruneau, M., 2007. Design recommendations for perforated steel plate shear walls. Technical Report MCEER-07-0011, MCEER, University at Buffalo, Buffalo, NY.

— 2009. Finite-element investigation and design recommendations for perforated steel plate shear walls. J. Struct. Eng., ASCE, 135, 11, 1367-1376.

Qu, B., and Bruneau, M. 2009. Design of steel plate shear walls considering boundary frame moment resisting action. J. Struct. Eng., ASCE, 135, 12, 1511-1521.

Qu, B., Bruneau, M., Lin, C.H., Tsai, K.C. 2008. Testing of full scale two-story steel plate shear walls, with RBS connections and composite floor. J. Struct. Eng., ASCE, 134, 3, 364-373.

Rainer, J. H. 1980. Dynamic tests on a steel-joist concrete-slab floor. Canadian Journal of Civil Engineering, 7(2).

RCSC. 2004. Specification for structural joints using ASTM A325 or A490 bolts. Research Council on Structural Connections. *www.boltcouncil.org/files/2004RCSCSpecification.pdf*

Redwood, R. G. 1971. Simplified plastic analysis for reinforced web holes. Eng. J., AISC, 8(4).

— 1972. Tables for plastic design of beams with rectangular holes. Engineering Journal, AISC, 9(1).

— 1973. Design of beams with web holes. Canadian Steel Industries Construction Council, Willowdale, Ont.

Redwood, R. G., and Channagiri, V.S. 1991. Earthquake resistant design of concentricity braced steel frames. Canadian Journal of Civil Engineering, 18(5).

Redwood, R. G., Feng Lu, Bouchard, G., and Paultre, P. 1991. Seismic response of concentrically braced steel frames. Canadian Journal of Civil Engineering, 18(6).

Redwood, R., and McCutcheon, J. 1968. Beam tests with unreinforced web openings. ASCE Journal of the Structural Division, 94(ST1).

Remennikov, A.M., and Walpole, W.R. 1998a. Seismic behavior and deterministic design procedures for steel V-braced frames. Earthquake Spectra, Vol. 14, No. 2, 335-355.

— 1998b. A note on compression strength reduction factor for a buckled strut in seismic-resisting braced system. Eng. Struct., 20, 8, 779–782.

Richards, P.W. 2009. Seismic column demands in ductile braced frames. J. Struct. Eng., ASCE, 135, 1, 33–41.

Ricles, J.M., and Yura, J.A. 1983. Strength of double-row bolted web connections. Journal of Structural Engineering, ASCE, 109(ST1): 126-142.

Ricles, J.M., Zhang, X., Lu, L.-W. and Fisher, J. 2004. Development of seismic guidelines for deep-column steel moment connections. ATLSS Report No. 04-13, Lehigh University, Bethlehem, PA.

Ritchie, J.K., and Chien, E.Y.L. 1980. Composite structural systems – Design, construction and cost considerations. Proceedings, Canadian Structural Engineering Conference, CISC.

Roberts, T., and Sabouri-Ghomi, S. 1992. Hysteretic characteristics of unstiffened perforated steel plate shear panels. Thin-Walled Struct., 14, 139–151.

Robinson, H. 1969. Composite beam incorporating cellular steel decking. ASCE Journal of the Structural Division, 95(ST3).

— 1988. Multiple stud shear connections in deep ribbed metal deck. Canadian Journal of Civil Engineering, 15(4).

Robinson, H., and Wallace, I. W. 1973. Composite beams with 1½ inch metal deck and partial and full shear connection. Transactions, Canadian Society for Civil Engineering, 16(A-8), published in the Engineering Journal, Engineering Institute of Canada.

Sabelli, R. 2004, Recommended provisions for buckling-restrained braced frames, AISC Eng. J., Vol. 41, No. 4, pp. 155-175.

Sabelli, R., and Bruneau, M. 2007. Steel plate shear walls. Steel Design Guide 20, AISC, Chicago, IL.

Sabelli, R. and Hohbach, D. 1999. Design of cross-braced frames for predictable buckling behavior. J. of Struct. Eng., ASCE, 125(ST2): 163-168.

Sabelli, R., Mahin, S., and Chang, C. 2003. Seismic demands on steel braced frame buildings with buckling restrained braces. Engineering Struct., 25, 5, 656-666.

Sabouri-Ghomi, S., Kharrazi M.H.K., Mam-Azizi, S.-E-D., and Sajadi, R.A. 2008. Buckling behavior improvement of steel plate shear wall systems. The Structural Design of Tall and Special Buildings, 17, 4, 823-837.

Sabouri-Ghomi, S., and Roberts, T.M. 1991. Nonlinear dynamic analysis of thin steel plate shear walls. Computers & Structures, 39, 1-2, 121-127.

