10.11 Design of HSS and Box Member Connections

This section covers member strength design considerations pertaining to connections to HSS members and box sections of uniform wall thickness. See also Section 10.10 for additional requirements for bolting to HSS.

10.11.1 Concentrated Forces on HSS

10.11.1.1 **Definitions of Parameters**

B = overall width of rectangular *HSS* member, measured 90 degrees to the plane of the *connection,* mm.

B_p = width of plate, measured 90 degrees to the plane of the connection, mm.

D = outside diameter of round HSS member, mm.

F_y = *specified minimum yield stress* of HSS member material, MPa.

F_{yp} = specified minimum yield stress of plate, MPa.

F_u = *specified minimum tensile strength* of HSS material, MPa.

H = overall height of rectangular HSS member, measured in the plane of the connection, mm.

N = bearing length of the *load,* measured parallel to the axis of the HSS member, (or measured across the width of the HSS in the case of loaded cap plates), mm.

t = *design wall thickness* of HSS member, mm.

t_p = thickness of plate, mm.

10.11.1.2 **Limits of Applicability**

The criteria herein are applicable only when the connection configuration is within the following limits of applicability:

Strength: Fy ≤ 360 MPa. for HSS

Ductility: Fy/Fu ≤ 0.8 for HSS

Other limits apply for specific criteria

10.11.1.3 **Concentrated Force Distributed Transversely**

10.11.1.3.1 Criterion for Round HSS

When a concentrated *force* is distributed transversely to the axis of the *HSS* the *design strength,* ΦR_n and the *allowable strength,* R_n/Ω , for the *limit state* of *local yielding* shall be determined as follows:

$$R_n = F_y t^2 [5.5/(1 - 0.81 B_P/D)] Q_f \tag{10.11.1.1}$$

$\emptyset = 0.90$ (LRFD) Ω=1.67 (ASD)

where Q_f is given by Equation 10.11.2.1. Additional limits of applicability are

$0.2 < B_p / D \leq 1.0$

$D/t \leq 50$ for *T-connections* and $D/t \leq 40$ for *cross-connections*

10.11.1.3.2 Criterion for Rectangular HSS

When a concentrated force is distributed transversely to the axis of the HSS the design strength, ΦRn and the allowable strength, Rn/Ω, shall be the lowest value according to the limit states of local yielding due to uneven load distribution, shear yielding (punching) and sidewall strength.

Additional limits of applicability are

0.25 < Bp / B ≤ 1.0

B /t for the loaded HSS wall ≤ 35

For the limit state of local yielding due to uneven load distribution in the loaded plate,

$$R_n = \left[10/F_y t/(B/t)\right] B_P \leq F_{yP} t_P B_P \tag{10.11.1.2}$$

$\emptyset = 0.95$ (LRFD) Ω=1.58 (ASD)

For the limit state of shear yielding (punching),

$$R_n = 0.6F_y t\left[2t_P + 2B_{ep}\right] \qquad (10.11.1.3)$$

$\emptyset = 0.95$ (LRFD) Ω=1.58 (ASD)

Where

$$B_{ep} = 10B_p/(B/t) \leq B_p$$

This limit state need not be checked when Bp > (B − 2t), nor when Bp < 0.85 B .

For the limit state of sidewall under tension loading, the available strength shall be taken as the strength for sidewall local yielding. For the limit state of sidewall under compression loading, available strength shall be taken as the lowest value obtained according to the limit states of sidewall local yielding, sidewall local crippling and sidewall local buckling.

This limit state need not be checked unless the chord member and branch member (connecting element) have the same width (β = 1.0).

For the limit state of sidewall local yielding,

$$R_n = 2F_y t[5k + N] \qquad (10.11.1.4)$$

$\emptyset = 1.0$ (LRFD) Ω=1.50 (ASD)

where

k = outside corner radius of the HSS, which is permitted to be taken as 1.5t if unknown, mm.

For the limit state of sidewall local crippling, in T-connections,

$$R_n = 1.6t^2[1 + 3N/(H - 3t)]\left(EF_y\right)^{0.5}Q_f \qquad (10.11.1.5)$$

$\emptyset = 0.75$ (LRFD) Ω=2.0 (ASD)

where Qf is given by Equation 10.11.2.10.

For the limit state of sidewall local buckling in cross-connections,

$$R_n = [48t^3/(H - 3t)]\left(EF_y\right)^{0.5}Q_f \qquad (10.11.1.6)$$

$\emptyset = 0.90$ (LRFD) Ω=1.67 (ASD)

where Qf is given by Equation 10.11.2.10

The nonuniformity of load transfer along the line of weld, due to the flexibility of the HSS wall in a transverse plate-to-HSS connection, shall be considered in proportioning such welds. This requirement can be satisfied by limiting the total effective weld length, Le , of groove and fillet welds to rectangular HSS as follows:

$$L_e = 2[10/(B/t)]\left[(F_t t)/\left(F_{yp} t_p\right)\right]B_p \leq 2B_p \qquad (10.11.1.7)$$

where

Le = total effective weld length for welds on both sides of the transverse plate, mm.

In lieu of Equation 10.11.17, this requirement may be satisfied by other rational approaches.

10.11.1.4 **Concentrated Force Distributed Longitudinally at the Center of the HSS Diameter or Width and Acting Perpendicular to the HSS Axis**

When a concentrated *force* is distributed longitudinally along the axis of the *HSS* at the center of the HSS diameter or width, and also acts perpendicular to the axis direction of the HSS (or has a component perpendicular to the axis direction of the HSS), the *design strength,* ΦR_n and the *allowable strength,* R_n/Ω , perpendicular to the HSS axis shall be determined for the *limit state* of *chord plastification* as follows.

10.11.1.4.1 **Criterion for Round HSS**

An additional limit of applicability is:

D/t ≤ 50 for *T-connections* and *D/t* ≤ 40 for *cross-connections*

$$R_n = 5.5F_y t^2(1 + 0.25N/D)Q_f \qquad (10.11.1.8)$$

$\emptyset = 0.90$ (LRFD) Ω=1.67 (ASD)

where Q_f is given by Equation 10.11.2.1.

10.11.1.4.2 Criterion for Rectangular HSS

An additional limit of applicability is:

B/t for the loaded HSS wall ≤ 40

$$R_n = \left[F_y t^2/(1 - t_P/B)\right]\left[2N/B + 4(1 - t_P/B)^{0.5} Q_f\right] \tag{10.11.1.9}$$

$\emptyset = 1.00$ (LRFD) Ω=1.50 (ASD)

where

Q_f = $(1 - U^2)^{0.5}$

U is given by Equation 10.11.2.12

10.11.1.5 **Concentrated Force Distributed Longitudinally at the Center of the HSS Width and Acting Parallel to the HSS Axis**

When a concentrated *force* is distributed longitudinally along the axis of a rectangular *HSS* and also acts parallel but eccentric to the axis direction of the member, the *connection* shall be verified as follows:

$$F_{yP} t_P \le F_u t \tag{10.11.1.10}$$

10.11.1.6 **Concentrated Axial Force on the End of a Rectangular HSS with a Cap Plate**

When a concentrated *force* acts on the end of a capped *HSS* and the force is in the direction of the HSS axis, the *design strength,* ΦR_n and the allowable strength, R_n/Ω , shall be determined for the *limit states* of wall local *yielding* (due to tensile or compressive *forces*) and wall *local crippling* (due to compressive forces only), with consideration for shear lag, as follows.

If $(5t_p + N) \ge B$, the *available strength* of the HSS is computed by summing the contributions of all four HSS walls.

If $(5t_p + N) < B$, the available strength of the HSS is computed by summing the contributions of the two walls into which the *load* is distributed.

For the limit state of wall local yielding, for one wall,

$$R_n = F_y t[5t_P + N] \le BF_y t \tag{10.11.1.11}$$

$\emptyset = 1.00$ (LRFD) Ω=1.50 (ASD)

For the limit state of wall local crippling, for one wall,

$$R_n = 0.8t^2[1 + (6N/B)(t/t_P)^{1.5}]\left[EF_y t_P/t\right]^{0.5} \tag{10.11.1.12}$$

$\emptyset = 0.75$ (LRFD) Ω = 2.00 (ASD)

10.11.2 HSS-To-HSS Truss Connections

HSS-to-*HSS* truss *connections* are defined as connections that consist of one or more *branch members* that are directly welded to a continuous chord that passes through the connection and shall be classified as follows:

a) When the punching *load* (P_r sinθ) in a branch member is equilibrated by *beam* shear in the *chord member*, the connection shall be classified as a *T-connection* when the branch is perpendicular to the chord and a *Y-connection* otherwise.

b) When the punching load (P_r sinθ) ina branch member is essentially equilibrated (within 20 percent) by *loads* in other branch member(s) on the same side of the connection, the connection shall be classified as a *K-connection*. The relevant gap is between the primary branch members whose loads equilibrate. An *N-connection* can be considered as a type of K-connection.

c) When the punching load ($P_r$sinθ) is transmitted through the chord member and is equilibrated by branch member(s) on the opposite side, the connection shall be classified as a *cross-connection.*

d) When a connection has more than two primary branch members or branch members in more than one plane, the connection shall be classified as a general or multiplanar connection.

e) When branch members transmit part of their load as K-connections and part of their load as T-, Y-, or cross-connections, the *nominal strength* shall be determined by interpolation on the proportion of each in total.

For the purposes of this Specification, the centerlines of branch members and chord members shall lie in a common plane. Rectangular HSS connections are further limited to have all members oriented with walls parallel to the plane. For trusses that are made with HSS that are connected by welding branch members to chord members, eccentricities within the limits of applicability are permitted without consideration of the resulting moments for the design of the connection.

10.11.2.1 **Definitions of Parameters**

B = overall width of rectangular HSS main member, measured 90 degrees to the plane of the connection, mm.

B_b = overall width of rectangular HSS branch member, measured 90 degrees to the plane of the connection, mm.

D = outside diameter of round HSS main member, mm.

D_b = outside diameter of round HSS branch member, mm.

E = eccentricity in a truss connection, positive being away from the branches, mm.

F_y = *specified minimum yield stress of HSS* main member material, MPa.

F_{yb} = specified minimum yield stress of HSS branch member material, MPa.

F_u = *specified minimum tensile strength* of HSS material, MPa.

G =gap between toes of branch members in a gapped K-connection, neglecting the welds, mm.

H = overall height of rectangular HSS main member, measured in the plane of the connection, mm.

H_b = overall height of rectangular HSS branch member, measured in the plane of the connection, mm.

t = *design wall thickness* of HSS main member, mm.

t_b = design wall thickness of HSS branch member, mm.

β = the width ratio; the ratio of branch diameter to chord diameter = D_b / D for round HSS; the ratio of overall branch width to chord width = B_b / B for rectangular HSS

β_{eff} = the *effective width* ratio; the sum of the perimeters of the two branch members in a K-connection divided by eight times the chord width

γ = the chord slenderness ratio; the ratio of one-half the diameter to the wall thickness = $D/(2t)$ for round HSS; the ratio of one-half the width to wall thickness = $B/(2t)$ for rectangular HSS

H = the *load* length parameter, applicable only to rectangular *HSS*; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width = N/B, where $N = H_b /\sin\theta$

θ = acute angle between the branch and chord (degrees)

ξ = the gap ratio; the ratio of the gap between the branches of a gapped K-connection to the width of the chord = g/ B for rectangular HSS

10.11.2.2 **Criteria for Round HSS**

The interaction of stress due to *chord member forces* and local branch connection forces shall be incorporated through the chord-stress interaction parameter Q_f.

When the chord is in tension,

$$Q_f = 1$$

When the chord is in compression,

$$Q_f = 1.0 - 0.3U(1+U) \tag{10.11.2.1}$$

where U is the utilization ratio given by

$$U = \left|P_r/\left(A_g F_c\right) + M_r/(SF_c)\right| \tag{10.11.2.2}$$

and

P_r = required axial strength in chord, N; for K-connections, P_r is to be determined on the side of the *joint* that has the lower compression stress (lower U)

M_r = required flexural strength in chord, N-mm.

A_g = chord gross area, mm^2

F_c = *available stress*, MPa.

S = chord elastic section modulus, mm^3

For design according to Section 10.2.3.3 (LRFD):

$P_r = P_u$ = required axial strength in chord, using *LRFD load combinations*, N

$M_r = M_u$ = required flexural strength in chord, using LRFD load combinations, N-mm.

$F_c = F_y$, MPa.

For design according to Section 10.2.3.4 (ASD):

$P_r = P_a$ = required axial strength in chord, using *ASD load combinations*, N

$M_r = M_a$ = required flexural strength in chord, using ASD load combinations, N-mm.

$F_c = 0.6\ F_y$, MPa.

10.11.2.2.1 **Limits of Applicability**

The criteria herein are applicable only when the *connection* configuration is within the following limits of applicability:

Joint eccentricity: $-0.55\ D \le e \le 0.25\ D$, where D is the chord diameter and e is positive away from the branches

Branch angle: $\theta \ge 30^0$

Chord wall slenderness: ratio of diameter to wall thickness less than or equal to 50 for *T* -, *Y* - and *K-connections*; less than or equal to 40 for *cross-connections*

Tension branch wall slenderness: ratio of diameter to wall thickness less than or equal to 50

Compression branch wall slenderness: ratio of diameter to wall thickness less than or equal to $0.05\ E / F_y$

Width ratio: $0.2 < D_b / D \le 1.0$ in general, and $0.4 \le D_b / D \le 1.0$ for gapped K-connections

If a *gap connection:* g greater than or equal to the sum of the branch wall thicknesses

If an *overlap connection:* $25\% \le O_v \le 100\%$, where $O_v = (q / p) \times 100\%$. P is the projected length of the overlapping branch on the chord; q is the overlap length measured along the connecting face of the chord beneath the two branches. For overlap connections, the larger (or if equal diameter, the thicker) branch is a "thru member" connected directly to the chord.

Branch thickness ratio for overlap connections: thickness of overlapping branch to be less than or equal to the thickness of the overlapped branch

Strength: $F_y \le 360$ MPa. for chord and branches

Ductility: $F_y / F_u \le 0.8$

10.11.2.2.2 **Branches with Axial Loads in T-, Y- and Cross-Connections**

For T- and Y- connections, the *design strength* of the branch $\emptyset P_n$ or the *allowable strength* of the branch, P_n / Ω , shall be the lower value obtained according to the *limit states* of *chord plastification* and *shear yielding* (*punching*).

For the limit state of chord plastification in T- and Y-connections,

$$P_n \sin\theta = F_y t^2 [3.1 + 15.6\beta^2] \gamma^{0.2} Q_f \qquad (10.11.2.3)$$

$\emptyset = 0.90$ (LRFD) $\quad \Omega = 1.67$ (ASD)

For the limit state of shear yielding (punching),

$$P_n = 0.6 F_y t \pi D_b [(1 + \sin\theta)/2\sin^2\theta] \qquad (10.11.2.4)$$

$\emptyset = 0.95$ (LRFD) $\quad \Omega = 1.58$ (ASD)

This limit state need not be checked when $\beta > (1 - 1/\gamma)$.

For the limit state of chord plastification in cross-connections,

$$P_n \sin\theta = F_y t^2 [5.7/(1 - 0.81\beta)] Q_f \qquad (10.11.2.5)$$

$\emptyset = 0.90$ (LRFD) Ω = 1.67 (ASD)

10.11.2.2.3 **Branches with Axial Loads in K-Connections**

For K-connections, the design strength of the branch, ΦP_n and the allowable strength of the branch, P_n/Ω , shall be the lower value obtained according to the limit states of chord plastification for gapped and overlapped connections and shear yielding (punching) for gapped connections only.

For the limit state of chord plastification,

Φ = 0.90 (LRFD) Ω = 1.67 (ASD)

For the compression branch:

$$P_n \sin\theta = F_y t^2 [2.0 + 11.33 D_b/D] Q_g Q_f \qquad (10.11.2.6)$$

where D_b refers to the compression branch only, and

$$Q_g = \gamma^{0.2}\left[1 + \frac{0.024\gamma^{1.2}}{e^{\left(\frac{0.5g}{t} - 1.33\right)} + 1}\right] \qquad (10.11.2.7)$$

In gapped connections, g (measured along the crown of the chord neglecting weld dimensions) is positive. In overlapped connections, g is negative and equals q .

For the tension branch,

$$P_n \sin\theta = (P_n \sin\theta)_{\text{compression branch}} \qquad (10.11.2.8)$$

For the limit state of shear yielding (punching) in gapped K-connections,

$$P_n = 0.6 F_y t \pi D_b [(1 + \sin\theta)/2\sin^2\theta] \qquad (10.11.2.9)$$

ϕ = 0.95 (LRFD) Ω = 1.58 (ASD)

10.11.2.3 **Criteria for Rectangular HSS**

The interaction of *stress* due to *chord member forces* and local branch connection forces shall be incorporated through the chord-stress interaction parameter Q_f .

When the chord is in tension,

Q_f = 1

When the chord is in compression in *T -, Y -*, and *cross-connections,*

$$Q_f = 1.3 - 0.4U/\beta \leq 1 \qquad (10.11.2.10)$$

When the chord is in compression in gapped *K -connections,*

$$Q_f = 1.3 - 0.4U/\beta_{eff} \leq 1 \qquad (10.11.2.11)$$

where U is the utilization ratio given by

$$U = \left|P_r/(A_g F_c) + M_r/(S F_c)\right| \qquad (10.11.2.12)$$

and

P_r = required axial strength in chord, N. For gapped K-connections, P_r is to be determined on the side of the *joint* that has the higher compression stress (higher U).

M_r = required flexural strength in chord, N-mm.

A_g = chord gross area, mm^2

F_c = available stress, MPa.

S = chord elastic section modulus, mm^3

For design according to Section 10.2.3.3 (LRFD):

P_r = P_u = required axial strength in chord, using *LRFD load combinations*, N

M_r = M_u = required flexural strength in chord, using LRFD load combinations, N-mm.

F_c = F_y , MPa.

For design according to Section 10.2.3.4 (ASD):

P_r = P_a = required axial strength in chord, using *ASD load combinations*, N.

$M_r = M_a$ = required flexural strength in chord, using ASD load combinations, N-mm.

F_c = 0.6 F_y , MPa.

10.11.2.3.1 Limits of Applicability

The criteria herein are applicable only when the connection configuration is within the following limits:

Joint eccentricity: $-0.55\ H \le e \le 0.25H$, where H is the chord depth and e is positive away from the branches

Branch angle: $\theta \ge 30^0$

Chord wall slenderness: ratio of overall wall width to thickness less than or equal to 35 for gapped K-connections and T-, Y- and cross-connections; less than or equal to 30 for overlapped K-connections

Tension branch wall slenderness: ratio of overall wall width to thickness less than or equal to 35

Compression branch wall slenderness: ratio of overall wall width to thickness less than or equal to $1.25(E/F_{yb})^{0.5}$ and also less than 35 for gapped K-connections and T-, Y- and cross-connections; less than or equal to $1.1(E/F_{yb})^{0.5}$ for overlapped K-connections

Width ratio: ratio of overall wall width of branch to overall wall width of chord greater than or equal to 0.25 for T-, Y-, cross- and overlapped K-connections; greater than or equal to 0.35 for gapped K-connections

Aspect ratio: 0.5 ≤ ratio of depth to width ≤ 2.0

Overlap: $25\% \le O_v \le 100\%$, where $O_v = (q\ /\ p) \times 100\%$. p is the projected length of the overlapping branch on the chord; q is the overlap length measured along the connecting face of the chord beneath the two branches. For overlap connections, the larger (or if equal width, the thicker) branch is a “thru member” connected directly to the chord

Branch width ratio for *overlap connections*: ratio of overall wall width of overlapping branch to overall wall width of overlapped branch greater than or equal to 0.75

Branch thickness ratio for overlap connections: thickness of overlapping branch to be less than or equal to the thickness of the overlapped branch

Strength: $F_y \le 360$ MPa. for chord and branches

Ductility: $F_y / F_u \le 0.8$

Other limits apply for specific criteria

10.11.2.3.2 Branches with Axial Loads in T-,Y- and Cross-Connections

For T-, Y- and cross-connections, the *design strength* of the branch, $\emptyset\ P_n$ or the *allowable strength* of the branch, P_n/Ω , shall be the lowest value obtained according to the *limit states* of *chord wall plastification, shear yielding* (*punching*), sidewall strength and *local yielding* due to uneven load distribution. In addition to the limits of applicability in Section 10.11.2.3a, β shall not be less than 0.25.

For the limit state of chord wall plastification,

$$P_n \sin\theta = F_y t^2 [2\eta/(1-\beta) + 4/(1-\beta)^{0.5}] Q_f \qquad (10.11.2.13)$$

Φ = 1.00 (LRFD) Ω = 1.50 (ASD)

This limit state need not be checked when β > 0.85.

For the limit state of shear yielding (punching),

$$P_n \sin\theta = 0.6 F_y t B [2\eta + 2\beta_{eoP}] \qquad (10.11.2.14)$$

Φ = 0.95 (LRFD) Ω = 1.58 (ASD)

In Equation 10.11.2.14, the effective outside punching parameter $\beta_{eop} = 5\beta/\gamma$ shall not exceed β .

This limit state need not be checked when $\beta > (1 - 1/\gamma)$, nor when $\beta < 0.85$ and $B/t \ge 10$.

For the limit state of sidewall strength, the *available strength* for branches in tension shall be taken as the available strength for sidewall local yielding. For the limit state of sidewall strength, the available strength for branches in compression shall be taken as the lower of the strengths for sidewall local yielding and sidewall local crippling. For cross-connections with a branch angle less than 90 degrees, an additional check for chord sidewall shear failure must be made in accordance with Section 10.7.5.

This limit state need not be checked unless the chord member and branch member have the same width (β = 1.0)

For the limit state of local yielding,

$$P_n \sin\theta = 2F_y t[5k + N] \qquad (10.11.2.15)$$

Φ = 1.00 (LRFD) Ω = 1.50 (ASD)

where

k = outside corner radius of the HSS, which is permitted to be taken as 1.5*t* if unknown, mm.

N = bearing length of the *load*, parallel to the axis of the HSS main member, $H_b/\sin\theta$, mm.

For the limit state of sidewall local crippling, in T- and Y-connections,

$$P_n \sin\theta = 1.6t^2[1 + 3N/(H - 3t)]\left(EF_y\right)^{0.5} Q_f \qquad (10.11.2.16)$$

Φ = 0.75 (LRFD) Ω = 2.00 (ASD)

For the limit state of sidewall local crippling in cross-connections,

$$P_n \sin\theta = [48t^3/(H - 3t)]\left(EF_y\right)^{0.5} Q_f \qquad (10.11.2.17)$$

Φ = 0.90 (LRFD) Ω = 1.67 (ASD)

For the limit state of local yielding due to uneven load distribution,

$$P_n = F_{yb} t_b [2H_b + 2b_{eoi} - 4t_b] \qquad (10.11.2.18)$$

Φ = 0.95 (LRFD) Ω = 1.58 (ASD)

where

$$b_{eoi} = [10/(B/t)]\left[F_y t/(F_{yb} t_b)\right] B_b \leq B_b \qquad (10.11.2.19)$$

This limit state need not be checked when $\beta < 0.85$.

10.11.2.3.3 Branches with Axial Loads in Gapped K-Connections

For gapped K-connections, the design strength of the branch, ΦP_n or the allowable strength of the branch, P_n/Ω, shall be the lowest value obtained according to the limit states of chord wall plastification, *shear yielding* (punching), shear yielding and local yielding due to uneven load distribution. In addition to the limits of applicability in Section 10.11.2.3a, the following limits shall apply:

a) $B_b/B \geq 0.1 + \gamma/50$
b) $\beta_{eff} \geq 0.35$
c) $\zeta \leq 0.5(1 - \beta_{eff})$
d) Gap: *g* greater than or equal to the sum of the branch wall thicknesses
e) The smaller $B_b > 0.63$ times the larger B_b

For the limit state of chord wall plastification,

$$P_n \sin\theta = F_y t^2\left[9.8\beta_{eff}\gamma^{0.5}\right] Q_f \qquad (10.11.2.20)$$

Φ = 0.90 (LRFD) Ω = 1.67 (ASD)

For the limit state of shear yielding (punching),

$$P_n \sin\theta = 0.6F_y tB\left[2\eta + \beta + \beta_{eop}\right] \qquad (10.11.2.21)$$

Φ = 0.95 (LRFD) Ω = 1.58 (ASD)

In the above equation, the effective outside punching parameter $\beta_{eop} = 5\,\beta/\gamma$ shall not exceed β.

This limit state need only be checked if $B_b < (B - 2t)$ or the branch is not square.

For the limit state of shear yielding of the chord in the gap, available strength shall be checked in accordance with Section 10.7. This limit state need only be checked if the chord is not square.

For the limit state of local yielding due to uneven load distribution,

$$P_n = F_{yb} t_b [2H_b + B_b + b_{eoi} - 4t_b] \qquad (10.11.2.22)$$

Φ = 0.95 (LRFD) Ω = 1.58 (ASD)

where

$$b_{eoi} = [10/(B/t)]\left[F_y t/(F_{yb} t_b)\right] B_b \leq B_b \qquad (10.11.2.23)$$

This limit state need only be checked if the branch is not square or B/t <15.

10.11.2.3.4 Branches with Axial Loads in Overlapped K-Connections

For overlapped K-connections, the design strength of the branch, ΦP_n or the allowable strength of the branch, P_n/Ω shall be determined from the limit state of local yielding due to uneven load distribution,

Φ = 0.95 (LRFD) Ω = 1.58 (ASD)

For the overlapping branch and for overlap 25% ≤ O_v ≤ 50% measured with respect to the overlapping branch,

$$P_n = F_{ybi} t_{bi} [(O_v/50)(2H_{bi} - 4t_{bi}) + b_{eoi} + b_{eov}] \tag{10.11.2.24}$$

For the overlapping branch, and for overlap 50% ≤ O_v < 80% measured with respect to the overlapping branch,

$$P_n = F_{ybi} t_{bi} [2H_{bi} - 4t_{bi} + b_{eoi} + b_{eov}] \tag{10.11.2.25}$$

For the overlapping branch and for overlap 80% ≤ O_v ≤ 100% measured with respect to the overlapping branch,

$$P_n = F_{ybi} t_{bi} [2H_{bi} - 4t_{bi} + B_{bi} + b_{eov}] \tag{10.11.2.26}$$

where

b_{eoi} is the *effective width* of the *branch face* welded to the chord,

$$b_{eoi} = [10/(B/t)]\big[(F_y t)/(F_{ybi} t_{bi})\big] B_{bi} \le B_{bi} \tag{10.11.2.27}$$

b_{eov} is the effective width of the branch face welded to the overlapped brace,

$$b_{eov} = \big[10/\big(B_{bj}/t_{bj}\big)\big]\big[(F_{ybj} t_{bj})/(F_{ybi} t_{bi})\big] B_{bi} \le B_{bi} \tag{10.11.2.28}$$

B_{bi} = overall branch width of the overlapping branch, mm.

B_{bj} = overall branch width of the overlapped branch, mm.

F_{ybi} = *specified minimum yield stress* of the overlapping branch material, MPa.

F_{ybj} = specified minimum yield stress of the overlapped branch material, MPa.

H_{bi} = overall depth of the overlapping branch, mm.

T_{bi} = thickness of the overlapping branch, mm.

t_{bj} = thickness of the overlapped branch, mm.

For the overlapped branch, P_n shall not exceed P_n of the overlapping branch, calculated using Equation 10.11.2.24, 10.11.2.25, or 10.11.2.26, as applicable, multiplied by the factor ($A_{bj} F_{ybj} / A_{bi} F_{ybi}$),

where

A_{bi} = cross-sectional area of the overlapping branch

A_{bj} = cross-sectional area of the overlapped branch

10.11.2.3.5 Welds to Branches

The nonuniformity of load transfer along the line of weld, due to differences in relative flexibility of HSS walls in HSS-to-HSS connections, shall be considered in proportioning such welds. This can be considered by limiting the total effective weld length, L_e , of groove and *fillet welds* to rectangular HSS as follows:

In T-, Y- and cross-connections,

for $\theta \le 50$ degrees

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} + (B_b - 1.2t_b) \tag{10.11.2.29}$$

for $\theta \ge 60$ degrees

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} \tag{10.11.2.30}$$

Linear interpolation shall be used to determine L_e for values of θ between 50 and 60 degrees.

In gapped K-connections, around each branch,

for $\theta \le 50$ degrees

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} + 2(B_b - 1.2t_b) \qquad (10.11.2.31)$$

for θ ≥ 60 degrees

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} + (B_b - 1.2t_b) \qquad (10.11.2.32)$$

Linear interpolation shall be used to determine L_e for values of θ between 50 and 60 degrees.

In lieu of the above criteria in Equations 10.11.2.29 to 10.11.2.32, other rational criteria are permitted.

10.11.3 HSS-To-HSS Moment Connections

HSS-to-HSS *moment connections* are defined as *connections* that consist of one or two *branch members* that are directly welded to a continuous chord that passes through the connection, with the branch or branches loaded by bending moments. A connection shall be classified

As a *T-connection* when there is one branch and it is perpendicular to the chord and as a *Y-connection* when there is one branch but not perpendicular to the chord.

As a *cross-connection* when there is a branch on each (opposite) side of the chord.

For the purposes of this Specification, the centerlines of the branch member(s) and the *chord member* shall lie in a common plane.

10.11.3.1 **Definitions of Parameters**

B = overall width of rectangular HSS main member, measured 90 degrees to the plane of the connection, mm.

B_b = overall width of rectangular HSS branch member, measured 90 degrees to the plane of the connection, mm.

D = outside diameter of round HSS main member, mm.

D_b = outside diameter of round HSS branch member, mm.

F_y = *specified minimum yield stress* of HSS main member, MPa.

F_{yb} = specified minimum yield stress of HSS branch member, MPa.

F_u = ultimate strength of HSS member, MPa.

H = overall height of rectangular HSS main member, measured in the plane of the connection, mm.

H_b = overall height of rectangular HSS branch member, measured in the plane of the connection, mm.

t = *design wall thickness* of HSS main member, mm.

t_b = design wall thickness of HSS branch member, mm.

β = the width ratio; the ratio of branch diameter to chord diameter = D_b / D for round HSS; the ratio of overall branch width to chord width = B_b / B for rectangular HSS

γ = the chord slenderness ratio; the ratio of one-half the diameter to the wall thickness = $D/(2t)$ for round HSS; the ratio of one-half the width to wall thickness = $B/(2t)$ for rectangular HSS

η = the *load* length parameter, applicable only to rectangular HSS; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width = N / B, where $N = H_b / \sin\theta$

θ = acute angle between the branch and chord (degrees)

10.11.3.2 **Criteria for Round HSS**

The interaction of *stress* due to *chord member forces* and local branch *connection forces* shall be incorporated through the chord-stress interaction parameter Q_f.

When the chord is in tension,

$Q_f = 1$

When the chord is in compression,

$$Q_f = 1.0 - 0.3U(1 + U) \qquad (10.11.3.1)$$

where U is the utilization ratio given by

$$U = \left|P_r/A_gF_c + M_r/SF_c\right| \qquad (10.11.3.2)$$

and

P_r = required axial strength in chord, N.

M_r = required flexural strength in chord, N-mm.

A_g = chord gross area, mm^2

F_c = available stress, MPa.

S = chord elastic section modulus, mm^3

For design according to Section 10.2.3.3 (LRFD):

$P_r = P_u$ = required axial strength in chord, using *LRFD load combinations*, N

$M_r = M_u$ = required flexural strength in chord, using LRFD load combinations, N-mm.

$F_c = F_y$, MPa.

For design according to Section 10.2.3.4 (ASD):

$P_r = P_a$ = required axial strength in chord, using *ASD load combinations*, N

$M_r = M_a$ = required flexural strength in chord, using ASD load combinations, N-mm.

$F_c = 0.6\ F_y$, MPa.

10.11.3.2.1 Limits of Applicability

The criteria herein are applicable only when the connection configuration is within the following limits of applicability:

Branch angle: $\theta \geq 30^0$

Chord wall slenderness: ratio of diameter to wall thickness less than or equal to 50 for *T* - and *Y-connections;* less than or equal to 40 for *cross-connections*

Tension branch wall slenderness: ratio of diameter to wall thickness less than or equal to 50

Compression branch wall slenderness: ratio of diameter to wall thickness less than or equal to 0.05 E/F_y

Width ratio: $0.2 < D_b/D \leq 1.0$

Strength: $F_y \leq$ 360 MPa. for chord and branches

Ductility: $F_y/F_u \leq 0.8$

10.11.3.2.2 Branches with In-Plane Bending Moments in T-, Y- and Cross-Connections

The *design strength,* $\emptyset M_n$ and the *allowable strength,* M_n/Ω, shall be the lowest value obtained according to the *limit states* of *chord plastification* and *shear yielding* (*punching*).

For the limit state of chord plastification,

$$M_n \sin\theta = 5.39F_yt^2\gamma^{0.5}\beta D_bQ_f \qquad (10.11.3.3)$$

$\emptyset = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

For the limit state of shear yielding (punching),

$$M_n = 0.6F_ytD_b^2[(1+3\sin\theta)/4\sin^2\theta] \qquad (10.11.3.4)$$

$\emptyset = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)

This limit state need not be checked when $\beta > (1-1/\gamma)$.

10.11.3.2.3 Branches with Out-of-Plane Bending Moments in T-, Y- and Cross-Connections

The *design strength,* ΦM_n and the allowable strength, M_n/Ω, shall be the lowest value obtained according to the limit states of chord plastification and shear yielding (punching).

For the limit state of chord plastification,

$$M_n \sin\theta = F_yt^2D_b[3.0/(1-0.81\beta)]Q_f \qquad (10.11.3.5)$$

$\emptyset = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

For the limit state of shear yielding (punching),

$$M_n = 0.6F_ytD_b^2[(3+\sin\theta)/4\sin^2\theta]Q_f \qquad (10.11.3.6)$$

$\emptyset$ = 0.95 (LRFD) Ω = 1.58 (ASD)

This limit state need not be checked when $\beta > (1 - 1/\gamma)$.

10.11.3.2.4 Branches with Combined Bending Moment and Axial Force in T-, Y- and Cross-Connections

Connections subject to branch axial load, branch in-plane bending moment, and branch out-of-plane bending moment, or any combination of these *load effects,* should satisfy the following.

For design according to Section 10.2.3.3 (LRFD):

$$(P_r/\emptyset P_n) + (M_{r-iP}/\emptyset M_{n-iP})^2 + (M_{r-oP}/\emptyset M_{n-oP}) \leq 1.0 \qquad (10.11.3.7)$$

where

P_r = P_u = required axial strength in branch, using LRFD load combinations, N

$\emptyset P_n$= design strength obtained from Section 10.11.2.2.2

M_{r-ip}= required in-plane flexural strength in branch, using LRFD load combinations, N-mm.

$\emptyset M_{n-ip}$ = design strength obtained from Section 10.11.3.2.2

M_{r-op} = required out-of-plane flexural strength in branch, using LRFD load combinations, N-mm.

$\emptyset M_{n-op}$ = design strength obtained from Section 10.11.3.2.3

For design according to Section 10.2.3.4 (ASD):

$$(P_r/(P_n/\Omega)) + (M_{r-iP}/(M_{n-iP}/\Omega))^2 + (M_{r-oP}/(M_{n-oP}/\Omega)) \leq 1.0 \qquad (10.11.3.8)$$

where

P_r = P_a = required axial strength in branch, using ASD load combina- tions, N

P_n/Ω = *allowable strength* obtained from Section 10.11.2.2.2

M_{r-ip} =required in-plane flexural strength in branch, using ASD load combinations, N-mm.

M_{n-ip}/Ω = allowable strength obtained from Section 10.11.3.2.2

M_{r-op} =required out-of-plane flexural strength in branch, using ASD load combinations, N-mm.

M_{n-op}/Ω = allowable strength obtained from Section 10.11.3.2.3

10.11.3.3 **Criteria for Rectangular HSS**

The interaction of *stress* due to *chord member forces* and local branch *connection* forces shall be incorporated through the chord-stress interaction parameter Q_f .

When the chord is in tension,

Q_f = 1

When the chord is in compression,

$$Q_f = (1.3 - 0.4U/\beta) \leq 1 \qquad (10.11.3.9)$$

where U is the utilization ratio given by

$$U = \left|P_r/A_g F_c + M_r/SF_c\right| \qquad (10.11.3.10)$$

and

P_r = required axial strength in chord, N.

M_r = required flexural strength in chord, N-mm.

A_g = chord gross area, mm^2

F_c = *available stress*, MPa.

S = chord elastic section modulus, mm^3.

For design according to Section 10.2.3.3 (LRFD):

P_r = P_u = required axial strength in chord, using *LRFD load combinations*, N

M_r = M_u = required flexural strength in chord, using LRFD load combinations, N-mm.

F_c = F_y , MPa.

For design according to Section 10.2.3.4 (ASD):

P_r = P_a = required axial strength in chord, using *ASD load combinations*, N

$M_r = M_a$ = required flexural strength in chord, using ASD load combinations, N-mm.

$F_c = 0.6\ F_y$, MPa.

10.11.3.3.1 Limits of Applicability

The criteria herein are applicable only when the *connection* configuration is within the following limits:

Branch angle is approximately 90^0

Chord wall slenderness: ratio of overall wall width to thickness less than or equal to 35

Tension branch wall slenderness: ratio of overall wall width to thickness less than or equal to 35

Compression branch wall slenderness: ratio of overall wall width to thickness less than or equal to $1.25(E/F_{yb})^{0.5}$ and also less than 35

Width ratio: ratio of overall wall width of branch to overall wall width of chord greater than or equal to 0.25

Aspect ratio: 0.5 ≤ ratio of depth to width ≤ 2.0

Strength: F_y ≤ 360 MPa. for chord and branches

Ductility: F_y / F_u ≤ 0.8

Other limits apply for specific criteria

10.11.3.3.2 Branches with In-Plane Bending Moments in T- and Cross-Connections

The *design strength,* $\emptyset M_n$ and the *allowable strength,* M_n/Ω, shall be the lowest value obtained according to the *limit states* of *chord wall plastification,* sidewall *local yielding* and local yielding due to *uneven load distribution.*

For the limit state of chord wall plastification,

$$M_n = F_y t^2 H_b[(1/2\eta) + 2/(1-\beta)^{0.5} + \eta/(1-\beta)]Q_f \qquad (10.11.3.11)$$

∅ = 1.00 (LRFD) Ω = 1.50 (ASD)

This limit state need not be checked when β > 0.85.

For the limit state of sidewall local yielding,

$$M_n = 0.5F_y^* t(H_b + 5t)^2 \qquad (10.11.3.12)$$

∅ = 1.00 (LRFD) Ω = 1.50 (ASD)

where

$F_y^* = F_y$ for *T-connections*

$F_y^* = 0.8\ F_y$ for *cross-connections*

This limit state need not be checked when β < 0.85.

For the limit state of local yielding due to uneven load distribution,

$$M_n = F_{yb}[Z_b - (1 - b_{eoi}/B_b)B_b H_b t_b] \qquad (10.11.3.13)$$

∅ = 0.95 (LRFD) Ω = 1.58 (ASD)

Where

$$b_{eoi} = [10/(B/t)][F_y t/(F_{yb} t_b)]B_b \le B_b \qquad (10.11.3.14)$$

Z_b = branch plastic section modulus about the axis of bending, mm^3.

This limit state need not be checked when β < 0.85.

10.11.3.3.3 **Branches with Out-of-Plane Bending Moments in T- and Cross-Connections**

The *design strength,* $\emptyset M_n$ and the *allowable strength,* M_n/Ω, shall be the lowest value obtained according to the limit states of chord wall plastification, sidewall local yielding, local yielding due to uneven load distribution and chord *distortional failure.*

For the limit state of chord wall plastification,

$$M_n = F_y t^2[0.5H_b(1+\beta)/(1-\beta) + [2BB_b(1+\beta)/(1-\beta)]^{0.5}]Q_f \qquad (10.11.3.15)$$

∅ = 1.00 (LRFD) Ω = 1.50 (ASD)

This limit state need not be checked when β > 0.85.

For the limit state of sidewall local yielding,

$$M_n = F_y^* t(B-t)(H_b + 5t) \quad (10.11.3.16)$$

$\emptyset$ = 1.00 (LRFD) Ω = 1.50 (ASD)

where

F_y^* = F_y for T-connections

F_y^* = 0.8 F_y for cross-connections

This limit state need not be checked when β < 0.85.

For the limit state of local yielding due to uneven load distribution,

$$M_n = F_{yb}\left[Z_b - 0.5(1 - b_{eoi}/B_b)^2 B_b^2 t_b\right] \quad (10.11.3.17)$$

$\emptyset$ = 0.95 (LRFD) Ω = 1.58 (ASD)

where

$$b_{eoi} = [10/(B/t)]\left[F_y t/(F_{yb} t_b)\right] B_b \le B_b \quad (10.11.3.18)$$

Z_b = branch plastic section modulus about the axis of bending, mm^3.

This limit state need not be checked when β < 0.85.

For the limit state of chord distortional failure,

$$M_n = 2F_y t[H_b t + [BHt(B+H)]^{0.5}] \quad (10.11.3.19)$$

$\emptyset$ = 1.00 (LRFD) Ω = 1.50 (ASD)

This limit state need not be checked for cross-connections or for T-connections if chord distortional failure is prevented by other means.

10.11.3.3.4 **Branches with Combined Bending Moment and Axial Force in T- and Cross-Connections**

Connections subject to branch axial *load*, branch in-plane bending moment and branch out-of-plane bending moment, or any combination of these *load effects,* should satisfy

For design according to Section 10.2.3.3 (LRFD)

$$(P_r/\emptyset P_n) + (M_{r-iP}/\emptyset M_{n-iP}) + (M_{r-oP}/\emptyset M_{n-oP}) \le 1.0 \quad (10.11.3.20)$$

where

P_r = P_u = required axial strength in branch, using LRFD load combinations, N

$\emptyset P_n$= design strength obtained from Section 10.11.2.3.2

M_{r-ip}= required in-plane flexural strength in branch, using LRFD load combinations, N-mm.

$\emptyset M_{n-ip}$ = design strength obtained from Section 10.11.3.3.2

M_{r-op} = required out-of-plane flexural strength in branch, using LRFD load combinations, N-mm.

$\emptyset M_{n-op}$ = design strength obtained from Section 10.11.3.3.3

For design according to Section 10.2.3.4 (ASD)

$$(P_r/(P_n/\Omega)) + (M_{r-iP}/(M_{n-iP}/\Omega)) + (M_{r-oP}/(M_{n-oP}/\Omega)) \le 1.0 \quad (10.11.3.21)$$

where

P_r = P_a = required axial strength in branch, using ASD load combinations, N

P_n/Ω = allowable strength obtained from Section 10.11.2.3.2

M_{r-ip} = required in-plane flexural strength in branch, using ASD load combinations, N-mm.

M_{n-ip}/Ω = allowable strength obtained from Section 10.11.3.3.2

M_{r-op} = required out-of-plane flexural strength in branch, using ASD load combinations, N-mm.

M_{n-op}/Ω = allowable strength obtained from Section 10.11.3.3.3

10.12 Design for Serviceability

This chapter addresses *serviceability* performance design requirements.

10.12.1 General Provisions

Serviceability is a state in which the function of a building, its appearance, maintainability, durability, and comfort of its occupants are preserved under normal usage. Limiting values of structural behavior for serviceability (for example, maximum deflections, accelerations) shall be chosen with due regard to the intended function of the structure. Serviceability shall be evaluated using appropriate *load combinations* for the serviceability *limit states* identified.

10.12.2 Camber

Where *camber* is used to achieve proper position and location of the structure, the magnitude, direction and location of camber shall be specified in the structural drawings in accordance with the provisions of Chapter 1.

10.12.3 Deflections

Deflections in structural members and structural systems under appropriate *service load combinations* shall not impair the *serviceability* of the structure. Limiting values of deflections of various structural members shall be in accordance with those specified in Section 1.4 of Chapter 1.

10.12.4 Drift

Drift of a structure shall be evaluated under *service loads* to provide for *serviceability* of the structure, including the integrity of interior partitions and exterior *cladding*. *Drift* under strength *load combinations* shall not cause collision with adjacent structures or exceed the limiting values specified in Section 1.5.6 of Chapter 1.

10.12.5 Vibration

The effect of vibration on the comfort of the occupants and the function of the structure shall be considered. Sources of vibration to be considered include pedestrian loading, vibrating machinery and others identified for the structure. It must be shown by any rational method of analysis that the vibrations induced by any source including the above mentioned ones is within tolerable limit and shall not cause any adverse effect on the safety, stability and durability of the structure.

10.12.6 Wind-Induced Motion

The effect of wind-induced motion of buildings on the comfort of occupants shall be considered. For flexible building and structures as defined in Sec. 2.4.2, it must be shown by a rational dynamic analysis that wind induced vibration does not cause any discomfort to occupants as well as the wind induced dynamic effect does not cause any adverse effect on the safety, stability and durability of the structure.

10.12.7 Expansion and Contraction

The effects of thermal expansion and contraction of a building shall be considered. Damage to building *cladding* can cause water penetration and may lead to corrosion.

10.12.8 Connection Slip

The effects of *connection* slip shall be included in the design where slip at bolted connections may cause deformations that impair the *serviceability* of the structure. Where appropriate, the connection shall be designed to preclude slip. For the design of slip-critical connections see Sections 10.10.3.8 and 10.10.3.9.

10.13 Fabrication, Erection and Quality Control

This chapter addresses requirements for design and shop drawings, fabrication, shop painting, erection and quality control.

10.13.1 DESIGN DRAWINGS AND SPECIFICATIONS

Structural Design Drawings and Specifications

Unless otherwise indicated in the contract documents, the structural design drawings shall be based upon consideration of the design loads and forces to be resisted by the structural steel frame in the completed project.

The structural design drawings shall clearly show the work that is to be performed and shall give the following information with sufficient dimensions to accurately convey the quantity and nature of the structural steel to be fabricated:

a) The size, section, material grade and location of all members;
b) All geometry and working points necessary for layout;
c) Floor elevations;
d) Column centers and offsets;
e) The camber requirements for members;
f) Joining requirements between elements of built-up members; and,
g) The information that is required in Sections 10.13.1.1.1 through 10.13.1.1.6.

The structural steel specifications shall include any special requirements for the fabrication and erection of the structural steel.

The structural design drawings, specifications and addenda shall be numbered and dated for the purposes of identification.

10.13.1.1 **Detailing of Components**

Permanent bracing, column stiffeners, column web doubler plates, bearing stiffeners in beams and girders, web reinforcement, openings for other trades and other special details, where required, shall be shown in sufficient detail in the structural design drawings so that the quantity, detailing and fabrication requirements for these items can be readily understood.

10.13.1.2 **Designer's Responsibility**

The owner's designated representative for design shall indicate one of the following options for each connection:

(1) The complete connection design shall be shown in the structural design drawings;

(2) In the structural design drawings or specifications, the connection shall be designated to be selected or completed by an experienced steel detailer; or,

(3) In the structural design drawings or specifications, the connection shall be designated to be designed by a licensed professional engineer working for the fabricator.

In all of the above options,

(a) The requirements of Section 10.13.1.1 shall apply; and,

(b) The approvals process in Section 10.13.2.4 shall be followed.

When option (2) above is specified:

The experienced steel detailer shall utilize tables or schematic information provided in the structural design drawings in the selection or completion of the connections. When such information is not provided, standard reference information as approved by the owner's designated representative for design, shall be used.

When option (2) or (3) above is specified

The owner's designated representative for design shall provide the following information in the structural design drawings and specifications:

a) Any restrictions on the types of connections that are permitted;
b) Data concerning the loads, including shears, moments, axial forces and transfer forces, that are to be resisted by the individual members and their connections, sufficient to allow the selection, completion, or design of the connection details while preparing the shop and erection drawings;
c) Whether the data required in (b) is given at the service-load level or the factored-load level;
d) Whether LRFD or ASD is to be used in the selection, completion, or design of connection details; and,
e) What substantiating connection information, if any, is to be provided with the shop and erection drawings to the owner's designated representative for design.

When option (3) above is specified:

a) The fabricator shall submit in a timely manner representative samples of the required substantiating connection information to the owner's designated representatives for design and construction. The owner's designated representative for design shall confirm in writing in a timely manner that these representative samples are consistent with the requirements in the contract documents, or shall advise what modifications are required to bring the representative samples into compliance with the requirements in the contract documents. This initial submittal and review is in addition to the requirements in Section 10.13.2.4.
b) The licensed professional engineer in responsible charge of the connection design shall review and confirm in writing as part of the substantiating connection information, that the shop and erection drawings properly incorporate the connection designs. However, this review by the licensed professional engineer in responsible charge of the connection design does not replace the approval process of the shop and erection drawings by the owner's designated representative for design in Section 10.13.2.4.
c) The fabricator shall provide a means by which the substantiating connection information is referenced to the related connections on the shop and erection drawings for the purpose of review.

10.13.1.2.1 Levelling Plates

When leveling plates are to be furnished as part of the contract requirements, their locations and required thickness and sizes shall be specified in the contract documents.

10.13.1.2.2 Non-Structural Elements

When the structural steel frame, in the completely erected and fully connected state, requires interaction with non-structural steel elements (see Section 2) for strength and/or stability, those non-structural steel elements shall be identified in the contract documents as required in Section 10.13.5.10.

10.13.1.2.3 Camber

When camber is required, the magnitude, direction and location of camber shall be specified in the structural design drawings.

10.13.1.2.4 Painting Information

Specific members or portions thereof that are to be left unpainted shall be identified in the contract documents. When shop painting is required, the painting requirements shall be specified in the contract documents, including the following information:

a) The identification of specific members or portions thereof to be painted;
b) The surface preparation that is required for these members;
c) The paint specifications and manufacturer's product identification that are required for these members; and,
d) The minimum dry-film shop-coat thickness that is required for these members.

10.13.1.3 **Architectural, Electrical and Mechanical Design Drawings and Specifications**

All requirements for the quantities, sizes and locations of structural steel shall be shown or noted in the structural design drawings. The use of architectural, electrical and/or mechanical design drawings as a supplement to the structural design drawings is permitted for the purposes of defining detail configurations and construction information.

10.13.1.4 **Discrepancies**

When discrepancies exist between the design drawings and specifications, the design drawings shall govern. When discrepancies exist between scale dimensions in the design drawings and the figures written in them, the figures shall govern. When discrepancies exist between the structural design drawings and the architectural, electrical or mechanical design drawings or design drawings for other trades, the structural design drawings shall govern.

When a discrepancy is discovered in the contract documents in the course of the fabricator's work, the fabricator shall promptly notify the owner's designated representative for construction so that the discrepancy can be resolved by the owner's designated representative for design. Such resolution shall be timely so as not to delay the fabricator's work. See Sections 10.13.1.5 and 10.13.7.3.

10.13.1.5 **Legibility of Design Drawings**

Design drawings shall be clearly legible and drawn to an identified scale that is appropriate to clearly convey the information.

10.13.1.6 **Revisions to the Design Drawings and Specifications**

Revisions to the design drawings and specifications shall be made either by issuing new design drawings and specifications or by reissuing the existing design drawings and specifications. In either case, all revisions, including revisions that are communicated through responses to RFIs or the annotation of shop and/or erection drawings (see Section 10.13.2.4.2), shall be clearly and individually indicated in the contract documents. The contract documents shall be dated and identified by revision number. Each design drawings shall be identified by the same drawing number throughout the duration of the project, regardless of the revision. See also Section 10.13.7.3.

10.13.2 Shop and Erection Drawings

Shop drawings shall be prepared in advance of fabrication and give complete information necessary for the fabrication of the component parts of the structure, including the location, type and size of welds and bolts. Erection drawings shall be prepared in advance of erection and give information necessary for erection of the structure. Shop and erection drawings shall clearly distinguish between shop and field welds and bolts and shall clearly identify pretensioned and slip-critical high-strength bolted *connections*. Shop and erection drawings shall be made with due regard to speed and economy in fabrication and erection.

10.13.2.1 **Owner Responsibility**

The owner shall furnish, in a timely manner and in accordance with the contract documents, complete structural design drawings and specifications that have been released for construction. Unless otherwise noted, design drawings that are provided as part of a contract bid package shall constitute authorization by the owner that the design drawings are released for construction.

Records of the meetings should be written and distributed to all parties. Subsequent meetings to discuss progress and issues that arise during construction also can be helpful, particularly when they are held on a regular schedule.

10.13.2.2 **Fabricator Responsibility**

Except as provided in Section 10.13.2.5, the fabricator shall produce shop and erection drawings for the fabrication and erection of the structural steel and is responsible for the following:

(a) The transfer of information from the contract documents into accurate and complete shop and erection drawings; and,

(b) The development of accurate, detailed dimensional information to provide for the fit-up of parts in the field.

Each shop and erection drawing shall be identified by the same drawing number throughout the duration of the project and shall be identified by revision number and date, with each specific revision clearly identified.

When the fabricator submits a request to change connection details that are described in the contract documents, the fabricator shall notify the owner's designated representatives for design and construction in writing in advance of the submission of the shop and erection drawings. The owner's designated representative for design shall review and approve or reject the request in a timely manner.

When requested to do so by the owner's designated representative for design, the fabricator shall provide to the owner's designated representatives for design and construction its schedule for the submittal of shop and erection drawings so as to facilitate the timely flow of information between all parties.

10.13.2.3 Use of CAD Files and/or Copies of Design Drawings

The fabricator shall neither use nor reproduce any part of the design drawings as part of the shop or erection drawings without the written permission of the owner's designated representative for design. When CAD files or copies of the design drawings are made available for the fabricator's use, the fabricator shall accept this information under the following conditions:

a) All information contained in the CAD files or copies of the design drawings shall be considered instruments of service of the owner's designated representative for design and shall not be used for other projects, additions to the project or the completion of the project by others. CAD files and copies of the design drawings shall remain the property of the owner's designated representative for design and in no case shall the transfer of these CAD files or copies of the design drawings be considered a sale.
b) The CAD files or copies of the design drawings shall not be considered to be contract documents. In the event of a conflict between the design drawings and the CAD files or copies thereof, the design drawings shall govern;
c) The use of CAD files or copies of the design drawings shall not in any way obviate the fabricator's responsibility for proper checking and coordination of dimensions, details, member sizes and fit-up and quantities of materials as required to facilitate the preparation of shop and erection drawings that are complete and accurate as required in Section 4.2; and,
d) The fabricator shall remove information that is not required for the fabrication or erection of the structural steel from the CAD files or copies of the design drawings.

10.13.2.4 Approval

Except as provided in Section 10.13.2.5, the shop and erection drawings shall be submitted to the owner's designated representatives for design and construction for review and approval. The shop and erection drawings shall be returned to the fabricator within 14 calendar days.

Final substantiating connection information, if any, shall also be submitted with the shop and erection drawings. The owner's designated representative for design is the final authority in the event of a disagreement between parties regarding connection design.

Approved shop and erection drawings shall be individually annotated by the owner's designated representatives for design and construction as either approved or approved subject to corrections noted. When so required, the fabricator shall subsequently make the corrections noted and furnish corrected shop and erection drawings to the owner's designated representatives for design and construction.

10.13.2.4.1 Constituents of Approval

Approval of the shop and erection drawings, approval subject to corrections noted and similar approvals shall constitute the following:

a) Confirmation that the fabricator has correctly interpreted the contract documents in the preparation of those submittals;
b) Confirmation that the owner's designated representative for design has reviewed and approved the connection details shown on the shop and erection drawings and submitted in accordance with Sections 10.13.1 and 10.13.2, if applicable; and,
c) Release by the owner's designated representatives for design and construction for the fabricator to begin fabrication using the approved submittals.

Such approval shall not relieve the fabricator of the responsibility for either the accuracy of the detailed dimensions in the shop and erection drawings or the general fit-up of parts that are to be assembled in the field.

The fabricator shall determine the fabrication schedule that is necessary to meet the requirements of the contract.

10.13.2.4.2 Authorization by Owner

Unless otherwise noted, any additions, deletions or revisions that are indicated in responses to RFIs or on the approved shop and erection drawings shall constitute authorization by the owner that the additions, deletions or revisions are released for construction. The fabricator and the erector shall promptly notify the owner's designated representative for construction when any direction or notation in responses to RFIs or on the shop or erection drawings or other information will result in an additional cost and/or a delay. See Sections 10.13.1.5 and 10.13.7.3.

10.13.2.5 **Shop and/or Erection Drawings Not Furnished by the Fabricator**

When the shop and erection drawings are not prepared by the fabricator, but are furnished by others, they shall be delivered to the fabricator in a timely manner. These shop and erection drawings shall be prepared, insofar as is practical, in accordance with the shop fabrication and detailing standards of the fabricator. The fabricator shall neither be responsible for the completeness or accuracy of shop and erection drawings so furnished, nor for the general fit-up of the members that are fabricated from them.

10.13.2.6 **The RFI Process**

When requests for information (RFIs) are issued, the process shall include the maintenance of a written record of inquiries and responses related to interpretation and implementation of the contract documents, including the clarifications and/or revisions to the contract documents that result, if any. RFIs shall not be used for the incremental release for construction of design drawings. When RFIs involve discrepancies or revisions, see Sections 10.13.1.3, 10.13.1.5, and 10.13.2.4.2.

10.13.2.7 **Erection Drawings**

Erection drawings shall be provided to the erector in a timely manner so as to allow the erector to properly plan and perform the work.

10.13.3 MATERIALS

10.13.3.1 **Mill Materials**

Unless otherwise noted in the contract documents, the fabricator is permitted to order the materials that are necessary for fabrication when the fabricator receives contract documents that have been released for construction.

Unless otherwise specified by means of special testing requirements in the contract documents, mill testing shall be limited to those tests that are required for the material in the ASTM specifications indicated in the contract documents. Materials ordered to special material requirements shall be marked by the supplier as specified in ASTM A6/A6M Section 12 prior to delivery to the fabricator's shop or other point of use. Such material not so marked by the supplier, shall not be used until:

a) Its identification is established by means of testing in accordance with the applicable ASTM specifications; and,
b) A fabricator's identification mark, as described in Section 10.13.2 has been applied.
When mill material does not satisfy ASTM A6/A6M tolerances for camber, profile, flatness or sweep, the fabricator shall be permitted to perform corrective procedures, including the use of controlled heating and/or mechanical straightening, subject to the limitations in the AISC Specification.

10.13.3.2 **Stock Materials**

If used for structural purposes, materials that are taken from stock by the fabricator shall be of a quality that is at least equal to that required in the ASTM specifications indicated in the contract documents.

Material test reports shall be accepted as sufficient record of the quality of materials taken from stock by the fabricator. The fabricator shall review and retain the material test reports that cover such stock materials. However, the fabricator need not maintain records that identify individual pieces of stock material against individual material test reports, provided the fabricator purchases stock materials that meet the requirements for material grade and quality in the applicable ASTM specifications.

Stock materials that are purchased under no particular specification, under a specification that is less rigorous than the applicable ASTM specifications or without material test reports or other recognized test reports shall not be used without the approval of the owner's designated representative for design.

10.13.4 Fabrication

10.13.4.1 **Cambering, Curving and Straightening**

Local application of heat or mechanical means is permitted to be used to introduce or correct *camber*, curvature and straightness. The temperature of heated areas, as measured by approved methods, shall not exceed 593^0 C for A514/A514M and A852/A852M steel nor $1,200^0$ F (649^0 C) for other steels.

10.13.4.2 **Thermal Cutting**

Thermally cut edges shall meet the requirements of AWS D1.1, Sections 5.15.1.2, 5.15.4.3 and 5.15.4.4 with the exception that thermally cut free edges that will be subject to calculated static tensile *stress* shall be free of round-bottom *gouges* greater than 5 mm deep and sharp V-shaped notches. *Gouges* deeper than 5 mm and notches shall be removed by grinding or repaired by welding.

Reentrant corners, except reentrant corners of *beam copes* and weld access holes, shall meet the requirements of AWS D1.1, Section A5.16. If another specified contour is required it must be shown on the contract documents.

Beam copes and weld access holes shall meet the geometrical requirements of Section 10.10.1.6. Beam copes and weld access holes in shapes that are to be galvanized shall be ground. For shapes with a flange thickness not exceeding 50 mm the roughness of *thermally cut* surfaces of copes shall be no greater than a surface roughness value of 50 µm as defined in ASME B46.1 Surface Texture (*Surface Roughness, Waviness, and Lay*). For beam copes and weld access holes in which the curved part of the access hole is thermally cut in ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 50 mm and welded built-up shapes with material thickness greater than 50 mm, a preheat temperature of not less than 66^0 C shall be applied prior to thermal cutting. The thermally cut surface of access holes in ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 50 mm and built-up shapes with a material thickness greater than 50 mm shall be ground and inspected for cracks using magnetic particle inspection in accordance with ASTM E709. Any crack is unacceptable regardless of size or location.

10.13.4.3 **Planing of Edges**

Planing or finishing of sheared or *thermally cut* edges of plates or shapes is not required unless specifically called for in the contract documents or included in a stipulated edge preparation for welding.

10.13.4.4 **Welded Construction**

The technique of welding, the workmanship, appearance and quality of welds, and the methods used in correcting nonconforming work shall be in accordance with AWS D1.1 except as modified in Section J2.

10.13.4.5 **Bolted Construction**

Parts of bolted members shall be pinned or bolted and rigidly held together during assembly. Use of a *drift* pin in bolt holes during assembly shall not distort the metal or enlarge the holes. Poor matching of holes shall be cause for rejection.

Bolt holes shall comply with the provisions of the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, Section 10.13.3.3 except that *thermally cut* holes shall be permitted with a surface roughness profile not exceeding 25 µm as defined in ASME B46.1. *Gouges* shall not exceed a depth of 2 mm.

Fully inserted finger *shims*, with a total thickness of not more than 6 mm within a *joint* are permitted in *joints* without changing the strength (based upon hole type) for the design of *connections*. The orientation of such *shims* is independent of the direction of application of the *load.*

The use of high-strength bolts shall conform to the requirements of the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, except as modified in Section 10.10.3.

10.13.4.6 **Compression Joints**

Compression *joints* that depend on contact bearing as part of the *splice* strength shall have the bearing surfaces of individual fabricated pieces prepared by milling, sawing, or other suitable means.

10.13.4.7 **Dimensional Tolerances**

Dimensional tolerances shall be in accordance with ASTM A6/A6M.

10.13.4.8 **Finish of Column Bases**

Column bases and base plates shall be finished in accordance with the following requirements:

(1) Steel bearing plates 50 mm or less in thickness are permitted without milling, provided a satisfactory contact bearing is obtained. Steel bearing plates over 50 mm but not over 100 mm in thickness are permitted to be straightened by pressing or, if presses are not available, by milling for bearing surfaces (except as noted in subparagraphs 2 and 3 of this section), to obtain a satisfactory contact bearing. Steel bearing plates over 100 mm in thickness shall be milled for bearing surfaces (except as noted in subparagraphs 2 and 3 of this section).

(2) Bottom surfaces of bearing plates and *column* bases that are grouted to ensure full bearing contact on foundations need not be milled.

(3) Top surfaces of bearing plates need not be milled when complete-*joint*- penetration *groove welds* are provided between the *column* and the bearing plate.

10.13.4.9 **Holes for Anchor Rods**

Holes for anchor rods shall be permitted to be *thermally cut* in accordance with the provisions of Section 10.13.2.2.

10.13.4.10 **Drain Holes**

When water can collect inside *HSS* or box members, either during construction or during service, the member shall be sealed, provided with a drain hole at the base, or protected by other suitable means.

10.13.4.11 **Requirements for Galvanized Members**

Members and parts to be galvanized shall be designed, detailed and fabricated to provide for flow and drainage of pickling fluids and zinc and to prevent pressure build-up in enclosed parts.

10.13.5 Shop Painting

10.13.5.1 **General Requirements**

Shop paint is not required unless specified by the contract documents.

10.13.5.2 **Inaccessible Surfaces**

Except for contact surfaces, surfaces inaccessible after shop assembly shall be cleaned and painted prior to assembly, if required by the design documents.

10.13.5.3 **Contact Surfaces**

Paint is permitted in bearing-type connections. For slip-critical connections, the faying surface requirements shall be in accordance with the RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts, Section 3.2.2(b).

10.13.5.4 **Finished Surfaces**

Machine-finished surfaces shall be protected against corrosion by a rust inhibitive coating that can be removed prior to erection, or which has characteristics that make removal prior to erection unnecessary.

10.13.5.5 **Surfaces Adjacent to Field Welds**

Unless otherwise specified in the design documents, surfaces within 50 mm of any field weld location shall be free of materials that would prevent proper welding or produce objectionable fumes during welding.

10.13.6 Erection

10.13.6.1 **Alignment of Column Bases**

Column bases shall be set level and to correct elevation with full bearing on concrete or masonry.

10.13.6.2 **Bracing**

The frame of steel skeleton buildings shall be carried up true and plumb**.** Temporary bracing shall be provided, wherever necessary to support the loads to which the structure may be subjected, including equipment and the operation of same. Such bracing shall be left in place as long as required for safety.

10.13.6.3 **Alignment**

No permanent bolting or welding shall be performed until the adjacent affected portions of the structure have been properly aligned.

10.13.6.4 **Fit of Column Compression Joints and Base Plates**

Lack of contact bearing not exceeding a gap of 2 mm, regardless of the type of *splice* used (*partial-joint-penetration groove welded* or bolted), is permitted. If the gap exceeds 2 mm, but is less than 6 mm, and if an engineering investigation shows that sufficient contact area does not exist, the gap shall be packed out with nontapered steel *shims*. Shims need not be other than mild steel, regardless of the grade of the main material.

10.13.6.5 **Field Welding**

Shop paint on surfaces adjacent to *joints* to be field welded shall be wire brushed if necessary to assure weld quality.

Field welding of attachments to installed embedments in contact with concrete shall be done in such a manner as to avoid excessive thermal expansion of the embedment which could result in spalling or cracking of the concrete or excessive stress in the embedment anchors.

10.13.6.6 **Field Painting**

Responsibility for touch-up painting, cleaning and field painting shall be allocated in accordance with accepted local practices, and this allocation shall be set forth explicitly in the design documents.

10.13.6.7 **Connections**

As erection progresses, the structure shall be securely bolted or welded to support the dead, wind and erection *loads.*

10.13.7 Quality Control

The fabricator shall provide *quality control* procedures to the extent that the fabricator deems necessary to assure that the work is performed in accordance with this Specification. In addition to the fabricator's quality control procedures, material and workmanship at all times may be subject to inspection by qualified inspectors representing the purchaser. If such inspection by representatives of the purchaser will be required, it shall be so stated in the design documents.

10.13.7.1 **Cooperation**

As far as possible, the inspection by representatives of the purchaser shall be made at the fabricator's plant. The fabricator shall cooperate with the inspector, permitting access for inspection to all places where work is being done. The purchaser's inspector shall schedule this work for minimum interruption to the work of the fabricator.

10.13.7.2 **Rejections**

Material or workmanship not in conformance with the provisions of this Specification may be rejected at any time during the progress of the work.

The fabricator shall receive copies of all reports furnished to the purchaser by the inspection agency.

10.13.7.3 **Inspection of Welding**

The inspection of welding shall be performed in accordance with the provisions of AWS D1.1 except as modified in Section 10.10.2.

When visual inspection is required to be performed by AWS certified welding inspectors, it shall be so specified in the design documents.

When nondestructive testing is required, the process, extent and standards of acceptance shall be clearly defined in the design documents.

10.13.7.4 **Inspection of Slip-Critical High-Strength Bolted Connections**

The inspection of slip-critical high-strength bolted *connections* shall be in accordance with the provisions of the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts.*

10.13.7.5 **Identification of Steel**

The fabricator shall be able to demonstrate by a written procedure and by actual practice a method of material identification, visible at least through the "fit-up" operation, for the main structural elements of each shipping piece.

10.14 Direct Analysis Method

This section addresses the *direct analysis method* for structural systems comprised of *moment frames*, *braced frames*, *shear walls*, or combinations thereof.

10.14.1 General Requirements

Members shall satisfy the provisions of Section 10.8.1 with the nominal *column* strengths, P_n, determined using K = 1.0. The *required strengths* for members, *connections* and other structural elements shall be determined using a second-order *elastic analysis* with the constraints presented in Section 10.14.3. All component and *connection* deformations that contribute to the lateral displacement of the structure shall be considered in the analysis.

10.14.2 Notional Loads

Notional loads shall be applied to the lateral framing system to account for the effects of geometric imperfections, inelasticity, or both. *Notional loads* are *lateral loads* that are applied at each framing level and specified in terms of the *gravity loads* applied at that level. The *gravity load* used to determine the *notional load* shall be equal to or greater than the *gravity load* associated with the *load combination* being evaluated. *Notional loads* shall be applied in the direction that adds to the destabilizing effects under the specified *load combination*.

10.14.3 Notional Loads

(1) The *second-order analysis* shall consider both P-δ and P-Δ *effects*. It is permitted to perform the analysis using any general second-order analysis method, or by the amplified *first-order analysis* method of Section 10.3.2, provided that the B_1 and B_2 factors are based on the reduced *stiffness*es defined in Equations 10.14.3.2 and 10.14.3.3. Analyses shall be conducted according to the design and loading requirements specified in either Section 10.2.3.3 (LRFD) or Section 10.2.3.4 (ASD). For ASD, the second-order analysis shall be carried out under 1.6 times the ASD *load combinations* and the results shall be divided by 1.6 to obtain the *required strengths*.

Methods of analysis that neglect the effects of P - δ on the lateral displacement of the structure are permitted where the axial *loads* in all members whose flexural stiffnesses are considered to contribute to the lateral *stability* of the structure satisfy the following limit:

$$\alpha P_r < 0.15\, P_{eL} \tag{10.14.3.1}$$

where

P_r = required axial compressive strength under LRFD or ASD *load combinations*, N

$P_{eL} = \pi^2 EI/L^2$, evaluated in the plane of bending

And α = 1.0 (LRFD) α = 1.6 (ASD)

(2) A *notional load*, N_i = 0.002Y_i, applied independently in two orthogonal directions, shall be applied as a *lateral load* in all load combinations. This load shall be in addition to other lateral loads, if any,

where

N_i = notional lateral load applied at level i, N

Y_i = *gravity load* from the *LRFD load combination* or 1.6 times the ASD load combination applied at level i, N

The notional load coefficient of 0.002 is based on an assumed initial story out-of-plumbness ratio of 1/500. Where a smaller assumed out-of-plumbness is justified, the notional load coefficient may be adjusted proportionally.

For frames where the ratio of second-order drift to first-order drift is equal to or less than 1.5, it is permissible to apply the notional load, N_i , as a minimum lateral load for the gravity-only load combinations and not in combination with other lateral loads.

For all cases, it is permissible to use the assumed out-of-plumbness geometry in the analysis of the structure in lieu of applying a notional load or a minimum lateral load as defined above.

(3) A reduced flexural *stiffness*, EI^*,

$$EI^* = 0.08\,\tau_b EI \qquad (10.14.3.2)$$

shall be used for all members whose flexural stiffness is considered to contribute to the lateral *stability* of the structure,

where

I = moment of inertia about the axis of bending, mm^4

τ_B = 1.0 for $\alpha P_r / P_y \le 0.5$

= $4[\alpha P_r / P_y\ (1 - \alpha P_r / P_y)]$ for $\alpha P_r / P_y > 0.5$

P_r = required axial compressive strength under *LRFD* or *ASD load combinations*, N

$P_y = AF_y$, member *yield strength*, N

And α = 1.0 (LRFD) α = 1.6 (ASD)

In lieu of using $\tau_b < 1.0$ where $\alpha P_r / P_y > 0.5$, τ_b = 1.0 may be used for all members, provided that an additive *notional load* of $0.001 Y_i$ is added to the notional load required in (2).

(4) A reduced flexural *stiffness*, EA^*,

$$EA^* = 0.8\,EA \qquad (10.14.3.3)$$

shall be used for members whose axial stiffness is considered to contribute to the lateral *stability* of the structure, where A is the cross-sectional member area.

10.15 Inelastic Analysis and Design

10.15.1 General Provisions

Inelastic analysis is permitted for design according to the provisions of Section 10.2.3.3 (LRFD). Inelastic analysis is not permitted for design according to the provisions of Section 10.2.3.4 (ASD) except as provided in Section 10.15.3.

10.15.2 Materials

Members undergoing plastic hinging shall have a *specified minimum yield stress* not exceeding 450 MPa.

10.15.3 Moment Redistribution

*Beam*s and girders composed of *compact sections* as defined in Section 10.2.4 and satisfying the *unbraced length* requirements of Section 10.15.7, including *composite* members, may be proportioned for nine-tenths of the negative moments at points of support, produced by the *gravity loading* computed by an *elastic analysis*, provided that the maximum positive moment is increased by one-tenth of the average negative moments. This reduction is not permitted for moments produced by loading on cantilevers and for design according to Sections 10.15.4 through 10.15.8 of this section.

If the negative moment is resisted by a *column* rigidly framed to the beam or girder, the one-tenth reduction may be used in proportioning the column for combined axial *force* and flexure, provided that the axial force does not exceed $0.15\varphi_c F_y A_g$ for LRFD or $0.15 F_y A_g / \Omega_c$ for ASD,

where

A_g = gross area of member, mm^2

F_y = *specified minimum yield stress* of the compression flange, MPa.

φ_c = *resistance factor* for compression = 0.90

Ω_c = *safety factor* for compression = 1.67

10.15.4 Local Buckling

Flanges and webs of members subject to plastic hinging in combined flexure and axial compression shall be compact with width-thickness ratios less than or equal to the limiting λ_p defined in Table 10.2.4.1 or as modified as follows:

a) For webs of doubly symmetric wide flange members and rectangular *HSS* in combined flexure and compression

(i) For $P_u/(\phi_b P_y) \leq 0.125$

$$h/t_w \leq 3.76\sqrt{\frac{E}{F_y}}\left(1-\frac{2.75\,P_u}{\varphi_b\,P_y}\right) \qquad (10.15.4.1)$$

(ii) For $P_u/\phi_b P_y > 0.125$

$$h/t_w \leq 1.12\sqrt{\frac{E}{F_y}}\left(2.33-\frac{P_u}{\varphi_b\,P_y}\right) \geq 1.49\sqrt{\frac{E}{F_y}} \qquad (10.15.4.2)$$

where

E = modulus of elasticity of steel 200 000 MPa.

F_y = specified minimum yield stress of the type of steel being used, MPa.

h = as defined in Section 10.2.4.2, mm.

P_u = required axial strength in compression, N.

P_y = member yield strength, N.

t_w = web thickness, mm.

ϕ_b = resistance factor for flexure = 0.90

b) For flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression, flange *cover plates*, and *diaphragm plates* between lines of *fasteners* or welds

$$b/t \leq 0.94\sqrt{E/F_y} \qquad (10.15.4.3)$$

where

b = as defined in Section 10.2.4.2, mm.

t = as defined in Section 10.2.4.2, mm.

(c) For circular hollow sections in flexure

$$D/t \leq 0.045\ E/F_y \qquad (10.15.4.4)$$

where

D = outside diameter of round HSS member, mm.

10.15.5 Stability and Second-Order Effects

Continuous *beams* not subjected to axial *loads* and that do not contribute to lateral *stability* of framed structures may be designed based on a *first-order inelastic analysis* or a plastic *mechanism* analysis.

Braced frames and *moment frames* may be designed based on a *first-order inelastic analysis* or a plastic *mechanism* analysis provided that *stability* and *second-order effects* are taken into account.

Structures may be designed on the basis of a second-order *inelastic analysis*. For *beam-columns*, *connections* and connected members, the *required strengths* shall be determined from a second-order inelastic analysis, where equilibrium is satisfied on the deformed geometry, taking into account the change in *stiffness* due to yielding

10.15.5.1 **Braced Frames**

In *braced frames* designed on the basis of *inelastic analysis*, braces shall be designed to remain elastic under the *design loads*. The required axial strength for *columns* and compression braces shall not exceed $\varphi_c\,(0.85\,F_y\,A_g)$, where

φ_c = 0.90 (LRFD)

10.15.5.2 **Moment Frames**

In *moment frames* designed on the basis of *inelastic analysis*, the required axial strength of columns shall not exceed $\varphi_c\,(0.75\ F_y\,A_g)$

where

φ_c = 0.90 (LRFD)

10.15.6 Columns and Other Compression Members

In addition to the limits set in Sections 10.15.5.1 and 10.15.5.2, the required axial strength of columns designed on the basis of inelastic analysis shall not exceed the design strength, ϕc Pn , determined according to the provisions of Section 10.5.3.

Design by inelastic analysis is permitted if the column slenderness ratio, L/r, does not exceed $4.71\sqrt{E/F_y}$,

where

L = laterally unbraced length of a member, mm.

r = governing radius of gyration, mm.

10.15.7 Beams and Other Flexural Members

The required moment strength, M_u, of *beams* designed on the basis of *inelastic analysis* shall not exceed the *design strength*, φM_n , where

$$M_n = M_p = F_y Z < 1.6 F_y S \qquad (10.15.7.1)$$

$$\varphi_c = 0.90\ (LRFD)$$

a) For doubly symmetric and singly symmetric I-shaped members with the compression flange equal to or larger than the tension flange loaded in the plane of the web:

$$L_{pd} = \left[0.12 + 0.076\left(\frac{M_1}{M_2}\right)\right]\left(\frac{E}{F_y}\right) r_y \qquad (10.15.7.2)$$

where

M_1 = smaller moment at end of unbraced length of beam, N-mm

M_2 = larger moment at end of unbraced length of beam, N-mm

r_y = radius of gyration about minor axis, mm

(M_1 / M_2) is positive when moments cause *reverse curvature* and negative for *single curvature*.

b) For solid rectangular bars and symmetric box beams:

$$L_{pd} = \left[0.17 + 0.10\left(\frac{M_1}{M_2}\right)\right]\left(\frac{E}{F_y}\right) r_y \geq 0.10\left(\frac{E}{F_y}\right) r_y \qquad (10.15.7.3)$$

There is no limit on L_b for members with circular or square cross sections or for any beam bent about its minor axis.

10.15.8 Beams and Other Flexural Members

When inelastic analysis is used for symmetric members subject to bending and axial force, the provisions in Section 10.8.1 apply.

Inelastic analysis is not permitted for members subject to torsion and combined torsion, flexure, shear and/or axial force.

10.15.9 Connections

Connections adjacent to plastic hinging regions of connected members shall be designed with sufficient strength and ductility to sustain the *forces* and deformations imposed under the required *loads*.

10.16 Design for Ponding

This section provides methods for determining whether a roof system has adequate strength and stiffness to resist ponding.

10.16.1 Simplified Design for Ponding

The roof system shall be considered stable for *ponding* and no further investigation is needed if both of the following two conditions are met:

$$C_p + 0.9\,C_s \leq 0.25 \qquad (10.16.1.1)$$

$$I_d \geq 3940\,S^4 \qquad (10.16.1.2)$$

where

$$C_p = \frac{504\,L_s\,{L_p}^4}{I_p}$$

$$C_s = \frac{504\,S{L_s}^4}{I_s}$$

Lp = column spacing in direction of girder (length of primary members), m.

Ls = column spacing perpendicular to direction of girder (length of secondary members), m.

S = spacing of secondary members, m.

Ip = moment of inertia of primary members, mm4.

Is = moment of inertia of secondary members, mm4

Id = moment of inertia of the steel deck supported on secondary members, mm4 per m.

For trusses and steel joists, the moment of inertia Is shall be decreased 15 percent when used in the above equation. A steel deck shall be considered a secondary member when it is directly supported by the primary members.

10.16.2 Improved Design for Ponding

The provisions given below are permitted to be used when a more exact determination of framing *stiffness* is needed than that given in Section 10.16.1.

For primary members, the stress index shall be

$$U_p = \left(\frac{0.8F_y - f_o}{f_o}\right)_p \qquad (10.16.2.1)$$

For secondary members, the stress index shall be

$$U_s = \left(\frac{0.8F_y - f_o}{f_o}\right)_s \qquad (10.16.2.2)$$

where

f_o = *stress* due to the load combination (*D* + *R*)

D = nominal dead *load*

R = *nominal load* due to rainwater or snow, exclusive of the *ponding* contribution, MPa.

For roof framing consisting of primary and secondary members, the combined stiffness shall be evaluated as follows: enter Figure 10.16.1 at the level of the computed stress index U_p determined for the primary beam; move horizontally to the computed C_s value of the secondary beams and then downward to the abscissa scale. The combined stiffness of the primary and secondary framing is sufficient to prevent ponding if the flexibility constant read from this latter scale is more than the value of C_p computed for the given primary member; if not, a stiffer primary or secondary beam, or combination of both, is required.

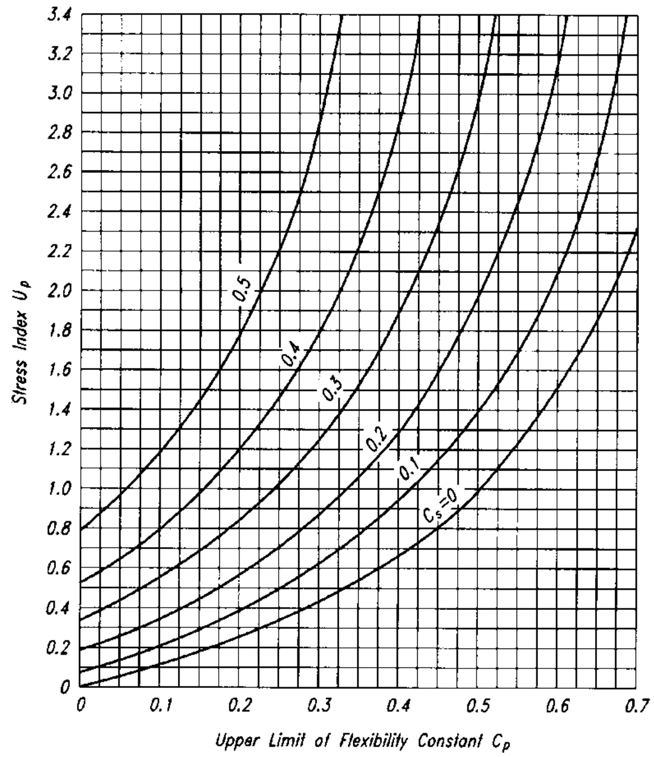


Fig.10.16.1. Limiting flexibility coefficient for the primary systems

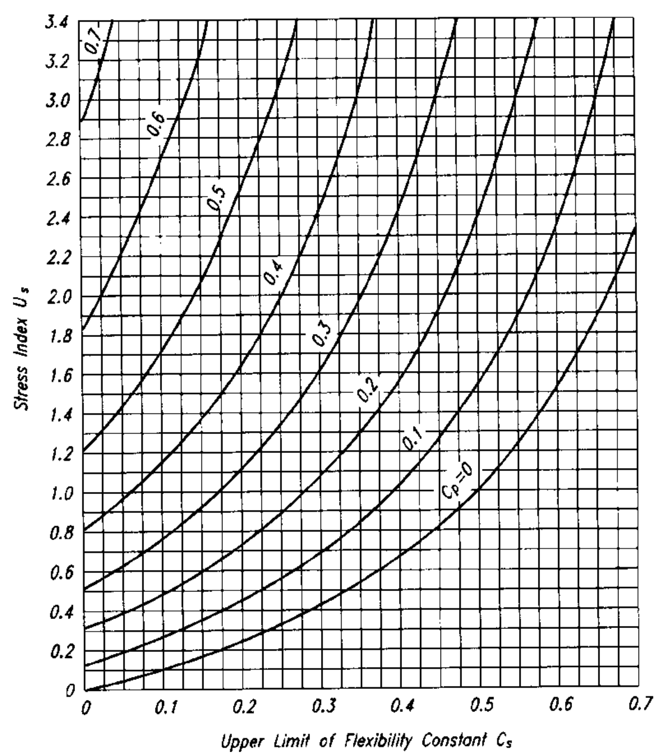


Fig.10.16.2. Limiting flexibility coefficient for the secondary systems.

A similar procedure must be followed using Figure 10.16.2.

For roof framing consisting of a series of equally spaced wall-bearing beams, the stiffness shall be evaluated as follows. The beams are considered as secondary members supported on an infinitely stiff primary member. For this case, enter Figure 10.16.2 with the computed stress index Us. The limiting value of Cs is determined by the intercept of a horizontal line representing the Us value and the curve for Cp = 0.

For roof framing consisting of metal deck spanning between beams supported on columns, the stiffness shall be evaluated as follows. Employ Figure 10.16.1 or 10.16.2 using as Cs the flexibility constant for a 1 m width of the roof deck (S = 1.0).

10.17 Design for Fatigue

This section applies to members and *connections* subject to high cyclic loading within the elastic range of stresses of frequency and magnitude sufficient to initiate cracking and progressive failure, which defines the *limit state* of *fatigue*

10.17.1 General

The provisions of this Section apply to stresses calculated on the basis of service loads. The maximum permitted stress due to unfactored loads is 0.66 Fy.

Stress range is defined as the magnitude of the change in stress due to the application or removal of the service live load. In the case of a stress reversal, the stress range shall be computed as the numerical sum of maximum repeated tensile and compressive stresses or the numerical sum of maximum shearing stresses of opposite direction at the point of probable crack initiation.

In the case of complete-joint-penetration butt welds, the maximum design stress range calculated by Equation 10.17.3.1 applies only to welds with internal soundness meeting the acceptance requirements of Section 6.12.2 or 6.13.2 of AWS D1.1.

No evaluation of fatigue resistance is required if the live load stress range is less than the threshold stress range, FTH. See Table 10.17.1.

No evaluation of fatigue resistance is required if the number of cycles of application of live load is less than 20,000.

The cyclic load resistance determined by the provisions of this Section is applicable to structures with suitable corrosion protection or subject only to mildly corrosive atmospheres, such as normal atmospheric conditions.

The cyclic load resistance determined by the provisions of this Section is applicable only to structures subject to temperatures not exceeding 150 oC.

The engineer of record shall provide either complete details including weld sizes or shall specify the planned cycle life and the maximum range of moments, shears and reactions for the connections.

10.17.2 Calculation of Maximum Stresses and Stress Ranges

Calculated stresses shall be based upon elastic analysis. Stresses shall not be amplified by stress concentration factors for geometrical discontinuities.

For bolts and threaded rods subject to axial tension, the calculated stresses shall include the effects of prying action, if any. In the case of axial stress combined with bending, the maximum stresses, of each kind, shall be those determined for concurrent arrangements of the applied load.

For members having symmetric cross sections, the fasteners and welds shall be arranged symmetrically about the axis of the member, or the total stresses including those due to eccentricity shall be included in the calculation of the stress range.

For axially loaded angle members where the center of gravity of the connecting welds lies between the line of the center of gravity of the angle cross section and the center of the connected leg, the effects of eccentricity shall be ignored. If the center of gravity of the connecting welds lies outside this zone, the total stresses, including those due to joint eccentricity, shall be included in the calculation of stress range.

10.17.3 Design Stress Range

The range of *stress* at *service loads* shall not exceed the *design stress range* computed as follows.

(a) For stress categories A, B, B', C, D, E and E' (see table 10.17.1) the design stress range, F_{SR}, shall be determined by Equation 10.17.3.1 or 10.17.3.2.

$$F_{SR} = \left(\frac{C_f \times 329}{N}\right)^{0.333} \geq F_{TH} \qquad (10.17.3.1)$$

where

F_{SR} = design stress range, MPa.

C_f = constant from Table 10.17.1 for the category

N = number of stress range fluctuations in design life

= number of stress range fluctuations per day × 365 × years of design life

F_{TH} = threshold *fatigue stress* range, maximum *stress* range for indefinite design life from Table 10.17.1, MPa.

(b) For stress category F, the design stress range, F_{SR}, shall be determined by Equation 10.17.3.2.

$$F_{SR} = \left(\frac{C_f \times 11 \times 10^4}{N}\right)^{0.167} \geq F_{TH} \qquad (10.17.3.2)$$

(c) For tension-loaded plate elements connected at their end by cruciform, T, or corner details with *complete-joint-penetration* (CJP) *groove welds* or *partial- joint-penetration* (PJP) *groove welds*, *fillet welds*, or combinations of the preceding, transverse to the direction of stress, the design stress range on the cross section of the tension-loaded plate element at the toe of the weld shall be determined as follows:

(i) Based upon crack initiation from the toe of the weld on the tension loaded plate element the design stress range, F_{SR}, shall be determined by Equation 10.17.3.3 for stress category C which is equal to

$$F_{SR} = \left(\frac{14.4 \times 10^{11}}{N}\right)^{0.333} \geq 68.9 \qquad (10.17.3.3)$$

(ii) Based upon crack initiation from the root of the weld the design stress range, F_{SR}, on the tension loaded plate element using transverse PJP groove welds, with or without reinforcing or contouring fillet welds, the design stress range on the cross section at the toe of the weld shall be determined by Equation 10.17.3.4, stress category C' as follows:

$$F_{SR} = R_{PJP}\left(\frac{14.4 \times 10^{11}}{N}\right)^{0.333} \qquad (10.17.3.8)$$

where

R_{PJP} is the reduction factor for reinforced or nonreinforced transverse PJP groove welds determined as follows:

$$R_{PJP} = \left(\frac{1.12 - 1.01\left(\frac{2a}{t_p}\right) + 1.24\left(\frac{w}{t_p}\right)}{t_p^{0.167}}\right) \leq 1.0 \qquad (10.17.3.9)$$

If R_{PJP} = 1.0, use stress category C.

$2a$ = the length of the nonwelded root face in the direction of the thickness of the tension-loaded plate, mm.

W = the leg size of the reinforcing or contouring fillet, if any, in the direction of the thickness of the tension-loaded plate, mm.

t_p = thickness of tension loaded plate, mm.

(iii) Based upon crack initiation from the roots of a pair of transverse fillet welds on opposite sides of the tension loaded plate element the design stress range, F_{SR} , on the cross section at the toe of the welds shall be determined by Equation 10.17.3.5, stress category C" as follows:

$$F_{SR} = R_{FIL}\left(\frac{14.4 \times 10^{11}}{N}\right)^{0.333} \qquad (10.17.3.10)$$

where

R_{FIL} is the reduction factor for joints using a pair of transverse fillet welds only.

$$R_{FIL} = \left(\frac{0.10 + 1.24(w/t_p)}{t_p^{0.167}}\right) \leq 1.0 \qquad (10.17.3.11)$$

If R_{FIL} = 1.0, use stress category C.

10.17.4 Bolts and Threaded Parts

The range of stress at service loads shall not exceed the stress range computed as follows.

a) For mechanically fastened connections loaded in shear, the maximum range of stress in the connected material at service loads shall not exceed the design stress range computed using Equation 10.17.3.1 where Cf and FTH are taken from Section 2 of Table 10.17.1.

b) For high-strength bolts, common bolts, and threaded anchor rods with cut, ground or rolled threads, the maximum range of tensile stress on the net tensile area from applied axial load and moment plus load due to prying action shall not exceed the design stress range computed using Equation 10.17.3.1. The factor Cf shall be taken as 3.9 × 108 (as for stress category E'). The threshold stress, FTH shall be taken as 48 MPa (as for stress category D). The net tensile area is given by Equation 10.17.4.1.

$$A_t = \frac{\pi}{4}(d_b - 0.9382P)^2 \qquad (10.17.4.1)$$

where

P = pitch, mm per thread

db = the nominal diameter (body or shank diameter), mm.

n = threads per mm.

For joints in which the material within the grip is not limited to steel or joints which are not tensioned to the requirements of Table 10.10.3.1, all axial load and moment applied to the joint plus effects of any prying action shall be assumed to be carried exclusively by the bolts or rods.

For joints in which the material within the grip is limited to steel and which are tensioned to the requirements of Table 10.10.3.1, an analysis of the relative stiffness of the connected parts and bolts shall be permitted to be used to determine the tensile stress range in the pretensioned bolts due to the total service live load and moment plus effects of any prying action. Alternatively, the stress range in the bolts shall be assumed to be equal to the stress on the net tensile area due to 20 percent of the absolute value of the service load axial load and moment from dead, live and other loads.

10.17.5 Special Fabrication and Erection Requirements

Longitudinal backing bars are permitted to remain in place, and if used, shall be continuous. If splicing is necessary for long *joints*, the bar shall be joined with complete penetration butt joints and the reinforcement ground prior to assembly in the joint.

In transverse joints subject to tension, backing bars, if used, shall be removed and the joint back gouged and welded.

In transverse complete-joint-penetration T and corner joints, a reinforcing *fillet weld*, not less than 6 mm in size shall be added at re-entrant corners.

The surface roughness of flame cut edges subject to significant cyclic tensile *stress* ranges shall not exceed 25 μm, where ASME B46.1 is the reference standard.

Reentrant corners at cuts, *copes* and weld access holes shall form a radius of not less than 10 mm by predrilling or subpunching and reaming a hole, or by thermal cutting to form the radius of the cut. If the radius portion is formed by thermal cutting, the cut surface shall be ground to a bright metal surface.

For transverse butt joints in regions of high tensile stress, run-off tabs shall be used to provide for cascading the weld termination outside the finished joint. End dams shall not be used. Run-off tabs shall be removed and the end of the weld finished flush with the edge of the member.

See Section 10.10.2.2 for requirements for *end returns* on certain fillet welds subject to cyclic *service loading*.

TABLE 10.17.1 Fatigue Design Parameters

Description	Stress Category	Constant Cf	Threshold FTH (MPa)	Potential Crack Initiation Point
SECTION 1 – PLAIN MATERIAL AWAY FROM ANY WELDING				
1.1 Base metal, except non- coated weathering steel, with rolled or cleaned surface. Flame-cut edges with surface roughness value of 25 m or less, but without reentrant corners.	A	250×10^8	165	Away from all welds or structural *connections*
1.2 Non-coated weathering steel base metal with rolled or cleaned surface. Flame-cut edges with surface roughness value of 25 m or less, but without reentrant corners.	B	120×10^8	110	Away from all welds or structural *connections*
1.3 Member with drilled or reamed holes. Member with re- entrant corners at *copes*, cuts, block-outs or other geometrical discontinuities made to requirements of Section 10.17.3.5, except weld access holes.	B	120×10^8	110	At any external edge or at hole perimeter
1.4 Rolled cross sections with weld access holes made to re- quirements of Section 10.10.1.6 and Section 10.17.3.5. Members with drilled or reamed holes con- taining bolts for attachment of light bracing where there is a small longitudinal component of brace *force*.	C	44×10^8	69	At *reentrant* corner of weld access hole or at any small hole (may contain bolt for minor *connections*)
SECTION 2 – CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS				
2.1 Gross area of base metal in lap joints connected by high- strength bolts in joints satis- fying all requirements for slip-critical connections.	B	120×10^8	110	Through gross section near hole
2.2 Base metal at net section of high-strength bolted joints, de- signed on the basis of bearing resistance, but fabricated and installed to all requirements for slip-critical connections.	B	120×10^8	110	In net section originat- ing at side of hole

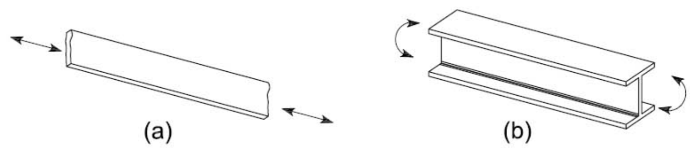
2.3 Base metal at the net sec- tion of other mechanically fas- tened joints except eye bars and pin plates.	D	22×10^8	48	In net section originat- ing at side of hole
2.4 Base metal at net section of *eyebar* head or pin plate.	E	11×10^8	31	In net section originat- ing at side of hole

TABLE 10.17.1 (Cont.)
Fatigue Design Parameters

Illustrative Typical Examples

SECTION 1 – PLAIN MATERIAL AWAY FROM ANY WELDING

1.1 and 1.2



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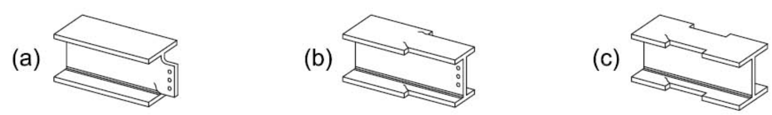

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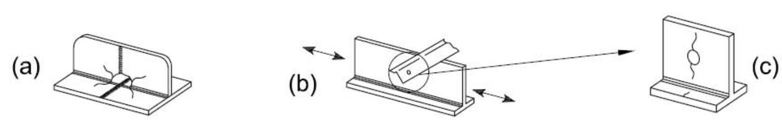
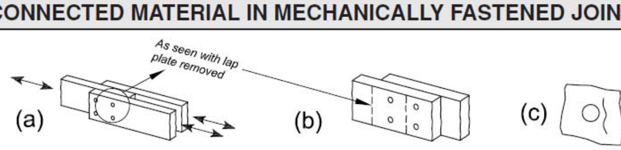


SECTION 2 – CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS

2.1



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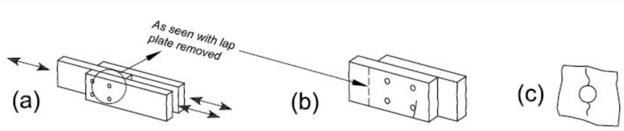


2.3

2.4

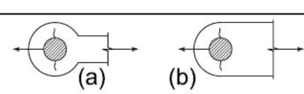


TABLE 10.17.1 (cont.) Fatigue Design Parameters

Description	Stress Category	Constant Cf	Threshold FTH (MPa)	Potential Crack Initiation Point
SECTION 3 – WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS				
3.1 Base metal and weld metal in members without attachments built-up of plates or shapes connected by continuous longitudinal complete-joint-penetration groove welds, back gouged and welded from second side, or by	B	120×10^8	110	From surface or internal discontinuities in weld away from end of weld
3.2 Base metal and weld metal in members without attachments built-up of plates or shapes, connected by continuous longitudinal complete-joint-penetration groove welds with backing bars not re- moved, or by continuous	B	61×10^8	83	From surface or internal discontinuities in weld, including weld attaching backing bars
3.3 Base metal and weld metal ter- mination of longitudinal welds at weld access holes in	D	22×10^8	48	From the weld termi- nation into the web or flange
3.4 Base metal at ends of lon- gitudinal intermittent fillet weld segments.	E	11×10^8	31	In connected material at start and stop locations of any weld deposit
3.5 Base metal at ends of partial length welded coverplates narrower than the flange having square or tapered ends, with or without welds across the ends of coverplates wider than the flange with welds across the ends. Flange thickness $\leq$ 20 mm Flange thickness $>$ 20 mm				In flange at toe of end weld or in flange at termination of longitudinal weld or in edge of flange with wide coverplates
3.6 Base metal at ends of partial length welded coverplates wider than the flange without	E'	3.9×10^8	18	In edge of flange at end of coverplate weld
SECTION 4 – LONGITUDINAL FILLET WELDED END CONNECTIONS				
4.1 Base metal at junction of axially loaded members with longitudinally welded end connections. Welds shall be on each side of the axis of the member to balance weld stresses. $t \leq 20$ mm $t > 20$ mm	 E E'	 11×10^8 3.9×10^8	 31 18	Initiating from end of any weld termination ex- tending into the base metal

TABLE 10.17.1 (Cont.) Fatigue Design Parameters

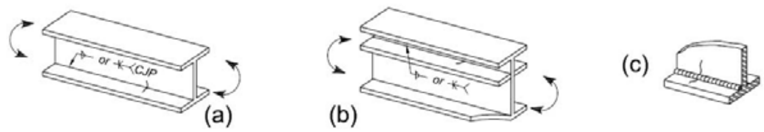
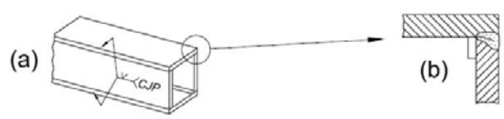
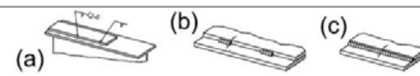
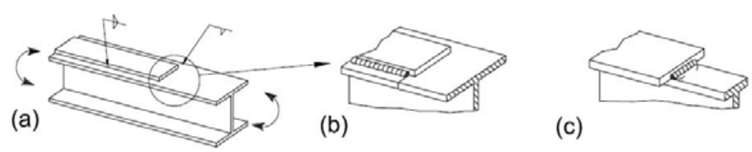
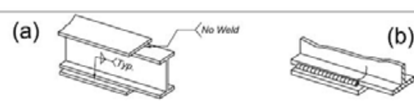
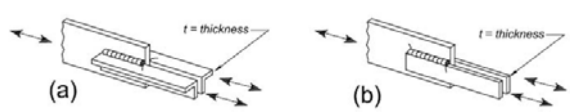
Illustrative Typical Examples
SECTION 3 – WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS
3.1 
3.2 
3.3 
3.4 
3.5 
3.6 
SECTION 4 – LONGITUDINAL FILLET WELDED END CONNECTIONS
4.1 

TABLE 10.17.1 (cont.): Fatigue Design Parameters

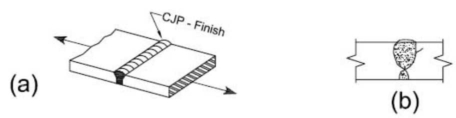
Description	Stress Category	Constant Cf	Threshold FTH (MPa)	Potential Crack Initiation Point
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS				
5.1 Base metal and weld metal in or adjacent to complete-joint-penetration groove welded splices in rolled or welded cross sections with welds ground essentially par- allel to the direction of stress.	B	120 × 108	110	From internal discon- tinuities in filler metal or along the fusion boundary
5.2 Base metal and weld metal in or adjacent to complete-joint-penetration groove welded splices with welds ground essentially parallel to the direction of stress at transitions in thickness or width made on a slope no greater than 8 to 20%. Fy < 620 MPa Fy ≥ 620 MPa	B B'	120 × 108 61 × 108	110 83	From internal disconti- nuities in filler metal or along fusion boundary or at start of transition when Fy ≥ 620 MPa
5.3 Base metal with Fy equal to or greater than 620 MPa and weld metal in or adja- cent to complete-joint-penetra- tion groove welded splices with welds ground essentially parallel to the direction of stress at tran- sitions in width made on a radius of not less than 600 mm with the point of tangency at the end of the groove weld.	B	120 × 108	110	From internal discon- tinuities in filler metal or discontinuities along the fusion boundary
5.4 Base metal and weld metal in or adjacent to the toe of complete- joint-penetration T or corner joints or splices, with or without transitions in thickness having slopes no greater than 8 to 20%, when weld reinforcement is not removed.	C	44 × 108	69	From surface discon- tinuity at toe of weld extending into base metal or along fusion boundary.
5.5 Base metal and weld metal at transverse end connections of tension-loaded plate elements using partial-joint-penetration butt or T or corner joints, with reinforcing or contouring fillets, FSR shall be the smaller of the toe crack or root crack stress range. Crack initiating from weld toe: Crack initiating from weld root:	C C'	44 × 108 Eqn. 10.17.3.4	69 None provided	Initiating from geomet- rical discontinuity at toe of weld extending into base metal or, initiat- ing at weld root subject to tension extending up and then out through weld

TABLE 10.17.1 (Cont.)
Fatigue Design Parameters

Illustrative Typical Examples

SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS

5.1



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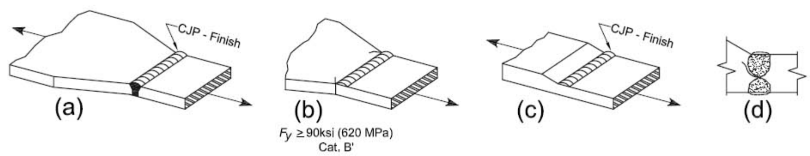

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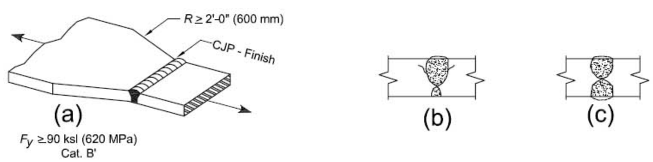

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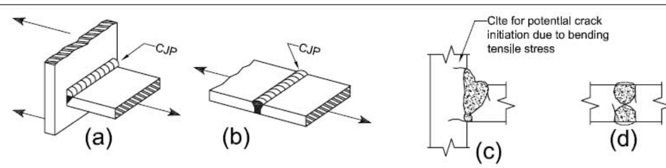

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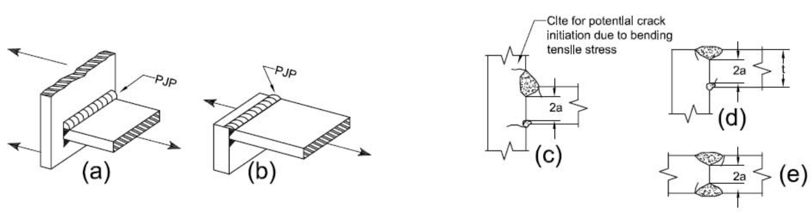


TABLE 10.17.1 (cont.) : Fatigue Design Parameters

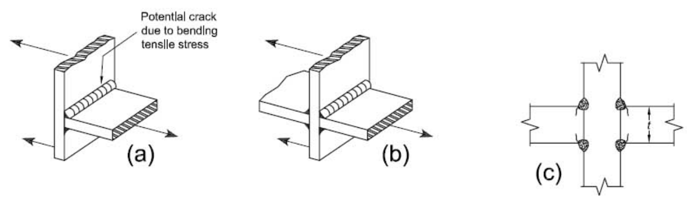
Description	Stress Category	Constant	Threshold F_{TH} (MPa)	Potential Crack Initiation Point
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (cont'd)				
5.6 Base metal and filler metal at transverse end connections of tension-loaded plate elements using a pair of fillet welds on opposite sides of the plate. F_{SR} shall be the smaller of the toe crack or root crack stress range.	C	44×10^8	69	Initiating from geomet- rical discontinuity at toe of weld extend- ing into base metal or, initiating at *weld root* subject to tension ex- tending up and then out through weld
Crack initiating from weld toe:	C'	Eqn. 10.17.3.5	None provided	
5.7 Base metal of tension loaded plate elements and on girders and rolled beam webs or flanges at toe of transverse fillet welds adjacent to welded transverse stiffeners.	C	44×10^8	69	From geometrical dis- continuity at *toe of fil- let* extending into base metal
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS				
6.1 Base metal at details attached by complete-joint- penetration groove welds subject to longitudinal loading only when the detail embodies a transition radius R with the weld termination ground smooth.				Near point of tangency of radius at edge of member
$R \geq 600$ mm	B	120×10^8	110	
600 mm $> R \geq 150$ mm	C	44×10^8	69	
150 mm $> R \geq 50$ mm	D	22×10^8	48	
50 mm $> R$	E	11×10^8	31	

TABLE 10.17.1 (Cont.)
Fatigue Design Parameters

Illustrative Typical Examples

SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (cont'd)

5.6



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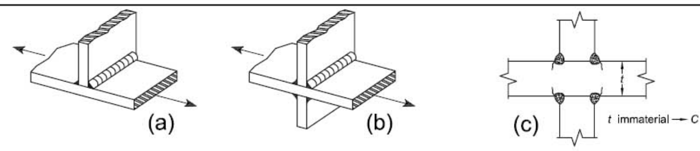


SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS

6.1

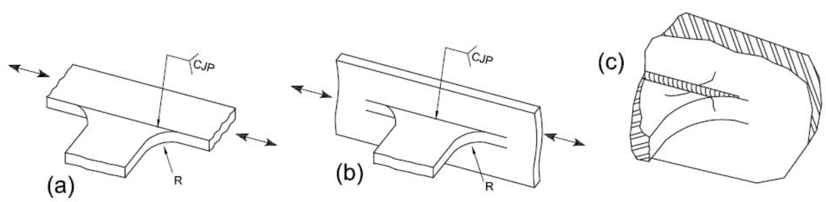


TABLE 10.17.1 (cont.): Fatigue Design Parameters

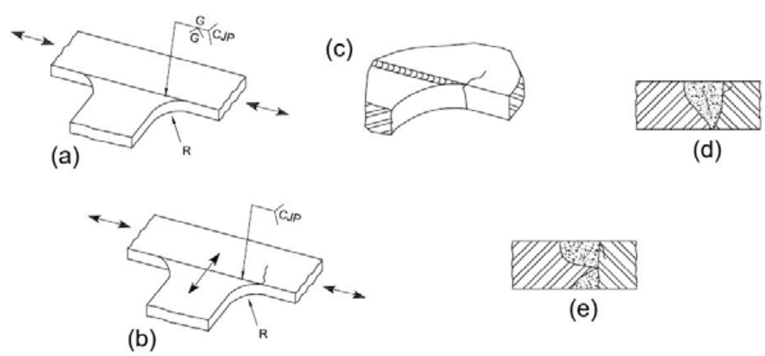
Description	Stress Category	Constant Cf	Threshold FTH (MPa)	Potential Crack Initiation Point
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)				
6.2 Base metal at details of equal thickness attached by complete-joint-penetration groove welds subject to transverse loading with or without longitudinalloading when the detail embodies a tran- sition radius *R* with the weld termination ground smooth:				
When weld reinforcement is removed:				
$R \geq 600$ mm	B	120×10^8	110	
600 mm $> R \geq 150$ mm	C	44×10^8	69	Near points of tangency of radius or in the weld or at fusion boundary or member or attachment
150 mm $> R \geq 50$ mm	D	22×10^8	48	
50 mm $> R$	E	11×10^8	31	
When weld reinforcement is not removed:				
$R \geq 600$ mm	C	44×10^8	69	
600 mm $> R \geq 150$ mm	C	44×10^8	69	At toe of the weld either along edge of member or the attachment
150 mm $> R \geq 50$ mm	D	22×10^8	48	
50 mm $> R$	E	11×10^8	31	
6.3 Base metal at details of unequal thickness attached by complete-joint-penetration groove welds subject to transverse loading with or without longitudinal loading when the detail embodies a transition ra- dius *R* with the weld termina-tion ground smooth.				
When weld reinforcement is removed:				At toe of weld along edge of thinner material
$R > 50$ mm	D	22×10^8	48	In weld termination in small radius
$R \leq 50$ mm	E	11×10^8	31	
When reinforcement is not removed:				At toe of weld along edge of thinner material
Any radius	E	11×10^8	31	

TABLE 10.17.1 (Cont.) Fatigue Design Parameters

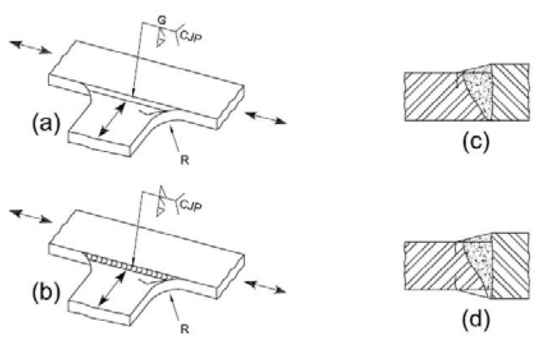
Illustrative Typical Examples

SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)

6.2



6.3



Description	Stress Category	Constant Cf	Threshold FTH (MPa)	Potential Crack Initiation Point
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)				
6.4 Base metal subject to longitudinal stress at transverse members, with or without transverse stress, attached by fillet or partial penetration groove welds parallel to direction of stress when the detail embodies a transition radius, *R*, with weld termination ground smooth:				In weld termination or from the toe of the weld extending into member
$R > 50$ mm	D	22×10^8		
$R \leq 50$ mm	E	11×10^8		
SECTION 7 – BASE METAL AT SHORT ATTACHMENTS[1]				
7.1 Base metal subject to longitu- dinal loading at details attached by fillet welds parallel or trans- verse to the direction of stress where the detail embodies no transition radius and with detail length in direction of stress, *a*, and attachment height normal to the surface of the member, *b*:				In the member at the end of the weld
$a < 50$ mm	C	44×10^8	69	
50 mm $\leq a \leq 12\,b$ or 100 mm	D	22×10^8	48	
$a > 12b$ or100 mm when b is ≤ 25 mm	E	11 10^8	31	
$a > 12b$ or 100 mm when b is > 25 mm	E'	3.9×10^8	18	
7.2 Base metal subject to longitudinal *stress* at details attached by fillet or partial-joint-penetration groove welds, with or without transverse *load* on detail, when the detail embodies a transition radius, *R*, with weld termination ground smooth:				In weld termination ex- tending into member
$R > 50$ mm	D	22×10^8		
$R \leq 50$ mm	E	11 10^8		

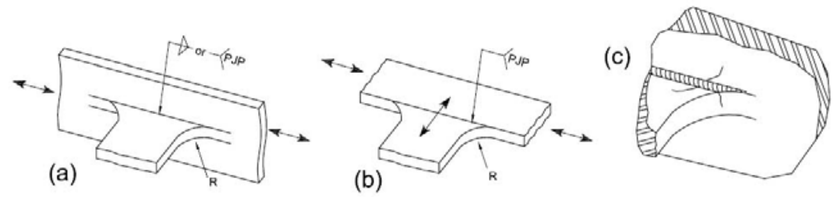
[1] "Attachment" as used herein, is defined as any steel detail welded to a member which, by its mere presence and independent of its loading, causes a discontinuity in the *stress* flow in the member and thus reduces the *fatigue* resistance.

TABLE 10.17.1 (Cont.)
Fatigue Design Parameters

Illustrative Typical Examples

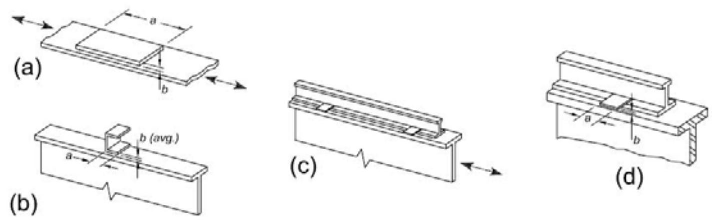
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)

6.4



SECTION 7 – BASE METAL AT SHORT ATTACHMENTS

7.1



7.2

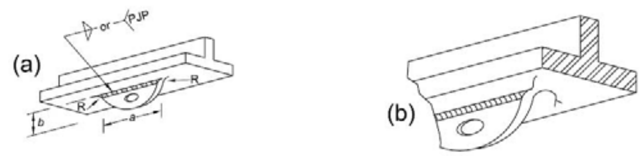


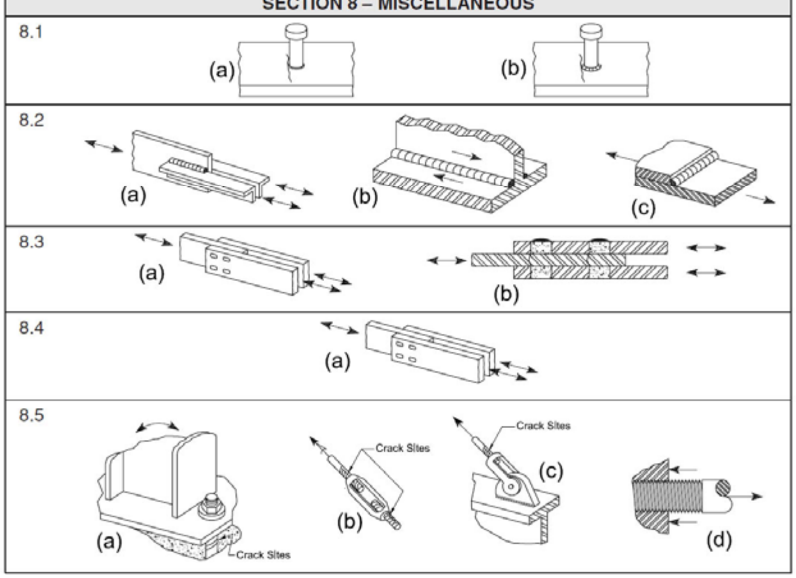
TABLE 10.17.1 (cont.): Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH} (MPa)	Potential Crack Initiation Point
SECTION 8 - MISCELLANEOUS				
8.1 Base metal at stud-type shear connectors attached by fillet or electric stud welding.	C	44×10^8	69	At toe of weld in base metal
8.2 Shear on throat of continu- ous or intermittent longitudinal or transverse fillet welds.	F	150×10^{10} (Eqn. 10.17.3.2)	55	In throat of weld
8.3 Base metal at plug or slot welds.	E	11×10^8	31	At end of weld in base metal
8.4 Shear on plug or slot welds.	F	150×10^{10} (Eqn. 10.17.3.2)	55	At *faying surface*
8.5 Not fully tightened high- strength bolts, common bolts, threaded anchor rods and hanger rods with cut, ground or rolled threads. Stress range on tensile stress area due to live load plus prying action when applicable.	E'	3.9×10^8	48	At the root of the threads extending into the tensile *stress* area

TABLE 10.17.1 (Cont.)
Fatigue Design Parameters

Illustrative Typical Examples

SECTION 8 – MISCELLANEOUS



10.18 Structural Design for Fire Conditions

This section provides criteria for the design and evaluation of *structural steel* components, systems and frames for fire conditions. These criteria provide for the determination of the heat input, thermal expansion and degradation in mechanical properties of materials at elevated temperatures that cause progressive decrease in strength and *stiffness* of structural components and systems at elevated temperatures.

10.18.1 General Provisions

The methods contained in this section provide regulatory evidence of compliance in accordance with the design applications outlined in this section.

The section uses the following terms in addition to the terms in the Glossary.

Active fire protection: Building materials and systems that are activated by a fire to mitigate adverse effects or to notify people to take some action to mitigate adverse effects.

Compartmentation: The enclosure of a building space with elements that have a specific fire endurance.

Convective heat transfer: The transfer of thermal energy from a point of higher temperature to a point of lower temperature through the motion of an intervening medium.

Design-basis fire: A set of conditions that define the development of a fire and the spread of combustion products throughout a building or portion thereof.

Elevated temperatures: Heating conditions experienced by building elements or structures as a result of fire, which are in excess of the anticipated ambient conditions.

Fire: Destructive burning, as manifested by any or all of the following: light, flame, heat, or smoke.

Fire barrier: Element of construction formed of fire-resisting materials and tested in accordance with ASTM Standard E119, or other approved standard fire resistance test, to demonstrate compliance with the Building Code.

Fire endurance: A measure of the elapsed time during which a material or assembly continues to exhibit fire resistance.

Fire resistance: That property of assemblies that prevents or retards the pas- sage of excessive heat, hot gases or flames under conditions of use and enables them to continue to perform a stipulated function.

Fire resistance rating: The period of time a building element, component or assembly maintains the ability to contain a fire, continues to perform a given structural function, or both, as determined by test or methods based on tests.

Flashover: The rapid transition to a state of total surface involvement in a fire of combustible materials within an enclosure.

Heat flux: Radiant energy per unit surface area.

Heat release rate: The rate at which thermal energy is generated by a burning material.

Passive fire protection: Building materials and systems whose ability to resist the effects of fire does not rely on any outside activating condition or mechanism.

Performance-based design: An engineering approach to structural design that is based on agreed-upon performance goals and objectives, engineering analysis and quantitative assessment of alternatives against those design goals and objectives using accepted engineering tools, methodologies and performance criteria.

Prescriptive design: A design method that documents compliance with general criteria established in a building code.

Restrained construction: Floor and roof assemblies and individual beams in buildings where the surrounding or supporting structure is capable of resisting substantial thermal expansion throughout the range of anticipated elevated temperatures.

Unrestrained construction: Floor and roof assemblies and individual beams in buildings that are assumed to be free to rotate and expand throughout the range of anticipated elevated temperatures.

10.18.1.1 Performance Objective

Structural components, members and building frame systems shall be designed so as to maintain their load-bearing function during the design-basis fire and to satisfy other performance requirements specified for the building occupancy. Deformation criteria shall be applied where the means of providing structural fire resistance, or the design criteria for fire barriers, requires consideration of the deformation of the load-carrying structure. Within the compartment of fire origin, forces and deformations from the design basis fire shall not cause a breach of horizontal or vertical compartmentation.

10.18.1.2 Design by Engineering Analysis

The analysis methods in Section 10.18.2 are permitted to be used to document the anticipated performance of steel framing when subjected to design-basis fire scenarios. Methods in Section 10.18..2 provide evidence of compliance with performance objectives established in Section 10.18.1.1.

The analysis methods in Section 10.18.2 are permitted to be used to demonstrate an equivalency for an alternative material or method, as permitted by the building code.

10.18.1.3 Design by Qualification Testing

The qualification testing methods in Section 10.18.3 are permitted to be used to document the fire resistance of steel framing subject to the standardized fire testing protocols required by building codes.

10.18.1.4 Load Combinations and Required Strength

The *required strength* of the structure and its elements shall be determined from the following *gravity load combination*:

$$[0.9\ or\ 1.2]D + T + 0.5L + 0.2S \qquad (10.18.1.1)$$

where

D = *nominal dead load*

L = nominal occupancy live load

S = nominal snow load

T = nominal *forces* and deformations due to the design-basis fire defined in Section 4.2.1

A lateral *notional load*, N_i = $0.002Y_i$, as defined in Section 10.20, where N_i = notional *lateral load* applied at framing level i and Y_i = *gravity load* from combination 10.18.1.1 acting on framing level i, shall be applied in combination with the loads stipulated in Equation 10.18.1.1. Unless otherwise stipulated by the *authority having jurisdiction*, *D*, *L* and *S* shall be the *nominal loads* specified in Chapter 2 of Part 6 of this code.

10.18.2 Structural Design for Fire Conditions By Analysis

It is permitted to design structural members, components and building frames for elevated temperatures in accordance with the requirements of this section.

10.18.2.1 **Design-Basis Fire**

A design-basis fire shall be identified to describe the heating conditions for the structure. These heating conditions shall relate to the fuel commodities and compartment characteristics present in the assumed fire area. The fuel *load* density based on the occupancy of the space shall be considered when determining the total fuel *load*. Heating conditions shall be specified either in terms of a heat flux or temperature of the upper gas layer created by the fire. The variation of the heating conditions with time shall be determined for the duration of the fire.

When the analysis methods in Section 10.18.2 are used to demonstrate an equivalency as an alternative material or method as permitted by a building code, the design-basis fire shall be determined in accordance with ASTM E119.

10.18.2.1.1 Localized Fire

Where the heat release rate from the fire is insufficient to cause flashover, a localized fire exposure shall be assumed. In such cases, the fuel composition, arrangement of the fuel array and floor area occupied by the fuel shall be used to determine the radiant heat flux from the flame and smoke plume to the structure.

10.18.2.1.2 Post-Flashover Compartment Fires

Where the heat release rate from the fire is sufficient to cause flashover, a post-flashover compartment fire shall be assumed. The determination of the temperature versus time profile resulting from the fire shall include fuel *load*, ventilation characteristics to the space (natural and mechanical), compartment dimensions and thermal characteristics of the compartment boundary.

10.18.2.1.3 Exterior Fires

The exposure of exterior structure to flames projecting from windows or other wall openings as a result of a post-flashover compartment fire shall be considered along with the radiation from the interior fire through the opening. The shape and length of the flame projection shall be used along with the distance between the flame and the exterior steelwork to determine the heat flux to the steel. The method identified in Section 10.18.2.1.2 shall be used for describing the characteristics of the interior compartment fire.

10.18.2.1.4 Fire Duration

The fire duration in a particular area shall be determined by considering the total combustible mass, in other words, fuel *load* available in the space. In the case of either a localized fire or a post-flashover compartment fire, the time duration shall be determined as the total combustible mass divided by the mass loss rate, except where determined from Section 10.18.2.1.2.

10.18.2.1.5 Active Fire Protection Systems

The effects of active fire protection systems shall be considered when describing the design-basis fire.

Where automatic smoke and heat vents are installed in nonsprinklered spaces, the resulting smoke temperature shall be determined from calculation.

10.18.2.2 **Temperatures in Structural Systems under Fire Conditions**

Temperatures within structural members, components and frames due to the heating conditions posed by the design-basis fire shall be determined by a heat transfer analysis.

Table 10.18.2.1
Properties of Steel at Elevated Temperatures

Steel Temperature [°C]	$k_E = E_m/E$	$k_y = F_{ym}/F_y$	$k_u = F_{um}/F_y$
20	*	*	*
93	1.00	*	*
204	0.90	*	*
316	0.78	*	*
399	0.70	1.00	1.00
427	0.67	0.94	0.94
538	0.49	0.66	0.66
649	0.22	0.35	0.35
760	0.11	0.16	0.16
871	0.07	0.07	0.07
982	0.05	0.04	0.04
1093	0.02	0.02	0.02
1204	0.00	0.00	0.00

*Use ambient properties.

10.18.2.3 **Material Strengths at Elevated Temperatures**

Material properties at elevated temperatures shall be determined from test data. In the absence of such data, it is permitted to use the material properties stipulated in this section. These relationships do not apply for steels with a *yield strength* in excess of 448 MPa or concretes with specified compression strength in excess of 55 MPa.

10.18.2.3.1 Thermal Elongation

Thermal expansion of structural and reinforcing steels: For calculations at temperatures above 65°C, the coefficient of thermal expansion shall be 1.4×10^{-5} /°C.

Thermal expansion of normal weight concrete: For calculations at temperatures above 65 °C, the coefficient of thermal expansion shall be 1.8×10^{-5} /°C.

Thermal expansion of lightweight concrete: For calculations at temperatures above 65°C, the coefficient of thermal expansion shall be 7.9×10^{-6} /°C.

10.18.2.3.2 Mechanical Properties at Elevated Temperatures

The deterioration in strength and *stiffness* of structural members, components, and systems shall be taken into account in the *structural analysis* of the frame.

The values F_{ym}, F_{um}, E_m, f'_{cm}, E_{cm} and ε_{cu} at elevated temperature to be used in structural analysis, expressed as the ratio with respect to the property at ambient, assumed to be 20° C, shall be defined as in Tables 10.18.2.1 and 10.18.2.2. It is permitted to interpolate between these values.

Table 10.18.2.2
Properties of Concrete at Elevated Temperatures

Concrete Temperature [°C]	$k_c = f'_{cm}/f'_c$		E_{cm}/E_c	ε_{cu}(%)
	NWC	LWC		LWC
20	1.00	1.00	1.00	0.25
93	0.95	1.00	0.93	0.34
204	0.90	1.00	0.75	0.46
288	0.86	1.00	0.61	0.58
316	0.83	0.98	0.57	0.62
427	0.71	0.85	0.38	0.80
538	0.54	0.71	0.20	1.06
649	0.38	0.58	0.092	1.32
760	0.21	0.45	0.073	1.43
871	0.10	0.31	0.055	1.49
982	0.05	0.18	0.036	1.50
1093	0.01	0.05	0.018	1.50
1204	0.00	0.00	0.00	–

10.18.2.4 **Structural Design Requirements**

10.18.2.4.1 General Structural Integrity

The structural frame shall be capable of providing adequate strength and deformation capacity to withstand, as a system, the structural actions developed during the fire within the prescribed limits of deformation. The *structural system* shall be designed to sustain local damage with the structural system as a whole remaining stable.

Continuous *load* paths shall be provided to transfer all *forces* from the ex- posed region to the final point of resistance. The foundation shall be designed to resist the forces and to accommodate the deformations developed during the design-basis fire.

10.18.2.4.2 Strength Requirements and Deformation Limits

Conformance of the structural system to these requirements shall be demonstrated by constructing a mathematical model of the structure based on principles of structural mechanics and evaluating this model for the internal forces and deformations in the members of the structure developed by the temperatures from the design-basis fire.

Individual members shall be provided with adequate strength to resist the shears, axial forces and moments determined in accordance with these provisions.

Connections shall develop the strength of the connected members or the forces indicated above. Where the means of providing fire resistance requires the consideration of deformation criteria, the deformation of the structural system, or members thereof, under the design-basis fire shall not exceed the prescribed limits.

10.18.2.4.3 Methods of Analysis

10.18.2.4.3a Advanced Methods of Analysis

The methods of analysis in this section are permitted for the design of all steel building structures for fire conditions. The design-basis fire exposure shall be that determined in Section 10.18.2.1. The analysis shall include both a thermal response and the mechanical response to the design-basis fire.

The *thermal response* shall produce a temperature field in each structural element as a result of the design-basis fire and shall incorporate temperature- dependent thermal properties of the structural elements and fire-resistive materials as per Section 10.18.2.2.

The *mechanical response* results in forces and deflections in the structural sys- tem subjected to the thermal response calculated from the design-basis fire. The mechanical response shall take into account explicitly the deterioration in strength and *stiffness* with increasing temperature, the effects of thermal expansions and large deformations. Boundary conditions and connection fixity must represent the proposed structural design. Material properties shall be defined as per Section 10.18.2.3.

The resulting analysis shall consider all relevant *limit states*, such as excessive deflections, connection fractures, and overall or *local buckling*.

10.18.2.4.3a Advanced Methods of Analysis

The methods of analysis in this section are applicable for the evaluation of the performance of individual members at elevated temperatures during exposure to fire.

The support and restraint conditions (forces, moments and boundary conditions) applicable at normal temperatures may be assumed to remain unchanged throughout the fire exposure.

(1) Tension members

It is permitted to model the thermal response of a tension element using a one-dimensional heat transfer equation with heat input as directed by the design-basis fire defined in Section 10.18.2.1.

The *design strength* of a tension member shall be determined using the provisions of Section 10.4, with steel properties as stipulated in Section 10.18.2.3 and assuming a uniform temperature over the cross section using the temperature equal to the maximum steel temperature.

(2) Compression members

It is permitted to model the thermal response of a compression element using a one-dimensional heat transfer equation with heat input as directed by the design-basis fire defined in Section 10.18.2.1.

The design strength of a compression member shall be determined using the provisions of Section 10.5 with steel properties as stipulated in Section 10.18.2.3.

(3) Flexural members

It is permitted to model the thermal response of flexural elements using a one-dimensional heat transfer equation to calculate bottom flange temperature and to assume that this bottom flange temperature is constant over the depth of the member. The design strength of a flexural member shall be determined using the provisions of Section 10.6 with steel properties as stipulated in Section 10.18.2.3.

(4) Composite floor members

It is permitted to model the thermal response of flexural elements supporting a concrete slab using a one-dimensional heat transfer equation to calculate bottom flange temperature. That temperature shall be taken as constant between the bottom flange and mid-depth of the web and shall decrease linearly by no more than 25 percent from the mid-depth of the web to the top flange of the *beam*.

The design strength of a *composite* flexural member shall be determined using the provisions of Chapter 13 of Part 6 of this code, with reduced *yield stresses* in the steel consistent with the temperature variation described under thermal response.

10.18.2.4.4 Design Strength

The design strength shall be determined as in Section 10.2.3.3. The *nominal strength*, R_n , shall be calculated using material properties, as stipulated in Section 10.18.2.3, at the temperature developed by the design-basis fire.

10.18.3 Design By Qualification Testing

10.18.3.1 **Design Strength**

Structural members and components in steel buildings shall be qualified for the rating period in conformance with ASTM E119. It shall be permitted to demonstrate compliance with these requirements using the procedures specified for steel construction in Section 5 of ASCE/SFPE 29.

10.18.3.2 **Restrained Construction**

For floor and roof assemblies and individual *beams* in buildings, a restrained condition exists when the surrounding or supporting structure is capable of resisting actions caused by thermal expansion throughout the range of anticipated elevated temperatures.

Steel beams, girders and frames supporting concrete slabs that are welded or bolted to integral framing members (in other words, *columns*, girders) shall be considered restrained construction.

10.18.3.3 **Unrestrained Construction**

Steel beams, girders and frames that do not support a concrete slab shall be considered unrestrained unless the members are bolted or welded to surrounding construction that has been specifically designed and detailed to resist actions caused by thermal expansion.

A steel member bearing on a wall in a single span or at the end span of multiple spans shall be considered unrestrained unless the wall has been designed and detailed to resist effects of thermal expansion.

10.19 Stability Bracing For Columns and Beams

This section addresses the minimum brace strength and *stiffness* necessary to provide member *strengths* based on the *unbraced length* between braces with an *effective length factor*, K, equal to 1.0.

10.19.1 General Provisions

Bracing is assumed to be perpendicular to the members to be braced; for inclined or diagonal bracing, the brace strength (force or moment) and stiffness (force per unit displacement or moment per unit rotation) shall be adjusted for the angle of inclination. The evaluation of the stiffness furnished by a brace shall include its member and geometric properties, as well as the effects of connections and anchoring details.

Two general types of bracing systems are considered, relative and nodal. A relative brace controls the movement of the brace point with respect to adjacent braced points. A nodal brace controls the movement at the braced point without direct interaction with adjacent braced points. The available strength and stiffness of the bracing shall equal or exceed the required limits unless analysis indicates that smaller values are justified by analysis.

A second-order analysis that includes an initial out-of-straightness of the member to obtain brace strength and stiffness is permitted in lieu of the requirements of this section.

10.19.2 Columns

It is permitted to brace an individual column at end and intermediate points along its length by either relative or nodal bracing systems. It is assumed that nodal braces are equally spaced along the column.

10.19.2.1 **Relative Bracing**

The required brace strength is

$$P_{br} = 0.004\,P_r \tag{10.19.2.1}$$

The required brace stiffness is

$$\beta_{br} = \frac{1}{\varphi}\left(\frac{2P_r}{L_b}\right)(LRFD) \qquad \beta_{br} = \Omega\left(\frac{2P_r}{L_b}\right)(ASD) \tag{10.19.2.2}$$

where

$\phi = 0.75$ (LRFD) $\quad \Omega = 2.00$ (ASD)

For design according to Section 10.2.3.3 (LRFD)

Pr = required axial compressive strength using LRFD load combinations, N

For design according to Section 10.2.3.4 (ASD)

Pr = required axial compressive strength using ASD load combinations, N

10.19.2.2 Nodal Bracing

The required brace strength is

$$P_{br} = 0.01\, P_r \qquad (10.19.2.3)$$

The required brace stiffness is

$$\beta_{br} = \frac{1}{\varphi}\left(\frac{8P_r}{L_b}\right)(LRFD) \qquad \beta_{br} = \Omega\left(\frac{8P_r}{L_b}\right)(ASD) \qquad (10.19.2.4)$$

where

ϕ = 0.75 (LRFD) Ω = 2.00 (ASD)

For design according to Section 10.2.3.3 (LRFD)

Pr = required axial compressive strength using LRFD load combinations, N

For design according to Section 10.2.3.4 (ASD)

Pr = required axial compressive strength using ASD load combinations, N

When Lb is less than Lq, where Lq is the maximum unbraced length for the required column force with K equal to 1.0, then Lb in Equation 10.19.2.4 is permitted to be taken equal to Lq.

10.19.3 Beams

At points of support for beams, girders and trusses, restraint against rotation about their longitudinal axis shall be provided. Beam bracing shall prevent the relative displacement of the top and bottom flanges, in other words, twist of the section. Lateral stability of beams shall be provided by lateral bracing, torsional bracing or a combination of the two. In members subjected to double curvature bending, the inflection point shall not be considered a brace point.

10.19.3.1 Lateral Bracing

Bracing shall be attached near the compression flange, except for a cantilevered member, where an end brace shall be attached near the top (tension) flange. Lateral bracing shall be attached to both flanges at the brace point nearest the inflection point for beams subjected to double curvature bending along the length to be braced.

a. Relative Bracing

The required brace strength is

$$P_{br} = 0.008\, M_r C_d / h_o \qquad (10.19.3.1)$$

The required brace *stiffness* is

$$\beta_{br} = \frac{1}{\varphi}\left(\frac{4M_r C_d}{L_b h_o}\right)(LRFD) \qquad \beta_{br} = \Omega\left(\frac{4M_r C_d}{L_b h_o}\right)(ASD) \qquad (10.19.3.2)$$

Where,

ϕ = 0.75 (LRFD) Ω = 2.00 (ASD)

h_o = distance between flange centroids, mm.

C_d = 1.0 for bending in *single curvature*; 2.0 for double curvature; C_d = 2.0 only applies to the brace closest to the inflection point.

L_b = laterally *unbraced length*, mm.

For design according to Section 10.2.3.3 (LRFD)

M_r = *required flexural strength* using *LRFD load combinations*, N-mm

For design according to Section 10.2.3.4 (ASD)

M_r = required flexural strength using *ASD load combinations*, N-mm

b. Nodal Bracing

The required brace strength is

$$P_{br} = 0.02\, M_r C_d / h_o \qquad (10.19.3.3)$$

The required brace *stiffness* is

$$\beta_{br} = \frac{1}{\varphi}\left(\frac{10 M_r C_d}{L_b h_o}\right)(LRFD) \qquad \beta_{br} = \Omega\left(\frac{10 M_r C_d}{L_b h_o}\right)(ASD) \qquad (10.19.3.4)$$

where

ϕ = 0.75 (LRFD) Ω = 2.00 (ASD)

For design according to Section 10.2.3.3 (LRFD)

M_r = *required flexural strength* using *LRFD load combinations*, N-mm

For design according to Section 10.2.3.4 (ASD)

M_r = required flexural strength using *ASD load combinations*, N-mm

When L_b is less than L_q, the maximum unbraced length for M_r, then L_b in Equation 10.19.3.4 shall be permitted to be taken equal to L_q.

10.19.3.2 **Torsional Bracing**

It is permitted to provide either nodal or continuous *torsional bracing* along the *beam* length. It is permitted to attach the bracing at any cross-sectional location and it need not be attached near the compression flange. The *connection* between a torsional brace and the beam shall be able to support the required moment given below.

a. Nodal Bracing

The required bracing moment is

$$M_{br} = \frac{0.024\, M_r L}{n C_b L_b} \qquad (10.19.3.5)$$

The required cross-frame or diaphragm bracing stiffness is

$$\beta_{Tb} = \frac{\beta_T}{\left(1 - \frac{\beta_T}{\beta_{sec}}\right)} \qquad (10.19.3.6)$$

where

$$\beta_T = \frac{1}{\varphi}\left(\frac{2.4 L M_r^2}{n E I_y C_b^2}\right)(LRFD) \qquad \beta_T = \Omega\left(\frac{2.4 L M_r^2}{n E I_y C_b^2}\right)(ASD) \qquad (10.19.3.7)$$

$$\beta_{sec} = \frac{3.3\, E}{h_o}\left(\frac{1.5 h_o t_w^3}{12} + \frac{t_s b_s^3}{12}\right) \qquad (10.19.3.8)$$

where

ϕ = 0.75 (LRFD) Ω = 3.00 (ASD)

L = span length, mm

n = number of nodal braced points within the span

E = modulus of elasticity of steel 200 000 MPa

Iy = out-of-plane moment of inertia, mm4

Cb = modification factor defined in Section 10.6

tw = beam web thickness, mm

ts = web stiffener thickness, mm

bs = stiffener width for one-sided stiffeners (use twice the individual stiffener width for pairs of stiffeners), mm.

βT = brace stiffness excluding web distortion, N-mm/radian

βsec = web distortional stiffness, including the effect of web transverse stiffeners,

if any, N-mm/radian

For design according to Section 10.2.3.3 (LRFD)

Mr = required flexural strength using LRFD load combinations, N-mm

For design according to Section 10.2.3.4 (ASD)

Mr = required flexural strength using ASD load combinations, N-mm

If $\beta_{sec} < \beta_T$, Equation 10.19.3.6 is negative, which indicates that torsional beam bracing will not be effective due to inadequate web distortional stiffness.

When required, the web stiffener shall extend the full depth of the braced member and shall be attached to the flange if the torsional brace is also attached to the flange. Alternatively, it shall be permissible to stop the stiffener short by a distance equal to $4t_w$ from any beam flange that is not directly attached to the torsional brace. When L_b is less than L_q , then L_b in Equation 10.19.3.5 shall be permitted to be taken equal to L_q .

b. Continuous Torsional Bracing

For continuous bracing, use Equations 10.19.3.5, 10.19.3.6 and 10.19.3.8 with L/n taken as 1.0 and Lb taken as Lq ; the bracing moment and stiffness are given per unit span length. The distortional stiffness for an unstiffened web is

$$\beta_{sec} = \frac{3.3\, E t_w^3}{12 h_o} \qquad (10.19.3.9)$$

10.19.4 Slenderness Limitations

The slenderness ratio, L/r, of any stability bracing shall not exceed 180 unless a comprehensive analysis including second order effects justifies a higher value.

10.20 Seismic Provisions for Structural Steel Buildings

10.20.1 Scope

The *Seismic Provisions for Structural Steel Buildings*, hereinafter referred to as these *Provisions* as outline in this Section 10.20, shall govern the design, fabrication and erection of structural steel members and connections in the *seismic load resisting systems* (SLRS) and splices in columns that are not part of the SLRS, in buildings and other structures, where other structures are defined as those structures designed, fabricated and erected in a manner similar to buildings, with building-like vertical and lateral load-resisting-elements.

These *Provisions* shall apply when the *seismic response modification coefficient*, *R*, (as specified in Chapter 2 of Part 6) is taken greater than 3, regardless of the *seismic design category*. When the seismic response modification coefficient, *R*, is taken as 3 or less, the structure is not required to satisfy the *Provisions* this Section 10.20, unless specifically required by the applicable authority

These Provisions shall be applied in conjunction with the specification set forth in Sections 10.1 through 10.19 whichever is applicable.

Loads, load combinations, system limitations and general design requirements shall be those in Chapter 2 of Part 6 of this code as well as those mentioned in Section 10.2.

10.20.2 Referenced Specifications, Codes and Standards

The documents referenced in these Provisions shall include those listed in Section 10.1.2 with the following additions and modifications:

American Institute *of Steel Construction (AISC):*

Specification for Structural Steel Buildings, ANSI/AISC 360-05

Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications, ANSI/AISC 358-05

American Society for Nondestructive Testing (ASNT):

Recommended Practice for the Training and Testing of Nondestructive Testing Personnel, ASNT SNT TC-1a-2001

Standard for the Qualification and Certification of Nondestructive Testing Personnel, ANSI/ASNT CP-189-2001

American Welding Society (AWS):

Standard Methods for Determination of the Diffusible Hydrogen Content of Martensitic, Bainitic, and Ferritic Steel Weld Metal Produced by Arc Welding, AWS A4.3-93R

Standard Methods for Mechanical Testing of Welds-U.S. Customary, ANSI/ AWS B4.0-98

Standard Methods for Mechanical Testing of Welds–Metric Only, ANSI/AWS B4.0M:2000

Standard for the Qualification of Welding Inspectors, AWS B5.1:2003

Oxygen Cutting Surface Roughness Gauge and Wall Chart for Criteria Describing Oxygen-Cut Surfaces, AWS C4.1

Federal Emergency *Management Agency (FEMA)*

Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings, FEMA 350, July 2000

Symbols

Symbols used in this section are listed below. Numbers in parentheses after the definition refer to the section in these Provisions in which the symbol is first used.

Symbol	**Description**	**Section**
A_b	Cross-sectional area of a horizontal boundary element (HBE), (mm^2)	10.20.17.2.1
A_c	Cross-sectional area of a vertical boundary element (VBE), (mm^2)	10.20.17.2.1
A_f	Flange area, (mm^2)	10.20.8
A_g	Gross area, (mm^2)	10.20.9
A_{sc}	Area of the yielding segment of steel core, (mm^2)	10.20.16
A_{st}	Area of link stiffener, (mm^2)	10.20.15
A_w	Link web area, (mm^2)	10.20.15
C_a	Ratio of required strength to available strength	Table 10.20.8.1
C_d	Coefficient relating relative brace stiffness and curvature	10.20.9
C_d	Deflection amplification factor	10.C.2
C_r	Parameter used for determining the approximate fundamental period	10.C.2
D	Dead load due to the weight of the structural elements and permanent features on the building, (N)	10.20.9
D	Outside diameter of round HSS, (mm)	Table 10.20.8.1
E	Earthquake load	10.20.4
E	Effect of horizontal and vertical earthquake-induced loads	10.20.9
E	Modulus of elasticity of steel, E = 200,000 MPa	10.20.8
EI	Flexural elastic stiffness of the chord members of the special segment, ($N\text{-}mm^2$)	10.20.12
F_y	Specified minimum yield stress of the type of steel to be used, (MPa). As used in the *Specification*, "yield stress" denotes either the minimum specified yield point (for those steels that have a yield point) or the specified yield strength (for those steels that do not have a yield point)	10.20.6
F_{yb}	F_y of a beam, (MPa)	10.20.9

Symbol	Description	Section
F_{yc}	F_y of a column, (MPa)	10.20.9
F_{ysc}	Specified minimum yield stress of the steel core, or actual yield stress of the steel core as determined from a coupon test, (MPa) .	10.20.16
F_u	Specified minimum tensile strength, (MPa)	10.20.6
H	Height of story, which may be taken as the distance between the centerline of floor framing at each of the levels above and below, or the distance between the top of floor slabs at each of the levels above and below, (mm)	10.20.8
I	Moment of inertia, (mm^4)	10.20.12
I_c	Moment of inertia of a vertical boundary element (VBE) taken perpendicular to the direction of the web plate line, (mm^4)	10.20.17
K	Effective length factor for prismatic member	10.20.13
L	Span length of the truss, (mm)	10.20.12
L	Distance between VBE centerlines, (mm)	10.20.17
L_b	Length between points which are either braced against lateral displacement of compression flange or braced against twist of the cross section, (mm)	10.20.13
L_b	Link length, (mm)	10.20.15
L_{cf}	Clear distance between VBE flanges, (mm)	10.20.17
L_h	Distance between plastic hinge locations, (mm)	10.20.9
L_p	Limiting laterally unbraced length for full plastic flexural strength, uniform moment case, (mm)	10.20.12
L_{pd}	Limiting laterally unbraced length for plastic analysis, (mm)	10.20.13
L_s	Length of the special segment, (mm)	10.20.12
M_a	Required flexural strength, using ASD load combinations, (N-mm)	10.20.9
M_{av}	Additional moment due to shear amplification from the location of the plastic hinge to the column centerline based on ASD load combinations, (N-mm)	10.20.9
M_n	Nominal flexural strength, (N-mm)	10.20.11
M_{nc}	Nominal flexural strength of the chord member of the special segment, (N-mm)	10.20.12
M_p	Nominal plastic flexural strength, (N-mm)	Table 10.20.8.1
M_{pa}	Nominal plastic flexural strength modified by axial load, (N-mm)	10.20.15
M_{pb}	Nominal plastic flexural strength of the beam, (N-mm)	10.20.9
$M_{p,exp}$	Expected plastic moment, (N-mm)	10.20.9
M_{pc}	Nominal plastic flexural strength of the column, (N-mm)	10.20.8
M_r	Expected flexural strength, (N-mm)	10.20.9
M_{uv}	Additional moment due to shear amplification from the location of the plastic hinge to the column centerline based on LRFD load combinations, (N-mm)	10.20.9
M_u	Required flexural strength, using LRFD load combinations, (N-mm)	10.20.9
$M_{u,exp}$	Expected required flexural strength, (N-mm)	10.20.15

Symbol	Description	Section
P_a	Required axial strength of a column using ASD load combinations, (N)	10.20.8
P_{ac}	Required compressive strength using ASD load combinations, (N)	10.20.9
P_b	Required strength of lateral brace at ends of the link, (N)	10.20.15
P_c	Available axial strength of a column, (N)	10.20.9
P_n	Nominal axial strength of a column, (N)	10.20.8
P_{nc}	Nominal axial compressive strength of diagonal members of the special segment, (N)	10.20.12
P_{nt}	Nominal axial tensile strength of diagonal members of the special segment, (N)	10.20.12
P_r	Required compressive strength, (N)	10.20.15
P_{rc}	Required compressive strength using ASD or LRFD load combinations, (N)	10.20.9
P_u	Required axial strength of a column or a link using LRFD load combinations, (N)	10.20.8
P_{uc}	Required compressive strength using LRFD load combinations, (N)	10.20.9
P_y	Nominal axial yield strength of a member, equal to $F_y A_g$, (N)	Table 10.20.8.1
P_{ysc}	Axial yield strength of steel core, (N)	10.20.16
Q_b	Maximum unbalanced vertical load effect applied to a beam by the braces, (N)	10.20.13
Q_1	Axial forces and moments generated by at least 1.25 times the expected nominal shear strength of the link	10.20.15
R	Seismic response modification coefficient	10.20.1
R_n	Nominal strength, (N)	10.20.6
R_t	Ratio of the expected tensile strength to the specified minimum tensile strength F_u, as related to overstrength in material yield stress R_y	10.20.6
R_u	Required strength	10.20.9
R_v	Panel zone nominal shear strength	10.20.9
R_y	Ratio of the expected yield stress to the specified minimum yield stress, F_y	10.20.6
S	Snow load, (N)	10.20.9
V_a	Required shear strength using ASD load combinations, (N)	10.20.9
V_n	Nominal shear strength of a member, (N)	10.20.15
V_{ne}	Expected vertical shear strength of the special segment, (N)	10.20.12
V_p	Nominal shear strength of an active link, (N)	Table 10.20.8.1
V_{pa}	Nominal shear strength of an active link modified by the axial load magnitude, (N)	10.20.15
V_u	Required shear strength using LRFD load combinations, (N)	10.20.10
Z	Plastic section modulus of a member, (mm^3)	10.20.9
Z_b	Plastic section modulus of the beam, (mm^3)	10.20.9
Z_c	Plastic section modulus of the column, (mm^3)	10.20.9

Symbol	Description	Section
Z_x	Plastic section modulus x-axis, (mm^3)	10.20.8
Z_{RBS}	Minimum plastic section modulus at the reduced beam section, (mm^3)	10.20.9
a	Angle that diagonal members make with the horizontal	10.20.12
b	Width of compression element as defined in *Specification* Section 10.2.4.1, (mm)	Table 10.20.8.1
b_{cf}	Width of column flange, (mm)	10.20.9
b_f	Flange width, (mm)	10.20.9
d	Nominal fastener diameter, (mm)	10.20.7
d	Overall beam depth, (mm)	10.20.15
d_c	Overall column depth, (mm)	10.20.9
d_z	Overall panel zone depth between continuity plates, (mm)	10.20.9
e	EBF link length, (mm)	10.20.15
h	Clear distance between flanges less the fillet or corner radius for rolled shapes; and for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used; for tees, the overall depth; and for rectangular HSS, the clear distance between the flanges less the inside corner radius on each side, (mm)	Table 10.20.8.1
h	Distance between horizontal boundary element centerlines, (mm)	10.20.17
h_o	Distance between flange centroids, (mm)	10.20.9
l	Unbraced length between stitches of built-up bracing members, (mm) .	10.20.13
l	Unbraced length of compression or bracing member, (mm)	10.20.13
r	Governing radius of gyration, (mm)	10.20.13
r_y	Radius of gyration about y-axis, (mm)	10.20.9
t	Thickness of connected part, (mm)	10.20.7
t	Thickness of element, (mm)	Table 10.20.8.1
t	Thickness of column web or doubler plate, (mm)	10.20.9
t_{bf}	Thickness of beam flange, (mm)	10.20.9
t_{cf}	Thickness of column flange, (mm)	10.20.9
t_f	Thickness of flange, (mm)	10.20.17
t_p	Thickness of panel zone including doubler plates, (mm)	10.20.9
t_w	Thickness of web, (mm)	Table 10.20.8.1
w_z	Width of panel zone between column flanges, (mm)	10.20.9
x	Parameter used for determining the approximate fundamental period	10.C.2
z_b	Minimum plastic section modulus at the reduced beam section, (mm^3)	10.20.9
ΣM^*_{pc}	Moment at beam and column centerline determined by projecting the sum of the nominal column plastic moment strength, reduced by the axial stress P_{uc}/A_g, from the top and bottom of the beam moment connection	10.20.9
ΣM^*_{pb}	Moment at the intersection of the beam and column centerlines determined by projecting the beam maximum	10.20.9

Symbol	Description	Section
	developed moments from the column face. Maximum developed moments shall be determined from test results	
β	Compression strength adjustment factor	10.20.16
Δ	Design story drift	10.20.15
Δ_b	Deformation quantity used to control loading of test specimen (total brace end rotation for the subassemblage test specimen; total brace axial deformation for the brace test specimen)	10.E.2
Δ_{bm}	Value of deformation quantity, Δ_b, corresponding to the design story drift	10.E.6
Δ_{by}	Value of deformation quantity, Δ_b, at first significant yield of test specimen	10.E.6
Ω	Safety factor	10.20.6
Ω_b	Safety factor for flexure = 1.67	10.20.8
Ω_c	Safety factor for compression = 1.67	10.20.8
Ω_o	Horizontal seismic overstrength factor	10.20.4
Ω_v	Safety factor for shear strength of panel zone of beam-to-column connections	10.20.9
α	Angle of diagonal members with the horizontal	10.20.12
α	Angle of web yielding in radians, as measured relative to the vertical	10.20.17
δ	Deformation quantity used to control loading of test specimen	10.D.6
δ_y	Value of deformation quantity δ at first significant yield of test specimen	10.D.6
ρ'	Ratio of required axial force P_u to required shear strength V_u of a link	10.20.15
λ_p, λ_{ps}	Limiting slenderness parameter for compact element	10.20.8
ϕ	Resistance factor	10.20.6
ϕ_b	Resistance factor for flexure	10.20.8
ϕ_c	Resistance factor for compression	10.20.8
ϕ_v	Resistance factor for shear strength of panel zone of beam-to-column connections	10.20.9
ϕ_v	Resistance factor for shear	10.20.15
θ	Interstory drift angle, radians	10.D.3
γ_{total}	Link rotation angle	10.D.2
ω	Strain hardening adjustment factor	10.20.16

Glossary

Notes:

(1) Terms designated with * are usually qualified by the type of *load effect,* for example, *nominal tensile strength*, *available compressive strength*, *design flexural strength*.

(2) Terms designated with ** are usually qualified by the type of component, for example, web *local buckling*, flange *local bending*.

Adjusted brace strength. Strength of a brace in a *buckling-restrained braced frame* at deformations corresponding to 2.0 times the *design story drift*.

*Allowable strength**. Nominal strength divided by the safety factor, R_n / Ω.

Applicable building code (ABC) . Building code under which the structure is designed.

Amplified seismic load. Horizontal component of earthquake load E multiplied by Ω_o, where E and the horizontal component of E are specified in the *applicable building code*.

Authority having jurisdiction (AHJ). Organization, political subdivision, office or individual charged with the responsibility of administering and enforcing the provisions of this standard.

Available strength.* Design strength or allowable strength, as appropriate.

ASD (Allowable Strength Design). Method of proportioning structural components such that the *allowable strength* equals or exceeds the *required strength* of the component under the action of the ASD load combinations.

ASD load combination. Load combination in the *applicable building code* intended for allowable strength design (allowable stress design*)*.

Buckling-restrained braced frame (BRBF). Diagonally braced frame safisfying the requirements of Section 16 in which all members of the bracing system are subjected primarily to axial forces and in which the limit state of compression buckling of braces is precluded at forces and deformations corresponding to 2.0 times the *design story drift.*

Buckling-restraining system. System of restraints that limits buckling of the steel core in BRBF. This system includes the casing on the steel core and structural elements adjoining its connections. The buckling-restraining system is intended to permit the transverse expansion and longitudinal contraction of the steel core for deformations corresponding to 2.0 times the *design story drift*.

Casing. Element that resists forces transverse to the axis of the brace thereby restraining buckling of the core. The casing requires a means of delivering this force to the remainder of the buckling-restraining system. The casing resists little or no force in the axis of the brace.

Column base. Assemblage of plates, connectors, bolts, and rods at the base of a column used to transmit forces between the steel superstructure and the foundation.

Continuity plates. Column stiffeners at the top and bottom of the *panel zone*; also known as transverse stiffeners.

Contractor. Fabricator or erector, as applicable*.*

Demand critical weld. Weld so designated by these *Provisions.*

Design earthquake. The earthquake represented by the *design response spectrum* as specified in the *applicable building code*.

Design story drift. Amplified story drift (drift under the *design earthquake*, including the effects of inelastic action), determined as specified in the *applicable building code*.

Design strength. Resistance factor* multiplied by the *nominal strength*, ϕR_n.

Diagonal bracing. Inclined structural members carrying primarily axial load that are employed to enable a structural frame to act as a truss to resist lateral loads.

Dual system. Structural system with the following features: (1) an essentially complete space frame that provides support for gravity loads; (2) resistance to lateral load provided by moment frames (SMF, IMF or OMF) that are capable of resisting at least 25 percent of the base shear, and concrete or steel shear walls, or steel braced frames (EBF, SCBF or OCBF); and (3) each system designed to resist the total lateral load in proportion to its relative rigidity.

Ductile limit state. Ductile limit states include member and connection yielding, bearing deformation at bolt holes, as well as buckling of members that conform to the width- thickness limitations of Table I-8-1. Fracture of a member or of a connection, or buckling of a connection element, is not a ductile limit state.

Eccentrically braced frame (EBF). Diagonally braced frame meeting the requirements of Section 15 that has at least one end of each bracing member connected to a beam a short distance from another beam-to-brace connection or a beam-to-column connection.

Exempted column. Column not meeting the requirements of Equation 9-3 for SMF.

Expected yield strength. Yield strength in tension of a member, equal to the expected yield stress multiplied by A_g.

*Expected tensile strength *.* Tensile strength of a member, equal to the specified minimum tensile strength, F_u, multiplied by R_t.

Expected yield stress. Yield stress of the material, equal to the specified minimum yield stress, F_y, multiplied by R_y .

Intermediate moment frame (IMF). Moment frame system that meets the requirements of Section 10.20.10

Interstory drift angle. Interstory displacement divided by story height, radians.

Inverted-V-braced frame. See *V-braced frame*.

k-area. The *k-area* is the region of the web that extends from the tangent point of the web and the flange-web fillet (AISC "*k*" dimension) a distance of 38 mm into the web beyond the "*k*" dimension.

K-braced frame. A bracing configuration in which braces connect to a column at a location with no diaphragm or other out-of-plane support.

Lateral bracing member. Member that is designed to inhibit lateral buckling or lateral- torsional buckling of primary framing members.

Link. In EBF, the segment of a beam that is located between the ends of two diagonal braces or between the end of a diagonal brace and a column. The length of the *link* is defined as the clear distance between the ends of two diagonal braces or between the diagonal brace and the column face.

Link intermediate web stiffeners. Vertical web stiffeners placed within the link in EBF.

Link rotation angle. Inelastic angle between the *link* and the beam outside of the link when the total story drift is equal to the *design story drift.*

Link shear design strength. Lesser of the available shear strength of the *link* developed from the moment or shear strength of the link.

Lowest Anticipated Service Temperature (LAST). The lowest 1-hour average temperature with a 100-year mean recurrence interval.

LRFD (Load and Resistance Factor Design). Method of proportioning structural components such that the *design strength* equals or exceeds the *required strength* of the component under the action of the *LRFD load combinations*.

LRFD Load Combination. Load combination in the *applicable building code* intended for strength design (*load and resistance factor design).*

Measured flexural resistance. Bending moment measured in a beam at the face of the column, for a beam-to-column test specimen tested in accordance with Appendix S.

Nominal load. Magnitude of the *load* specified by the *applicable building code*.

Nominal strength.* Strength of a structure or component (without the *resistance factor* or *safety factor* applied*)* to resist the load effects, as determined in accordance with this *Specification.*

Ordinary concentrically braced frame (OCBF). Diagonally braced frame meeting the requirements of Section 14 in which all members of the bracing system are subjected primarily to axial forces.

Ordinary moment frame (OMF). Moment frame system that meets the requirements of Section 10.20.11.

Overstrength factor, Ω_o. Factor specified by the *applicable building code* in order to determine the amplified seismic load, where required by these *Provisions*.

Prequalified connection. Connection that complies with the requirements of Appendix 10.A

Protected zone. Area of members in which limitations apply to fabrication and attachments.

Prototype. The connection or brace design that is to be used in the building (SMF, IMF, EBF, and BRBF).

Quality assurance plan. Written description of qualifications, procedures, quality inspections, resources, and records to be used to provide assurance that the structure complies with the engineer's quality requirements, specifications and contract documents.

Reduced beam section. Reduction in cross section over a discrete length that promotes a zone of inelasticity in the member.

Required strength.* Forces, stresses, and deformations produced in a structural component, determined by either structural analysis, for the *LRFD* or *ASD load combinations*, as appropriate, or as specified by the *Specification* and these *Provisions.*

Resistance factor, ϕ. Factor that accounts for unavoidable deviations of the *nominal strength*

from the actual strength and for the manner and consequences of failure.

Safety factor, Ω. Factor that accounts for deviations of the actual strength from the nominal strength, deviations of the actual load from the *nominal load*, uncertainties in the analysis that transforms the load into a load effect, and for the manner and consequences of failure.

Seismic design category. Classification assigned to a building by the *applicable building code* based upon its *seismic use group* and the design spectral response acceleration coefficients.

Seismic load resisting system (SLRS). Assembly of structural elements in the building that resists seismic loads, including struts, collectors, chords, diaphragms and trusses.

Seismic response modification coefficient, *R*. Factor that reduces seismic load effects to strength level as specified by the *applicable building code.*

Seismic use group. Classification assigned to a structure based on its use as specified by the *applicable building code.*

Special concentrically braced frame (SCBF). Diagonally braced frame meeting the requirements of Section 10.20.13 in which all members of the bracing system are subjected primarily to axial forces.

Special moment frame (SMF). Moment frame system that meets the requirements of Section 10.20.9.

Special plate shear wall (SPSW). Plate shear wall system that meets the requirements of Section 10.20.17.

Special truss moment frame (STMF). Truss moment frame system that meets the requirements of Section 10.20.12.

Static yield strength. Strength of a structural member or connection determined on the basis of testing conducted under slow monotonic loading until failure.

Steel core. Axial-force-resisting element of braces in BRBF. The steel core contains a yielding segment and connections to transfer its axial force to adjoining elements; it may also contain projections beyond the casing and transition segments between the projections and yielding segment.

Tested connection. Connection that complies with the requirements of Appendix 10.B.

V-braced frame. Concentrically braced frame (SCBF, OCBF or BRBF) in which a pair of diagonal braces located either above or below a beam is connected to a single point within the clear beam span. Where the diagonal braces are below the beam, the system is also referred to as an *inverted-V-braced frame*.

X-braced frame. Concentrically braced frame (OCBF or SCBF) in which a pair of diagonal braces crosses near the mid-length of the braces.

Y-braced frame. Eccentrically braced frame (EBF) in which the stem of the Y is the *link* of the EBF system.

10.20.3 General Seismic Design Requirements

The required *strength* and other seismic provisions and the limitations on height and irregularity are specified in Chapter 2 of Part 6 of this code.

The design *story drift s*hall be in accordance with the requirements set forth in Chapter 2 of Part 6 of this code.

10.20.4 Loads, Load Combinations, and Nominal Strengths

10.20.4.1 **Loads and Load Combinations**

The loads and load combinations shall be as stipulated in Chapter 2 of Part 6 of this code. Where *amplified seismic loads* are required by these *Provisions*, the horizontal portion of the earthquake load *E* (as defined in Chapter 2 of Part 6) shall be multiplied by the overstrength factor, Ω_o, as applicable.

10.20.4.2 **Nominal Strength**

The *nominal strength* of systems, members and connections shall comply with the *Specification*, except as modified throughout these *Provisions.*

10.20.5 Structural Design Drawings and Specifications, Shop Drawings, and Erection Drawings

10.20.5.1 **Structural Design Drawings and Specifications**

Structural design drawings and specifications shall show the work to be performed, and include items required by the *Specification* and the following, as applicable:

(1) Designation of the seismic load resisting system (SLRS)

(2) Designation of the members and connections that are part of the SLRS

(3) Configuration of the connections

(4) Connection material specifications and sizes

(5) Locations of demand critical welds

(6) Lowest anticipated service temperature (LAST) of the steel structure, if the structure is not enclosed and maintained at a temperature of 10 °C or higher.

(7) Locations and dimensions of protected zones

(8) Locations where gusset plates are to be detailed to accommodate inelastic rotation

(9) Welding requirements as specified in Appendix 10.F, Section 10.F.2.1.

Shop Drawings

Shop drawings shall include items required by the *Specification* and the following, as applicable:

(1) Designation of the members and connections that are part of the SLRS

(2) Connection material specifications

(3) Locations of *demand critical* shop welds

(4) Locations and dimensions of *protected zones*

(5) Gusset plates drawn to scale when they are detailed to accommodate inelastic rotation

Welding requirements as specified in Appendix 10.F, Section 10.F.2.2.

10.20.5.2 **Erection Drawings**

Erection drawings shall include items required by the *Specification* and the following, as applicable:

(1) Designation of the members and connections that are part of the SLRS

(2) Field connection material specifications and sizes

(3) Locations of *demand critical* field welds

(4) Locations and dimensions of *protected zones*

(5) Locations of pretensioned bolts

(6) Field welding requirements as specified in Appendix 10.F, Section 10.F.2.3

10.20.6 Materials

10.20.6.1 **Material Specifications**

Structural steel used in the seismic *load resisting system* (SLRS) shall meet the requirements of Section 10.1.3.1a, except as modified in present Section 10.20. The specified minimum yield stress of steel to be used for members in which inelastic behavior is expected shall not exceed 345 MPa for systems defined in Sections 10.20.9, 10.20.10, 10.20.12, 10.20.13, 10.20.15, 10.20.16, and 10.20.17 nor 380 MPa for systems defined in Sections 10.20.11 and 10.20.14, unless the suitability of the material is determined by testing or other rational criteria. This limitation does not apply to columns for which the only expected inelastic behavior is yielding at the *column base*.

The structural steel used in the SLRS described in Sections 10.20.9 through 10.20.17 shall meet one of the following ASTM Specifications: A36/ A36M, A53/A53M, A500 (Grade B or C), A501, A529/A529M, A572/A572M [Grade 290, 345 or 380], A588/A588M, A913/A913M [Grade 345, 415 or 450], A992/A992M, or A1011 HSLAS Grade 380. The structural steel used for column base plates shall meet one of the preceding ASTM specifications or ASTM A283/A283M Grade D.

Other steels and non-steel materials in *buckling-restrained braced frames* are permitted to be used subject to the requirements of Section 10.20.16 and Appendix 10.E.

Material Properties for Determination of Required Strength of Members and Connections

The required strength of an element (a member or a connection) shall be determined from the *expected yield stress*, $R_y F_y$, of an adjoining member, where F_y is the specified minimum yield stress of the grade of steel to be used in the adjoining members and R_y is the ratio of the expected yield stress to the specified minimum yield stress, F_y , of that material.

The available strength of the element, ϕR_n for LRFD and R_n / Ω for ASD, shall be equal to or greater than the required strength, where R_n is the nominal strength of the connection. The expected tensile strength, $R_t F_u$, and the expected yield stress, $R_y F_y$, are permitted to be used in lieu of F_u and F_y , respectively, in determining the nominal strength, R_n, of rupture and yielding limit states within the same member for which the required strength is determined.

The values of R_y and R_t for various steels are given in Table 10.20.1. Other values of R_y and R_t shall be permitted if the values are determined by testing of specimens similar in size and source conducted in accordance with the requirements for the specified grade of steel.

10.20.6.2 **Heavy Section CVN Requirements**

For structural steel in the SLRS, in addition to the requirements of Section 10.1.3.1c, hot rolled shapes with flange thickness 38 mm and thicker shall have a minimum Charpy V-Notch toughness of 27 J at 21°C, tested in the alternate core location as described in ASTM A6 Supplementary Requirement S30. Plates 50 mm thick and thicker shall have a minimum Charpy V-Notch toughness of 27 J at 21 °C, measured at any location permitted by ASTM A673, where the plate is used in the following:

1. Members built-up from plate
2. Connection plates where inelastic strain under seismic loading is expected
3. As the *steel core* of buckling-restrained braces

TABLE 10.20.1: R_y and R_t Values for Different Member Types

Application	R_y	R_t
Hot-rolled structural shapes and bars:		
• ASTM A36/A36M	1.5	1.2
• ASTM A572/572M Grade 42 (290)	1.3	1.1
• ASTM A572/572M Grade 50 (345) or 55 (380), ASTM A913/A913M Grade 50 (345), 60 (415), or 65 (450), ASTM A588/A588M, ASTM A992/A992M, A1011 HSLAS Grade 55 (380)	1.1	1.1
• ASTM A529 Grade 50 (345)	1.2	1.2
• ASTM A529 Grade 55 (380)	1.1	1.2
Hollow structural sections (HSS):		
• ASTM A500 (Grade B or C), ASTM A501	1.4	1.3
Pipe:		
• ASTM A53/A53M	1.6	1.2
Plates:		
• ASTM A36/A36M	1.3	1.2
• ASTM A572/A572M Grade 50 (345), ASTM A588/A588M	1.1	1.2

10.20.7 Connections, Joints and Fasteners

10.20.7.1 Scope

Connections, joints and fasteners that are part of the *seismic load resisting system* (SLRS) shall comply with Section 10.10, and with the additional requirements of this Section.

The design of connections for a member that is a part of the SLRS shall be configured such that a *ductile limit state* in either the connection or the member controls the design.

10.20.7.2 Bolted Joints

All bolts shall be pre-tensioned high strength bolts and shall meet the requirements for *slip-critical* faying surfaces in accordance with Section 10.10.3.8 with a Class A surface. Bolts shall be installed in standard holes or in short-slotted holes perpendicular to the applied load. For brace diagonals, oversized holes shall be permitted when the connection is designed as a slip-critical joint, and the oversized hole is in one ply only. Alternative hole types are permitted if determined in a connection prequalification in accordance with Appendix 10.A, or if determined in a program of qualification testing in accordance with Appendix 10.D or 10.E. The *available shear strength* of bolted joints using standard holes shall be calculated as that for bearing-type joints in accordance with Sections 10.10.3.7 and 10.10.3.10, except that the nominal bearing strength at bolt holes shall not be taken greater than $2.4dtF_u$.

Exception: The faying surfaces for end plate moment connections are permitted to be coated with coatings not tested for slip resistance, or with coatings with a slip coefficient less than that of a Class A faying surface.

Bolts and welds shall not be designed to share force in a joint or the same force component in a connection.

10.20.7.3 **Welded Joints**

Welding shall be performed in accordance with Appendix 10.F. Welding shall be performed in accordance with a welding procedure specification (WPS) as required in AWS D1.1. The WPS variables shall be within the parameters established by the filler metal manufacturer.

General Requirements

All welds used in members and connections in the SLRS shall be made with a filler metal that can produce welds that have a minimum Charpy V-Notch toughness of 27 J at minus 18 °C, as determined by the appropriate AWS A5 classification test method or manufacturer certification. This requirement for notch toughness shall also apply in other cases as required in these *Provisions*.

Demand Critical Welds

Where welds are designated as *demand critical*, they shall be made with a filler metal capable of providing a minimum Charpy V-Notch (CVN) toughness of 27 J at 29 °C as determined by the appropriate AWS classification test method or manufacturer certification, and 54 J at 21 °C as determined by Appendix 10.G or other approved method, when the steel frame is normally enclosed and maintained at a temperature of 10 °C or higher. For structures with service temperatures lower than 10 °C, the qualification temperature for Appendix 10.G shall be 11 °C above the *lowest anticipated service temperature*, or at a lower temperature.

SMAW electrodes classified in AWS A5.1 as E7018 or E7018-X, SMAW electrodes classified in AWS A5.5 as E7018-C3L or E8018-C3, and GMAW solid electrodes are exempted from production lot testing when the CVN toughness of the electrode equals or exceeds 27 J at a temperature not exceeding 29 °C as determined by AWS classification test methods. The manufacturer's certificate of compliance shall be considered sufficient evidence of meeting this requirement.

Protected Zone

Where a *protected zone* is designated by these *Provisions*, it shall comply with the following:

(1) Within the protected zone, discontinuities created by fabrication or erection operations, such as tack welds, erection aids, air-arc gouging and thermal cutting shall be repaired as required by the engineer of record.

(2) Welded shear studs and decking attachments that penetrate the beam flange shall not be placed on beam flanges within the protected zone. Decking arc spot welds as required to secure decking shall be permitted.

(3) Welded, bolted, screwed or shot-in attachments for perimeter edge angles, exterior facades, partitions, duct work, piping or other construction shall not be placed within the protected zone.

Exception: Welded shear studs and other connections shall be permitted when determined in accordance with a connection prequalification in accordance with Appendix 10.A, or as determined in a program of qualification testing in accordance with Appendix 10.D.

Outside the protected zone, calculations based upon the expected moment shall be made to demonstrate the adequacy of the member net section when connectors that penetrate the member are used.

Continuity Plates and Stiffeners

Corners of *continuity plates* and stiffeners placed in the webs of rolled shapes shall be clipped as described below. Along the web, the clip shall be detailed so that the clip extends a distance of at least 38 mm beyond the published k detail dimension for the rolled shape. Along the flange, the clip shall be detailed so that the clip does not exceed a distance of 12 mm beyond the published k_1 detail dimension. The clip shall be detailed to facilitate suitable weld terminations for both the flange weld and the web weld. If a curved clip is used, it shall have a minimum radius of 12 mm.

At the end of the weld adjacent to the column web/flange juncture, weld tabs for continuity plates shall not be used, except when permitted by the engineer of record. Unless specified by the engineer of record that they be removed, weld tabs shall not be removed when used in this location.

10.20.8 Members

10.20.8.1 **Scope**

Members in the seismic load resisting system (SLRS) shall comply with the specifications of Sections 10.1 through 10.11 and Section 10.20.8. For columns that are not part of the SLRS, see Section 10.20.8.4.2.

10.20.8.2 Classification of Sections for Local Buckling

Compact

When required by these Provisions, members of the SLRS shall have flanges continuously connected to the web or webs and the width-thickness ratios of its compression elements shall not exceed the limiting width-thickness ratios, λ_p, from Specification Table B4.1.

Seismically Compact

When required by these Provisions, members of the SLRS must have flanges continuously connected to the web or webs and the width-thickness ratios of its compression elements shall not exceed the limiting width-thickness ratios, λ_{ps}, from Provisions Table 10.2.4.1.

Column Strength

When $P_u/\phi_c P_n$ (LRFD) > 0.4 or $\Omega_c P_a/P_n$ (ASD) > 0.4, as appropriate, without consideration of the amplified seismic load,

where

ϕ_c = 0.90 (LRFD) Ω_c = 1.67 (ASD)

P_a = required axial strength of a column using ASD load combinations, N

P_n = nominal axial strength of a column, N

P_u = required axial strength of a column using LRFD load combinations, N

The following requirements shall be met:

The *required axial compressive* and *tensile strength*, considered in the absence of any applied moment, shall be determined using the load combinations stipulated by the *applicable building code* including the amplified seismic load.

The required axial compressive and tensile strength shall not exceed either of the following:

(a) The maximum load transferred to the column considering $1.1R_y$ (LRFD) or $(1.1/1.5)R_y$ (ASD), as appropriate, times the *nominal strengths* of the connecting beam or brace elements of the building.

(b) The limit as determined from the resistance of the foundation to over-turning uplift.

TABLE 10.20.2: Limiting Width-Thickness Ratios for Compression Elements

	Description of Element	Width- Thickness Ratio	Limiting Width- Thickness Ratios λ_{ps} (seismically compact)
Unstiffened Elements	Flexure in flanges of rolled or built-up I-shaped sections [a], [c], [e], [g], [h]	b/t	$0.30\sqrt{(E/F_y)}$
	Uniform compression in flanges of rolled or built-up I-shaped sections [b], [h]	b/t	$0.30\sqrt{(E/F_y)}$
	Uniform compression in flanges of rolled or built-up I-shaped sections [d]	b/t	$0.38\sqrt{(E/F_y)}$
	Uniform compression in flanges of channels, outstanding legs of pairs of angles in continuous contact, and braces [c], [g]	b/t	$0.30\sqrt{(E/F_y)}$
	Uniform compression in flanges of H-pile sections	b/t	$0.45\sqrt{(E/F_y)}$
	Flat bars [f]	b/t	2.5
	Uniform compression in legs of single angles, legs of double angle members with separators, or flanges of tees [g]	b/t	$0.30\sqrt{(E/F_y)}$
	Uniform compression in stems of tees [g]	d/t	$0.30\sqrt{(E/F_y)}$

Note: See continued Table 10.20.2 for stiffened elements.

TABLE 10.20.2 (cont.) Limiting Width-Thickness Ratios for Compression Elements

	Description of Element	Width-Thickness Ratio	Limiting Width-Thickness Ratios: λ_{ps} (seismically compact)
Stiffened Elements	Webs in flexural compression in beams in SMF, Section 10.20.9, unless noted otherwise	h/t_w	$2.45\sqrt{E/F_y}$
	Webs in flexural compression or combined flexure and axial compression [a], [c], [g], [h], [i], [j]	h/t_w	for $C_a \leq 0.125$ [k] $3.14\sqrt{\frac{E}{F_y}}(1-1.54C_a)$ for $C_a > 0.125$ [k] $1.12\sqrt{\frac{E}{F_y}}(2.33-C_a) \geq 1.49\sqrt{\frac{E}{F_y}}$
	Round HSS in axial and/or flexural compression [c], [g]	D/t	$0.044\ E/F_y$
	Rectangular HSS in axial and/or flexural compression [c], [g]	b/t or h/t_w	$0.64\sqrt{E/F_y}$
	Webs of H-Pile sections	h/t_w	$0.94\sqrt{E/F_y}$

[a] Required for beams in SMF, Section 10.20.9 and SPSW, Section 10.20.17.
[b] Required for columns in SMF, Section 10.20.9, unless the ratios from Equation 10.20.9.3 are greater than 2.0 where it is permitted to use λ_p in Specification Table 10.2.4.1.
[c] Required for braces and columns in SCBF, Section 10.20.13 and braces in OCBF, Section 10.20.14.
[d] It is permitted to use λ_p in Specification Table 10.2.4.1 for columns in STMF, Section 10.20.12 and columns in EBF, Section 10.20.15.
[e] Required for link in EBF, Section 10.20.15, except it is permitted to use λ_p in Table 10.2.4.1 of the Specification for flanges of links of length $1.6M_p/V_p$ or less, where M_p and V_p are defined in Section 10.20.15.
[f] Diagonal web members within the special segment of STMF, Section 10.20.12.
[g] Chord members of STMF, Section 10.20.12.
[h] Required for beams and columns in BRBF, Section 10.20.16.
[i] Required for columns in SPSW, Section 10.20.17.
[j] For columns in STMF, Section 10.20.12; columns in SMF, if the ratios from Equation 10.20.9.3 are greater than 2.0; columns in EBF, Section 10.20.15; or EBF webs of links of length 1.6 M_p/V_p or less, it is permitted to use the following for λ_p:

$$\text{for } C_a \leq 0.125,\ \lambda_p = 3.76\sqrt{\frac{E}{F_y}}\left(1-275C_a\right)$$

$$\text{for } C_a > 0.125,\ \lambda_p = 1.12\sqrt{\frac{E}{F_y}}\left(2.33-C_a\right) \geq 1.49\sqrt{\frac{E}{F_y}}$$

[k] For LFRD, $C_a = \dfrac{P_u}{\phi_b P_y}$

For ASD, $C_a = \dfrac{\Omega_b P_a}{P_y}$

where
P_a = required compressive strength (ASD), N
P_u = *required compressive strength* (LRFD), N
P_y = axial yield strength, N
ϕ_b = 0.90
Ω_b = 1.67

10.20.8.3 **Column Splices**

General

The *required strength* of column splices in the *seismic load resisting system (SLRS)* shall equal the required strength of the columns, including that determined from Sections 10.20.8.3, 10.20.9.9, 10.20.10.9, 10.20.11.9, 10.20.13.5 and 10.20.16.5.2.

In addition, welded column splices that are subject to a calculated net tensile *load effect* determined using the load combinations stipulated by the *applicable building code* including the *amplified seismic load,* shall satisfy both of the following requirements:

(1) The *available strength* of partial-joint-penetration (PJP) groove welded joints, if used, shall be at least equal to 200 percent of the required strength.

(2) The available strength for each flange splice shall be at least equal to 0.5 $R_y F_y A_f$ (LRFD) or $(0.5/1.5)R_y F_y A_f$ (ASD), as appropriate, where $R_y F_y$ is the *expected yield stress* of the column material and A_f is the flange area of the smaller column connected.

Beveled transitions are not required when changes in thickness and width of flanges and webs occur in column splices where PJP groove welded joints are used.

Column web splices shall be either bolted or welded, or welded to one column and bolted to the other. In *moment frames* using bolted splices, plates or channels shall be used on both sides of the column web.

The centerline of column splices made with fillet welds or partial-joint-penetra-tion groove welds shall be located 1.2 m or more away from the beam-to-column connections. When the column clear height between beam-to-column connections is less than 2.4 m, splices shall be at half the clear height.

Columns Not Part of the Seismic Load Resisting System

Splices of columns that are not a part of the SLRS shall satisfy the following:

(1) Splices shall be located 1.2m or more away from the beam-to-column connections. When the column clear height between beam-to-column connections is less than 2.4m, splices shall be at half the clear height.

(2) The *required shear strength* of column splices with respect to both orthogonal axes of the column shall be M_{pc}/H (LRFD) or $M_{pc}/1.5H$ (ASD), as appropriate, where M_{pc} is the lesser nominal plastic flexural strength of the column sections for the direction in question, and H is the story height.

10.20.8.4 **Column Bases**

The *required strength* of *column bases* shall be calculated in accordance with Sections 10.20.8.5.1, 10.20.8.5.2, and 10.20.8.5.3. The *available strength* of anchor rods shall be determined in accordance with *Specification* Section 10.10.3.

The available strength of concrete elements at the column base, including anchor rod embedment and reinforcing steel, shall be in accordance with Appendix D of Chapter 6 of Part 6 of this code.

Exception: The special requirements in Appendix D of Chapter 6 of Part 6 of this code, for "regions of moderate or high seismic risk, or for structures assigned to intermediate or high seismic performance or design categories" need not be applied.

Required Axial Strength

The required axial strength of column bases, including their attachment to the foundation, shall be the summation of the vertical components of the required strengths of the steel elements that are connected to the column base.

Required Shear Strength

The required shear strength of column bases, including their attachments to the foundations, shall be the summation of the horizontal component of the required strengths of the steel elements that are connected to the column base as follows:

(1) For diagonal bracing, the horizontal component shall be determined from the required strength o f bracing connections for the *seismic load resisting system (SLRS).*

(2) For columns, the horizontal component shall be at least equal to the lesser of the following:

(a) $2R_y F_y Z_x/H$ (LRFD) or $(2/1.5) R_y F_y Z_x/H$ (ASD), as appropriate, of the column where

H = height of story, which may be taken as the distance between the centerline of floor framing at each of the levels above and below, or the distance between the top of floor slabs at each of the levels above and below, mm.

(b) The shear calculated using the load combinations of the *applicable building code,* including the *amplified seismic load.*

Required Flexural Strength

The *required flexural strength* of column bases, including their attachment to the foundation, shall be the summation of the required strengths of the steel elements that are connected to the column base as follows:

(1) For diagonal bracing, the required flexural strength shall be at least equal to the required strength of bracing connections for the SLRS.

(2) For columns, the required flexural strength shall be at least equal to the lesser of the following:

(a) $1.1R_y F_y Z$ (LRFD) or $(1.1/1.5)R_y F_y Z$ (ASD), as appropriate, of the column or

(b) the moment calculated using the load combinations of the applicable building code, including the amplified seismic load.

10.20.8.5 **H-Piles**

Design of H-Piles

Design of H-piles shall comply with the provisions of the *Specification* regarding design of members subjected to combined loads. H-piles shall meet the requirements of Section 10.20.8.2.2.

Battered H-Piles

If battered (sloped) and vertical piles are used in a pile group, the vertical piles shall be designed to support the combined effects of the dead and live loads without the participation of the battered piles.

Tension in H-Piles

Tension in each pile shall be transferred to the pile cap by mechanical means such as shear keys, reinforcing bars or studs welded to the embedded portion of the pile. Directly below the bottom of the pile cap, each pile shall be free of attachments and welds for a length at least equal to the depth of the pile cross section.

10.20.9 Special Moment Frames (SMF)

10.20.9.1 **Scope**

Special moment frames (SMF) are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the *design earthquake.* SMF shall satisfy the requirements in this Section.

10.20.9.2 **Beam-to-Column Connections**

Requirements

Beam-to-column connections used in the *seismic load resisting system* (SLRS) shall satisfy the following three requirements:

(1) The connection shall be capable of sustaining an *interstory drift angle* of at least 0.04 radians.

(2) The *measured flexural resistance* of the connection, determined at the column face, shall equal at least $0.80M_p$ of the connected beam at an interstory drift angle of 0.04 radians.

(3) The *required shear strength* of the connection shall be determined using the following quantity for the earthquake load effect E:

$$E = 2[1.1Ry\ Mp]/Lh \qquad (10.20.9.1)$$

where

R_y = ratio of the expected yield stress to the specified minimum yield stress, F_y

M_p = nominal plastic flexural strength, (N-mm)

L_h = distance between plastic hinge locations, (mm)

When E as defined in Equation 10.20.9.1 is used in *ASD load combinations* that are additive with other transient loads and that are based on Chapter 2 (of Part 6), the 0.75 combination factor for transient loads shall not be applied to E.

Connections that accommodate the required interstory drift angle within the connection elements and provide the measured flexural resistance and shear strengths specified above are permitted. In addition to satisfying the requirements noted above, the design shall demonstrate that any additional drift due to connection deformation can be accommodated by the structure. The design shall include analysis for stability effects of the overall frame, including second-order effects.

Conformance Demonstration

Beam-to-column connections used in the SLRS shall satisfy the requirements of Section 10.20.9.2.1 by one of the following:

(a) Use of SMF connections designed in accordance with ANSI/AISC 358.

(b) Use of a connection prequalified for SMF in accordance with Appendix 10.A.

(c) Provision of qualifying cyclic test results in accordance with Appendix 10.D.

Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:

(i) Tests reported in the research literature or documented tests performed for other projects that represent the project conditions, within the limits specified in Appendix 10.D.

(ii) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Appendix 10.D.

Welds

Unless otherwise designated by ANSI/AISC 358, or otherwise determined in a connection prequalification in accordance with Appendix 10.A, or as determined in a program of qualification testing in accordance with Appendix 10.D, complete-joint-penetration groove welds of beam flanges, shear plates, and beam webs to columns shall be *demand critical welds* as described in Section 10.20.7.3.2.

Protected Zones

The region at each end of the beam subject to inelastic straining shall be designated as a *protected zone*, and shall meet the requirements of Section 10.20.7.4. The extent of the protected zone shall be as designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix 10.A, or as determined in a program of qualification testing in accordance with Appendix 10.D.

10.20.9.3 Panel Zone of Beam-to-Column Connections (beam web parallel to column web)

Shear Strength

The required thickness of the panel zone shall be determined in accordance with the method used in proportioning the panel zone of the tested or *prequalified connection*. As a minimum, the *required shear strength* of the panel zone shall be determined from the summation of the moments at the column faces as determined by projecting the expected moments at the plastic hinge points to the column faces. The design shear strength shall be $\phi_v R_v$ and the allowable shear strength shall be R_v/Ω_v where

$\phi_v = 1.0$ (LRFD) $\Omega_v = 1.50$ (ASD)

and the *nominal shear strength*, R_v, according to the limit state of shear yielding, is determined as specified in *Specification* Section 10.10.10.6.

Panel Zone Thickness

The individual thicknesses, t, of column webs and doubler plates, if used, shall conform to the following requirement:

$$t \geq (d_z + w_z)/90 \qquad (10.20.9.2)$$

where

t = thickness of column web or doubler plate, mm

d_z = panel zone depth between *continuity plates*, mm

w_z = panel zone width between column flanges, mm

Alternatively, when local buckling of the column web and doubler plate is prevented by using plug welds joining them, the total panel zone thickness shall satisfy Equation 10.20.9.2.

Panel Zone Doubler Plates

Doubler plates shall be welded to the column flanges using either a complete-joint-penetration groove-welded or fillet-welded joint that develops the available shear strength of the full doubler plate thickness. When doubler plates are placed against the column web, they shall be welded across the top and bottom edges to develop the proportion of the total force that is transmitted to the doubler plate. When doubler plates are placed away from the column web, they shall be placed symmetrically in pairs and welded to continuity plates to develop the proportion of the total force that is transmitted to the doubler plate.

Beam and Column Limitations

The requirements of Section 10.20.8.1 shall be satisfied, in addition to the following.

Width-Thickness Limitations

Beam and column members shall meet the requirements of Section 10.20.8.2.2, unless otherwise qualified by tests.

Beam Flanges

Abrupt changes in beam flange area are not permitted in plastic hinge regions. The drilling of flange holes or trimming of beam flange width is permitted if testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges. The configuration shall be consistent with a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix 10A, or in a program of qualification testing in accordance with Appendix 10.D.

Continuity Plates

Continuity plates shall be consistent with the prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix 10A, or as determined in a program of qualification testing in accordance with Appendix 10.D.

Column-Beam Moment Ratio

The following relationship shall be satisfied at beam-to-column connections:

$$\frac{\sum M^*_{pc}}{\sum M^*_{pb}} > 1.0 \qquad (10.20.9.3)$$

$\sum M^*_{pc}$ = the sum of the moments in the column above and below the joint at the intersection of the beam and column centerlines. $\sum M^*_{pc}$ is deter-mined by summing the projections of the nominal flexural strengths of the columns (including haunches where used) above and below the joint to the beam centerline with a reduction for the axial force in the column. It is permitted to take $\sum M^*_{pc} = \Sigma Z_c(F_{yc} - P_{uc} / A_g)$ (LRFD) or $\Sigma Z_c[(F_{yc}/1.5) - P_{ac} / A_g]$ (ASD), as appropriate. When the centerlines of opposing beams in the same joint do not coincide, the mid-line between centerlines shall be used.

$\sum M^*_{pb}$ = the sum of the moments in the beams at the intersection of the beam and column centerlines. $\sum M^*_{pb}$ is determined by summing the projections of the expected flexural strengths of the beams at the plastic hinge locations to the column centerline. It is permitted to take $\sum M^*_{pb} = \Sigma(1.1R_yF_{yb}Z_b + M_{uv})$ (LRFD) or $\Sigma[(1.1/1.5)R_yF_{yb}Z_b + M_{av}]$ (ASD), as appropriate. Alternatively, it is permitted to determine $\sum M^*_{pb}$ consistent with a prequalified connection design as designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix 10.A, or in a program of qualification testing in accordance with Appendix 10.D. When connections with reduced beam sections are used, it is permitted to take $\sum M^*_{pb} = \Sigma(1.1R_yF_{yb}Z_{RBS} + M_{uv})$ (LRFD) or $\Sigma[(1.1/1.5)R_yF_{yb}Z_{RBS} + M_{av}]$ (ASD), as appropriate.

A_g = gross area of column, mm

F_{yc} = specified minimum yield stress of column, MPa

M_{av} = the additional moment due to shear amplification from the location of the plastic hinge to the column centerline, based on ASD load combinations, N-mm.

M_{uv} = the additional moment due to shear amplification from the location of the plastic hinge to the column centerline, based on LRFD load combinations, N-mm

P_{ac} = required compressive strength using ASD load combinations, (positive number) N.

P_{uc} = required compressive strength using LRFD load combinations, (positive number) N

Z_b = plastic section modulus of the beam, mm^3

Z_c = plastic section modulus of the column, mm^3

Z_{RBS} = minimum plastic section modulus at the reduced beam section, mm^3

Exception: This requirement does not apply if either of the following two conditions is satisfied:

(a) Columns with $P_{rc} < 0.3P_c$ for all load combinations other than those determined using the *amplified seismic load* that satisfy either of the following:

(i) Columns used in a one-story building or the top story of a multistory building.

(ii) Columns where: (1) the sum of the *available shear strengths* of all *exempted columns* in the story is less than 20 percent of the sum of the available shear strengths of all moment frame columns in the story acting in the same direction; and (2) the sum of the available shear strengths of all exempted columns on each moment frame column line within that story is less than 33 percent of the available shear strength of all moment frame columns on that column line. For the purpose of this exception, a column line is defined as a single line of columns or parallel lines of columns located within 10 percent of the plan dimension perpendicular to the line of columns.

where

For design according to *Specification* Section 10.2.3.3 (LRFD),

P_c = $F_{yc} A_g$, N

P_{rc} = P_{uc}, required compressive strength, using LRFD load combinations, N

For design according to *Specification* Section 10.2.3.4 (ASD),

P_c = $F_{yc} A_g /1.5$, N

P_{rc} = P_{ac}, required compressive strength, using ASD load combinations, N

(b) Columns in any story that has a ratio of available shear strength to *required shear strength* that is 50 percent greater than the story above.

10.20.9.4 **Lateral Bracing at Beam-to-Column Connections**

Braced Connections

Column flanges at beam-to-column connections require lateral bracing only at the level of the top flanges of the beams, when the webs of the beams and column are co-planar, and a column is shown to remain elastic outside of the panel zone. It shall be permitted to assume that the column remains elastic when the ratio calculated using Equation 10.20.9.3 is greater than 2.0.

When a column cannot be shown to remain elastic outside of the panel zone, the following requirements shall apply:

The column flanges shall be laterally braced at the levels of both the top and bottom beam flanges. Lateral bracing shall be either direct or indirect.

Each column-flange lateral brace shall be designed for a *required strength* that is equal to 2 percent of the available beam flange strength $F_y b_f t_{bf}$ (LRFD) or $F_y b_f t_{bf} /1.5$ (ASD), as appropriate.

Unbraced Connections

A column containing a beam-to-column connection with no lateral bracing transverse to the seismic frame at the connection shall be designed using the distance between adjacent lateral braces as the column height for buckling transverse to the seismic frame and shall conform to *Specification* Section 10.8, except that:

(1) The required column strength shall be determined from the appropriate load combinations, except that E shall be taken as the lesser of:

(a) The amplified seismic load.

(b) 125 percent of the frame *available strength* based upon either the beam available flexural strength or panel zone available shear strength.

(2) The slenderness L/r for the column shall not exceed 60.

The column *required flexural strength* transverse to the seismic frame shall include that moment caused by the application of the beam flange force specified in Section 10.20.9.7.1.(2) in addition to the second-order moment due to the resulting column flange displacement.

Lateral Bracing of Beams

Both flanges of beams shall be laterally braced, with a maximum spacing of L_b = $0.086 r_y E/F_y$. Braces shall meet the provisions of Equations 10.19.3.3 and 10.19.3.4 of Section 10.19, where $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_a = R_y Z F_y /1.5$ (ASD), as appropriate, of the beam and $C_d = 1.0$.

In addition, lateral braces shall be placed near concentrated forces, changes in cross-section, and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the SMF. The placement of lateral bracing shall be consistent with that documented for a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix 10.A, or in a program of qualification testing in accordance with Appendix 10.D.

The *required strength* of lateral bracing provided adjacent to plastic hinges shall be $P_u = 0.06 M_u /h_o$ (LRFD) or $P_a = 0.06 M_a /h_o$ (ASD), as appropriate, where h_o is the distance between flange centroids; and the required stiffness shall meet the provisions of Equation 10.19.3.4 of Section 10.19.

Column Splices

Column splices shall comply with the requirements of Section 10.20.8.4.1. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds that meet the requirements of Section 10.20.7.3.2. Weld tabs shall be removed. When column splices are not made with groove welds, they shall have a *required flexural strength* that is at least equal to $R_y F_y Z_x$ (LRFD) or $R_y F_y Z_x /1.5$ (ASD), as appropriate, of the smaller column. The required shear strength of column web splices shall be at least equal to ΣM_{pc} /H (LRFD) or ΣM_{pc} /1.5H (ASD), as appropriate, where ΣM_{pc} is the sum of the nominal plastic flexural strengths of the columns above and below the splice.

Exception: The *required strength* of the column splice considering appropriate stress concentration factors or fracture mechanics stress intensity factors need not exceed that determined by inelastic analyses.

10.20.10 Intermediate Moment Frames (IMF)

10.20.10.1 **Scope**

Intermediate moment frames (IMF) are expected to withstand limited inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the *design earthquake*. IMF shall meet the requirements in this Section.

10.20.10.2 **Beam-to-Column Connections**

Requirements

Beam-to-column connections used in the *seismic load resisting system (SLRS)* shall satisfy the requirements of Section 10.20.9.2.1, with the following exceptions:

(1) The required *interstory drift angle* shall be a minimum of 0.02 radian.

(2) The *required strength* in shear shall be determined as specified in Section 10.20.9.2.1, except that a lesser value of V_u or V_a, as appropriate, is permitted if justified by analysis. The *required shear strength* need not exceed the shear resulting from the application of appropriate *load combinations* using the *amplified seismic load.*

Conformance Demonstration

Conformance demonstration shall be as described in Section 10.20.9.2.2 to satisfy the requirements of Section 10.20.10.2.1 for IMF, except that a connection prequalified for IMF in accordance with ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix 10.A, or as determined in a program of qualification testing in accordance with Appendix 10.D.

Welds

Unless otherwise designated by ANSI/AISC 358, or otherwise determined in a connection prequalification in accordance with Appendix 10.A, or as determined in a program of qualification testing in accordance with

Appendix 10.D, complete joint penetration groove welds of beam flanges, shear plates, and beam webs to columns shall be *demand critical welds* as described in Section 10.20.7.3.2.

Protected Zone

The region at each end of the beam subject to inelastic straining shall be treated as a *protected zone*, and shall meet the requirements of Section 10.20.7.4. The extent of the protected zone shall be as designated in ANSI/AISC 358, or as other wise determined in a connection prequalification in accordance with Appendix 10.A, or as determined in a program of qualification testing in accordance with Appendix 10.D.

Panel Zone of Beam-to-Column Connections (beam web parallel to column web)

No additional requirements beyond the *Specification*.

Beam and Column Limitations

The requirements of Section 10.20.8.1 shall be satisfied, in addition to the following.

Width-Thickness Limitations

Beam and column members shall meet the requirements of Section 10.20.8.2.1, unless otherwise qualified by tests.

Beam Flanges

Abrupt changes in beam flange area are not permitted in plastic hinge regions. Drilling of flange holes or trimming of beam flange width is permitted if testing or qualification demonstrates that the resulting configuration can develop stable plastic hinges. The configuration shall be consistent with a prequalified connection designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix 10.A, or in a program of qualification testing in accordance with Appendix 10.D.

Continuity Plates

Continuity plates shall be provided to be consistent with the prequalified connections designated in ANSI/AISC 358, or as otherwise determined in a connection prequalification in accordance with Appendix 10.A, or as determined in a program of qualification testing in accordance with Appendix 10.D.

Column-Beam Moment Ratio

No additional requirements beyond the *Specification*.

Lateral Bracing at Beam-to-Column Connections

No additional requirements beyond the *Specification*.

Lateral Bracing of Beams

Both flanges shall be laterally braced directly or indirectly. The unbraced length between lateral braces shall not exceed $0.17 r_y E/F_y$. Braces shall meet the provisions of Equations 10.19.3.3 and 10.19.3.4 of Section 10.19, where $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_a = R_y Z F_y /1.5$ (ASD), as appropriate, of the beam, and $C_d = 1.0$.

In addition, lateral braces shall be placed near concentrated loads, changes in cross-section and other locations where analysis indicates that a plastic hinge will form during inelastic deformations of the IMF. Where the design is based upon assemblies tested in accordance with Appendix 10.D, the placement of lateral bracing for the beams shall be consistent with that used in the tests or as required for prequalification in Appendix 10.A. The *required strength* of lateral bracing provided adjacent to plastic hinges shall be $P_u = 0.06 M_u /h_o$ (LRFD) or $P_a = 0.06 M_a / h_o$ (ASD), as appropriate, where h_o = distance between flange centroids; and the required stiffness shall meet the provisions of Equation 10.19.3.4 of Section 10.19.

Column Splices

Column splices shall comply with the requirements of Section 10.20.8.4.1. Where groove welds are used to make the splice, they shall be complete-joint-penetration groove welds that meet the requirements of Section 10.20.7.3.2.

10.20.11 Ordinary Moment Frames (OMF)

10.20.11.1 **Scope**

Ordinary moment frames (OMF) are expected to withstand minimal inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the *design earthquake*. OMF shall meet the requirements of this Section. Connections in conformance with Sections 10.20.9.2.1 and

10.20.9.5 or Sections 10.20.10.2.1 and 10.20.10.5 shall be permitted for use in OMF without meeting the requirements of Sections 10.20.11.2.1, 10.20.11.2.3, and 10.20.11.5

10.20.11.2 **Beam-to-Column Connections**

Beam-to-column connections shall be made with welds and/or high-strength bolts. Connections are permitted to be fully restrained (FR) or partially restrained (PR) moment connections as follows.

Requirements for FR Moment Connections

FR moment connections that are part of the *seismic load resisting system (SLRS)* shall be designed for a *required flexural strength* that is equal to 1.1$R_y M_p$ (LRFD) or (1.1/1.5)$R_y M_p$ (ASD), as appropriate, of the beam or girder, or the maximum moment that can be developed by the system, whichever is less.

FR connections shall meet the following requirements.

(1) Where steel backing is used in connections with complete-joint-penetration (CJP) beam flange groove welds, steel backing and tabs shall be removed, except that top-flange backing attached to the column by a continuous fillet weld on the edge below the CJP groove weld need not be removed. Removal of steel backing and tabs shall be as follows:

(i) Following the removal of backing, the root pass shall be backgouged to sound weld metal and backwelded with a reinforcing fillet. The reinforcing fillet shall have a minimum leg size of 8 mm.

(ii) Weld tab removal shall extend to within 3 mm of the base metal surface, except at *continuity plates* where removal to within 6 mm of the plate edge is acceptable. Edges of the weld tab shall be finished to a surface roughness value of 13 µm or better. Grinding to a flush condition is not required. Gouges and notches are not permitted. The transitional slope of any area where gouges and notches have been removed shall not exceed 1:5. Material removed by grinding that extends more than 2 mm below the surface of the base metal shall be filled with weld metal. The contour of the weld at the ends shall provide a smooth transition, free of notches and sharp corners.

(2) Where weld access holes are provided, they shall be as shown in Figure 10.20.11.1. The weld access hole shall have a surface roughness value not to exceed 13 µm, and shall be free of notches and gouges. Notches and gouges shall be repaired as required by the engineer of record. Weld access holes are prohibited in the beam web adjacent to the end-plate in bolted moment end-plate connections.

(3) The *required strength* of double-sided partial-joint-penetration groove welds and double-sided fillet welds that resist tensile forces in connections shall be 1.1$R_y F_y A_g$ (LRFD) or (1.1/1.5)$R_y F_y A_g$ (ASD), as appropriate, of the connected element or part. Single-sided partial-joint-penetration groove welds and single-sided fillet welds shall not be used to resist tensile forces in the connections.

(4) For FR moment connections, *the required shear strength,* V_u or V_a, as appropriate, of the connection shall be determined using the following quantity for the earthquake load effect *E:*

$$E = 2[1.1 R_y M_p]/L_h \qquad (10.20.11.1)$$

Where this *E* is used in *ASD load combinations* that are additive with other transient loads and that are based on Chapter 2 (of Part 6), the 0.75 combination factor for transient loads shall not be applied to *E*.

Alternatively, a lesser value of V_u or V_a is permitted if justified by analysis. The required shear strength need not exceed the shear resulting from the application of appropriate load combinations in the *applicable building code* using the *amplified seismic load*

Requirements for PR Moment Connections

PR moment connections are permitted when the following requirements are met:

(1) Such connections shall be designed for the *required strength* as specified in Section 10.20.11.2.1 above.

(2) The *nominal flexural strength* of the connection, M_n, shall be no less than 50 percent of M_p of the connected beam or column, whichever is less.

(3) The stiffness and strength of the PR moment connections shall be considered in the design, including the effect on overall frame stability.

(4) For PR moment connections, V_u or V_a, as appropriate, shall be determined from the load combination above plus the shear resulting from the maximum end moment that the connection is capable of resisting.

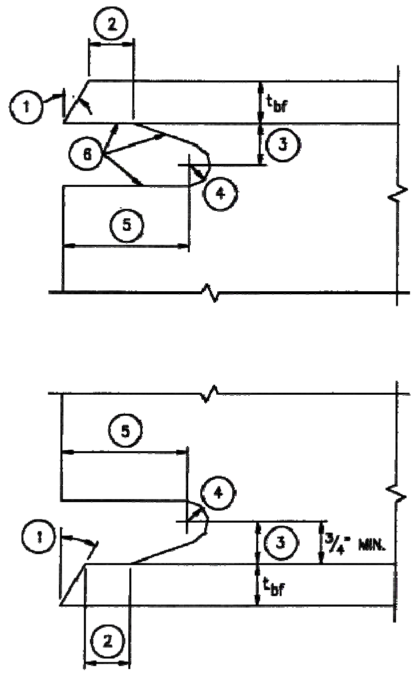


Fig. 10.20.11.1. Weld access hole detail (FEMA 350)

Notes: 1. Bevel as required for selected groove weld.

2. Larger of t_{bf} or 13 mm (plus ½ t_{bf}, or minus ¼t_{bf})

3. ¾ t_{bf} to t_{bf}, 19 mm minimum (± 6 mm)

4. 10 mm minimum radius (plus not limited, minus 0)

5. 3 t_{bf} (±13 mm)

Tolerances shall not accumulate to the extent that the angle of the access hole cut to the flange surface exceeds 25°.

Welds

Complete-joint-penetration groove welds of beam flanges, shear plates, and beam webs to columns shall be *demand critical welds* as described in Section 10.20.7.3.2.

Panel Zone of Beam-to-Column Connections (beam web parallel to column web)

No additional requirements beyond the *Specification*.

Beam and Column Limitations

No requirements beyond Section 10.20.8.1.

Continuity Plates

When FR moment connections are made by means of welds of beam flanges or beam-flange connection plates directly to column flanges, *continuity plates* shall be provided in accordance with Section J10 of the *Specification.* Continuity plates shall also be required when:

$t_{cf} < 0.54\ (b_f\, t_{bf}\, F_{yb}/F_{yc})^{1/2}$

or when,

$t_{cf} < b_f/6$

Where continuity plates are required, the thickness of the plates shall be determined as follows:

For one-sided connections, continuity plate thickness shall be at least one half of the thickness of the beam flange.

For two-sided connections the continuity plates shall be at least equal in thickness to the thicker of the beam flanges.

The welded joints of the continuity plates to the column flanges shall be made with either complete-joint-penetration groove welds, two-sided partial-joint-penetration groove welds combined with reinforcing fillet welds, or two-sided fillet welds. The *required strength* of these joints shall not be less than the *available* *strength* of the contact area of the plate with the column flange. The required strength of the welded joints of the continuity plates to the column web shall be the least of the following:

The sum of the available strengths at the connections of the continuity plate to the column flanges.

The *available shear strength* of the contact area of the plate with the column web.

The weld available strength that develops the available shear strength of the column panel zone.

The actual force transmitted by the stiffener.

Column-Beam Moment Ratio

No requirements.

Lateral Bracing at Beam-to-Column Connections

No additional requirements beyond the *Specification*.

Lateral Bracing of Beams

No additional requirements beyond the *Specification*.

Column Splices

Column splices shall comply with the requirements of Section 10.20.8.4.1.

10.20.12 Special Truss Moment Frames (STMF)

10.20.12.1 **Scope**

Special truss moment frames (STMF) are expected to withstand significant inelastic deformation within a specially designed segment of the truss when subjected to the forces from the motions of the *design earthquake.* STMF shall be limited to span lengths between columns not to exceed 20 m and overall depth not to exceed 1.8 m. The columns and truss segments outside of the special segments shall be designed to remain elastic under the forces that can be generated by the fully yielded and strain-hardened special segment. STMF shall meet the requirements in this Section.

10.20.12.2 **Special Segment**

Each horizontal truss that is part of the *seismic load resisting system (SLRS)* shall have a *special segment* that is located between the quarter points of the span of the truss. The length of the special segment shall be between 0.1 and 0.5 times the truss span length. The length-to-depth ratio of any panel in the special segment shall neither exceed 1.5 nor be less than 0.67.

Panels within a special segment shall either be all Vierendeel panels or all X-braced panels; neither a combination thereof nor the use of other truss diagonal configurations is permitted. Where diagonal members are used in the special segment, they shall be arranged in an X pattern separated by vertical members. Such diagonal members shall be interconnected at points where they cross. The interconnection shall have a *required strength* equal to 0.25 times the *nominal tensile strength* of the diagonal member. Bolted connections shall not be used for web members within the special segment. Diagonal web members within the special segment shall be made of flat bars of identical sections.

Splicing of chord members is not permitted within the special segment, nor within one-half the panel length from the ends of the special segment. The *required axial strength* of the diagonal web members in the special segment due to dead and live loads within the special segment shall not exceed $0.03F_yA_g$ (LRFD) or $(0.03/1.5)F_yA_g$ (ASD), as appropriate.

The special segment shall be a *protected zone* meeting the requirements of Section 10.20.7.4.

Strength of Special Segment Members

The *available shear strength* of the *special segment* shall be calculated as the sum of the available shear strength of the chord members through flexure, and the shear strength corresponding to the *available tensile strength* and 0.3 times the *available compressive strength* of the diagonal members, when they are used. The top and bottom chord members in the special segment shall be made of identical sections and shall provide at least 25 percent of the *required vertical shear strength*. The *required axial strength* in the chord members, determined according to the limit state of tensile yielding, shall not exceed 0.45 times ϕP_n (LRFD) or P_n/Ω (ASD), as appropriate,

ϕ = 0.90 (LRFD) Ω = 1.67 (ASD)

where,

$P_n = F_y A_g$

The end connection of diagonal web members in the special segment shall have a *required strength* that is at least equal to the *expected yield strength*, in tension, of the web member, $R_y F_y A_g$ (LRFD) or $R_y F_y A_g/1.5$ (ASD), as appropriate.

Strength of Non-Special Segment Members

Members and connections of STMF, except those in the *special segment* specified in Section 10.20.12.2, shall have a *required strength* based on the appropriate load combinations in the *applicable building code*, replacing the earthquake load term E with the lateral loads necessary to develop the *expected vertical shear strength* of the special segment V_{ne} (LRFD) or $V_{ne}/1.5$ (ASD), as appropriate, at mid-length, given as:

$$V_{ne} = \frac{3.75 R_y M_{nc}}{L_s} + 0.075 EI \frac{L - L_s}{L_s^3} + R_y (P_{nt} + 0.3 P_{nc}) \sin\alpha \tag{10.20.12.1}$$

where

M_{nc} = *nominal flexural strength* of a chord member of the special segment, N-mm

EI = flexural elastic stiffness of a chord member of the special segment, N-mm^2

L = span length of the truss, mm

L_s = length of the special segment, mm

P_{nt} = *nominal tensile strength* of a diagonal member of the special segment, N

P_{nc} = *nominal compressive strength* of a diagonal member of the special segment, N

α = angle of diagonal members with the horizontal

Width-Thickness Limitations

Chord members and diagonal web members within the special segment shall meet the requirements of Section 10.20.8.2.2.

Lateral Bracing

The top and bottom chords of the trusses shall be laterally braced at the ends of the *special segment*, and at intervals not to exceed L_p according to *Specification* Section 10.6 along the entire length of the truss. The *required strength* of each lateral brace at the ends of and within the special segment shall be

$P_u = 0.06 R_y P_{nc}$ (LRFD) or

$P_a = (0.06/1.5) R_y P_{nc}$ (ASD), as appropriate,

where P_{nc} is the *nominal compressive strength* of the special segment chord member. Lateral braces outside of the special segment shall have a required strength of

$P_u = 0.02 R_y P_{nc}$ (LRFD) or

$P_a = (0.02/1.5) R_y P_{nc}$ (ASD), as appropriate.

The required brace stiffness shall meet the provisions of Equation 10.19.2.4 of Section 10.19, where

$P_r = P_u = R_y P_{nc}$ (LRFD) or

$P_r = P_a = R_y P_{nc}/1.5$ (ASD), as appropriate.

10.20.13 Special Concentrically Braced Frames (SCBF)

10.20.13.1 Scope

Special concentrically braced frames (SCBF) are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the *design earthquake.* SCBF shall meet the requirements in this Section.

10.20.13.2 Members Slenderness

Bracing members shall have $Kl/r \le 4\sqrt{E/F_y}$.

Exception: Braces with $4\sqrt{E/F_y} < Kl/r \le 200$ are permitted in frames in which the available strength of the column is at least equal to the maximum load transferred to the column considering R_y (LRFD) or $(1/1.5)R_y$ (ASD), as appropriate, times the nominal strengths of the connecting brace elements of the building. Column forces need not exceed those determined by inelastic analysis, nor the maximum load effects that can be developed by the system.

Required Strength

Where the effective net area of bracing members is less than the gross area, the *required tensile strength* of the brace based upon the limit state of fracture in the net section shall be greater than the lesser of the following:

(a) The *expected yield strength*, in tension, of the bracing member, determined as $R_y F_y A_g$ (LRFD) or $R_y F_y A_g /1.5$ (ASD), as appropriate.

(b) The maximum load effect, indicated by analysis that can be transferred to the brace by the system.

Lateral Force Distribution

Along any line of bracing, braces shall be deployed in alternate directions such that, for either direction of force parallel to the bracing, at least 30 percent but no more than 70 percent of the total horizontal force along that line is resisted by braces in tension, unless the available strength of each brace in compression is larger than the *required strength* resulting from the application of the appropriate load combinations stipulated by the *applicable building code* including the *amplified seismic load*. For the purposes of this provision, a line of bracing is defined as a single line or parallel lines with a plan offset of 10 percent or less of the building dimension perpendicular to the line of bracing.

Width-Thickness Limitations

Column and brace members shall meet the requirements of Section 10.20.8.2.2.

Built-up Members

The spacing of stitches shall be such that the slenderness ratio *l*/*r* of individual elements between the stitches does not exceed 0.4 times the governing slenderness ratio of the built-up member.

The sum of the *available shear strengths* of the stitches shall equal or exceed the available tensile strength of each element. The spacing of stitches shall be uniform. Not less than two stitches shall be used in a built-up member. Bolted stitches shall not be located within the middle one-fourth of the clear brace length.

Exception: Where the buckling of braces about their critical bucking axis does not cause shear in the stitches, the spacing of the stitches shall be such that the slenderness ratio *l*/*r* of the individual elements between the stitches does not exceed 0.75 times the governing slenderness ratio of the built-up member.

Required Strength of Bracing Connections

Required Tensile Strength

The *required tensile strength* of bracing connections (including beam-to-column connections if part of the bracing system) shall be the lesser of the following:

(a) The *expected yield strength*, in tension, of the bracing member, determined as $R_y F_y A_g$ (LRFD) or $R_y F_y A_g /1.5$ (ASD), as appropriate.

(b) The maximum *load effect*, indicated by analysis that can be transferred to the brace by the system.

Required Flexural Strength

The *required flexural strength* of bracing connections shall be equal to $1.1R_y M_p$ (LRFD) or $(1.1/1.5)R_y M_p$ (ASD), as appropriate, of the brace about the critical buckling axis.

Exception: Brace connections that meet the requirements of Section 10.20.13.3.1 and can accommodate the inelastic rotations associated with brace post-buckling deformations need not meet this requirement.

Required Compressive Strength

Bracing connections shall be designed for a *required compressive strength* based on buckling limit states that is at least equal to $1.1R_yP_n$ (LRFD) or $(1.1/1.5)R_yP_n$ (ASD), as appropriate, where P_n is the *nominal compressive strength* of the brace.

Special Bracing Configuration Requirements

V-Type and Inverted-V-Type Bracing

V-type and inverted V-type SCBF shall meet the following requirements:

(1) The *required strength* of beams intersected by braces, their connections, and supporting members shall be determined based on the load combinations of the *applicable building code* assuming that the braces provide no support for dead and live loads. For load combinations that include earthquake effects, the earthquake effect, *E*, on the beam shall be determined as follows:

(a) The forces in all braces in tension shall be assumed to be equal to $R_y F_y A_g$.

(b) The forces in all adjoining braces in compression shall be assumed to be equal to $0.3P_n$.

(2) Beams shall be continuous between columns. Both flanges of beams shall be laterally braced, with a maximum spacing of $L_b = L_{pd}$, as specified by Equation 10.15.7.2 and 10.15.7.3 of Section 10.15. Lateral braces shall meet the provisions of Equations 10.19.3.3 and 10.19.3.4 of Section 10.19, where $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_a = R_y Z F_y /1.5$ (ASD), as appropriate, of the beam and $C_d = 1.0$.

As a minimum, one set of lateral braces is required at the point of intersection of the V-type (or inverted V-type) bracing, unless the beam has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

K-Type Bracing

K-type braced frames are not permitted for SCBF.

Column Splices

In addition to meeting the requirements in Section 10.20.8.4, column splices in SCBF shall be designed to develop 50 percent of the lesser available flexural strength of the connected members. The required shear strength shall be $\Sigma M_{pc} /H$ (LRFD) or $\Sigma M_{pc} /1.5H$ (ASD), as appropriate, where ΣM_{pc} is the sum of the nominal plastic flexural strengths of the columns above and below the splice.

Protected Zone

The *protected zone* of bracing members in SCBF shall include the center one-quarter of the brace length, and a zone adjacent to each connection equal to the brace depth in the plane of buckling. The protected zone of SCBF shall include elements that connect braces to beams and columns and shall satisfy the requirements of Section 10.20.7.4.

10.20.14 Ordinary Concentrically Braced Frames (OCBF)

10.20.14.1 Scope

Ordinary concentrically braced frames (OCBF) are expected to withstand limited inelastic deformations in their members and connections when subjected to the forces resulting from the motions of the *design earthquake*. OCBF shall meet the requirements in this Section. OCBF above the isolation system in *seismically isolated structures* shall meet the requirements of Sections 10.20.14.4 and 10.20.14.5 and need not meet the requirements of Sections 10.20.14.2 and 10.20.14.3.

10.20.14.2 Bracing Members

Bracing members shall meet the requirements of Section 10.20.8.2.2.

Exception: HSS braces that are filled with concrete need not comply with this provision.

Bracing members in K, V, or inverted-V configurations shall have $KL/r \le 4\sqrt{(E/F_y)}$.

Special Bracing Configuration Requirements

Beams in V-type and inverted V-type OCBF and columns in K-type OCBF shall be continuous at bracing connections away from the beam-column connection and shall meet the following requirements:

(1) The *required strength* shall be determined based on the load combinations of the *applicable building code* assuming that the braces provide no support of dead and live loads. For load combinations that include earthquake effects, the earthquake effect, E, on the member shall be determined as follows:

(a) The forces in braces in tension shall be assumed to be equal to $R_y F_y A_g$. For V-type and inverted V-type OCBF, the forces in braces in tension need not exceed the maximum force that can be developed by the system.

(b) The forces in braces in compression shall be assumed to be equal to $0.3P_n$.

(2) Both flanges shall be laterally braced, with a maximum spacing of $L_b = L_{pd}$, as specified by Equations 10.15.7.2 and 10.15.7.3 of Section 10.15.. Lateral braces shall meet the provisions of Equations 10.19.3.3 and 10.19.3.4 of Section 10.19, where $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_a = R_y Z F_y /1.5$ (ASD), as appropriate, of the beam and $C_d = 1.0$. As a minimum, one set of lateral braces is required at the point of intersection of the bracing, unless the member has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

Bracing Connections

The *required strength* of bracing connections shall be determined as follows.

(1) For the limit state of bolt slip, the required strength of bracing connections shall be that determined using the load combinations stipulated by the *applicable building code*, not including the *amplified seismic load.*

(2) For other limit states, the required strength of bracing connections is the *expected yield strength,* in tension, of the brace, determined as $R_y F_y A_g$ (LRFD) or $R_y F_y A_g/1.5$ (ASD), as appropriate.

Exception: The required strength of the brace connection need not exceed either of the following:

(a) The maximum force that can be developed by the system

(b) A load effect based upon using the amplified seismic load

OCBF above Seismic Isolation Systems

Bracing Members

Bracing members shall meet the requirements of Section 10.20.8.2.2 and shall have

$KL/r \leq 4\sqrt{(E/F_y)}$.

K-Type Bracing

K-type braced frames are not permitted.

V-Type and Inverted-V-Type Bracing

Beams in V-type and inverted V-type bracing shall be continuous between columns.

10.20.15 Eccentrically Braced Frames (EBF)

10.20.15.1 **Scope**

Eccentrically braced frames (EBFs) are expected to withstand significant inelastic deformations in the *links* when subjected to the forces resulting from the motions of the *design earthquake*. The diagonal braces, columns, and beam segments outside of the links shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded and strain-hardened links, except where permitted in this Section. In buildings exceeding five stories in height, the upper story of an EBF system is permitted to be designed as an OCBF or a SCBF and still be considered to be part of an EBF system for the purposes of determining system factors in the *applicable building code*. EBF shall meet the requirements in this Section.

10.20.15.2 **Links**

Limitations

Links shall meet the requirements of Section 10.20.8.2.2.

The web of a link shall be single thickness. Doubler-plate reinforcement and web penetrations are not permitted.

Shear Strength

Except as limited below, the link design shear strength, $\phi_v V_n$, and the allowable shear strength, V_n/Ω_v, according to the limit state of shear yielding shall be determined as follows:

V_n = *nominal shear strength* of the link, equal to the lesser of V_p or $2M_p/e$, N

$\phi_v = 0.90$ (LRFD) $\Omega_v = 1.67$ (ASD)

where

$M_p = F_y Z$, N-mm

$V_p = 0.6F_y A_w$, N

e = link length, mm

$A_w = (d - 2t_f)t_w$

The effect of axial force on the link available shear strength need not be considered if

$P_u \leq 0.15P_y$ (LRFD) or $P_a \leq (0.15/1.5)P_y$ (ASD), as appropriate.

Where,

P_u = required axial strength using LRFD load combinations, N

P_a = required axial strength using ASD load combinations, N

P_y = nominal axial yield strength = $F_y A_g$, N

If $P_u > 0.15P_y$ (LRFD) or $P_a > (0.15/1.5)P_y$ (ASD), as appropriate, the following additional requirements shall be met:

(1) The available shear strength of the link shall be the lesser of

$\phi_v V_{pa}$ and $2\phi_v M_{pa}/e$ (LRFD)

or

V_{pa}/Ω_v and $2(M_{pa}/e)/\Omega_v$ (ASD), as appropriate, where

ϕ_v = 0.90 (LRFD), Ω_v = 1.67 (ASD)

$$V_{pa} = V_P\sqrt{(1-(P_r/P_c)^2} \quad (10.20.15.1)$$

$$M_{pa} = 1.18M_p[1-(P_r/P_c)] \quad (10.20.15.2)$$

$P_r = P_u$ (LRFD) or P_a (ASD), as appropriate

$P_c = P_y$ (LRFD) or $P_y/1.5$ (ASD), as appropriate

(2) The length of the link shall not exceed:

(a) $[1.15 - 0.5\rho'(A_w/A_g)]1.6M_p/V_p$ when $\rho'(A_w/A_g) \geq 0.3$ (10.20.15.3)

Nor,

(b) $1.6\,M_p/V_p$ when $\rho'(A_w/A_g) < 0.3$ (10.20.15.4)

Where,

$A_w = (d - 2t_f)t_w$

$\rho' = P_r/V_r$

and where,

$V_r = V_u$ (LRFD) or V_a (ASD), as appropriate

V_u = required shear strength based on LRFD load combinations.

V_a = required shear strength based on ASD load combinations.

Link Rotation Angle

The link rotation angle is the inelastic angle between the link and the beam outside of the link when the total story drift is equal to the design story drift, Δ. The link rotation angle shall not exceed the following values:

(a) 0.08 radians for links of length $1.6M_p/V_p$ or less.

(b) 0.02 radians for links of length $2.6M_p/V_p$ or greater.

(c) The value determined by linear interpolation between the above values for links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$.

Link Stiffeners

Full-depth web stiffeners shall be provided on both sides of the link web at the diagonal brace ends of the link. These stiffeners shall have a combined width not less than $(b_f - 2t_w)$ and a thickness not less than $0.75t_w$ or 10 mm, whichever is larger, where b_f and t_w are the link flange width and link web thickness, respectively.

Links shall be provided with intermediate web stiffeners as follows:

(a) Links of lengths $1.6M_p/V_p$ or less shall be provided with intermediate web stiffeners spaced at intervals not exceeding $(30t_w - d/5)$ for a link rotation angle of 0.08 radian or $(52t_w - d/5)$ for link rotation angles of 0.02 radian or less. Linear interpolation shall be used for values between 0.08 and 0.02 radian.

(b) Links of length greater than $2.6M_p/V_p$ and less than $5M_p/V_p$ shall be provided with intermediate web stiffeners placed at a distance of 1.5 times b_f from each end of the link.

(c) Links of length between $1.6M_p/V_p$ and $2.6M_p/V_p$ shall be provided with intermediate web stiffeners meeting the requirements of (a) and (b) above.

(d) Intermediate web stiffeners are not required in links of lengths greater than $5M_p/V_p$.

(e) Intermediate web stiffeners shall be full depth. For links that are less than 635 mm in depth, stiffeners are required on only one side of the link web. The thickness of one-sided stiffeners shall not be less than t_w or 10 mm, whichever is larger, and the width shall be not less than $(b_f/2)\ t_w$. For links that are 635 mm in depth or greater, similar intermediate stiffeners are required on both sides of the web.

The *required strength* of fillet welds connecting a link stiffener to the link web is $A_{st}F_y$ (LRFD) or $A_{st}F_y/1.5$ (ASD), as appropriate, where A_{st} is the area of the stiffener. The required strength of fillet welds connecting the stiffener to the link flanges is $A_{st}F_y/4$ (LRFD) or $A_{st}F_y/4(1.5)$ (ASD).

Link-to-Column Connections

Link-to-column connections must be capable of sustaining the maximum *link rotation angle* based on the length of the link, as specified in Section 10.20.15.2.3. The strength of the connection measured at the column face shall equal at least the nominal shear strength of the link, V_n, as specified in Section 10.20.15.2.2 at the maximum link rotation angle.

Link-to-column connections shall satisfy the above requirements by one of the following:

(a) Use a connection *prequalified* for EBF in accordance with Appendix 10A.

(b) Provide qualifying cyclic test results in accordance with Appendix 10.D. Results of at least two cyclic connection tests shall be provided and are permitted to be based on one of the following:

(i) Tests reported in research literature or documented tests performed for other projects that are representative of project conditions, within the limits specified in Appendix 10.D.

(ii) Tests that are conducted specifically for the project and are representative of project member sizes, material strengths, connection configurations, and matching connection processes, within the limits specified in Appendix 10.D.

Exception: Where reinforcement at the beam-to-column connection at the link end precludes yielding of the beam over the reinforced length, the link is permitted to be the beam segment from the end of the reinforcement to the brace connection. Where such links are used and the link length does not exceed $1.6M_p/V_p$, cyclic testing of the reinforced connection is not required if the *available strength* of the reinforced section and the connection equals or exceeds the *required strength* calculated based upon the strain-hardened link as described in Section 10.20.15.6. Full depth stiffeners as required in Section 10.20.15.3 shall be placed at the link-to-reinforcement interface.

Lateral Bracing of Link

Lateral bracing shall be provided at both the top and bottom *link* flanges at the ends of the link. The *required strength* of each lateral brace at the ends of the link shall be $P_b = 0.06\ M_r/h_o$, where h_o is the distance between flange centroids in mm.

For design according to S*pecification* Section 10.2.3.3 (LRFD)

$$M_r = M_{u,exp} = R_y Z F_y$$

For design according to S*pecification* Section 10.2.3.4 (ASD)

$$M_r = M_{u,exp}/1.5$$

The required brace stiffness shall meet the provisions of Equation 10.19.3.4 of Section 10.19, where M_r is defined above, $C_d = 1$, and L_b is the link length.

10.20.15.3 Diagonal Brace and Beam Outside of Link

Diagonal Brace

The *required combined axial and flexural strength* of the diagonal brace shall be determined based on load combinations stipulated by the *applicable building code*. For load combinations including seismic effects, a load Q_1 shall be substituted for the term *E,* where Q_1 is defined as the axial forces and moments generated by at least 1.25 times the *expected nominal shear strength* of the *link* R_yV_n, where V_n is as defined in Section 10.20.15.2.2. The *available strength* of the diagonal brace shall comply with *Specification* Section 10.10. Brace members shall meet the requirements of Section 10.20.8.2.1.

Beam Outside Link

The required combined axial and flexural strength of the beam outside of the link shall be determined based on load combinations stipulated by the applicable building code. For load combinations including seismic effects, a load Q_1 shall be substituted for the term *E* where Q_1 is defined as the forces generated by at least 1.1 times the expected nominal shear strength of the link, $R_y V_n$, where V_n is as defined in Section 10.20.15.2.2. The available strength of the beam outside of the link shall be determined by the *Specification*, multiplied by R_y .

At the connection between the diagonal brace and the beam at the link end of the brace, the intersection of the brace and beam centerlines shall be at the end of the link or in the link.

Bracing Connections

The *required strength* of the diagonal brace connections, at both ends of the brace, shall be at least equal to the required strength of the diagonal brace, as defined in Section 10.20.15.6.1. The diagonal brace connections shall also satisfy the requirements of Section 10.20.13.3.3.

No part of the diagonal brace connection at the link end of the brace shall extend over the link length. If the brace is designed to resist a portion of the link end moment, then the diagonal brace connection at the link end of the brace shall be designed as a fully-restrained moment connection.

Beam-to-Column Connections

If the EBF system factors in the *applicable building code* require moment resisting connections away from the *link*, then the beam-to-column connections away from the link shall meet the requirements for beam-to-column connections for OMF specified in Sections 10.20.11.2 and 10.20.11.5.

If the EBF system factors in the applicable building code do not require moment resisting connections away from the link, then the beam-to-column connections away from the link are permitted to be designed as pinned in the plane of the web.

Required Strength of Columns

In addition to the requirements in Section 10.20.8.3, the *required strength* of columns shall be determined from load combinations as stipulated by the *applicable building code*, except that the seismic load *E* shall be the forces generated by 1.1 times the *expected nominal shear strength* of all links above the level under consideration. The expected nominal shear strength of a link is $R_y V_n$, where V_n is as defined in Section 10.20.15.2.2. Column members shall meet the requirements of Section 10.20.8.2.2.

Protected Zone

Links in EBFs are a *protected zone*, and shall satisfy the requirements of Section 10.20.7.4. Welding on links is permitted for attachment of link stiffeners, as required in Section 10.20.15.3.

Demand Critical Welds

Complete-joint-penetration groove welds attaching the *link* flanges and the link web to the column are *demand critical welds*, and shall satisfy the requirements of Section 10.20.7.3.2.

10.20.16 Buckling-Restrained Braced Frames (BRBF)

10.20.16.1 **Scope**

Buckling-restrained braced frames (BRBF) are expected to withstand significant inelastic deformations when subjected to the forces resulting from the motions of the *design earthquake*. BRBF shall meet the requirements in this Section. Where the *applicable building code* does not contain design coefficients for BRBF, the provisions of Appendix 10.C shall apply.

10.20.16.2 **Bracing Members**

Bracing members shall be composed of a structural steel core and a system that restrains the steel core from buckling.

Steel Core

The *steel core* shall be designed to resist the entire axial force in the brace.

The brace *design axial strength*, ϕP_{ysc} (LRFD), and the brace *allowable axial strength*, P_{ysc}/Ω (ASD), in tension and compression, according to the limit state of yielding, shall be determined as follows:

$P_{ysc} = F_{ysc}\ A_{sc}$ (10.20.16.1)

$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

where

F_{ysc} = specified minimum yield stress of the steel core, or actual yield stress of the steel core as determined from a coupon test, MPa

A = net area of steel core, mm^2

Plates used in the steel core that are 50 mm thick or greater shall satisfy the minimum notch toughness requirements of Section 10.20.6.3.

Splices in the steel core are not permitted.

Buckling-Restraining System

The buckling-restraining system shall consist of the casing for the steel core. In stability calculations, beams, columns, and gussets connecting the core shall be considered parts of this system.

The buckling-restraining system shall limit local and overall buckling of the steel core for deformations corresponding to 2.0 times the *design story drift*. The buckling-restraining system shall not be permitted to buckle within deformations corresponding to 2.0 times the design story drift.

Testing

The design of braces shall be based upon results from qualifying cyclic tests in accordance with the procedures and acceptance criteria of Appendix 10.E. Qualifying test results shall consist of at least two successful cyclic tests: one is required to be a test of a brace sub-assemblage that includes brace connection rotational demands complying with Appendix 10.E, Section 10.E.4 and the other shall be either a uniaxial or a sub-assemblage test complying with Appendix 10.E, Section 10.E.5. Both test types are permitted to be based upon one of the following:

(a) Tests reported in research or documented tests performed for other projects.

(b) Tests that are conducted specifically for the project.

Interpolation or extrapolation of test results for different member sizes shall be justified by rational analysis that demonstrates stress distributions and magnitudes of internal strains consistent with or less severe than the tested assemblies and that considers the adverse effects of variations in material properties. Extrapolation of test results shall be based upon similar combinations of steel core and buckling-restraining system sizes. Tests shall be permitted to qualify a design when the provisions of Appendix 10.E are met.

Adjusted Brace Strength

Where required by these Provisions, bracing connections and adjoining members shall be designed to resist forces calculated based on the *adjusted brace strength.*

The adjusted brace strength in compression shall be $\beta\omega R_y P_{ysc}$. The adjusted brace strength in tension shall be $\omega R_y P_{ysc}$.

Exception: The factor R_y need not be applied if P_{ysc} is established using yield stress determined from a coupon test.

The compression strength adjustment factor, β, shall be calculated as the ratio of the maximum compression force to the maximum tension force of the test specimen measured from the qualification tests specified in Appendix 10.E, Section 10.E.6.3 for the range of deformations corresponding to 2.0 times the design story drift. The larger value of β from the two required brace qualification tests shall be used. In no case shall β be taken as less than 1.0. The strain hardening adjustment factor, ω, shall be calculated as the ratio of the maximum tension force measured from the qualification tests specified in Appendix 10.E, Section 10.E.6.3 (for the range of deformations corresponding to 2.0 times the design story drift) to F_{ysc} of the test specimen. The larger value of ω from the two required qualification tests shall be used. Where the tested steel core material does not match that of the prototype, ω shall be based on coupon testing of the prototype material.

10.20.16.3 **Bracing Connections**

Required Strength

The *required strength* of bracing connections in tension and compression (including beam-to-column connections if part of the bracing system) shall be 1.1 times the *adjusted brace strength* in compression (LRFD) or 1.1/1.5 times the adjusted brace strength in compression (ASD).

Gusset Plates

The design of connections shall include considerations of local and overall buckling. Bracing consistent with that used in the tests upon which the design is based is required.

Special Requirements Related to Bracing Configuration

V-type and inverted-V-type braced frames shall meet the following requirements:

(1) The *required strength* of beams intersected by braces, their connections, and supporting members shall be determined based on the load combinations of the *applicable building code* assuming that the braces provide no support for dead and live loads. For load combinations that include earthquake effects, the vertical and horizontal earthquake effect, *E*, on the beam shall be determined from the *adjusted brace strengths* in tension and compression.

(2) Beams shall be continuous between columns. Both flanges of beams shall be laterally braced. Lateral braces shall meet the provisions of Equations 10.19.3.3 and 10.19.3.4 of Section 10.19, where $M_r = M_u = R_y Z F_y$ (LRFD) or $M_r = M_a = R_y Z F_y/1.5$ (ASD), as appropriate, of the beam and $C_d = 1.0$. As a minimum, one set of lateral braces is required at the point of intersection of the V-type (or inverted V-type) bracing, unless the beam has sufficient out-of-plane strength and stiffness to ensure stability between adjacent brace points.

For purposes of brace design and testing, the calculated maximum deformation of braces shall be increased by including the effect of the vertical deflection of the beam under the loading defined in Section 10.20.16.4(1).

K-type braced frames are not permitted for BRBF.

Beams and Columns

Beams and columns in BRBF shall meet the following requirements.

Width-Thickness Limitations

Beam and column members shall meet the requirements of Section 10.20.8.2.2.

Required Strength

The *required strength* of beams and columns in BRBF shall be determined from load combinations as stipulated in the *applicable building code*. For load combinations that include earthquake effects, the earthquake effect, *E*, shall be determined from the *adjusted brace strengths* in tension and compression.

The required strength of beams and columns need not exceed the maximum force that can be developed by the system.

Splices

In addition to meeting the requirements in Section 10.20.8.4, column splices in BRBF shall be designed to develop 50 percent of the lesser *available flexural strength* of the connected members, determined based on the limit state of yielding. The *required shear strength* shall be $\Sigma M_{pc}/H$ (LRFD) or $\Sigma M_{pc}/1.5H$ (ASD), as appropriate, where ΣM_{pc} is the sum of the nominal plastic flexural strengths of the columns above and below the splice.

Protected Zone

The *protected zone* shall include the steel core of bracing members and elements that connect the steel core to beams and columns, and shall satisfy the requirements of Section 10.20.7.4.

10.20.17 Special Plate Shear Walls (SPSW)

10.20.17.1 **Scope**

Special plate shear walls (SPSW) are expected to withstand significant inelastic deformations in the webs when subjected to the forces resulting from the motions of the design earthquake. The horizontal boundary elements (HBEs) and vertical boundary elements (VBEs) adjacent to the webs shall be designed to remain essentially elastic under the maximum forces that can be generated by the fully yielded webs, except that plastic hinging at the ends of HBEs is permitted. SPSW shall meet the requirements of this Section. Where the applicable building code does not contain design coefficients for SPSW, the provisions of Appendix 10.C shall apply.

10.20.17.2 **Webs**

Shear Strength

The panel design shear strength, ϕV_n (LRFD), and the allowable shear strength, V_n/Ω (ASD), according to the limit state of shear yielding, shall be determined as follows:

$$V_n = 0.42 F_y t_w L_{cf} \sin 2\alpha \tag{10.20.17.1}$$

ϕ = 0.90 (LRFD) Ω = 1.67 (ASD)

where

t_w = thickness of the web, mm.

L_{cf} = clear distance between VBE flanges, mm

α is the angle of web yielding in radians, as measured relative to the vertical, and it is given by:

$$\tan^4 \alpha = \frac{1+\dfrac{t_w L}{2A_c}}{1+t_w h\left(\dfrac{1}{A_b}+\dfrac{h^3}{360 I_c L}\right)} \tag{10.20.17.2}$$

h = distance between HBE centerlines, mm

A_b = cross-sectional area of a HBE, mm^2

A_c = cross-sectional area of a VBE, mm^2

I_c = moment of inertia of a VBE taken perpendicular to the direction of the web plate line, mm^4

L = distance between VBE centerlines, mm

Panel Aspect Ratio

The ratio of panel length to height, L/h, shall be limited to $0.8 < L/h \leq 2.5$.

Openings in Webs

Openings in webs shall be bounded on all sides by HBE and VBE extending the full width and height of the panel, respectively, unless otherwise justified by testing and analysis.

Connections of Webs to Boundary Elements

The required strength of web connections to the surrounding HBE and VBE shall equal the expected yield strength, in tension, of the web calculated at an angle α, defined by Equation 10.20.17.2.

10.20.17.3 **Horizontal and Vertical Boundary Elements**

Required Strength

In addition to the requirements of Section 10.20.8.3, the *required strength* of VBE shall be based upon the forces corresponding to the *expected yield strength*, in tension, of the web calculated at an angle α.

The required strength of HBE shall be the greater of the forces corresponding to the expected yield strength, in tension, of the web calculated at an angle α or that determined from the load combinations in the *applicable building code* assuming the web provides no support for gravity loads.

The beam-column moment ratio provisions in Section 10.20.9.6 shall be met for all HBE/VBE intersections without consideration of the effects of the webs.

HBE-to-VBE Connections

HBE-to-VBE connections shall satisfy the requirements of Section 10.20.11.2. The required shear strength, V_u, of a HBE-to-VBE connection shall be determined in accordance with the provisions of Section 10.20.11.2, except that the required shear strength shall not be less than the shear corresponding to moments at each end equal to $1.1 R_y M_p$ (LRFD) or $(1.1/1.5) R_y M_p$ (ASD), as appropriate, together with the shear resulting from the *expected yield strength* in tension of the webs yielding at an angle α.

Width-Thickness Limitations

HBE and VBE members shall meet the requirements of Section 10.20.8.2.2.

Lateral Bracing

HBE shall be laterally braced at all intersections with VBE and at a spacing not to exceed $0.086r_y\, E/F_y$. Both flanges of HBE shall be braced either directly or indirectly. The required strength of lateral bracing shall be at least 2 percent of the HBE flange *nominal strength*, $F_y\, b_f\, t_f$. The required stiffness of all lateral bracing shall be determined in accordance with Equation 10.19.3.4 of Section 10.19. In these equations, M_r shall be computed as $R_y\, ZF_y$ (LRFD) or M_r shall be computed as $R_y\, ZF_y/1.5$ (ASD), as appropriate, and C_d = 1.0.

VBE Splices

VBE splices shall comply with the requirements of Section 10.20.8.4.

Panel Zones

The VBE panel zone next to the top and base HBE of the SPSW shall comply with the requirements in Section 10.20.9.3.

Stiffness of Vertical Boundary Elements

The VBE shall have moments of inertia about an axis taken perpendicular to the plane of the web, I_c, not less than $0.00307t_w h^4/L$.

10.20.18 Quality Assurance Plan

Scope

When required by the *applicable building code* or the engineer of record, a *quality assurance plan* shall be provided. The quality assurance plan shall include the requirements of Appendix 10.B.