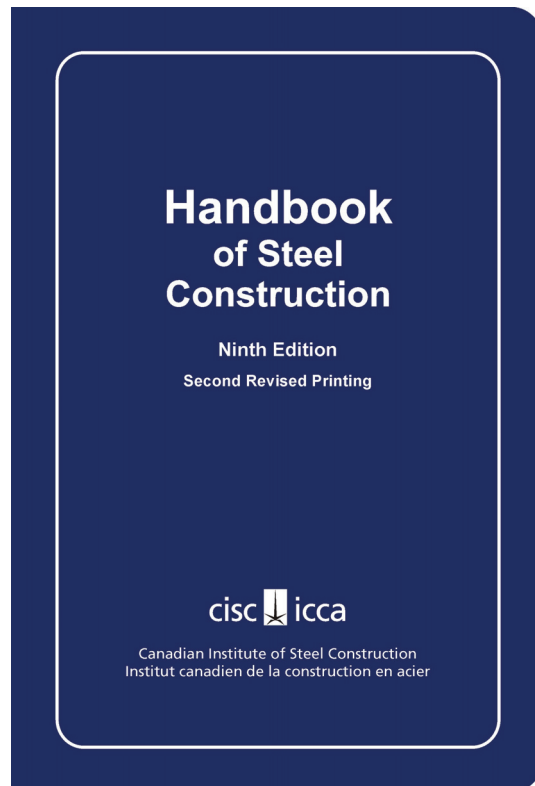


Handbook of Steel Construction

9th Edition, 2nd Revised Printing 2006

REVISIONS



Intended for users of the Handbook, 9th Edition, 1st printing, the following replacement pages contain the revisions appearing in the 2nd revised printing.

Most of the changes occur in Part 1 - CAN/CSA S16-01 “Limit States Design of Steel Structures” due to the incorporation of CSA Update #3, August 2006.

PART ONE
CAN/CSA-S16-01
LIMIT STATES DESIGN OF STEEL STRUCTURES
(Including S16S1-05, Supplement #1)

General

This Standard is reprinted with the permission of the Canadian Standards Association and contains all supplements, errata and revisions issued at time of printing.

The reprint is of the CSA publication titled “CAN/CSA-S16-01 CONSOLIDATION”. It consists of the CSA standard CAN/CSA-S16-01, Limit States Design of Steel Structures along with S16S1-05, Supplement #1 to CAN/CSA-S16-01 and replacement pages issued June 2003, December 2003 and August 2006 as Update #1, Update #2 and Update #3 to CAN/CSA-S16-01 incorporated into the original 2001 standard. The superseded pages are not included in the Handbook of Steel Construction.

The reference to the Standard in other parts of the 9th Edition of the Handbook of Steel Construction correctly remains as CAN/CSA-S16-01.

CSA Standards are subject to periodic review, and amendments will be published by CSA from time to time as warranted.

For information on requesting interpretations, see Note (5) to the Preface to CAN/CSA-S16-01.

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- A_w = web area; shear area; effective throat area of a weld
 a = centre-to-centre distance between transverse web stiffeners; depth of the concrete compression zone
 a' = length of cover plate termination
 a/h = aspect ratio; ratio of distance between stiffeners to web depth
 B = bearing force in a member or component under specified load; overstrength factor for ductile plate walls
 B_f = bearing force in a member or component under factored load
 B_r = factored bearing resistance of a member or component
 b = width of stiffened or unstiffened compression elements; design effective width of concrete or cover slab
 b_c = width of concrete at the neutral axis defined in Clause 18.2.3; width of column flange
 b_e = effective flange width in Clause 18.3.2
 b_f = width of flange
 C = compressive force in a member or component under specified load; axial load
 C_e = Euler buckling strength
 = π^2EI/L^2
 C_{ec} = Euler buckling strength of a concrete-filled hollow structural section
 C_f = compressive force in a member or component under factored load; factored axial load
 C_{fs} = sustained axial load on a composite column
 C_p = nominal compressive resistance of a composite column when $\lambda = 0$ (see Clause 18.3.2)
 C_r = factored compressive resistance of a member or component; factored compressive resistance of steel acting at the centroid of that part of the steel area in compression
 C_{rc} = factored compressive resistance of a composite column
 C_{rcm} = factored compressive resistance that can coexist with M_{rc} when all of the cross-section is in compression
 C_{rco} = factored compressive resistance with $\lambda = 0$
 C'_r = compressive resistance of concrete acting at the centroid of the concrete area assumed to be in uniform compression; compressive resistance of a concrete component of a composite column
 C_w = warping torsional constant (mm^6)
 C_y = axial compressive load at yield stress
 c_1 = coefficient used to determine slip resistance
 D = outside diameter of circular sections; diameter of rocker or roller; stiffener factor; dead load
 d = depth; overall depth of a section; diameter of a bolt or stud
 d_b = depth of beam
 d_c = depth of column
 E = elastic modulus of steel (200 000 MPa assumed); earthquake load and effects (see Clause 6.2.1)
 E_c = elastic modulus of concrete

- E_{ct} = effective modulus of concrete in tension
 e = end distance; lever arm between the compressive resistance, C_r , and the tensile resistance, T_r ; length of link in eccentrically braced frames
 e' = lever arm between the compressive resistance, C_r' , of concrete and tensile resistance, T_r , of steel
 F = strength or stress
 $\Delta\Delta$ F_a = acceleration-based site coefficient (see the *National Building Code of Canada, 2005*)
 F_{cr} = critical plate-buckling stress in compression, flexure, or shear
 F_{cre} = elastic critical plate-buckling stress in shear
 F_{cri} = inelastic critical plate-buckling stress in shear
 F_e = Euler buckling stress
 F_s = ultimate shear stress
 F_{sr} = allowable stress range in fatigue
 F_{srt} = constant amplitude threshold stress range
 F_{st} = factored axial force in the stiffener
 F_u = specified minimum tensile strength
 $\Delta\Delta$ F_v = velocity-based site coefficient (see the *National Building Code of Canada, 2005*)
 F_y = specified minimum yield stress, yield point, or yield strength
 F_y' = yield level, including effect of cold-working
 F_{yr} = specified yield strength of reinforcing steel
 f_c' = specified compressive strength of concrete at 28 days
 f_{sr} = calculated stress range at detail due to passage of the fatigue load
 G = shear modulus of steel (77 000 MPa assumed)
 g = transverse spacing between fastener gauge lines (gauge distance)
 H = weld leg size; permanent load due to lateral earth pressure (see Clause 6.2.1)
 h = clear depth of web between flanges; height of stud; storey height
 h_c = clear depth of column web
 h_d = depth of steel deck
 h_s = storey height
 I = importance factor; moment of inertia
 $\Delta\Delta$ I_E = earthquake importance factor of the structure (see the *National Building Code of Canada, 2005*)
 I_c = moment of inertia of a column
 I_e = effective moment of inertia of a composite beam
 I_g = moment of inertia of a cover-plated section
 I_s = moment of inertia of OWSJ or truss
 I_t = transformed moment of inertia of a composite beam
 J = St. Venant torsion constant
 K = effective length factor
 K_z = effective length factor for torsional buckling
 KL = effective length

