g) When columns frame under the beams, the roles of beam and column shall be reversed.

Note: *Beam-to-column connections with a welded web connection and complete-penetration groove welds made with matching electrodes in accordance with Clause 13.13.3.1 between the beam flanges and the column flanges are considered to have a moment-resistance equal to* R_yM_{pb}.

27.4.4.3

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

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 General

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);
d) multi-tiered bracing systems (see Clause 27.5.2.6); and
e) other systems, 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 horizontal diaphragms, are not considered to be moderately ductile concentrically braced frames. Multi-tiered bracing systems in which struts are used at every tier level can be considered to be moderately ductile concentrically braced frames provided that they meet the requirements in Clause 27.5.2.6.

27.5.2.2 Proportioning

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

27.5.2.3 Tension-compression bracing

In SC3 and SC4, tension-compression concentric bracing systems shall not exceed 40 m in height. In addition, when the height exceeds 32 m, the factored seismic forces for the ultimate limit states shall be increased by 3% per metre of height above 32 m.

27.5.2.4 Chevron bracing

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 T_{prob} and C'_{prob} in the tension and compression bracing members, respectively. In the case of buildings not exceeding four storeys, the tension brace force may be taken as $0.6T_{prob}$, provided that the beam is a Class 1 section. When braces are connected to the beam from above, the case where the brace compression force at first buckling is equal to C_{prob} shall also be considered.

The beam-to-column connections shall resist the forces corresponding to the loading described in Item c) for beams. However, when the tension brace force is less than T_{prob}, the connections shall resist the gravity loads combined with forces associated with the attainment of R_y times the 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: *See Clause 27.5.3.5 for T_{prob}, C_{prob}, and C'_{prob}.*

27.5.2.5 Tension-only bracing

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. In SC3 and SC4,

a) the structure shall not exceed 20 m in height and, when the height exceeds 16 m, the factored seismic forces for ultimate limit states shall be increased by 3% per metre of height above 16 m;
b) all columns shall be continuous and of constant cross-section over the building height; and
c) the column splices shall be proportioned for the full moment resistance of the cross-section and for a shear force of $2.0ZF_y/h_s$, 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 meet the other requirements of Clause 27, including Clauses 27.5.3 to 27.5.6.

27.5.2.6 Multi-tiered bracing

27.5.2.6.1 General

Multi-tiered bracing systems shall

a) have no more than three tiers between horizontal diaphragms;
b) have no more than one bay in width;
c) have braces at every tier level used in opposing pairs to form a tension-compression X-bracing system, a chevron bracing system, or a tension-only X-bracing system, as defined in Clause 27.5.2;
d) satisfy the requirements applicable to the bracing system used; and
e) satisfy the requirements of this Clause.

Multi-tiered bracing systems consisting of two or more adjacent bays may be used provided that the bracing system and geometry are the same in all bays.

27.5.2.6.2 Struts

Horizontal struts shall be provided between columns at every brace-to-column connection level to ensure that lateral loads can be transferred by the braces after buckling of the braces acting in compression.

The struts shall resist forces induced by gravity loads plus the brace forces in the tiers above and below when braces in tension reach their probable resistance, T_{prob}, and braces in compression reach their probable compressive resistance at first buckling, C_{prob}, in the tier below or above and their probable post-buckling compressive resistance, C'_{prob}, in the other tier, depending which condition produces the largest effects.

When a chevron bracing system is used, the struts shall also meet the requirements for beams in Clause 27.5.2.4, except that forces in the tension-acting braces are not permitted to be reduced to 0.6 T_{prob} in design. Out-of-plane lateral bracing of the struts shall be provided at the brace-to-strut connections.

Note: *Lateral bracing requirements may be satisfied by providing the struts with sufficient strength and stiffness to resist horizontal out-of-plane forces calculated in accordance with Clause 9 and applied in opposite directions at the top and bottom fibers of the struts. The out-of-plane forces must be determined assuming initial out-of-plane geometric imperfections equal to 1/500 of the strut length and reduced stiffnesses for the struts as defined in Clause O.2.3.*

When used to torsionally brace the columns at strut levels, the struts shall have the required horizontal out-of-plane flexural stiffness and strength, as required in this Clause. The horizontal component of out-of-plane moments induced by out-of-plane buckling of the braces shall also be resisted by the struts.

