

# Steel-Concrete Composite Structural Members

## 13.1 General Provisions for Steel-Concrete Composite Structural Members

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This section states the scope of the specification, summarizes referenced specifications, codes and standard documents and provide requirements for materials for steel-concrete composite members. General provisions for composite sections and shear connectors are also included.

### 13.1.1 Scope

The guidelines included in chapter 13 of part 6 of this code presents the design guidelines for steel concrete composite members frequently used in medium to high rise buildings. This chapter mainly addresses composite columns composed of rolled or built-up structural steel shapes or HSS, and structural concrete acting together, and steel beams supporting a reinforced concrete slab so interconnected that the beams and the slab act together to resist bending. Simple and continuous composite beams with shear connectors and concrete-encased beams, constructed with or without temporary shores, are included. Seismic provisions for steel-concrete composite members are also provided.

### 13.1.2 Referenced Specifications, Codes and Standards

The documents referenced in these provisions shall include those listed in Part 6 Chapter 10 Section 2 with the following additions and modifications:

American Society of Civil Engineers

*ASCE 3-91 Standard for the Structural Design of Composite Slabs*

*American Welding Society*

*AWS D1.1-04 Structural Welding Code- Steel*

*AWS D1.4-98 Structural Welding Code-Reinforcing Steel*

Canadian Standards Association

*CSA S16-01 Design of Steel Structures*

### 13.1.3 Material Limitations

Concrete and steel reinforcing bars in *composite* systems shall be subject to the following limitations.

- a) For the determination of the *available strength*, concrete shall have a compressive strength  $f'_c$  of not less than 21 MPa nor more than 70 MPa for normal weight concrete and not less than 21 MPa nor more than 42 MPa for lightweight concrete.
- b) The *specified minimum yield stress* of structural steel and reinforcing bars used in calculating the strength of a *composite column* shall not exceed 525 MPa.

Higher material strengths are permitted when their use is justified by testing or analysis.

### 13.1.4 General Provisions

In determining *load effects* in members and *connections* of a structure that includes *composite* members, consideration shall be given to the effective sections at the time each increment of *load* is applied. The design, detailing and material properties related to the concrete and reinforcing steel portions of composite construction shall comply with the reinforced concrete and reinforcing bar design specifications stipulated by the provisions in Part 6 Chapter 5.

#### 13.1.4.1 Resistance Prior to Composite Action

The factored resistance of the steel member prior to the attainment of composite action shall be determined in accordance with Chapter 10 of Part 6.

#### 13.1.4.2 Nominal Strength of Composite Sections

Two methods are provided for determining the nominal strength of composite sections: the plastic stress distribution method and the strain-compatibility method.

The tensile strength of the concrete shall be neglected in the determination of the nominal strength of composite members.

##### 13.1.4.2.1 Plastic Stress Distribution Method

For the plastic stress distribution method, the nominal strength shall be computed assuming that steel components have reached a *stress* of  $F_y$  in either tension or compression and concrete components in compression have reached a *stress* of  $0.85 f'_c$ . For round *HSS* filled with concrete, a *stress* of  $0.95 f'_c$  is permitted to be used for concrete components in uniform compression to account for the effects of concrete confinement.

##### 13.1.4.2.2 Strain-Compatibility Method

For the strain compatibility method, a linear distribution of strains across the section shall be assumed, with the maximum concrete compressive strain equal to 0.003 mm/mm (in/in). The stress-strain relationships for steel and concrete shall be obtained from tests or from published results for similar materials.

##### 13.1.4.2.3 Shear Connectors

*Shear connectors* shall be headed steel studs not less than four stud diameters in length after installation, or hot-rolled steel channels. Shear stud design values shall be taken as per Sections 13.2.1.7 and 13.3.2.4. Stud connectors shall conform to the requirements of Section 13.3.2.4(3) Channel connectors shall conform to the requirements of Section 13.3.2.4(4).