- k = distance from outer face of flange to web-toe of fillet of I-shaped sections; factor as defined in Clause 18.3.2
 k_a = coefficient used in determining inelastic shear resistance
 k_b = buckling coefficient; required stiffness of the bracing assembly
 k_s = mean slip coefficient
 k_v = shear buckling coefficient
 L = length; length of longitudinal or flare bevel groove weld; live load; length of connection in direction of loading
 L_c = length of channel shear connector
 L_{cr} = maximum unbraced length adjacent to a plastic hinge
 M = bending moment in a member or component under specified load
 M_f = bending moment in a member or component under factored load
 M_{fc} = bending moment in a girder, under factored load, at theoretical cut-off point
 M_{f1} = smaller factored end moment of a beam-column; factored bending moment at a point of concentrated load
 M_{f2} = larger factored end moment of a beam-column
 M_p = plastic moment resistance
 = ZF_y
 M_{pb} = plastic moment of a beam
 M_{pc} = plastic moment of a column
 M_r = factored moment resistance of a member or component
 M_{rc} = factored moment resistance of a composite beam; factored moment resistance of a column reduced for the presence of an axial load
 M_u = critical elastic moment of a laterally unbraced beam
 M_y = yield moment resistance
 = SF_y
 m = number of faying surfaces or shear planes in a bolted joint
 = 1.0 for bolts in single shear
 = 2.0 for bolts in double shear
 N = length of bearing of an applied load; number of passages of moving load
 N' = number of passages of moving load at which $F_{sr} = F_{srt}$
 N_{fi} = number of cycles that would cause failure at stress range level i
 n = number of bolts; number of shear connectors required between the point of maximum positive bending moment and the adjacent point of zero moment; parameter for compressive resistance; number of threads per inch; number of stress range cycles at a given detail for each passage of the moving load; modular ratio, E/E_c
 n' = number of shear connectors required between any concentrated load and nearest point of zero moment in a region of positive bending moment
 n_t = modular ratio, E/E_{ct}

- P = force to be developed in a cover plate; pitch of threads; permanent effects caused by prestress (see Clause 6.2.1)
 P_f = axial force
 p = fraction of full shear connection
 Q_r = sum of the factored resistances of all shear connectors between points of maximum and zero moment
 q_r = factored resistance of a shear connector
 q_{rr} = factored resistance of a shear connector in a ribbed slab
 q_{rs} = factored resistance of a shear connector in a solid slab
 R = end reaction or concentrated transverse load applied to a flexural member; nominal resistance of a member, connection, or structure; transition radius
 R_d = ductility-related force modification factor that reflects the capability of a structure to dissipate energy through inelastic behaviour (see the *National Building Code of Canada, 2005*)
 R_o = overstrength-related force modification factor that accounts for the dependable portion of reserve strength in a structure (see the *National Building Code of Canada, 2005*)
 R_y = factor applied to F_y to estimate the probable yield stress
 r = radius of gyration
 r_y = radius of gyration of a member about its weak axis
 S = elastic section modulus of a steel section; variable load due to snow (see Clause 6.2.1)
 $\Delta\Delta S_a(T)$ = 5% damped spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period of T in seconds (see the *National Building Code of Canada, 2005*)
 s = centre-to-centre longitudinal spacing (pitch) of any two successive fastener holes; longitudinal stud spacing; vertical spacing of tie bars (see Clause 18.3.1)
 T = tensile force in a member or component under specified load; load effects due to contraction, expansion, or deflection (see Clause 6.2.1)
 T_f = tensile force in a member or component under factored load
 T_r = factored tensile resistance of a member or component; in composite construction, factored tensile resistance of the steel acting at the centroid of that part of the steel area in tension
 T_y = axial tensile load at yield stress
 t = thickness; thickness of flange; average flange thickness of channel shear connector
 t_b = thickness of beam flange
 t_c = concrete or cover slab thickness; thickness of column flange
 t_p = thickness of plate
 U_1 = factor to account for moment gradient and for second-order effects of axial force acting on the deformed member
 U_2 = amplification factor to account for second-order effects of gravity loads acting on the laterally displaced storey
 V = shear force in a member or component under specified load
 V_f = shear force in a member or component under factored load
 V_h = total horizontal shear to be resisted at the junction of the steel section or joist and the slab or steel deck; shear acting at plastic hinge locations when plastic hinging occurs

- (l) the specified yield strength of reinforcement, F_{yr} , does not exceed 400 MPa; and
 (m) the clear height-to-width ratio of the column does not exceed 14.

18.3.2 Compressive Resistance

The factored compressive resistance of a partially encased three-plate built-up composite column shall be taken as

$$C_{rc} = (\phi A_{se} F_y + 0.80 \phi_c A_c f'_c + \phi_r A_r F_{yr}) (1 + \lambda^{2n})^{-1/n}$$

where

A_{se} = the effective steel area of the steel section

$$A = (d - 2t + 2b_e)t$$

where

$$b_e = \frac{b_f}{(1 + \lambda_p^3)^{1/1.5}} \leq b_f$$

where

$$\lambda_p = \frac{b_f}{t} \sqrt{\frac{F_y}{720\,000\,k}}$$

where

$$k = \frac{0.9}{(s/b_f)^2} + 0.2(s/b_f)^2 + 0.75$$

A_r = the area of longitudinal reinforcement

$$\lambda = \sqrt{\frac{C_p}{C_{ec}}}$$

where

C_p = C_{rc} computed with ϕ , ϕ_c , and $\phi_r = 1.0$ and $\lambda = 0$

C_{ec} = value defined in Clause 18.2.2

$$n = 1.34$$

18.3.3 Special Reinforcement for Seismic Zones

ΔΔ 18.3.3.1

Columns larger than 500 mm in depth in buildings where the specified one-second spectral acceleration ratio ($I_E F_v S_a(1.0)$) is greater than 0.30 shall be reinforced with longitudinal and transverse bars.

18.3.3.2

The longitudinal bars shall

- have an area not less than 0.005 times the total gross cross-sectional area;
- be at least two in number in each cell; and
- be positioned against the tie bars and at a spacing not greater than the tie spacing, s .

18.3.3.3

The transverse bars shall

- (a) be U-shaped 15M bars arranged to provide corner support to at least every alternate longitudinal bar in such a way that no unsupported longitudinal bar is farther than 150 mm clear from a laterally supported bar;
- (b) have ends welded to the web of the steel shape, in line with the ends of the transverse bars located in the opposite cell, or ends anchored within the concrete core located on the opposite side of the web; and
- (c) have a vertical spacing not greater than the tie spacing, s , or 16 times the diameter of the smallest longitudinal bar.

18.4 Encased Composite Columns**18.4.1 Scope**

Clause 18.4 applies to doubly symmetrical steel columns encased in concrete, provided that

- (a) the steel shape is a Class 1, 2, or 3 section;
- (b) $A_s \geq 0.04$ of the gross cross-sectional area;
- (c) $A_s + A_r \leq 0.20$ of the gross cross-sectional area;
- (d) the concrete is of normal density and has a compressive strength, f'_c , between 20 and 55 MPa;
- (e) the specified yield strength of structural steel, F_y , does not exceed 350 MPa; and
- (f) the specified yield strength of reinforcement, F_{yr} , does not exceed 400 MPa.

18.4.2 Compressive Resistance

The factored compressive resistance of a steel concrete encased composite column shall be taken as

$$C_{rc} = (\phi A_s F_y + \phi_c 0.85 A_c f'_c + \phi_r A_r F_{yr}) (1 + \lambda^{2n})^{-1/n}$$

where

A_r = value defined in Clause 18.3.2

λ = value as defined in Clause 18.3.2

n = value defined in Clause 18.3.2

18.4.3 Reinforcement**18.4.3.1**

The concrete encasement shall be reinforced with longitudinal bars and lateral ties extending completely around the structural steel core. The clear cover shall not be less than 40 mm.

The longitudinal bars shall

- (a) be continuous at framed levels when considered to carry load;
- (b) have an area not less than 0.01 times the total gross cross-sectional area; and
- (c) be located at each corner, and spaced on all sides not further apart than one-half of the least dimension of the composite section.

18.4.3.2

The lateral ties shall

- (a) be 15M bars except that 10M bars may be used when no side dimension of the composite section exceeds 500 mm; and

26.3.5 Limited Number of Cycles

Except for fatigue-sensitive details with high stress ranges (probably with stress reversal), no special considerations beyond those given in Clause 26.1 need apply in the event that the number of stress range cycles, nN , over the life of the structure, expected to be applied at a given detail is less than the greater of

$$\frac{\gamma}{f_{sr}^3} \text{ or } 20\,000$$

26.4 Distortion-Induced Fatigue

26.4.1

Members and connections shall be detailed to minimize distortion-induced fatigue that may occur in regions of high strain at the interconnection of members undergoing differential displacements. Whenever practicable, all components that make up the cross-section of the primary member shall be fastened to the interconnection member.

26.4.2

Plate girders with $h/w > 3150/\sqrt{F_y}$ shall not be used under fatigue conditions.

27. Seismic Design Requirements

27.1 General

ΔΔ 27.1.1

Clause 27 provides requirements for the design of members and connections in the seismic-force-resisting system of steel-framed buildings. With the exception of Clause 27.10, Clause 27 applies to buildings for which seismic design loads are based on a ductility-related force modification factor, R_d , greater than 1.5. Clause 27 is to be applied in conjunction with the requirements of Subsection 4.1.8 of the *National Building Code of Canada, 2005*. Alternatively, the maximum anticipated seismic loads may be determined from non-linear time-history analyses using appropriate structural models and ground motions. No height restrictions are applicable when the seismic forces are determined from non-linear time-history analyses or for buildings with specified short-period spectral acceleration ratios ($I_E F_a S_a(0.2)$) less than 0.35, unless specifically stated in Clause 27 or in the *National Building Code of Canada, 2005*.

27.1.2

Unless otherwise specified, seismic-force-resisting systems shall be designed according to capacity design principles to resist the maximum anticipated seismic loads, but such loads need not exceed the values corresponding to $R_d R_o = 1.3$.

In capacity design

- (a) specific elements or mechanisms are designed to dissipate energy;
- (b) all other elements are sufficiently strong for this energy dissipation to be achieved;
- (c) structural integrity is maintained;
- (d) elements and connections in the horizontal and vertical load paths are designed to resist these seismic loads;
- (e) diaphragms and collector elements are capable of transmitting the loads developed at each level to the vertical seismic-force-resisting system; and
- (f) these loads are transmitted to the foundation.

27.1.3

Any element that significantly affects the load path or the seismic response shall be considered in the analysis and shown on the structural drawings.

Δ 27.1.4

Structural members and their connections that are not considered to form part of the seismic-force-resisting system shall be capable of supporting gravity loads when subjected to seismically induced deformations.

Note: *The gravity loads to be supported are those considered in combination with the earthquake loading.*

27.1.5**27.1.5.1**

Steel used in the energy-dissipating elements described in Clauses 27.2 to 27.8 shall conform to Clauses 5.1.3 and 8.6(a), and F_y shall not exceed 350 MPa, unless the suitability of the steel is determined by testing or by other rational means. F_y shall not exceed 480 MPa in columns in which the only expected inelastic behaviour is at the column base. Other material may be used if approved by the regulatory authority.

Note: F_y is the specified minimum yield stress as defined in Clause 2.2. See Clause 5.1.2.

ΔΔ 27.1.5.2

For buildings with specified short-period spectral acceleration ratios ($I_E F_a S_a(0.2)$) greater than 0.55, rolled shapes with flanges 40 mm or thicker, or plates and built-up shapes over 51 mm in thickness, used in energy-dissipating elements or welded parts shall have a minimum average Charpy V-Notch impact test value of 27 J at 20°C, unless it can be demonstrated that tensile stresses, including local effects, are not critical. The impact tests shall be conducted in accordance with CSA Standard G40.20, with the following exceptions:

- (a) the central longitudinal axis of the test specimens in rolled shapes shall be located as near as practicable to midway between the inner flange surface and the centre of the flange thickness at the intersection with the web mid-thickness; and
- (b) one impact test sample shall be taken from each 15 tonnes or less of shapes produced from each heat, or from each ingot for shapes rolled from ingots.

ΔΔ 27.1.5.3

Welds of primary members and connections in buildings with specified short-period spectral acceleration ratios ($I_E F_a S_a(0.2)$) greater than 0.35 shall be made with filler metals that have a minimum average Charpy V-Notch impact test value of 27 J at -30°C as certified in accordance with CSA Standard W48 or a manufacturer's certificate of conformance. This requirement may be waived for buildings with specified short-period spectral acceleration ratios ($I_E F_a S_a(0.2)$) less than or equal to 0.55 when the welds are loaded primarily in shear.

Δ 27.1.6

Bolted connections shall

- (a) have pretensioned high-strength bolts;
- (b) have surfaces of Class A or better, when designed as bearing-type connections;
- (c) not be considered to share load with welds;
- (d) not have long slotted holes;
- (e) not have short slotted holes unless the load is normal to the slot; and
- (f) have end distances in the line of seismic force not less than two bolt diameters when the bearing force due to seismic load exceeds 75% of the bearing resistance. (See Clause 13.10(c).)

The requirements of Clause 27.1.6 may be waived when fastener and connection details conform to those of a tested assembly.

(c) be located at least one-fourth of the clear distance between beams but not less than 1 m from the beam-to-column joint.

27.2.4 Column Joint Panel Zone

27.2.4.1

When plastic hinges form in adjacent beams, the panel zone shall resist forces arising from beam moments of

$$\Sigma \left(1.1 R_y M_{pb} + V_h \left(x + \frac{d_c}{2} \right) \right)$$

where the summation is for both beams at a joint, and x , M_{pb} , and V_h are as defined in Clause 27.2.3.2.

In single-storey buildings, when plastic hinges form near the top of columns, panel zones shall resist forces arising from moments corresponding to plastic hinge moments of $1.1R_y$ times the nominal flexural resistance of the column.

27.2.4.2

The horizontal shear resistance of the column joint panel zone shall be taken as

(a) when detailed in accordance with Clause 27.2.4.3

$$V_r = 0.55 \phi d_c w' F_{yc} \left[1 + \frac{3 b_c t_c^2}{d_c d_b w'} \right] \leq 0.66 \phi d_c w' F_{yc}$$

(b) if Item (a) does not apply

$$V_r = 0.55 \phi d_c w' F_{yc}$$

where the subscripts b and c denote the beam and the column, respectively.