Note: *See Clause 27.5.3.5 for T_{prob}, C_{prob}, and C'_{prob}.*

27.5.2.6.3 Columns

Columns shall satisfy the requirements of Clause 27.5.5 except as otherwise specified in this Clause.

The additional in-plane bending moment equal to $0.2ZAF_y$ of the column may be neglected when verifying the columns for the loading conditions where the compression acting braces attain their probable compressive resistance at first buckling, C_{prob}, in all tiers.

For the loading condition where the compression acting braces attain their probable buckled resistance, C'_{prob}, the columns resist the simultaneous effects of

a) the gravity loads;
b) out-of-plane shears and bending moments due to an out-of-plane transverse load at each strut level equal to 2% of the factored axial compression load in the columns below the strut; and
c) the axial loads, in-plane shear forces, and in-plane bending moments induced by yielding and buckling of the bracing members when the frame reaches the design storey drift, assuming that the tension-acting brace in any of the critical tiers along the storey height reaches its probable resistance, T_{prob}, the compression-acting brace in that critical tier reaches its probable post-buckling resistance, C'_{prob}, and the compression braces in the other tiers reach their probable compressive resistances at first buckling, C_{prob}.

   Note: *In Item c):*
   a) *A tier is critical when its storey shear resistance determined with brace resistances equal to 0.9 T_{prob} and 0.9 C_{prob} is less than the probable storey shear resistance in any of the other tiers determined with brace resistances equal to T_{prob} and C_{prob}.*
   b) *The possibility of brace tension yielding propagating in non-critical tiers before the frame attains the design storey drift may be considered in the determination of column forces. Guidance on methods required to account for propagation of brace tension yielding is given in the CISC Commentary.*

Columns shall be torsionally braced at every strut level.

Note: *Torsional bracing at tier levels may be achieved by connecting the strut to the column to restrain torsional deformation of the column. The strut must have sufficient flexural stiffness and strength, in accordance with Clause 9, and appropriate connections to the columns to provide adequate torsional bracing to the column.*

Splices in columns shall resist the anticipated axial forces, shears, and bending moments. For this verification, in-plane bending moments shall be taken equal to the larger of 1.2 times the bending moments from analysis, and the moment equal to $0.2ZF_y$ of the columns.

27.5.2.6.4

The maximum anticipated drift in any tier at the design storey drift shall not exceed 0.02 times the height of the tier.

Note: *For the purpose of verifying the requirement of this clause, tier drifts may be taken as the design storey drift multiplied by the ratio of the frame height to tier heights. Alternatively, more realistic tier drift estimates when the frame attains the design storey drift may be obtained from the analysis described in Clause 27.5.2.6.3 for determining in-plane shears and bending moments in columns.*

27.5.3 Diagonal bracing members

Note: *Where possible, at every storey, the two discontinuous bracing members in every X-bracing bay should be fabricated and installed from the same heat.*

27.5.3.1 Factored brace resistance

The factored resistance of bracing members shall be determined using the minimum specified yield strength. For bracing members made from HSS in compliance with Clause 5.1.3 with minimum specified yield stress of 345 or 350 MPa, the factored tension resistance T_r as determined using Clause 13.2 a) i) may be increased by 15% for load combinations that include seismic effects.

Note: *The increase in T_r does not apply to circular HSS conforming to ASTM A500.*

27.5.3.2 Brace slenderness

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

In SC4, the slenderness ratio of HSS bracing members shall not be less than 70.

Note: *The effects of translational and rotational restraints at the brace ends or along the brace length should be accounted for in the calculation of KL.*

27.5.3.3 Width (diameter)-to-thickness ratios

In SC3 and SC4, width-to-thickness ratios shall not exceed the following limits:

a) when $KL/r \leq 100$:
   i) for rectangular and square HSS: $330 / \sqrt{F_y}$;
   ii) for circular HSS: $10\,000/F_y$;
   iii) for legs of angles and flanges of channels: $145 / \sqrt{F_y}$; and
   iv) for other elements: Class 1 limits in Table 2;

b) when $KL/r = 200$
   i) for HSS members: Class 1 limits in Table 2;
   ii) for legs of angles: $170/\sqrt{F_y}$; and
   iii) for other elements: Class 2 limits in Table 2; and

c) when $100 < KL/r < 200$, linear interpolation may be used.