Schmidt, B.J. 2000. Review of the resistance factor for steel. M.Sc. thesis, Department of Civil and Environmental Engineering, University of Western Ontario, London, Ontario.

Schmidt B.J., and Bartlett, F.M. 2002. Review of resistance factor for steel: data collection. Can. J. Civ. Eng., 29:(1) 98–108.

Schmidt, B.J., and Bartlett, F.M. 2002. Review of resistance factor for steel: resistance distributions and resistance factor calibration. Canadian Journal of Civil Engineering, 29:(1) 109–118.

Schmitke, C.D., and Kennedy, D.J.L. 1985. Effective lengths of laterally continuous, laterally unsupported steel beams. Canadian Journal of Civil Engineering. 12(3): 603-616.

Schumacher, A. S., Grondin, G. Y., and Kulak, G. L. 1999. Connection of infill panels in steel plate shear walls. Canadian Journal of Civil Engineering, 26(5): 549-563.

Seaburg, P.A., and Carter, C.J. 1997. Torsional analysis of structural steel members, Steel design guide series 9, AISC, Chicago, IL.

Seeley, F. B., and Smith, J. O. 1957. Advanced mechanics of materials (2nd ed.). John Wiley & Sons, Inc., New York, NY. pp. 365-367.

Shaker, A.F., and Kennedy, D.J.L. 1991. The effective modulus of elasticity of concrete in tension. Structural Engineering Report 172, Department of Civil Engineering, University of Alberta, Edmonton, Alberta.

Sherman, D.R. 1996. Designing with Structural Tubing. Engineering Journal, AISC, 33: 101-109.

Sherman, D. R., and Ghorbanpoor, A. 2002. Final report: Design of extended shear tabs. American Institute of Steel Construction (*available via download from AISC website*).

Sherman, D. R., and Tanavde, A. S. 1984. Comparative study of flexural capacity of pipes. Department of Civil Engineering, University of Wisconsin-Milwaukee.

Shishkin, J.J., Driver, R.G., and Grondin, G.Y. 2005. Analysis of steel plate shear walls using the modified strip model. Structural Engineering Report No. 261, Dept. of Civ. and Env. Eng., Univ. of Alberta, Edmonton, AB.

— 2009. Analysis of steel plate shear walls using the modified strip model. Journal of Structural Engineering, American Society of Civil Engineers, vol. 135, no. 11, pp. 1357-1366.

Shrivastava, S. C., Redwood, R. G., Harris, P. J., and Ettehadieh, A. A. 1979. End moments in open web steel tie joists. McGill University, June.

Skarborn, S., and Daneff G. 1998. Shear resistance of flare bevel and puddle welds in open-web steel joist applications. Canadian Society for Civil Engineering, Annual Conference, Halifax NS, 10 pp.

Slutter, R. G., and Driscoll, G. C. 1965. Flexural strength of steel concrete composite beams. ASCE Journal of the Structural Division, 95(ST2).

Sourochnikoff, B. 1950. Wind-stresses in semi-rigid connections of steel framework. Transactions, American Society of Civil Engineers.

SSPC. Good Painting Practice, SSPC Painting Manual, Vol. 1, Fourth Edition, Society for Protective Coatings, Pittsburgh, PA.

— Systems and Specifications, SSPC Painting Manual, Vol. 2, Eighth Edition, Society for Protective Coatings, Pittsburgh, PA.

Stankevicius, J., Josi, G., Grondin, G.Y., and Kulak, G.L. 2009. Measurement of slip coefficient for grade ASTM A588 steel. Structural Engineering Report 268, Department of Civil and Environmental Engineering, University of Alberta.

STELCO. 1973. Hollow structural sections – Design manual for columns and beams. The Steel Company of Canada Limited, Hamilton, Ont.

— 1981. Hollow structural sections – Design manual for concrete-filled HSS columns. The Steel Company of Canada Limited, Hamilton, Ont.

Stratan, A., and Dubina, D. 2004. Bolted links for eccentrically braced steel frames. Proceedings 5th Int. Workshop on Connections in Steel Structures V, Delft Univ. of Technology, Netherlands, 223–232.

Tall, L., *et al*. 1974. Structural steel design (2nd ed.). The Ronald Press Company, New York, NY.

Tang, X., and Goel, S.C. 1989. Brace fractures and analysis of phase I structure. J. of Struct. Engrg., ASCE, 115, 1960-1976.

Temple, M.C., Schepers, J.A., and Kennedy, D.J.L. 1986. Interconnection of starred angle compression members. Canadian Journal of Civil Engineering, 13(6).

Thorburn, L. J., Kulak, G. L., and Montgomery, C. J. 1983. Analysis of steel plate shear walls. Structural Engineering. Report No. 107, Department of Civil Engineering, Univ. of Alberta, Edmonton, Alberta.