ΔΔ 27.2.4.3

The following requirements shall apply:

(a) Where the specified short-period spectral acceleration ratio ($I_E F_a S_a(0.2)$) is equal to or greater than 0.55, joint panel zones designed according to Clause 27.2.4.2(a) shall be detailed in such a way that the sum of panel zone depth and width divided by the panel zone thickness shall not exceed 90.

(b) Joint panel zones designed according to Clause 27.2.4.2(b) shall satisfy the width-to-thickness limit of Clause 13.4.1.1(a).

(c) Doubler plates shall be groove- or fillet-welded to develop their full shear resistance. In calculating width-to-thickness ratios, doubler plate thickness may be included with web thickness only when the doubler plate is connected to the column web near the centre of the panel.

27.2.4.4

When connections and associated design procedures referenced in Appendix J are selected, the provisions of Clauses 27.2.4.1, 27.2.4.2, and 27.2.4.3 need not apply.

27.2.5 Beam-to-Column Joints and Connections

27.2.5.1

The beam-to-column joint shall maintain a strength at the column face of at least the nominal plastic moment resistance of the beam, M_{pb} , through a minimum interstorey drift angle of 0.04 radians under cyclic loading. Satisfaction of these criteria shall be demonstrated by physical testing.

When reduced beam sections are used, or when local buckling limits the flexural strength of the beam, the beam need only achieve $0.8 M_{pb}$ at the column face when an interstorey drift angle of 0.04 radians is developed under cyclic loading.

Note: Physical testing procedures used to demonstrate the required behaviour and specific details and design procedures for connections that will achieve the specified performance are referenced in Appendix J.

27.2.5.2

The factored resistance of the beam web-to-column connection shall equal or exceed the effects of gravity loads combined with shears induced by moments of $1.1R_yZF_y$ acting at plastic hinge locations, except when connections and associated design procedures referenced in Appendix J are selected.

27.2.5.3

In single-storey buildings, when the column frames into the underside of the beam and plastic hinging is expected near the top of a column, the connection shall meet the requirements of Clause 27.2.5.1.

27.2.6 Bracing

The following apply:

- (a) Beams, columns, and beam-to-column joints shall be braced by members proportioned according to Clause 9.2 where $C_f = 1.1R_yF_y$ times the cross-sectional area in compression. The possibility of complete load reversals shall be considered.
- (b) When plastic hinges occur in the beam, lateral bracing at the joints shall be provided at least at the level of one beam flange. If bracing is not provided at the level of both beam flanges, the transverse moments produced by the forces that would otherwise be resisted by the lateral bracing shall be included in the seismic load combinations. Attachments in the hinging area shall meet the requirements of Clause 27.2.8.
- (c) When plastic hinges occur near the top of the column, lateral bracing at the joints shall be provided at the level of both beam flanges.
- (d) When no lateral support can be provided to the joint at the level considered
 - (i) the column maximum slenderness ratio shall not exceed 60; and
 - (ii) transverse moments produced by the forces otherwise resisted by the lateral bracing shall be included in the seismic load combinations.

27.2.7 Fasteners

Fasteners connecting the separate elements of built-up flexural members shall have resistance adequate to support forces corresponding to moments of $1.1R_yZF_y$ at the plastic hinge locations.

27.2.8 Attachments in Hinging Areas

Structural and other attachments, such as shear connectors and bracing, that may introduce metallurgical notches or stress concentrations shall not be permitted in the hinging areas, unless they form part of a test assembly that satisfies the physical test requirements of Clause 27.2.5.1. The hinging area shall be taken as the area within the distance from the end of the member to one-half its depth beyond the adjacent expected hinge location.

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

Moderately ductile moment-resisting frames can develop a moderate amount of inelastic deformation through plastic hinging in the beams at a short distance from the face of columns. All requirements of Clause 27.2 are applicable, except that

- (a) with respect to Clause 27.2.2.1
 - (i) the beams shall be Class 1 or 2 sections; and

- (ii) the bracing shall meet the requirements of Clause 13.7(a);
- (b) with respect to Clause 27.2.3.1(b), the factored axial load shall not exceed $0.50AF_y$; and
- (c) with respect to Clause 27.2.5.1, the minimum interstorey drift angle shall be 0.03 radians.

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

ΔΔ **27.4.1 General**

Limited-ductility moment-resisting frames can develop a limited amount of inelastic deformation through plastic hinging in the beams, columns, or joints. This system may be used in buildings

- (a) not exceeding 60 m in height where the specified short-period spectral acceleration ratio ($I_E F_a S_a(0.2)$) is greater than or equal to 0.35; and
- (b) not exceeding 30 m in height where the specified short-period spectral acceleration ratio ($I_E F_a S_a(0.2)$) is greater than 0.75 or where the specified one-second spectral acceleration ratio ($I_E F_v S_a(1.0)$) is greater than 0.30.

Δ **27.4.2 Beams and Columns**

27.4.2.1

Beams shall be Class 1 or 2. Columns shall be Class 1. Except at roof level, beams shall frame into the columns.

ΔΔ **27.4.2.2**

When the specified short-period spectral acceleration ratio ($I_E F_a S_a(0.2)$) is greater than 0.55 or the building is greater than 60 m in height, columns shall satisfy the requirements of Clause 27.2.3.2; however, when applying Clause 27.2.3.2, the term $1.1R_y M_{pb}$ may be replaced by M_{pb} . In addition, the beams shall be designed so that for each storey, the storey shear resistance is not less than that of the storey above.

27.4.3 Column Joint Panel Zone

The horizontal shear resistance of the column joint panel zone shall be taken as that specified in Clause 27.2.4.2.

27.4.4 Beam-to-Column Connections

27.4.4.1

The beam-to-column joints shall meet the requirements of Clause 27.2.5.1, except that the minimum interstorey drift angle shall be 0.02 radians. Alternatively, beam-to-column joints shall meet the requirements of Clauses 27.4.4.2 to 27.4.4.6.

Δ **27.4.4.2**

Beam-to-column connections shall have a moment resistance equal to the lesser of

- (a) $1.1R_y M_{pb}$; or
- (b) the effect of the gravity loads combined with the seismic load multiplied by 2.0, provided that the controlling limit state is ductile.

Joints with welded flanges designed in accordance with Item (a) shall have a welded web connection.

Note: The following are considered to meet the requirement specified in Item (a):

- (a) complete-penetration groove welds made with matching electrodes in accordance with Clause 13.13.3.1; and
- (b) beam flanges welded directly to column flanges, with the beam web connected by a welded joint.

Δ **27.4.4.3**

Columns shall be I-shaped sections. The tensile resistance of the column flange shall be taken as $0.6 T_r$, as given in Clause 21.3.

27.4.4.4

Weld backing bars and run-off tabs shall be removed and repaired with reinforcing fillet welds. Top-flange backing bars may remain in place if continuously fillet welded to the column flange on the edge below the complete joint penetration groove weld. Neither partial-joint-penetration groove welds nor fillet welds shall be used to resist tensile forces in the connections.