In SC1 and SC2, HSS shall satisfy the Class 1 limits in Table 2, and all other sections shall satisfy the Class 2 limits. The width-to-thickness ratio for legs of angles shall not exceed $170 / \sqrt{F_y}$.

In SC1 to SC4, the width-to-thickness ratios of back-to-back legs of double-angle bracing members may exceed the limits for angles specified in this clause when buckling out of the plane of symmetry governs, but they shall not exceed $200 / \sqrt{F_y}$.

27.5.3.4 Built-up bracing members

In SC3 and SC4, the slenderness ratio of the individual components of built-up bracing members, as defined in Clause 19.2.4, 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 components of built-up bracing members, the slenderness ratio of the individual components 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 larger 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.

27.5.3.5 Probable brace resistances

For the purpose of evaluating forces on connections and other members upon yielding and buckling of the bracing members in capacity design, the probable tensile resistance of bracing members, T_{prob}, the probable compressive resistance at first buckling of bracing members, C_{prob}, and the probable post-buckling compressive resistance of bracing members, C'_{prob}, shall be taken as equal to

$T_{prob} = A_g R_{sh} R_y F_y$, with R_{sh} = 1.0;

$C_{\mathrm{prob}} = T_{prob}$, but not greater than $1.2C_r/\phi$, where C_r is computed using $R_y F_y$ and R_{sh} = 1.0; and

$C'_{prob} = 0.2T_{prob}$, but not greater than C_r/ϕ, where C_r is computed using $R_y F_y$ and R_{sh} = 1.0.

Each of the two loading conditions,

a) the compression acting braces attaining their probable compressive resistance at first buckling, C_{prob}; and
b) the compression acting braces attaining their probable post-buckling compressive resistance, C'_{prob},

shall be considered as occurring in conjunction with the tension acting braces developing their probable tensile resistance, T_{prob}.

For chevron bracing, when plastic hinging in the beam is permitted by Clause 27.5.2.4 c) or 27.6.2.2, the brace tensile force need not exceed the greater of that corresponding to plastic hinging in the beam and that corresponding to C_{prob} of the compression brace at first buckling.

When the forces corresponding to $R_d R_o$ = 1.3 or 1.0, as applicable, are computed, the redistribution of forces due to brace buckling shall be considered.

The probable resistances of HSS bracing members conforming to ASTM A500 shall be determined using the nominal section properties.

Note: *For ASTM A500 HSS members, nominal section properties are based on unreduced wall thickness. See Clause 5.1.3.*

27.5.4 Brace connections

27.5.4.1 Eccentricities

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

27.5.4.2 Resistance

The factored resistance of brace connections shall equal or exceed both the probable tensile resistance of the bracing members in tension, T_{prob}, and the probable compressive resistance of the bracing members in compression at first buckling, C_{prob}, specified in Clause 27.5.3.5. For chevron bracing, when plastic hinging is permitted by Clause 27.5.2.4 c) or Clause 27.6.2.2, the brace tension force may be reduced as specified in Clause 27.5.2.4.

The net section fracture resistance of the brace shall be adequate to resist the tension resistance, T_{prob}. The net section factored resistance of the brace may be multiplied by R_y/ϕ_u, where R_y shall not exceed 1.2 for HSS and 1.1 for other shapes. This multiplier shall not be applied to the factored resistance of any cross-section reinforcement.

27.5.4.3 Ductile hinge rotation

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 $1.1ZR_yF_y$ 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.

27.5.5 Columns, beams, and connections other than brace connections

27.5.5.1

Structural members and their connections that are not considered to form part of the seismic force resisting system shall meet the requirements of Clause 27.1.4. Structural members and their connections that do form part of the seismic force resisting system shall meet the requirements of this Clause.