## 13.2 Design of Composite Axial Members

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This section states the design guidelines for two types of composite axial members. These include— encased composite columns and concrete filled hollow structural sections.

### 13.2.1 Encased Composite Columns

#### 13.2.1.1 Scope

This section applies to doubly symmetric steel columns encased in concrete, provided that

- (1) the steel shape is a compact or non-compact section;
- (2) the cross-sectional area of the steel core comprises at least 1 percent of the total composite cross section.
- (3) concrete encasement of the steel core is reinforced with continuous longitudinal bars and lateral ties or spirals. The minimum *transverse reinforcement* shall be at least  $6 \text{ mm}^2$  per mm of tie spacing.
- (4) The minimum reinforcement ratio for continuous longitudinal reinforcing,

$\rho_{sr}$ , shall be 0.004, where  $\rho_{sr}$  is given by:

$$\rho_{sr} = \frac{A_{sr}}{A_g} \quad 13.2.1$$

Where

$A_{sr}$  = area of continuous reinforcing bars,  $\text{mm}^2$

$A_g$  = gross area of composite member,  $\text{mm}^2$

### 13.2.1.2 Compressive Strength

The *design compressive strength*,  $\phi_c P_n$ , and *allowable compressive strength*,  $P_n/\Omega_c$ , for axially loaded *encased composite columns* shall be determined for the limit state of *flexural buckling* based on column slenderness as follows:

$$\phi_c = 0.75 \text{ (LRFD)} \quad \Omega_c = 2.00 \text{ (ASD)}$$

(a) When  $P_e \geq 0.44P_o$

$$P_n = P_o \left[ 0.658 \left( \frac{P_o}{P_e} \right) \right] \quad 13.2.2$$

(b) When  $P_e < 0.44P_o$

$$P_n = 0.877P_e \quad 13.2.3$$

Where

$$P_o = A_s F_y + A_{sr} F_{yr} + 0.85 A_c f'_c \quad 13.2.4$$

$$P_e = \frac{\pi^2 (EI_{eff})}{(KL)^2} \quad 13.2.5$$

and where

$A_s$  = area of the steel section, mm<sup>2</sup>

$A_c$  = area of concrete, mm<sup>2</sup>

$A_{sr}$  = area of continuous reinforcing bars, mm<sup>2</sup>

$E_c$  = modulus of elasticity of concrete =  $0.043 w_c^{1.5} \sqrt{f'_c}$  MPa

$E_s$  = modulus of elasticity of steel = 210 MPa

$f'_c$  = specified compressive strength of concrete, MPa

$F_y$  = specified minimum yield stress of steel section, MPa

$F_{yr}$  = specified minimum yield stress of reinforcing bars, MPa

$I_c$  = moment of inertia of the concrete section, mm<sup>4</sup>

$I_s$  = moment of inertia of steel shape, mm<sup>4</sup>

$I_{sr}$  = moment of inertia of reinforcing bars, mm<sup>4</sup>

$K$  = the effective length factor determined in accordance with Chapter 10 Part 6

$L$  = laterally unbraced length of the member, mm

$w_c$  = weight of concrete per unit volume  $1500 \leq w_c \leq 2500$  kg/m<sup>3</sup>

Where

$EI_{eff}$  = effective stiffness of composite section, N-mm<sup>2</sup>

$$EI_{eff} = E_s I_s + 0.5 E_s I_{sr} + C_1 E_c I_c \quad 13.2.6$$

Where

$$C_1 = 0.1 + 2 \left( \frac{A_s}{A_c + A_s} \right) \leq 0.3 \quad 13.2.7$$

### 13.2.1.3 Tensile Strength

The *design tensile strength*,  $\phi_t P_n$ , and *allowable tensile strength*,  $P_n / \Omega_t$ , for encased composite columns shall be determined for the limit state of *yielding* as

$$P_n = A_s F_y + A_{sr} F_{yr} \quad 13.2.8$$

$$\phi_t = 0.90 \text{ (LRFD)} \quad \Omega_t = 1.67 \text{ (ASD)}$$

### 13.2.1.4 Shear Strength

The *available shear strength* shall be calculated based on either the shear strength of the steel section alone as specified in Section 10.7 plus the shear strength provided by tie reinforcement, if present, or the shear strength of the reinforced concrete portion alone.