Thornton, C. H. 1973. Quality control in design and supervision can eliminate lamellar tearing. Engineering Journal, AISC, Fourth Quarter.

Thornton, W.A., and Fortney, P.J. 2011. On the need for stiffeners for and the effect of lap eccentricity on extended single-plate connections. AISC Engineering Journal, 2nd Quarter, pp. 117-125.

Timler, P. A., and Kulak, G. L. 1983. Experimental study of steel plate shear walls. Structural Engrg. Report No. 114, Department of Civil Engineering, Univ. of Alberta, Edmonton, Alberta.

Timoshenko, S.P., and Gere, J.M. 1961. Theory of elastic stability, Second edition, McGraw-Hill, New York, NY .

Toprac, A., and Natarajan, M. 1971. Fatigue strength of hybrid plate girders. ASCE Journal of the Structural Division, 97(ST4).

Tousignant, K. and Packer, J.A. 2015. Investigation of weld effective length rules for RHS overlapped K-connections. Proceedings of the 15th. International Symposium on Tubular Structures, Rio de Janeiro, Brazil.

Trahair, N.S. 1968. Interaction buckling of narrow rectangular continuous beams. Civil Engineering Trans. Inst. of Eng. Australia.

Tremblay, R. 1997. Seismic performance of RBS connections for steel moment resisting frames: Influence of loading rate and floor slab. Proceedings of Second International Conference on Behaviour of Steel Structures in Seismic Areas, Kyoto, Japan.

— 1998. Conception des cadres rigides en acier – Développements survenus suite aux séismes de Northridge et de Kobe. Calcul parasismique des charpentes d'acier/Earthquake resistant design of steel structures, Seminar notes.

— 2000. Influence of brace slenderness on the seismic response of concentrically braced steel frames. Behaviour of steel structures in seismic areas. Proceedings of the Third STESSA Conference 2000, Montreal, A.A. Balkema, Rotterdam: 527-534.

— 2001. Seismic behaviour and design of concentrically braced steel frames. AISC Eng. J., 38 (3): 148-166.

— 2002. Inelastic seismic response of bracing members. J. of Const. Steel Research. 58: 665-701.

— 2003. Achieving a stable inelastic seismic response for concentrically braced steel frames. Eng. J., AISC, 40(2): 111-129.

Tremblay, R., Archambault, M.-H., and Filiatrault, A. 2003. Seismic response of concentrically braced steel frames made with rectangular hollow bracing members. Journal of Structural Engineering, ASCE, 129(12): 1626-1636.

Tremblay, R., Bolduc. P., Neville, R., and DeVall, R. 2006. Seismic testing and performance of buckling restrained bracing systems, Can. J. of Civ. Eng., 33, 2, 183-198.

Tremblay, R., Bruneau, M., Nakashima, M., Prion, H.G.L., Filiatrault, A. and DeVall, R. 1996. Seismic design of steel buildings: Lessons from the 1995 Hyogoken-Nanbu earthquake. Can. J. of Civ. Eng., 23: 727-756.

Tremblay, R., Castonguay, P.X., Guilini-Charette, K., and Koboevic, S. 2009. Seismic performance of conventional construction braced steel frames designed according to Canadian seismic provisions. Proc. 2009 ASCE Structures Congress, Austin, TX, 341, 87.

Tremblay, R., Chicoine, T., and Massicotte, B. 2000. Design equation for the axial capacity of partially encased non-compact columns. Proceedings Composite Construction in Steel and Concrete IV, Banff, Canada, June, United Engineering Foundation, ASCE, Reston, VA: 506-517.

Tremblay, R., Degrange, G., and Blouin, J. 1999. Seismic rehabilitation of a four-storey building with a stiffened bracing system. Proc. 8th Can. Conf. on Earthquake Eng., Vancouver, B.C., 549-554.

Tremblay, R., Haddad, M., Martinez, G., Richard, J., and Moffatt, K. 2008. Inelastic cyclic testing of large size steel bracing members. Proc. 14WCEE, Beijing, China, Paper No. 05-05-0071.

Tremblay, R., Massicotte, B., Filion, I., and Maranda, R. 1998. Experimental study on the behaviour of partially encased composite columns made of light welded H steel shapes under compressive axial loads. Proc. Annual Meeting, Structural Stability Research Council, Atlanta, Sept. 21-23: 195-204.

Tremblay, R., and Poncet, L. 2007. Improving the seismic stability of concentrically braced steel frames. Eng. J., AISC, 44, 2, 103-116.

Tremblay, R., and Robert, N. 2000. Design of low- and medium-rise chevron braced steel frames. Can. J. of Civ. Eng., 27(6): 1192-1206.

— 2001. Seismic performance of low- and medium-rise chevron braced steel frames. Can. J. of Civ. Eng. 28(4): 699-714.

Tremblay, R., Rogers, C., Martin, É., and Yang, W. 2004. Analysis, testing and design of steel roof deck diaphragms for ductile earthquake resistance. Journal of Earthquake Engineering, Imperial College Press, 8(5): 775-816.

Tremblay, R., Timler, P., Bruneau, M., and Filiatrault, A. 1995. Performance of steel structures during the January 17, 1994 Northridge earthquake. Can. J. of Civ. Eng., 22, 338-360.

Tromposch, E. W., and Kulak, G. L. 1987. Cyclic and static behaviour of thin panel steel plate shear walls. Structural Engineering Report No. 145, Department of Civil Engineering, University of Alberta.

Tsai, K.C., and Hsiao, P.-C. 2008. Pseudo-dynamic test of a full-scale CFT/BRB frame – Part II: Seismic performance of buckling-restrained braces and connections. Earthquake Eng. and Struct. Dyn., 37, 7, 1099–1115.

Uang, C.-M., and Nakashima, M. 2004. Steel buckling-restrained braced frames. In Earthquake Engineering – From Engineering Seismology to Performance-Based Engineering (ed. Y. Bozorgnia and V. V. Bertero), Chapter 16, CRC Press, Boca Raton.

Vian, D., and Bruneau, M. 2004. Testing of special LYS steel plate shear walls. Proc. 13th World Conf. on Earthquake Eng., Vancouver, Canada, Paper No. 978.

Vian, D., Bruneau, M., Tsai, K.-C., and Lin, U.C. 2009. Special perforated steel plate shear walls with reduced beam section anchor beams. I: Experimental investigation, & II: Analysis and design recommendations, J. of Struct. Eng., ASCE, 135, 3, 211-228.

Vincent, G. S. 1969. Tentative criteria for load factor design of steel highway bridges. Steel Research Construction Bulletin No. 15, AISI, Washington, DC.

Virdi, K. S., and Dowling, P. J. 1976. A unified design method for composite columns. Memoires, IABSE, No. 36-11.

Wakabayashi, M. 1977 A new design method of long composite beam-columns. Proceedings, ASCE International Colloquium on Stability of Structures under Static and Dynamic Loads, Washington, DC., May.

White, D.W., and Jung, S.-K. 2004. Unified flexural resistance equations for stability design of steel I-section members – uniform bending tests. Structural Engineering, Mechanics and Materials Report No. 04-28. Georgia Institute of Technology.

Winter, G. 1958. Lateral bracing of columns and beams. ASCE Journal of the Structural Division, 84(ST2).

— 1970. Commentary on the 1968 edition of the specification for the design of cold-formed steel structural members. American Iron and Steel Institute, Washington, DC.

Wong, E., and Driver, R.G. 2010. Critical evaluation of equivalent moment factor procedures for laterally unsupported beams. Engineering Journal, American Institute of Steel Construction, vol. 47, no. 1.

Wong, E., Driver, R.G., and Heal, T. 2014. Simplified approach to estimating the elastic lateral–torsional buckling capacity of steel beams with top-flange loading. Canadian Journal of Civil Engineering, 42: in press.

Wu, Y., and Kulak, G. L. 1993. Shear lag in bolted single and double angle tension members. Structural Engineering Report, Department of Civil Engineering, University of Alberta, SER 187.

Yang, C.H., and Beedle, L.S. 1951. Behavior of I and WF beams in shear. Fritz Engineering Lab. Report No. 205B21, Lehigh Univ., Bethlehem, Pa.

Yang, F., and Mahin, S. 2005. Limiting net section fracture in slotted tube braces. Steel Tips, Technical Information and Product Service, Structural Steel Educational Council, Moraga, CA.

Yong Lin Pi and Trahair, N.S. 1995. Inelastic torsion of steel I-beams. ASCE Journal of Structural Engineering, 121(ST4).

Yura, J.A. 1993. Fundamentals of Beam Bracing. Proceedings, Structural Stability Research Council Conference "Is Your Structure Suitably Braced?", Milwaukee, WI, April.

— 1995. Bracing for Stability – State-of-the-Art. Proceedings, Structures Congress XIII, American Society of Civil Engineers, Boston, MA, April: 88-103.

Yura, J.A., Birkemoe, P.C., and Ricles, J.M. 1982. Web shear connections: an experimental study. American Society of Civil Engineers. Journal of the Structural Division, 108(ST2).

Zhao, Q., and Astaneh-Asl, A. 2004. Cyclic behavior of traditional and innovative composite shear walls. J. Struct. Eng., ASCE, 130, 2, 271–284.

Ziemian, R.D. (Editor). 2010. Guide to stability design criteria for metal structures, 6th Edition. John Wiley & Sons, Inc., Hoboken, NJ.

Zuk, W. 1956. Lateral bracing forces on beams and columns. Engineering Mechanics Division, American Society of Civil Engineers, 82(EM3).