27.5.5.2

The factored resistance of columns, beams, and connections other than brace connections shall equal or exceed the effects of gravity loads and the brace forces corresponding to the brace probable resistances specified in Clause 27.5.3.5. For chevron bracing, the beams shall be designed in accordance with Clause 27.5.2.4 and the brace tension force may be reduced as specified in Clause 27.5.3.5.

27.5.5.3

Columns shall meet the requirements of Class 1 or 2 beam-columns in Table 2.

Columns in multi-storey buildings using the systems specified in Items a) to d) of Clause 27.5.2.1 shall
a) be continuous and of constant cross-section over a minimum of two storeys, except as required by Clause 27.5.2.5; and
b) satisfy the requirements of Clause 13.8 including an additional bending moment in the direction of the braced bay of $0.2ZF_y$ in combination with the computed bending moments and axial loads.

Splices in columns shall be designed to provide the required axial, shear, and flexural resistances including the effects of the additional bending moments in the direction of the braced bays of $0.2ZF_y$ acting either in the same or the opposite directions at the column ends.

27.5.5.4

Splices in columns in braced bays shall be designed to provide the required axial, shear, and flexural resistances including the effects of the additional bending moments in the direction of the braced bays of $0.2ZF_y$ acting either in the same or the opposite directions at the column ends.

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

27.5.6 Protected zones

The protected zone of bracing members shall

a) be designated to include the full brace length;
b) be designated to include elements that connect braces to beams and columns; and
c) meet the requirements of Clause 27.1.9.

Splices shall not be used in bracing members.

Note: *Holes for erection bolts in brace connections may be used, provided that they are considered in design.*

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 by Clauses 27.6.2 to 27.6.5.

27.6.2 Bracing systems

27.6.2.1 Tension-compression bracing

In SC3 and SC4, tension-compression concentric bracing systems shall not exceed 60 m in height. In addition, when the height exceeds 48 m, the factored seismic forces for ultimate limit states shall be increased by 2% per metre of height above 48 m.

27.6.2.2 Chevron bracing

In SC3 and SC4, chevron braced systems shall not exceed 60 m in height. In addition, when the height exceeds 48 m, the factored seismic forces for ultimate limit states shall be increased by 2% per metre of height above 48 m.

Beams in chevron bracing of 20 m or less in height need not meet the requirements of Clause 27.5.2.4 c) provided that the beams and the beam-to-column connections are proportioned to resist the forces that develop when buckling of the compression brace occurs and provided that when the braces are connected to the beam from below, the beam 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.

27.6.2.3 Tension-only bracing

In SC3 and SC4, tension-only systems:

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

27.6.2.4 Multi-tiered bracing

Multi-tiered bracing systems having no more than five tiers between horizontal diaphragms may be used, provided that they meet the requirements of Clause 27.5.2.6.

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 may exceed 200 but shall not exceed 300.

27.6.3.2

The requirements of Clause 27.5.3.3 may be modified as follows:

a) when the brace slenderness ratio exceeds 200 (as permitted by Clause 27.6.3.1), the width-to-thickness limits of Clause 27.5.3.3 need not apply; and

b) for buildings less than 40 m in height and in SC1 and SC2, braces need not be more compact than the Class 2 limits in Table 2. The width-to-thickness ratio of the legs of angles shall not exceed $170 / \sqrt{F_y}$.

27.6.4 Bracing connections

The requirements of Clause 27.5.4.3 need not apply in SC1 and SC2 if the brace slenderness ratio is greater than 100.

27.6.5 Columns, beams, and other connections

In SC1 to SC3, the design forces for column splices in Clause 27.1.4 need not be taken into account.

27.7 Type D (ductile) eccentrically braced frames, R_d = 4.0, R_o = 1.5

27.7.1 General

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

Multi-tiered eccentrically braced frames are not considered to be ductile eccentrically braced frames unless justified by rational analysis accounting for the effects of uneven tier drifts over the frame height at the seismic design storey drift, and lateral bracing satisfying the strength requirements of Clause 27.7.7 is provided to both top and bottom flanges at the ends of the links.

Note: *Possible effects of uneven tier drifts include bending moments in columns, higher plastic link rotation demands in weaker tiers. Guidance on rational analysis is given in the CISC* Commentary.