### 13.2.1.5 Load Transfer

*Loads* applied to axially loaded encased composite columns shall be transferred between the steel and concrete in accordance with the following requirements:

- a) When the external *force* is applied directly to the steel section, *shear connectors* shall be provided to transfer the required shear force,  $V'$ , as follows:

$$V' = V \left( 1 - \frac{A_s F_y}{P_o} \right) \quad 13.2.9$$

Where

$V$  = required shear force introduced to *column*, N

$A_s$  = area of steel cross section, mm<sup>2</sup>

$P_o$  = nominal axial compressive strength without consideration of length effects, N

When the external force is applied directly to the concrete encasement, shear connectors shall be provided to transfer the required shear force,  $V'$ , as follows:

$$V' = V \left( \frac{A_s F_y}{P_o} \right) \quad 13.2.10$$

- b) When load is applied to the concrete of an encased composite column by direct bearing the *design bearing strength*,  $\phi_B P_p$ , and the *allowable bearing strength*,  $P_p / \Omega_B$ , of the concrete shall be:

$$P_p = 1.7 f' A_B \quad 13.2.11$$

Where

$$\phi_B = 0.65 \text{ (LRFD)} \quad \Omega_B = 2.31 \text{ (ASD)}$$

where

$A_B$  = loaded area of concrete, mm<sup>2</sup>

### 13.2.1.6 Detailing Requirements

#### 13.2.1.6.1 Longitudinal Bars

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

The longitudinal bars shall

- Be continuous at framed levels when considered to carry load;
- Have an area not less than 0.01 times the total gross cross-sectional area;
- Be located at each corner; and
- Spaced on all sides not further apart than 525t/fy times one-half the least dimension of the composite section.

### 13.2.1.6.2 Lateral ties

The lateral ties shall

- a) Be 15M bars, except that 10M bars may be used when no side dimension of the composite section exceeds 500 mm; and
- b) Have a vertical spacing not exceeding the least of the following:
  - i) Two-thirds of the least side dimension of the cross-section;
  - ii) 16 longitudinal bar diameters; or
  - iii) 500 mm.

Where required, shear connectors transferring the required shear force shall be distributed along the length of the member at least a distance of 2.5 times the width of a rectangular HSS or 2.5 times the diameter of a round HSS both above and below the load transfer region. The maximum connector spacing shall be 405 mm.

### 13.2.1.6.3 Shear Connectors

Shear connectors shall be provided to transfer the required shear force Section specified in 13.2.2.5. The shear connectors shall be distributed along the length of the member at least a distance of 2.5 times the depth of the encased composite column above and below the load transfer region. The maximum connector spacing shall be 405 mm. Connectors to transfer axial load shall be placed on at least two faces of the steel shape in a configuration symmetrical about the steel shape axes.

### 13.2.1.6.4 Columns with Multiple Built-up Shapes

If the composite cross section is built up from two or more encased steel shapes, the shapes shall be interconnected with *lacing*, *tie plates*, *batten plates* or similar components to prevent buckling of individual shapes due to loads applied prior to hardening of the concrete.

### 13.2.1.7 Strength of Stud Shear Connectors

The *nominal strength* of one stud shear connector embedded in solid concrete is:

$$Q_n = 0.5A_{sc}\sqrt{f'_c E_c} \leq A_{sc} F_u \quad 13.2.12$$

where

$A_{sc}$  = cross-sectional area of stud shear connector, mm<sup>2</sup>

$F_u$  = specified minimum tensile strength of a stud shear connector, MPa

## 13.2.2 Concrete Filled Hollow Structural Section

### 13.2.2.1 Scope

Section 13.2.2 applies to composite members consisting of steel hollow structural sections completely filled with concrete, provided that

- a) The cross-sectional area of the steel HSS shall comprise at least 1 percent of the total composite cross-section.
- b) The width-to thickness ratio of the walls of rectangular hollow structural sections does not exceed  $1350/\sqrt{F_y}$
- c) The outside diameter-to-thickness ration of circular hollow structural sections does not exceed  $28000/F_y$
- d) The concrete strength is between 20 and 80 MPa for axially loaded columns and between 20 and 40 MPa for columns subjected to axial compression and bending.

### 13.2.2.2 Compressive Strength

The *design compressive strength*,  $\phi_c P_n$  and *allowable compressive strength*,  $P_n/\Omega_c$ , for axially loaded filled composite columns shall be determined for the *limit state of flexural buckling* based on Section 13.2.1.2 with the following modifications:

$$P_o = A_s F_y + A_{sr} F_{yr} + C_2 A_c f'_c \quad 13.2.13$$

$C_2 = 0.85$  for rectangular sections and  $0.95$  for circular sections

$$EI_{eff} = E_s I_s + E_s I_{sr} + C_3 E_c I_c \quad 13.2.14$$

$$C_3 = 0.6 + 2 \left( \frac{A_s}{A_c + A_s} \right) \leq 0.9 \quad 13.2.15$$

### 13.2.2.3 Tensile Strength

The *design tensile strength*,  $\phi_t P_n$ , and allowable tensile strength,  $P_n / \Omega_t$ , for filled composite columns shall be determined for the limit state of *yielding* as:

$$P_n = A_s F_y + A_{sr} F_{yr} \quad 13.2.16$$

$$\phi_t = 0.90 \text{ (LRFD)} \quad \Omega_t = 1.67 \text{ (ASD)}$$

### 13.2.2.4 Shear Strength

The *available shear strength* shall be calculated based on either the shear strength of the steel section alone as specified in Chapter 10 or the shear strength of the reinforced concrete portion alone. The shear strength of reinforced concrete portion may be determined according to Chapter 6 of Part 6.

### 13.2.2.5 Load Transfer

Loads applied to filled composite columns shall be transferred between the steel and concrete. When the external force is applied either to the steel section or to the concrete infill, transfer of force from the steel section to the concrete core is required from *direct bond interaction*, *shear connection* or *direct bearing*. The force transfer *mechanism* providing the largest *nominal strength* may be used. These force transfer mechanisms shall not be superimposed.

When load is applied to the concrete of an encased or filled composite column by *direct bearing* the *design bearing strength*,  $\phi_B P_p$ , and the *allowable bearing strength*,  $P_p / \Omega_B$ , of the concrete shall be:

$$P_p = 1.7 f'_c A_E \quad 13.2.17$$

$$\phi_B = 0.65 \text{ (LRFD)} \quad \Omega_B = 2.31 \text{ (ASD)}$$

where

$A_B$  is the loaded area,  $\text{mm}^2$

### 13.2.2.6 Detailing Requirements

Where required, shear connectors transferring the required shear force shall be distributed along the length of the member at least a distance of 2.5 times the width of a rectangular HSS or 2.5 times the diameter of a round HSS both above and below the load transfer region. The maximum connector spacing shall be 405 mm.

## 13.3 Design of Composite Flexural Members

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This section applies to composite beams consisting of steel sections interconnected with either a reinforced concrete slab or a steel deck with a concrete cover slab. The steel *beams* and the reinforced concrete slab are so interconnected that the beams and the slab act together to resist bending. Simple and continuous *composite beams* with *shear connectors* and *concrete-encased beams*, constructed with or without temporary shores, are included. Design philosophy for composite columns subjected to bending moments is also stated.

### 13.3.1 General

#### 13.3.1.1 Deflections

Calculation of deflections shall take into account the effects of creep of concrete, shrinkage of concrete, and increased flexibility resulting from partial shear connection and from interfacial slip. These effects shall be established by test or analysis, where practicable. Consideration shall also be given to the effects of full or partial continuity in the steel beams and concrete slabs in reducing calculated deflections.

In lieu of tests or analysis, the effects of partial shear connection and interfacial slip, creep, and shrinkage may be assessed as follows:

- a) For increased flexibility resulting from partial shear connection and interfacial slip, the deflections shall be calculated using an effective moment of inertia given by

$$I_e = I_s + 0.85p^{0.25}(I_t - I_s) \quad 13.3.1$$

Where

$I_s$  = moment of inertia of a steel beam, or of a steel joist or truss adjusted to include the effect of shear deformations, which may be taken into account by decreasing the moment of inertia based on the cross-sectional areas of the top and bottom chords by 15% or by a more detailed analysis

$P$  = fraction of full shear connection

=1.00 for full shear connection

$I_t$  = transformed moment of inertia of composite beam based on the modular ration  $n=E/E_c$

- b) For creep, elastic deflections caused by dead loads and long-term live loads, as calculated in Item (a), need to be increased by 15% and
- c) For shrinkage of concrete, using a selected free shrinkage strain, strain compatibility between the steel and concrete, and an age-adjusted effective modulus of elasticity of concrete as it shrinks and creeps, the deflection of a simply supported composite beam, joist, or truss shall be calculated as follows:

$$\Delta_s = \frac{L^2}{8} \psi = \frac{L^2}{8} c \frac{\epsilon_f A_c y}{n_s I_{eS}} \quad 13.3.2$$

Where

$L$  = span of the beam, joist, or truss

$\psi$  = curvature along length of the beam, joist, or truss due to shrinkage of concrete

$c$  = empirical coefficient used to match theory with test results (accounting for cracking of concrete in tension, the non-linear stress-strain relationship of concrete, and other factors)

= free shrinkage strain of concrete

$A_c$  = effective area of concrete slab

$Y$  = distance from centroid of effective of effective area of concrete slab to centroidal axis of the composite beam, joist, or truss

$n_s$  = modular ration,  $E/E'_c$

where

$$E'_c = E_c / (1 + \chi\phi) \quad 13.3.3$$

= age-adjusted effective modulus of elasticity of concrete

Where

$\chi$  = aging coefficient of concrete

$\phi$  = creep coefficient of concrete

$$I_{eS} = I_s + 0.85p^{0.25}(I_{tS} - I_s) \quad 13.3.4$$

= effective moment of inertia of composite beam, truss, or joist based on the modular ratio  $n_s$

Where

$I_{ts}$  = transformed moment of inertia based on the modular ratio  $n_s$

### 13.3.1.2 Design Effective Width of Concrete

The *effective width* of the concrete slab is the sum of the effective widths for each side of the beam centerline, each of which shall not exceed:

- one-eighth of the beam span, center-to-center of supports;
- one-half the distance to the centerline of the adjacent beam; or
- the distance to the edge of the slab.

### 13.3.1.3 Shear Strength

The available shear strength of *composite beams with shear connectors* shall be determined based upon the properties of the steel section alone in accordance with Part 6 Chapter 10 Section 10.7. The available shear strength of concrete-encased and filled composite members shall be determined based upon the properties of the steel section alone in accordance with Part 6 Chapter 10 Section 10.7 or based upon the properties of the concrete and longitudinal steel reinforcement.

### 13.3.1.4 Strength during Construction

When temporary shores are not used during construction, the steel section alone shall have adequate strength to support all *loads* applied prior to the concrete attaining 75 percent of its specified strength  $f'_c$ . The available flexural strength of the steel section shall be determined according to Part 6 Chapter 10 Section 10.6.

## 13.3.2 Strength of Composite Beams with Shear Connectors

### 13.3.2.1 Positive Flexural Strength

The design positive flexural strength,  $\phi_b M_n$ , and the allowable positive flexural strength,  $M_n / \Omega_b$ , shall be determined for the limit state of yielding as follows:

$$\phi_b = 0.90 \text{ (LRFD)} \quad \Omega_b = 1.67 \text{ (ASD)}$$

a) For  $h/t_w \leq 3.76 \sqrt{E/F_y}$

$M_n$  shall be determined from the plastic *stress* distribution on the *composite* section for the limit state of *yielding (plastic moment)*.

b) For  $h/t_w > 3.76 \sqrt{E/F_y}$

$M_n$  shall be determined from the superposition of elastic stresses, considering the effects of shoring, for the limit state of *yielding (yield moment)*.

### 13.3.2.2 Negative Flexural Strength

The *design negative flexural strength*,  $\phi_b M_n$ , and the *allowable negative flexural strength*,  $M_n / \Omega_b$ , shall be determined for the steel section alone, in accordance with the requirements of Part 6 Chapter 10 Section 10.6.

Alternatively, the available negative flexural strength shall be determined from the plastic stress distribution on the composite section, for the limit state of yielding (plastic moment), with

$$\phi_b = 0.90 \text{ (LRFD)} \quad \Omega_b = 1.67 \text{ (ASD)}$$

provided that:

- The steel *beam* is compact and is adequately braced according to Section 10.6.
- Shear connectors* connect the slab to the steel beam in the negative moment region.
- The slab reinforcement parallel to the steel beam, within the *effective width* of the slab, is *properly developed*.



### 13.3.2.3 Strength of Composite Beams with Formed Steel Deck

#### (1) General

The *available flexural strength* of composite construction consisting of concrete slabs on *formed steel deck* connected to steel beams shall be determined by the applicable portions of Section 13.3.2.1 and 13.3.2.2, with the following requirements:

- This section is applicable to decks with *nominal rib height* not greater than 75 mm. The average width of concrete rib or haunch,  $w_r$ , shall be not less than 50 mm, but shall not be taken in calculations as more than the minimum clear width near the top of the steel deck.
- The concrete slab shall be connected to the steel beam with welded stud shear connectors 19 mm or less in diameter (AWS D1.1). Studs shall be welded either through the deck or directly to the steel cross section. Stud shear connectors, after installation, shall extend not less than 38 mm above the top of the steel deck and there shall be at least 13 mm of concrete cover above the top of the installed studs.
- The slab thickness above the steel deck shall be not less than 50 mm.
- Steel deck shall be anchored to all supporting members at a spacing not to exceed 460 mm. Such anchorage shall be provided by stud connectors, a combination of stud connectors and arc spot (puddle) welds, or other devices specified by the designer.

#### (2) Deck Ribs Oriented Perpendicular to Steel Beam

Concrete below the top of the steel deck shall be neglected in determining composite section properties and in calculating  $A_c$  for deck ribs oriented perpendicular to the steel beams.

#### (3) Deck Ribs Oriented Parallel to Steel Beam

Concrete below the top of the steel deck may be included in determining composite section properties and shall be included in calculating  $A_c$ .

Formed steel deck ribs over supporting beams may be split longitudinally and separated to form a *concrete haunch*.

When the nominal depth of steel deck is 38 mm or greater, the average width,  $w_r$ , of the supported haunch or rib shall be not less than 50 mm for the first stud in the transverse row plus four stud diameters for each additional stud.