27.4.4.5

Beam-to-column connections shall resist shear forces resulting from the gravity load together with shears corresponding to moments at each end equal to those specified in Clause 27.4.4.2.

27.4.4.6

For single-storey buildings in which columns frame under the beam, the roles of beam and column are reversed.

△ **27.5 Type MD (Moderately Ductile) Concentrically Braced Frames, $R_d = 3.0$, $R_o = 1.3$**

27.5.1 General

Moderately ductile concentrically braced frames can dissipate moderate amounts of energy through yielding of bracing members.

27.5.2 Bracing Systems**27.5.2.1 Systems**

Moderately ductile concentrically braced frames include

- (a) tension-compression bracing systems (see Clause 27.5.2.3);
- (b) chevron braced systems (see Clause 27.5.2.4);
- (c) tension-only bracing systems (see Clause 27.5.2.5); and
- (d) systems other than those in Items (a), (b), and (c), provided that stable inelastic response can be demonstrated.

Knee bracing and K-bracing, including those systems in which pairs of braces meet a column on one side between floors, are not considered to be moderately ductile concentrically braced frames.

27.5.2.2 Proportioning

At all levels of any planar frame, the diagonal bracing members shall be proportioned in such a way that the ratio of the sum of the horizontal components of the factored tensile resistances in opposite directions is between 0.75 and 1.33.

△△ **27.5.2.3 Tension-Compression**

Except where the specified short-period spectral acceleration ratio ($I_E F_a S_a(0.2)$) is less than 0.35, tension-compression concentric bracing systems shall not exceed 40 m in height. In addition, when the height exceeds 32 m, the factored seismic forces shall be increased by 3% per metre of height above 32 m.

△ **27.5.2.4 Chevron**

Chevron bracing systems comprise pairs of braces, located either above or below a beam, that meet the beam at a single point within the middle half of the span. Chevron bracing systems shall meet the requirements of Clause 27.5.2.3.

The beams to which the chevron bracing is attached shall

- (a) be continuous between columns;
- (b) have both top and bottom flanges laterally braced at the brace connection; and
- (c) resist bending moments due to gravity loads (assuming no vertical support is provided by the bracing members) in conjunction with bending moments and axial forces induced by forces of $A_g R_y F_y$

and $0.2A_gR_yF_y$ in the tension and compression bracing members, respectively. When braces are connected to the beam from above, the brace compression force shall be taken as 1.2 times the probable compressive resistance of the bracing member. In the case of buildings up to four storeys, the tension brace force may be taken as $0.6A_gR_yF_y$, provided that the beam is a Class 1 section.

The beam-to-column connections shall resist the forces corresponding to the loading described in Item (c) for beams with the following proviso: when the tension brace force is less than $A_gR_yF_y$, the connections shall resist the gravity loads combined with forces induced by the probable nominal flexural resistance of the beam at the brace connection.

The lateral braces, at the brace connection, shall resist a transverse load of 0.02 times the beam flange yield force.

Note: The probable compressive resistance of a brace is equal to C_r/ϕ , where C_r is a function of R_yF_y .

Δ 27.5.2.5 Tension-Only

The braces in tension-only bracing systems are designed to resist, in tension, 100% of the seismic loads and are connected at beam-to-column intersections. These systems shall

- (a) not exceed 20 m in height and, when the height exceeds 16 m, the factored seismic forces shall be increased by 3% per metre of height above 16 m;
- (b) have columns that are all fully continuous and of constant cross-section; and
- (c) have column splices proportioned for the full moment resistance of the cross-section and for a shear force of $2.0 ZF_y/h_s$, where Z is the plastic modulus of the column and h_s is the storey height.

Although the braces are proportioned on the basis of tension loading only, this system shall satisfy the provisions of Clause 27, including Clauses 27.5.3, 27.5.4, and 27.5.5.

27.5.3 Diagonal Bracing Members

27.5.3.1

The slenderness ratio, KL/r , of bracing members shall not exceed 200.

ΔΔ 27.5.3.2

For buildings with specified short-period spectral acceleration ratios ($I_EF_aS_a(0.2)$) equal to or greater than 0.35, width-to-thickness ratios shall not exceed the following limits:

- (a) when $KL/r \leq 100$
 - (i) for rectangular and square HSS: $330/\sqrt{F_y}$, for circular HSS: $10\,000/F_y$;
 - (ii) for legs of angles and flanges of channels: $145/\sqrt{F_y}$; and
 - (iii) for other elements: Class 1;
- (b) when $KL/r = 200$
 - (i) for HSS members: Class 1;
 - (ii) for legs of angles: $170/\sqrt{F_y}$; and
 - (iii) for other elements: Class 2; and
- (c) when $100 < KL/r < 200$, linear interpolation may be used.

When the specified short-period acceleration ratio ($I_EF_aS_a(0.2)$) is less than 0.35, HSS shall be Class 1 and all other sections shall be Class 1 or 2. The width-to-thickness ratio for legs of angles shall not exceed $170/\sqrt{F_y}$.

In all the above cases, for back-to-back legs of double angle bracing members for which buckling out of the plane of symmetry governs, the width-to-thickness ratio shall not exceed $200/\sqrt{F_y}$.

ΔΔ 27.5.3.3

For buildings with specified short-period spectral acceleration ratios ($I_EF_aS_a(0.2)$) equal to or greater than 0.35, the slenderness ratio of the individual parts of built-up bracing members shall not be greater than 0.5 times the governing effective slenderness ratio of the member as a whole. If overall buckling of the brace does not induce shear in the stitch fasteners that connect the separate elements of built-up bracing members, the slenderness ratio of the individual parts shall not exceed 0.75 times the governing effective slenderness ratio of the member as a whole.

If overall buckling of the brace induces shear in the stitch fasteners, the stitch fasteners shall have a resistance adequate to support one-half of the yield load of the smaller component being joined, with this force assumed to act at the centroid of the smaller member. Bolted stitch connections shall not be located in the anticipated plastic hinge regions of bracing members.

27.5.4 Brace Connections

27.5.4.1

Eccentricities in connections of braces to gusset plates or other supporting elements shall be minimized.

Δ 27.5.4.2

The following apply:

- (a) The factored resistance of brace connections shall equal or exceed both $A_g R_y F_y$ in tension and 1.2 times the probable compressive resistance of the bracing members (see note to Clause 27.5.2.4) except that the tensile force need not exceed the combined effect of the gravity load in the bracing members and the effects of seismic loads corresponding to $R_d R_o = 1.3$. The net section fracture resistance of the brace shall also be adequate to resist this tensile force.
- (b) When computing the forces corresponding to $R_d R_o = 1.3$, the redistribution of load due to brace buckling shall be considered. Connections detailed for these forces shall have a ductile ultimate limit state.
- (c) For chevron bracing, when plastic hinging is permitted in the beam, the brace tensile force need not exceed the greater of
 - (i) that due to plastic hinging in the beam; and
 - (ii) that corresponding to 1.2 times the probable resistance of the compression brace.
- (d) When designing brace connections for loads of $A_g R_y F_y$, the net section factored resistance of an unreinforced brace may be multiplied by R_y / ϕ , where R_y shall not exceed 1.1.