27.7.2 Link beam

27.7.2.1

The link beam shall contain a segment (the link) designed to yield, either in flexure or in shear, prior to yielding of other parts of the eccentrically braced frame.

27.7.2.2

The link beam shall be either

a) a segment of the beam, for beams with an I-section or a built-up tubular rectangular cross-section; or

b) a modular link distinct from the rest of the beam. A modular link shall be either
   i) an end-plate connected link fabricated from a I-shaped section connected to the beam with unstiffened end-plate moment connections; or
   ii) a web connected link consisting of a built-up cross-section made of two C-sections connected back-to-back to the beam web, where the C-sections are channels or wide-flange cross-sections with the flanges cut flush with the web on one side.

27.7.2.3

A link shall be provided at least at one end of each brace. A link shall not be required in roof beams of frames over five storeys in height.

27.7.2.4

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 $e \leq 1.6M_p/V_p$, where e is the length of the link and $V_p = 0.55wdF_y$, for links with wide-flange cross-sections, or $0.55(2w)dF_y$, for links with built-up tubular cross-sections and links with back-to-back C-sections.

27.7.2.5

The web or webs of the link shall be of uniform depth and have no penetrations, splices, attachments, reinforcement, or doubler plates, other than the stiffeners required by Clause 27.7.6.

For links with built-up tubular rectangular cross-sections, complete-joint-penetration groove welds shall be used to connect the webs to the flanges. Inaccessible backing bars need not be removed in these joints.

27.7.2.6

Flanges of built-up tubular links shall satisfy $b / t \leq 285 / \sqrt{F_y}$, where b is the clear flange width. Webs shall satisfy $h / w \leq 750 / \sqrt{F_y}$. The moment of inertia of built-up tubular links associated to horizontal, out-of-plane bending shall not be less than 0.67 times the link moment of inertia associated to bending in the vertical plane.

27.7.2.7

For web connected modular links, the flanges of the two C-sections shall be interconnected at both flange levels such that the clear longitudinal spacing between interconnections does not exceed 2.0 times the width of the flange of the individual C-sections.

When plates are used to reinforce the flanges of the C-sections in web connected modular links,

a) the flange reinforcement plates shall be continuously welded along their two longitudinal edges over the full length of the C-sections; and
b) the reinforced flanges shall satisfy Class 1 limit for flanges of I-sections in Table 2, where b_{el} is taken as the average of the C-section flange width and the flange reinforcement plate width and t is taken as the thickness of an equivalent flange having a moment of inertia for bending in the plane of the frame equal to that of the reinforced flange.

27.7.3 Link resistance

27.7.3.1 Factored link resistance

The factored shear resistance of the link shall be taken as the lesser of

$\phi V'_p$ and $2\phi M'_p/e$
where

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

where
V_p = $0.55wdF_y$ for links with wide-flange cross-sections
= $0.55(2w)dF_y$ for links with built-up tubular cross-sections and modular links with back-to-back C-sections
P_f = axial force in the link
= C_f or T_f
A = gross area of the link beam

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

e = length of the link (see Clause 27.7.4)

27.7.3.2 Probable link resistance

The nominal shear resistance of the link shall be taken equal to the lesser of V'_p and $2M'_p/e$, as defined in Clause 27.7.3.1, except that when P_f is equal to T_f, V'_p is given by

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

The probable resistance of the link shall be taken equal to $R_{sh}R_y$ times the link nominal shear resistance, with R_{sh} equal to 1.3, except that R_{sh} shall be taken equal to 1.45 for links with built-up tubular cross-sections and may be taken equal to 1.1 when $e > 2.6\ V_P/M_P$.

27.7.4 Link length

27.7.4.1

For end-plate connected modular links, the length of the link e shall be taken as the distance between the end plates. For web connected modular links, the length of the link e shall be taken as the distance between the innermost rows of bolts of the web connections, except that e shall be taken as the distance between end stiffeners when determining inelastic link rotation.

For links that consist of a segment of the beams, the length of the link e shall be taken as the clear distance between the ends of two braces. When a link is directly connected to a column, the link length is measured from the column face or from the link-to-column connection reinforcement.

27.7.4.2

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

a) when $\frac{A_w}{A} \ge 0.3\frac{V_f}{P_f}$: