APPENDIX C

FEMA 302 AND OTHER STANDARD GUIDANCE FOR COLD-FORMED STEEL SEISMIC DESIGN

C1. INTRODUCTION. Seismic use groups (FEMA 302, 1.3) are used to determine occupancy importance factors (FEMA 302, Table 1.4). Seismic use group III is for the most critical facilities as defined in FEMA 302, 1.3. This table is reproduced below:

Table C-1 Occupancy Importance Factors

Seismic Use Group	I
I	1.0
II	1.25
III	1.5

TI 809-04 uses enhanced performance objectives to define seismic design forces for more critical facilities rather than the occupancy importance factors presented here.

C2. DEFINING GROUND MOTION. Seismic ground motions shall be defined according to FEMA 302 Chapter 4 and TI 809-04 Chapter 3. This paragraph defines ground motions for the maximum considered earthquake ground motions derived from Maps 1 through 24 (FEMA 302, Chapter 4). Spectral response acceleration at short periods, S_S and at 1 second, S_1 are obtained from Maps 1 through 24 of FEMA 302. For most regions of the nation, the maximum considered earthquake ground motion is defined with a uniform likelihood of exceedance of 2 percent in 50 years (2500 year return period)[1].

Site classifications shall be determined based on soil type (A through F), which may be based on shear wave velocity, ν_s average blow counts from standard penetration resistance test N [2] or unconfined shear strength, s_u of the soil (FEMA 302, 4.1.2). From the site classifications, values of site coefficients (F_a and F_v) are determined for the mapped spectral response acceleration values (FEMA 302, Table 4.1.1.4a and Table 4.1.1.4b). These tables are reproduced below:

Table C-2a Values of F_a as a Function of Site Class and Mapped Short-Period Maximum Considered Earthquake Spectral Acceleration[3]

Site Class	Short Period Maximum Considered Response Spectral Acceleration				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	a[4]
F	a	a	a	a	a

[1] 1997 NEHRP, Part 2: Commentary (FEMA 303), Chapter 4 – Ground Motion, p. 37.
[2] Defined in ASTM D1536-84.
[3] Use straight line interpolation for intermediate values of S_S.
[4] a indicates that a site-specific geotechnical investigation and dynamic site response analyses shall be performed.

Table C-2b Values of F_v as a Function of Site Class and Mapped 1 Second Period Maximum Considered Earthquake Spectral Acceleration[5]					
Site Class	1 Second Period Maximum Considered Response Spectral Acceleration				
	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \geq 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	a[6]
F	a	a	a	a	a

The maximum considered earthquake spectral response acceleration for short periods, S_{MS} and at 1 second, S_{M1} adjusted for site class effects are calculated as follows (FEMA 302, Eq. 4.1.2.4-1 and 4.1.2.4-2):

$$S_{MS} = F_a S_S \qquad \text{(Eq C-1)}$$

and

$$S_{M1} = F_v S_1 \qquad \text{(Eq C-2)}$$

These values define the elastic spectra. These values are reduced to define design earthquake spectral response acceleration at short periods, S_{DS} and at 1-second period, S_{D1} as follows (FEMA 302, Eq. 4.1.2.5-1 and 4.1.2.5-2):

$$S_{DS} = \frac{2}{3} S_{MS} \qquad \text{(Eq C-3)}$$

and

$$S_{D1} = \frac{2}{3} S_{M1} \qquad \text{(Eq C-4)}$$

From these terms a design response spectrum is developed as indicated in Figure C-1 (FEMA 302, Figure 4.1.2.6). For the natural period of the structure, T this spectrum defines values of effective acceleration. The three regions of this spectrum are defined as follows:

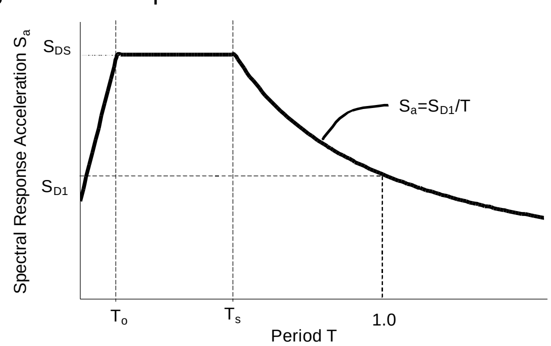

Figure C-1. Design response spectrum.

[5] Use straight-line interpolation for intermediate values of S_1.
[6] a indicates that a site-specific geotechnical investigation and dynamic site response analyses shall be performed.

For periods less than or equal to, T_0 the design spectral acceleration, S_a shall be (FEMA 302, Equation 4.1.2.6-1):

$$S_a = 0.6\frac{S_{DS}}{T_0}T + 0.4S_{DS} \qquad \text{(Eq C-5)}$$

For periods greater than or equal to T_0 and less than or equal to T_S, the design spectral response acceleration , S_a, shall be taken as equal to S_{DS}.

For periods greater than T_S, the design spectral response acceleration, S_a, shall be (FEMA 302, Equation 4.1.2.6-3):

$$S_a = \frac{S_{D1}}{T} \qquad \text{(Eq C-6)}$$

Where:

T = the fundamental period of the structure in seconds.
$T_0 = 0.2S_{D1}/S_{DS}$.
$T_S = S_{D1}/S_{DS}$.

C3. SEISMIC DESIGN CATEGORY. Each structure shall be assigned a seismic design category based on their Seismic Use Group and design response coefficients, S_{DS} and S_{D1} as indicated in the tables below (from FEMA 302 Tables 4.2.1a and 4.2.1b):

Table C-3a Seismic Design Category Based on Short Period Response Accelerations

Value of S_{DS}	Seismic Use Group		
	I	II	III
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

Table C-3b Seismic Design Category Based on 1 Second Period Response Accelerations

Value of S_{DS}	Seismic Use Group		
	I	II	III
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D
$0.75g \le S_1$	E	E	F

C4. STRUCTURAL CONFIGURATION AND REDUNDANCY. FEMA 302, Section 5.2.3 and 5.2.4 presents guidance on structural configurations and redundancy. Diaphragms are considered flexible if the maximum lateral deformation of the diaphragm exceeds twice the average story drift of the associated story (FEMA 302, 5.2.3.1).

A reliability factor, ρ, shall be defined for all structures based on the extent of structural redundancy in the lateral-force-resisting system. For structures in Seismic Design Categories A, B and C, the value for ρ shall be taken as 1.0. For structures in categories D, E and F, values for ρ shall be taken as the largest of the values of ρ_x calculated for each story of the structure "x" as follows (FEMA 302, Equation 5.2.4.2):

$$\rho_x = 2 - \frac{C1}{r_{max_x}\sqrt{A_x}} \qquad \text{(Eq C-7)}$$

Where:

r_{maxx} = the ratio of design story shear resisted by the single shear panel carrying the most shear force in the story to the total shear story, for a given direction of loading. Lateral loads shall be distributed to panels based on relative stiffness considering the interaction of panels with varying stiffness.

A_x = the floor area in m^2 (ft^2) of the diaphragm level immediately above the story.

ρ need not exceed 1.5, and may be used for any structure. The value of ρ shall not be taken as less than 1.0.

C1 = constant, 6.1 – metric, (20 – English)

C5. LOAD COMBINATIONS. Consideration of combinations of loads in the two orthogonal directions is not needed. The effects of gravity loads and seismic forces shall be combined in accordance with the factored load combinations as indicated below (ASCE 7[7]).

$$1.2D + 1.0E + 0.5L + 0.2S \qquad \text{(Eq C-8)}$$

and

$$0.9D + 1.0E \qquad \text{(Eq C-9)}$$

Where:

D = the dead load.

E = the effect of seismic load.

L = the live load – the load factor on L in Equation C-8 shall equal 1.0 for garages, areas occupied for public assembly, and all areas where the live load is greater than 4.79 kN/m^2 (100 psf).

S = the snow load – when the flat roof snow loads exceed 1.44 KN/m^2 (30 psf), the full design snow load shall be included in Equation C-8.

The effect of seismic loads, E shall be defined as follows, when the effect of gravity and seismic loads are additive (FEMA 302, 5.2.7):

$$E = \rho Q_E + 0.2 S_{DS} D \qquad \text{(Eq C-10)}$$

Where:

E = the effect of horizontal and vertical earthquake-induced forces.

ρQ_E = the maximum horizontal force that could be resisted by the bracing.

ρ = the system redundancy factor.

Q_E = the effect of horizontal seismic forces.

$0.2S_{DS}D$ = the vertical spectral acceleration effect of the seismic load.

S_{DS} = the design spectral response acceleration at short periods.

D = the effect of dead load.

The effect of seismic loads, E shall be defined as follows, when the effect of gravity and seismic loads counteract each other:

[7] Minimum Design Loads for Buildings and Other Structures, ASCE 7-95, Section 2.3.2.

$$E = \rho Q_E - 0.2S_{DS}D \qquad \text{(Eq C-11)}$$

The effects of gravity load (dead, live and snow load) and seismic forces shall be combined as follows when the effect of gravity and seismic loads are additive, by combining Equations C-8 and C-10:

$$(1.2 + 0.2S_{DS})D + 0.5L + 0.2S + \rho Q_E \qquad \text{(Eq C-12)}$$

The effects of gravity load and seismic forces shall be combined as follows when the effect of gravity and seismic loads counteract each other, by combining Equations C-9 and C-11:

$$(0.9 - 0.2S_{DS})D + \rho Q_E \qquad \text{(Eq C-13)}$$

For both expressions in Equations C-12 and C-13, the total horizontal force is ρQ_E. This force alone defines the total lateral load that must be resisted by the shear panel diagonal straps or full panel sheets, and these elements should be sized based on this force.

The effect of seismic loads, E shall be defined as follows, to account for diagonal strap overstrength when the effect of gravity and seismic loads are additive:

$$E = \Omega_0 Q_E + 0.2S_{DS}D \qquad \text{(Eq C-14)}$$

Where:

Ω_0 = the system overstrength.

The effect of seismic loads, E shall be defined as follows, when the effect of gravity and seismic loads counteract each other:

$$E = \Omega_0 Q_E - 0.2S_{DS}D \qquad \text{(Eq C-15)}$$

The term $\Omega_0 Q_E$ calculated in Equations C-14 and C-15 (TI 809-04, Equations 4-6 and 4-7) need not exceed the maximum force that can be developed in the diagonal straps, based on the maximum estimated ultimate strength of these elements. This is expressed as follows:

$$\Omega_0 Q_E \leq Q_u = F_{sumax} n_s b_s t_s \frac{W}{\sqrt{H^2 + W^2}} \qquad \text{(Eq C-16)}$$

Where:

F_{sumax} = the maximum ultimate stress of the diagonal straps, which equals 1.5 F_{su} for ASTM A653 Grade 33 steel (F_{su} = 310 MPa and 45 ksi) and 1.25 F_{su} for Grade 50 steel (F_{su} = 448 MPa and 65 ksi).
n_s = the number of diagonal straps.
b_s = the width of the diagonal straps.
t_s = the thickness of the diagonal straps.
W = the overall panel width.
H = the overall panel height (see Figure 3-2 for a schematic panel drawing showing W and H).

The effects of gravity load (dead, live and snow load) and seismic forces shall be combined as follows to account for diagonal strap overstrength, when the effect of gravity and seismic loads are additive, by combining Equations C-8 and C-14:

$$(1.2 + 0.2S_{DS})D + 0.5L + 0.2S + \Omega_0 Q_E \qquad \text{(Eq C-17)}$$

The effects of gravity load and seismic forces shall be combined as follows to account for diagonal strap overstrength, when the effect of gravity and seismic loads counteract each other, by combining Equations C-9 and C-15:

$$(0.9 - 0.2S_{DS})D + \Omega_0 Q_E \qquad \text{(Eq C-18)}$$

For both expressions in Equations C-17 and C-18, the total horizontal force is $\Omega_0 Q_E$. Every other term in these equations represent vertical loads. The shear panel systems should be analyzed based on the most critical load combination defined by either Equation C-17 or C-18. Each panel component (including all connections), other than the diagonal strap, should be designed based on these loads.

C6. EQUIVALENT LATERAL FORCE PROCEDURE. FEMA 302 and TI 809-04 present two methods for defining the structural response: the Equivalent Lateral Force Procedure (FEMA 302, 5.3) and the Modal Analysis Procedure (FEMA 302, 5.4). Paragraphs C7 through C9 presents the determination of base shear, period and vertical distribution of lateral forces using the Equivalent Lateral Force Procedure. Only this method is presented because of its simplicity and recognizing that typical cold-formed steel structures will likely be low rise construction so that first mode response will dominate the seismic response of the structures. However if deemed beneficial the modal analysis approach presented in FEMA 302 and TI 809-04 (Chapter 3-2.c.(2)) could be used.

C7. SEISMIC BASE SHEAR. Using the Equivalent Lateral Force Procedure, the seismic base shear, V in a given direction shall be determined according to the following equation (FEMA 302, 5.3.2):

$$V = C_s W \qquad \text{(Eq C-19)}$$

Where:

C_s = the seismic response coefficient.
W = the total dead load and applicable portions of other loads (see FEMA 302, 5.3.2).

The seismic response coefficient, C_s shall be determined according to the following equation:

$$C_s = \frac{S_{DS}}{R/I} \qquad \text{(Eq C-20)}$$

The value for C_s is calculated according to Equation C-20; and need not exceed the following:

$$C_s = \frac{S_{D1}}{T(R/I)} \qquad \text{(Eq C-21)}$$

but shall not be less than:

$$C_s = 0.1S_{D1}I \qquad \text{(Eq C-22)}$$

nor shall it be taken as less than the following equation for Seismic Design Categories E and F:

$$C_s = \frac{0.5S_1}{R/I} \qquad \text{(Eq C-23)}$$

C8. PERIOD DETERMINATION. The fundamental period of the building, T in the direction under consideration shall be defined using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis (FEMA 302, 5.3.3). Alternatively, T is permitted to be taken as the approximate fundamental period, T_a determined in accordance with the following requirements. The fundamental period, T shall not exceed the product of the coefficient for upper limit on calculated period, C_u from Table C-4 and the approximate fundamental period, T_a determined as follows:

$$T_a = C_T h_n^{3/4} \qquad \text{(Eq C-24)}$$

Where:

C_T = constant = 0.0731 – metric, (0.030 – English) for cold-formed steel shear panels with diagonal straps.

h_n = the height in meters (ft - English) above the base to the highest level in the structure.

Table C-4 Coefficient for Upper Limit on Calculated Period

Design Spectral Response Acceleration at 1 Second, S_{D1}	Coefficient C_u
$S_{D1} < 0.1g$	1.7
$0.1g \le S_{D1} < 0.15g$	1.7
$0.15g \le S_{D1} < 0.2g$	1.5
$0.2g \le S_{D1} < 0.3g$	1.4
$0.3g \le S_{D1} < 0.4g$	1.3
$0.4g \le S_{D1}$	1.2

C9. VERTICAL DISTRIBUTION OF LATERAL SEISMIC FORCES. The vertical distribution of lateral seismic forces, F_x (kN or kip), induced at any level shall be determined from the following equations (FEMA 302, 5.3.4):

$$F_x = C_{vx} V \qquad \text{(Eq C-25)}$$

and

$$C_{vx} = \frac{W_x h_x}{\sum_{i=1}^{n} w_i h_i} \qquad \text{(Eq C-26)}$$

Where:

C_{vx} = the vertical distribution factor.

V = the total design lateral force or shear at the base of the structure (kN or kip).

w_i and w_x = the portion of the total gravity load of the structure, W located or assigned to level i or x.

h_i and h_x = the height (m or ft) from the base to level i or x.

The horizontal distribution of seismic story shear in any story, V_x (kN or kips) shall be determined from the following equation (FEMA 302, 5.3.5):

$$V_x = \sum_{i=x}^{n} F_i \qquad \text{(Eq C-27)}$$

Where:

F_i = the portion of the seismic base shear, V (kN or kips) induced at level i.

The seismic design story shear, V_x (kN or kips) shall be distributed to the various vertical elements of the seismic-force-resisting system in the story under consideration based on the relative lateral stiffness of the vertical resisting elements and the diaphragm.

C10. STRUCTURAL OVERTURNING RESISTANCE. The structure shall be designed to resist overturning effects caused by the seismic forces determined from Equation C-25. At any story, the increment of overturning moment in the story under consideration shall be distributed to the various vertical force resisting elements in the same proportion as the distribution of the horizontal shears to those elements (FEMA 302, 5.3.6).

The overturning moments at Level x, M_x (kN-m or kip-ft), shall be determined from the following equation:

$$Mx = \sum_{i=x}^{n} F_i (h_i = h_x) \qquad \text{(Eq C-28)}$$

Where:

F_i = the portion of the seismic base shear, V, induced at Level i.
h_i and h_x = the height (m or ft) from the base to Level i or x.

Foundations shall be designed for the foundation overturning design moment, M_f (kN-m or kip-ft) at the foundation-soil interface determined using Equation C-28 at the foundation level, multiplied by a reduction factor of 0.75.

C11. STORY DRIFTS AND P-DELTA EFFECTS. The story drifts and, member forces and moments due to P-delta effects shall be determined in accordance with the following guidance (FEMA 302, 5.37). Story drifts shall be calculated based on the application of design seismic forces to a mathematical model of the structure. The model shall include the stiffness and strength of all elements that are significant to the distribution of forces and deformations in the structure and shall represent the spatial distribution of the mass and stiffness of the structure. The design story drift, Δ shall be computed as the difference of the deflections at the center of mass at the top and bottom of the story under consideration. The deflections of level x, δ_x (mm or in) shall be determined according to the following equation:

$$\delta_x = \frac{C_d \delta_{xe}}{I} \qquad \text{(Eq C-29)}$$

Where:

δ_{xe} = the deflections determined by an elastic analysis (mm or in.) based on the forces defined in Equation C-25.

For determining compliance with the story drift limitations in Table 3-2, the deflections of Level x, δ_x (mm or in) shall be calculated as expressed in Equation C-29. For the purposes of this drift analysis only, the computed fundamental period, T in seconds, of the structure may be used without the upper bound limitations specified in Table C-4, when determining drift level seismic design forces.

The design story drift, Δ (mm or in) shall be increased by the incremental factor relating to the P-delta effects if required by the following guidance (FEMA 302, 5.3.7.2). The P-delta effects on story shears and moments, the resulting member forces and moments, and the story drifts induced by these effects are not required to be considered when the stability coefficient, θ as determined by the following equation is equal to or less than 0.10:

$$\theta = \frac{P_x \Delta}{V_x h_{sx} C_d} \qquad \text{(Eq C-30)}$$

Where:

P_x = the total vertical design load at and above Level x (kN or kip). When calculating the vertical design load for purposes of determining P-delta, the individual load factors need not exceed 1.0.
Δ = the design story drift occurring simultaneously with V_x (mm or in).
V_x = the seismic shear force acting between Level x and x-1 (kN or kip).

The stability coefficient, θ shall not exceed θ_{max} determined as follows:

$$\theta_{max} = \frac{0.5}{\beta C_d} \le 0.25 \qquad \text{(Eq C-31)}$$

Where:

β = the ratio of shear demand to shear capacity for the story between Level x and x - 1. This ratio may conservatively be taken as 1.0.

When the stability coefficient, θ is greater than 0.10 but less than or equal to θ_{max} the incremental factor related to P-delta effects, a_d shall be determined by rational analysis (see FEMA 303, Commentary, 5.3.7). To obtain the story drift for including the P-delta effects, the design story drift, Δ shall be multiplied by 1.0/(1 - θ). When θ is greater than θ_{max} the structure is potentially unstable and shall be redesigned.

C12. DIAGONAL STRAP DESIGN. From the values of seismic story shear, V_x and additional shear force due to torsion, the shear panel dimensions are defined and diagonal straps designed. The straps are tension only members and their design strength is defined by the following equation (AISI, C2, p. V 45):

$$\phi_t A_n F_{sy} = \phi_t \sum(b_s t_s) F_{sy} \qquad \text{(Eq C-32)}$$

Where:

ϕ_t = the resistance factor for tensile members (0.95).
A_n = the cross-sectional area of the all diagonal straps in tension ($b_s t_s$).
b_s= the width of an individual diagonal strap.
t_s = the thickness of an individual strap.
F_{sy} = the design yield strength of the strap.

The shear panel lateral yield capacity, Q_{sy} when the diagonal straps are the sole lateral-load-resisting element is calculated as follows:

$$Q_{sy} = n_s b_s t_s F_{sy} \left(\frac{W}{\sqrt{H^2 + W^2}} \right) \qquad \text{(Eq C-33)}$$

Where:

W = the width of a trial shear panel.
H = the height of a trial shear panel.

The shear panel design strength, $\phi_t Q_{sy}$ must be greater than the seismic story shear, V_x and additional shear force due to torsion, Qsi for all shear panels resisting in the frame of the building for which these forces are applied. This is expressed as:

$$\phi_t Q_{sy} = \phi_t \sum\left[n_s b_s t_s F_{sy}\left(\frac{W}{\sqrt{H^2 + W^2}}\right)\right] \geq V_x + Q_{si} \qquad \text{(Eq C-34)}$$

The number of shear panels, panel width, height, and strap size and strength shall be designed according to Equation C-34 to meet minimum lateral yield capacity. All diagonal strap material must be ASTM A653 steel. Diagonal straps may not use re-rolled steel, because the re-rolling strain hardens the material, increasing material strength variability and reducing elongation (see USACERL TR FL-XX, Chapter 4 for a discussion of this concern).

C13. COLUMN AXIAL CAPACITY. The column axial design strength, P shall be determined as follows for columns built-up with cold-formed steel studs or individual structural tubing members (AISI, C4, Concentrically Loaded Compression Members):

$$P = \phi_c A_e F_n \qquad \text{(Eq C-35)}$$

Where:

ϕ_c = the resistance factor for compression, which equals 0.85.
A_e = the effective area at the stress F_n.
F_n = the nominal strength of the column, determined as follows:

For $\lambda_c \leq 1.5$

$$F_n = (0.658^{\lambda_c^2})F_{cy} \qquad \text{(Eq C-36)}$$

For $\lambda_c > 1.5$

$$F_n = \left[\frac{0.877}{\lambda_c^2}\right]F_{cy} \qquad \text{(Eq C-37)}$$

Where:

$$\lambda_c = \sqrt{\frac{F_{cy}}{F_e}} \qquad \text{(Eq C-38)}$$

Where:

F_{cy} = the column design yield strength.

$$F_e = \frac{\pi^2 E}{\left(KL/r\right)^2} \qquad \text{(Eq C-39)}$$

Where:

E = the modulus of elasticity.
K = the effective length factor.
L = the unbraced length of the column.
r = the radius of gyration of the full, unreduced column cross section, calculated as follows:

$$r = \sqrt{I/A} \quad \text{(Eq C-40)}$$

The effective area, A_e is calculated as follows for columns built-up from cold-formed steel studs such that they form a closed section or structural tube columns (AISI, C4):

$$A_e = A_c - nt_c(w - b) \quad \text{(Eq C-41)}$$

Where:

A_c = the nominal column area.
n = the number of studs making up the column, or is 2 when using structural tube columns.
t_c= the thickness of the stud material used in the built-up columns, or the thickness of the structural tube column.
w = the flat width of the stud web making up the built-up columns, or the width of the structural tube face perpendicular to the plane of the panel. Assuming the outside radius of the stud corners is twice the thickness, t_c this may be calculated as follows:

$$w = d_s - 4t_c \quad \text{(Eq C-42)}$$

d_s = the depth of the studs making up the built-up columns, or the structural tube width perpendicular to the plane of the panel,
b = the effective width and shall be determined as follows (AISI, B2.2):

$$\text{for} \quad 0.5 \geq \frac{d_h}{w} \geq 0, \text{and} \frac{w}{t_c} \leq 70 \quad \text{and}$$

the distance between centers of holes $\geq 0.5w$ and $\geq 3d_h$,

$$b = w - d_h \quad \text{when} \quad \lambda \leq 0.673 \quad \text{(Eq C-43)}$$

$$b = \frac{w\left[1 - \frac{0.22}{\lambda} - \frac{0.8d_h}{w}\right]}{\lambda} \quad \text{when } \lambda > 0.673 \quad \text{(Eq C-44)}$$

Where:

d_h = the diameter of holes.
λ = a slenderness factor defined as follows (AISI, B2.1):

$$\lambda = \frac{1.052}{\sqrt{k}}\left(\frac{w}{t_c}\right)\sqrt{\frac{F_n}{E}} \quad \text{(Eq C-45)}$$

Where:

k = the plate buckling coefficient, equal to 4 for the studs making up the built-up columns or structural tube columns.

C14. COLUMN SHEAR CAPACITY. The column shear design strength, V_c shall be determined as follows for columns built-up with cold-formed steel studs or individual structural tubing members (AISI, C3.2, Strength for Shear Only):

For $\quad \frac{h}{t_c} \le 0.96\sqrt{\frac{Ek_v}{F_{cy}}}$

$$V_C = \phi_v 0.60 F_{cy} h_c n_s t_c \qquad \text{(Eq C-46)}$$

Where:

h = the depth of the flat portion of the web, which equals the stud flange width for the built-up columns.
k_v = the shear buckling coefficient, which equals 5.34 for both built-up and structural tubing columns.
$\phi_v = 1.0$
h_c = the depth of the column, which is the column width in the in-plane direction of the panel.
n_s = the number of faces of a shear panel with diagonal straps (i.e., 1 or 2).

C15. CONNECTION SHEAR AND PULL-OVER. The design shear (AISI E4.3.1) and pull-over per screw (AISI E4.4.2), P_s shall be calculated as follows:

$$P_s = \phi_s \min(P_{ns} \text{and} P_{nov}) \qquad \text{(Eq C-47)}$$

Where, the nominal shear strength per screw, P_{ns}, shall be determined as follows:

For $t_2/t_1 \le 1.0$, P_{ns} shall be taken as the smallest of:

$P_{ns} = 4.2\sqrt{t_2^3 d}F_{u2}$	tilting mode of failure	(Eq C-48)
$P_{ns} = 2.7 t_1 d F_{u1}$	bearing mode of failure	(Eq C-49)
$P_{ns} = 2.7 t_2 d F_{u2}$	bearing mode of failure	(Eq C-50)

For $t_2/t_1 \ge 2.5$, P_{ns} shall be taken as the smaller of:

$P_{ns} = 2.7 t_1 d F_{u1}$	bearing mode of failure	(Eq C-51)
$P_{ns} = 2.7 t_2 d F_{u2}$	bearing mode of failure	(Eq C-52)

For $1.0 < t_2/t_1 < 2.5$, P_{ns} shall be determined by linear interpolation between the two cases above.

Where:

ϕ_s = the screw resistance factor for shear, equal to 0.5.
d = the nominal screw diameter.
t_1 = the thickness of the member in contact with the screw head.
t_2 = the thickness of the member not in contact with the screw head.
F_{u1} = the ultimate tensile strength of the member in contact with the screw head.
F_{u2} = the ultimate tensile strength of the member not in contact with the screw head.

The nominal shear strength per screw, P_{ns} may also be determined by AISI Tables IV-7a and IV-7b for connections to various sheet thicknesses for sheets with ultimate strengths of 310 MPa (45 ksi) and 448 MPa (65 ksi) (ASTM A653, Grade 33 and 50 respectively). These tables may only be used if ultimate strengths of the materials being connected are the same.

The nominal pull-over strength, P_{nov} shall be calculated as follows:

$$P_{nov} = 1.5 t_1 d_w F_{u1} \qquad \text{(Eq C-53)}$$

Where:

d_w = the larger of the screw head diameter or the washer diameter, and shall not be taken as larger than 13 mm (½ inch).

C16. WELDED CONNECTION DESIGN. Diagonal strap-to-column connection fillet weld design based on AISI (E2.4 Fillet Welds) is summarized below. The design shear strength for loading in the longitudinal direction, P_L shall be determined as follows:

For $L/t < 25$
$$P_L = \left(1 - \frac{0.01L}{t}\right)\phi t L F_u \qquad \text{(Eq C-54)}$$

Where:

ϕ = 0.60.
L = the length of fillet weld.
t = the least value of the thicknesses of the two members being welded.

For $L/t \geq 25$
$$P_L = 0.75\phi t L F_u \qquad \text{(Eq C-55)}$$

Where:

ϕ = 0.55

The design shear strength of loading in the transverse direction, P_T shall be determined as follows:

$$P_T = \phi t L F_u \qquad \text{(Eq C-56)}$$

Where:

ϕ = 0.60.

For fillet welds to heavy strap material, thicker than 0.150 inches, the design shear strength for both longitudinal and transverse loading due to weld failure, P_W[8] shall not exceed the following:

$$P_W = 0.75\phi t_w L F_{xx} \qquad \text{(Eq C-57)}$$

Where:

ϕ = 0.60.
t_w = the effective throat, equal to 0.707 times the least leg of the weld in-plane or out-of-plane of the materials being welded. A larger effective throat shall be permitted if measurement shows that the welding procedure to be used consistently yields a larger value of t_w.
F_{xx} = the weld metal strength designation in AWS electrode classification.

C17. ANCHOR FLARE BEVEL GROVE WELD DESIGN. Column-to-anchor flare bevel grove weld design based on AISI (E2.5 Flare Grove Welds) is summarized below. The design strength of the longitudinal loaded flare bevel grove weld, P_G shall be determined as follows:

[8] AISI Commentary, E2.4, indicates that this equation is needed to cover the possibility of weld failure through the throat of the weld material, because tests that ensured tearing in the plate were not conducted on plates thicker than 0.15 inches.

For $t_c \le t_w < 2t_c$ (single shear) $$P_G = 0.75\phi_G t_c L F_{cu}$$ (Eq C-58)

For $t_w \ge 2t_c$ (double shear) $$P_G = 1.5\phi_G t_c L F_{cu}$$ (Eq C-59)

Where:

ϕ_G = the resistance factor for flare grove welds, equal to 0.55.
t_c = the thickness of the column material.
L = the length of the flare bevel grove weld.
F_{cu} = the ultimate strength of the column steel.

C18. ANCHOR BOLT CONE FAILURE. Embedded anchors shall be used for all anchor bolts described here. The anchor bolt cone failure design strength, P_c shall exceed the applied tensile force per bolt, P_{tAB} (Equation 3-37). Either one or two anchor bolts may be installed on both sides of the columns (i.e., n_{AB} equal to 2 or 4). If only one bolt is installed, the cone failure surface will be that of a simple cone. If two bolts are installed, the critical failure surface will be the minimum failure surface defined by two independent cones or a surface that accounts for the overlap of two cones. If the two bolts on the same side of the column are close enough relative to the bolt embedment length, then the combined surface will control. The column anchor bolts at the outside of the shear panel will always be more highly stressed than those at the inside. These bolts will be more critically loaded by uplift forces due to the direction of diagonal strap forces and moment in the column. A cone failure surface including bolts on both sides of the columns will never be more critical than the cone for bolts only at the outside of the shear panel. Therefore, only the cone with bolts on the outside are considered. The anchor bolt cone failure design strength, P_c (in pounds) is determined by[9]:

$$P_c = 4\phi_c \sqrt{f'_c} A_c$$ (Eq C-60)

Where:

ϕ_c = the cone strength reduction factor - a value of 0.85 for uncracked concrete.
f'_c = the specified concrete compressive strength in psi
A_c = the minimum of the area of a single anchor bolt stress cone (A_{c1}) or the summation of the combined failure surface for two overlapping stress cones divided by two (A_{c2}), in inches^2. The area of individual stress cones is dependent on the angle of cone failure. In the case of expansion anchors this angle varies from about 60 degrees (measure from the axis of the anchor) for short embedments ($l_{AB} \le 2$ inches) to 45 degrees for $l_{AB} \ge 6$ inches. In this guidance this angle will conservatively be set equal to 45 degrees. Then the radius of the cone at the concrete surface, r_c is equal to anchor embedment length, l_{AB}. The area of an individual stress cone failure surface, A_{c1} shall be calculated as follows:

$$A_{c1} = \pi r_c \sqrt{r_c^2 + l_{AB}^2} = \sqrt{2}\pi l_{AB}^2$$ (Eq C-61)

The area of the combined failure surface for two overlapping stress cones, divided by two for an individual anchor bolt, A_{c2} shall be calculated as follows:

$$A_{c2} = \frac{\pi r_c + 2d_{cc}}{2}\sqrt{r_c^2 + l_{AB}^2} = \frac{\sqrt{2}}{2} l_{AB}\left(\pi l_{AB} + 2d_{cc}\right)$$ (Eq C-62)

[9] ACI 355.1R-91, Equation 3.2.

The anchor bolts shall not be installed too close to the edge of a concrete beam or slab, or edge failure could occur before developing the cone strength. The minimum distance from the center of an anchor bolt to the edge of the concrete to prevent side cone failure, m (in inches) is determined as follows:[10]

$$m = d_{AB}\sqrt{\frac{F_t}{73\sqrt{f'_c}}} \qquad \text{(Eq C-63)}$$

[10] ACI 355.1R-91, Equation 3.3.