Note: In computing the forces corresponding to $R_d R_o = 1.3$, the post-buckling resistance of bracing members may be taken as equal to the lesser of $0.2 A_g R_y F_y$ and the probable nominal compressive resistance.

27.5.4.3

Brace members or connections, including gusset plates, shall be detailed to provide ductile rotational behaviour, either in or out of the plane of the frame, depending on the governing effective brace slenderness ratio. When rotation is anticipated in the bracing member, the factored flexural resistance of the connections shall equal or exceed $1.1 Z R_y F_y$ of the bracing member and the net section factored bending resistance of an unreinforced brace may be multiplied by R_y / ϕ . This requirement may be satisfied in the absence of axial load.

ΔΔ 27.5.5 Columns, Beams, and Connections Other than Brace Connections

ΔΔ 27.5.5.1

The factored resistance of columns, beams, and connections other than brace connections shall equal or exceed the effects of gravity loads and the brace connection forces given in Clause 27.5.4.2. Redistributed loads due to brace buckling or yielding shall be considered.

ΔΔ 27.5.5.2

Except as required by Clause 27.5.2.5, all columns in multi-storey buildings using the systems listed in Clause 27.5.2.1(a) to (c) shall be continuous and of constant cross-section over a minimum of two storeys. Class 4 columns shall not be used. The factored shear resistance of all column splices shall equal or exceed $0.4/h_s$ times the nominal flexural resistance of the columns.

Columns in braced bays shall be Class 1 or 2 and have a bending resistance in the direction of the braced bay of not less than $0.2 Z F_y$ in combination with the computed axial loads.

27.5.5.3

Partial-joint-penetration groove weld splices in columns subject to tension shall meet the requirements of Clause 27.2.3.3(a) and (b).

Δ **27.6 Type LD (Limited-Ductility) Concentrically Braced Frames,**
 $R_d = 2.0$, $R_o = 1.3$

27.6.1 General

Concentrically braced frames of limited ductility can dissipate limited amounts of energy through yielding of bracing members. The requirements of Clause 27.5 shall be met except as modified in Clause 27.6.

27.6.2 Bracing SystemsΔΔ **27.6.2.1 Tension-Compression**

Except where the specified short-period spectral acceleration ratio ($I_E F_a S_a(0.2)$) is less than 0.35, tension-compression concentric bracing systems shall not exceed 60 m in height. In addition, when the height exceeds 48 m, the factored seismic forces shall be increased by 2% per metre of height above 48 m.

Δ **27.6.2.2 Chevron**

Chevron bracing systems shall not exceed 60 m in height.

Structures of 20 m or less in height need not meet the requirements of Clause 27.5.2.4(c), provided that the braces and beam-to-column connections are proportioned to resist the forces that develop when buckling of the compression brace occurs, and provided that when a beam is attached to braces from below, it is a Class 1 section and has adequate nominal resistance to support the tributary gravity loads assuming no vertical support is provided by the bracing members.

Note: Clause 27.6.2.1 also applies to chevron bracing systems.

Δ **27.6.2.3 Tension-Only**

Tension-only systems shall

- (a) not exceed 40 m in height and, when the height exceeds 32 m, the factored seismic forces shall be increased by 3% per metre of height above 32 m; and
- (b) in multi-storey structures, have all columns fully continuous and of constant cross-section over a minimum of two storeys.

27.6.3 Diagonal Bracing Members**27.6.3.1**

In single- and two-storey structures, the slenderness ratio of bracing members connected and designed in accordance with Clause 27.5.2.5 shall not exceed 300.

ΔΔ **27.6.3.2**

The requirements of Clause 27.5.3.2 may be modified as follows:

- (a) when the brace slenderness exceeds 200 as permitted in Clause 27.6.3.1, the width-to-thickness limits of Clause 27.5.3.2 shall not apply; and
- (b) for buildings with specified short-period spectral acceleration ratios ($I_E F_a S_a(0.2)$) less than 0.45, braces need not be more compact than Class 2. The width-to-thickness ratio of the legs of angles shall not exceed $170/\sqrt{F_y}$.

ΔΔ **27.6.4 Bracing Connections**

The requirements of Clause 27.5.4.3 shall be waived for buildings with specified short-period spectral acceleration ratios ($I_E F_a S_a(0.2)$) less than 0.55 if the brace slenderness ratio is greater than 100.

ΔΔ 27.6.5 Columns, Beams, and Other Connections

For buildings with specified one-second spectral acceleration ratios ($I_E F_v S_a(1.0)$) not greater than 0.30, the design forces for column splices in Clause 27.5.5.2 need not be taken into account in columns other than those in the braced bays.

Δ 27.7 Ductile Eccentrically Braced Frames, $R_d = 4.0$, $R_o = 1.5$

Ductile eccentrically braced frames can dissipate energy by yielding of links.

27.7.1 Link Beam**27.7.1.1**

The link beam contains a segment (the link) designed to yield, either in flexure or in shear, prior to yielding of other parts of the eccentrically braced frame. A link shall be provided at least at one end of each brace.

Δ 27.7.1.2

Link beams shall be Class 1 and designed for the coexisting shears, bending moments, and axial forces. Link beams may have Class 2 flanges and Class 1 webs when $e \leq 1.6 M_p / V_p$,

where

e = length of the link

V_p = $0.55wdF_y$

27.7.1.3

The web of the link shall be of uniform depth and shall have no penetrations, splices, attachments, reinforcement, or doubler plates, other than the stiffeners required by Clause 27.7.5.

27.7.2 Link Resistance

The shear resistance of the link shall be taken as the lesser of $\phi V_p'$ and $2\phi M_p' / e$

where

$$V_p' = V_p \sqrt{1 - \left(\frac{P_f}{AF_y} \right)^2}$$

$$M_p' = 1.18M_p \left(1 - \frac{P_f}{AF_y} \right) \leq M_p$$

where

V_p = $0.55wdF_y$

P_f = axial force in the link (= C_f or T_f)

A = gross area of the link beam

e = length of the link

27.7.3 Length of Link

The link length shall be not less than the depth of the link beam. When $P_f / (AF_y) > 0.15$, the length of link shall be as follows:

27.7.10 Brace-to-Beam Connection

No part of the brace-to-beam connection shall extend into the link. The intersection of the brace and beam centrelines shall be at the end of, or within, the link. If the brace is designed to resist a portion of the link end moment, full end restraint shall be provided. The beam shall not be spliced within or adjacent to the connection between beam and brace.

27.7.11 Columns

27.7.11.1 General

Columns shall be designed to resist the cumulative effect of yielding links together with the gravity loads. The link forces shall be taken as $1.15R_y$ times the nominal strength of the link, except that in the top two storeys the force shall be taken as $1.30R_y$ times the nominal value. Column resistances shall satisfy the requirements of Clause 13.8, except that the interaction value shall not exceed 0.65 for the top column tier in the braced bay and 0.85 for all other columns in the braced bay. Column sections shall be Class 1 or 2.

27.7.11.2 Column Splices

Column splices shall resist shear forces equal to $0.3/h_s$ times the average nominal flexural resistance of the columns, except that at the base of the top column tier the factor shall be $0.5/h_s$. Splices that incorporate partial-joint-penetration groove welds shall be located at least one-fourth of the clear distance between beams but not less than 1 m from the beam-to-column joints. When tension occurs in columns due to the link-induced forces, column splices having partial-joint-penetration groove welds shall be designed according to Clause 27.2.3.3(a) and (b).

27.7.12 Roof Link Beam

A link shall not be required in roof beams of frames over five storeys in height.

27.8 Plate Walls

27.8.1 General

27.8.1.1

Plate walls used to resist seismic forces shall be designed as either Type D or Type LD plate walls.

27.8.1.2

The requirements of Clause 20 shall apply to Clause 27.8 unless otherwise specified.

Δ 27.8.2 Type D (Ductile) Plate Walls, $R_d = 5.0$, $R_o = 1.6$

27.8.2.1 General

Ductile plate walls are vertical plate girders comprising web plates framed by rigidly connected columns and beams. Ductile plate walls can develop significant inelastic deformation by the yielding of the web plates and development of plastic hinges in the framing members.

27.8.2.2 Beams

Beams shall be Class 1 sections braced in accordance with Clause 13.7(b).

27.8.2.3 Columns

Columns shall be braced in accordance with Clause 13.7(b).

Column splices shall develop the full flexural resistance of the smaller section at the splice, together with the shear force consistent with plastic hinging at column ends assuming double curvature. Splices shall be located as close as practicable to one-fourth of the storey height above the floor.

Δ 27.8.2.4 Capacity Design

The following apply:

(a) Unless otherwise approved, capacity design shall be based on tension yielding of the web plate at the base of the wall prior to the columns attaining their factored resistances.

Loads in the framing members shall be determined from the gravity loads combined with the seismic loads increased by the amplification factor

$$B = V_{re} / V_f$$

where

V_{re} = probable shear resistance at the base of the wall, determined for the web plate thickness supplied

$$= 0.5 R_y F_y w L \sin 2\alpha$$

V_f = factored lateral seismic force at the base of the wall

(b) In determining the loads in columns, the amplification factor, B, need not be taken as greater than R_d .

(c) Notwithstanding Items (a) and (b), the column axial forces shall be determined from overturning moments defined as follows:

(i) the moment at the base is BM_f , where M_f is the factored seismic overturning moment at the base of the wall corresponding to the force V_f ;

(ii) the moment BM_f extends for a height L but not less than two storeys from the base; and

(iii) the moment decreases linearly above a height L to B times the overturning moment at one storey below the top of the wall, but need not exceed R_d times the factored seismic overturning moment at the storey under consideration corresponding to the force V_f .

The local bending moments in the columns due to tension field action in the web plate shall be multiplied by the amplification factor B.

27.8.2.5 Column Joint Panel Zones

The horizontal shear resistance of the column panel zone shall meet the requirements of Clauses 27.2.4.2 and 27.2.4.3.

27.8.2.6 Beam-to-Columns Joints and Connections

Beam-to-column joints and connections shall meet the requirements of Clause 27.4.4 except that Clause 27.4.4.2(b) shall not apply.

27.8.2.7 Column Base Plates

The column shall be stiffened so that the plastic hinge forms at a minimum distance of 1.5 times the column depth above the base plate. Anchorage details shall resist the greater of

(a) $1.1R_y$ times the nominal flexural resistance of the column; or

(b) the tensile column load computed from Clause 27.8.2.4.

Δ 27.8.3 Type LD (Limited-Ductility) Plate Walls, $R_d = 2.0$, $R_o = 1.5$

Limited-ductility plate walls dissipate a limited amount of energy by yielding of the web plates and supporting members. Type LD plate walls may be proportioned in accordance with Clause 20, without any other special requirements.

Type LD plate walls shall be limited to 60 m in height.

ΔΔ 27.9 — Deleted

ΔΔ 27.10 Conventional Construction, $R_d = 1.5$, $R_o = 1.3$

Note: Other provisions of Clause 27 do not apply to these systems.

27.10.1

Structural systems in this category have some capacity to dissipate energy through localized yielding and friction that, in general, are available through the use of traditional design and construction practices.

Diaphragms and connections of primary framing members and diaphragms forming the seismic-load-resisting system of steel-framed buildings with specified short-period spectral acceleration ratios ($I_E F_a S_a(0.2)$) greater than 0.45 that are designed to resist seismic loads based on a force reduction factor, R_d , of 1.5 shall either

- (a) be proportioned so that the expected connection failure mode is ductile; or
- (b) be designed to resist gravity loads combined with the seismic load multiplied by R_d .

The connection design load need not exceed R_y times the nominal gross section strength of the members being joined.

27.10.2

Cantilever column structures composed of single or multiple beam-columns fixed at the base and pin-connected or free at the upper ends shall

- (a) comprise Class 1 sections;
- (b) have U_2 not greater than 1.25; and
- (c) have base connections designed to resist a moment of $1.1R_y$ times the nominal flexural resistance of the column, but need not exceed the value corresponding to $R_d R_o = 1.0$.

27.11 Special Seismic Construction

Other framing systems and frames that incorporate special bracing, ductile truss segments, seismic isolation, or other energy-dissipating devices shall be designed on the basis of published research results or design guides, observed performance in past earthquakes, or special investigation. A level of safety and seismic performance comparable to that required by these provisions shall be provided.

28. Shop and Field Fabrication and Coating**28.1 Cambering, Curving, and Straightening**

Cambering, curving, and straightening may be done by mechanical means or local application of heat, or both. The temperature of heated areas as measured by approved methods shall not exceed the limits given in CSA Standard W59.

28.2 Thermal Cutting

Thermal cutting shall be performed by guided machine where practicable. Thermally cut edges shall conform to CSA Standard W59. Re-entrant corners shall be free from notches and shall have the largest practicable radii, with a minimum radius of 14 mm.

28.3 Sheared or Thermally Cut Edge Finish**28.3.1**

Planing or finishing of sheared or thermally cut edges of plates or shapes shall not be required, unless specifically noted on the drawings or included in a stipulated edge preparation for welding.

28.3.2

The use of sheared edges in the tension area shall be avoided in locations subject to plastic hinge rotation at factored loading. If used, such edges shall be finished smooth by grinding, chipping, or planing. These requirements shall be noted on design drawings and shop details where applicable.

28.3.3

Burrs shall be removed

- (a) as required in Clause 23.2;
- (b) when required for proper fit-up for welding; and
- (c) when they create a hazard during or after construction.

28.4 Fastener Holes

28.4.1 Drilled and Punched Holes

Unless otherwise shown on design documents or as specified in Clause 22.3.5, holes shall be made 2 mm larger than the nominal diameter of the fastener. Holes may be punched when the thickness of the material is not greater than the nominal fastener diameter plus 4 mm. For greater thicknesses, holes shall be either drilled from the solid or sub-punched or sub-drilled and reamed. The die for all sub-punched holes or the drill for all sub-drilled holes shall be at least 4 mm smaller than the required diameter of the finished hole. Holes in CSA Standard G40.21-700Q or ASTM Standard A 514 steels more than 13 mm thick shall be drilled.

28.4.2 Holes at Plastic Hinges

In locations subject to plastic hinge rotation at factored loading, fastener holes in the tension area shall be either sub-punched and reamed or drilled full size. This requirement shall be noted on design drawings and shop details.

28.4.3 Thermally Cut Holes

Thermally cut holes, produced by guided machine, shall be permitted in statically loaded structures, provided that the actual hole size does not exceed the nominal hole size by more than 1 mm. Gouges not exceeding 1.5 mm deep shall be permitted along edges of thermally cut slots. Manually cut fastener holes shall be permitted only with the approval of the designer.

28.4.4 Alignment

Drifting done during assembly to align holes shall not distort the metal or enlarge holes. Holes in adjacent parts shall match well enough to permit easy entry of bolts. Holes, except oversize or slotted holes, may be enlarged to admit bolts by a moderate amount of reaming. However, gross mismatch of holes shall be cause for rejection.

28.5 Joints in Contact Bearing

Joints in compression that depend on contact bearing shall have the bearing surfaces prepared to a common plane by milling, sawing, or other suitable means. Surface roughness shall have a roughness height rating not exceeding 500 (12.5 μm), as defined in CSA Standard B95, unless otherwise specified.

When shop assembled, such joints shall have at least 75% of the entire contact area in bearing. A separation not exceeding 0.5 mm shall be considered acceptable as bearing. The separation of any remaining portion shall not exceed 1 mm. A gap of up to 3 mm may be packed with non-tapered steel shims in order to meet the requirements of this clause. Shims need not be other than mild steel, regardless of the grade of the main material.

28.6 Member Tolerances

28.6.1

Structural members consisting primarily of a single rolled shape shall be straight within the tolerances allowed in CSA Standard G40.20, except as specified in Clause 28.6.4.

Table 9 (Concluded)

General condition	Situation	Detail category	Illustrative example (see Figure 2)
A 325, A 325M, and F 1852 bolts in axial tension	Tensile stress on area A_b		
A 490 and A 490M bolts in axial tension	Tensile stress on area A_b		

See Clause 13.12.1.2

*The fatigue resistance of fillet welds transversely loaded is a function of the effective throat and plate thickness. (See Frank and Fisher, Journal of the Structural Division, ASCE, Vol. 105, No. ST9, September 1979.)

$$F_{sr} = F_{sr}^c [(0.06 + 0.79 H/t_p) / (0.64 t_p^{1/6})]$$

where

F_{sr}^c = the fatigue resistance for Category C as determined in accordance with Clause 26.3.3. This assumes no penetration at the weld root

t_p = plate thickness

H = weld leg size

ΔΔ

Table 10
Fatigue Constants for Various Detail Categories
(See Clauses 26.3.3 and 26.3.4.)

Detail category	Fatigue life constant, γ	Constant amplitude threshold stress range, F_{sr} , MPa	nN'	Fatigue life constant, γ'
A	8190×10^9	165	1.82×10^6	223×10^{15}
B	3930×10^9	110	2.95×10^6	47.6×10^{15}
B1	2000×10^9	83	3.50×10^6	13.8×10^{15}
C	1440×10^9	69	4.38×10^6	6.86×10^{15}
C1	1440×10^9	83	2.52×10^6	9.92×10^{15}
D	721×10^9	48	6.52×10^6	1.66×10^{15}
E	361×10^9	31	12.1×10^6	0.347×10^{15}
E1	128×10^9	18	21.9×10^6	0.0415×10^{15}

Δ

Table 11
Importance Factors, I, for ULS and SLS
(See Clause 6.2.2.)

Importance category	Ultimate limit states			Serviceability limit states		
	Snow, I_s	Wind, I_w	Earthquake, I_e	Snow, I_s	Wind, I_w	Earthquake, I_e
Low	0.8	0.8	0.8	0.9	0.75	See NBCC 2005, Section 4.1.8.13 and Commentary J
Normal	1.0	1.0	1.0	0.9	0.75	
High	1.15	1.15	1.3	0.9	0.75	
Post-disaster	1.25	1.25	1.5	0.9	0.75	

△

Table 12
Building Importance Categories
(See Clause 6.2.2.)

Use and occupancy	Importance category
Buildings that represent a low direct or indirect hazard to human life in the event of failure, including <ul style="list-style-type: none"> • low human-occupancy buildings, where it can be shown that collapse is not likely to cause injury or other serious consequences • minor storage buildings 	Low
All buildings except those listed in low, high, and post-disaster categories	Normal
Buildings that are likely to be used as post-disaster shelters, including buildings whose primary function is as <ul style="list-style-type: none"> • an elementary, middle, or secondary school • a community centre Manufacturing and storage facilities containing toxic, explosive, or other hazardous substances in sufficient quantities to be dangerous to the public, if released	High
Post-disaster buildings that include <ul style="list-style-type: none"> • hospitals, emergency treatment facilities, and blood banks • telephone exchanges • power generating stations and electrical substations • control centres for air, land, and marine transportation • sewage treatment facilities • buildings of the following types, unless exempted from this designation by the authority having jurisdiction: <ul style="list-style-type: none"> • emergency response facilities • fire, rescue, and police stations and housing for the vehicles, aircraft, or boats used for these emergency services • communication facilities, including radio and television stations 	Post-disaster

△

Table 13
Load Combinations for Ultimate Limit States
(See Clauses 7.2.2, 7.2.3, 7.2.6, 7.2.7 and 7.2.8.)

Case	Load combination	
	Principal loads	Companion loads
1	1.4D	—
2	(1.25D or 0.9D) + 1.5L	0.5S or 0.4W
3	(1.25D or 0.9D) + 1.5S	0.5L or 0.4W
4	(1.25D or 0.9D) + 1.4W	0.5L or 0.5S
5	1.0D + 1.0E	0.5L + 0.25S

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, shear or both. In braced frames, they also include brace-to-beam, brace-to-column and brace-to-brace connections.

The use of Conventional Construction for steel buildings, in regions of moderate and high seismicity is restricted, in Table 4.1.8.9 of NBCC 2005 to buildings not exceeding 15 metres in height. This restriction was simply intended to retain the traditional 3-storey height limit stipulated in previous editions of the NBCC. However, this height limit was not intended for and does not apply to single-storey steel structures (See Commentary J to NBCC 2005). In particular, structures such as steel mills and aircraft hangers may well exceed 15 metres in height and Conventional Construction may be used for them. Structures such as stadia, large exhibition halls, arenas, convention centres and other similar structures must satisfy the height restrictions.

Update No. 3, August 2006, to S16, clarifies that Conventional Construction may also be used for cantilever column structures composed of single or multiple beam-columns fixed at the base and pin-connected or free at their upper ends, provided these structures:

- (a) comprise Class 1 sections,
- (b) have U_2 not greater than 1.25, and
- (c) have base connections designed to resist a moment of $1.1R_y$ times the nominal flexural resistance of the column, but need not exceed the value corresponding to $R_d R_o = 1.0$.

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 that are designed and constructed using connections that have been shown by testing to be ductile can be designed using the forces calculated for conventional construction while those diaphragms with connections that have not been shown to be ductile should be designed using forces modified by $R_d R_o = 1.3$. Button punched side lap connections or arc spot welded connections commonly used for steel decks have not been shown to exhibit ductile behaviour under cyclic loading. See Essa, *et al.* (2003) and Tremblay, *et al.* (2004).

27.11 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 2005) 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 S16-01 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, and erection, and inspection procedures. (See also CISC (2002)).