### 13.3.2.4 Shear Connectors

#### (1) Load Transfer for Positive Moment

The entire *horizontal shear* at the interface between the steel *beam* and the concrete slab shall be assumed to be transferred by shear connectors, except for *concrete-encased beams* as defined in Section 10.9.3.3. For *composite* action with concrete subject to flexural compression, the total horizontal shear force,  $V'$ , between the point of maximum positive moment and the point of zero moment shall be taken as the lowest value according to the *limit states* of *concrete crushing*, *tensile yielding* of the steel section, or strength of the shear connectors:

Concrete crushing

$$V' = 0.85 f'_c A_c \quad 13.3.5a$$

Tensile yielding of the steel section

$$V' = F_y A_s \quad 13.3.5b$$

Strength of shear connectors

$$V' = \sum Q_n \quad 13.3.5c$$

where

$A_c$  = area of concrete slab within *effective width*, mm<sup>2</sup>

$A_s$  = area of steel cross section, mm<sup>2</sup>

$\sum Q_n$  = sum of *nominal strengths* of shear connectors between the point of maximum positive moment and the point of zero moment, N

### (2) Load Transfer for Negative Moment

In continuous *composite beams* where longitudinal reinforcing steel in the negative moment regions is considered to act compositely with the steel *beam*, the total *horizontal shear force* between the point of maximum negative moment and the point of zero moment shall be taken as the lower value according to the limit states of *yielding* of the steel reinforcement in the slab, or strength of the shear connectors:

- a) Tensile yielding of the slab reinforcement

$$V' = A_r F_{yr} \quad 13.3.6a$$

where

$A_r$  = area of adequately developed longitudinal reinforcing steel within the *effective width* of the concrete slab, mm<sup>2</sup>

$F_{yr}$  = *specified minimum yield stress* of the reinforcing steel, MPa

- b) Strength of shear connectors

$$V' = \sum Q_n \quad 13.3.6b$$

### (3) Strength of Stud Shear Connectors

The *nominal strength* of one stud shear connector embedded in solid concrete or in a *composite slab* is

$$Q_n = 0.5 A_{sc} \sqrt{f'_c E_c} \leq R_g R_p A_{sc} F_u \quad 13.3.7$$

where

$A_{sc}$  = cross-sectional area of stud *shear connector*, mm<sup>2</sup>

$E_c$  = modulus of elasticity of concrete =  $0.043 w_c^{1.5} \sqrt{f'_c}$ , MPa

$F_u$  = *specified minimum tensile strength* of a stud shear connector

$R_g$  = 1.0; (a) for one stud welded in a steel deck rib with the deck oriented perpendicular to the steel shape; (b) for any number of studs welded in a row directly to the steel shape; (c) for any number of studs welded in a row through steel deck with the deck oriented parallel to the steel shape and the ratio of the *average rib width* to rib depth  $\geq 1.5$

= 0.85; (a) for two studs welded in a steel deck rib with the deck oriented perpendicular to the steel shape; (b) for one stud welded through steel deck with the deck oriented parallel to the steel shape and the ratio of the average rib width to rib depth  $< 1.5$

= 0.7 for three or more studs welded in a steel deck rib with the deck oriented perpendicular to the steel shape

$R_p$  = 1.0 for studs welded directly to the steel shape (in other words, not through steel deck or sheet) and having a haunch detail with not more than 50 percent of the top flange covered by deck or sheet steel closures

= 0.75; (a) for studs welded in a composite slab with the deck oriented perpendicular to the beam and  $e_{mid-ht} \geq 50$  mm; (b) for studs welded through steel deck, or steel sheet used as *girder filler* material and embedded in a *composite slab* with the deck oriented parallel to the *beam*

= 0.6 for studs welded in a composite slab with deck oriented perpendicular to the beam and  $e_{mid-ht} < 50$  mm

$e_{mid-ht}$  = distance from the edge of stud shank to the steel deck web, measured at mid-height of the deck rib, and in the *load bearing* direction of the stud (in other words, in the direction of maximum moment for a simply supported beam), mm

$w_c$  = weight of concrete per unit volume ( $1500 \leq w_c \leq 2500$  kg/m<sup>3</sup>)

#### (4) Strength of Channel Shear Connectors

The nominal strength of one channel shear connector embedded in a solid concrete slab is

$$Q_n = 0.3(t_f + 0.5t_w)L_c\sqrt{f'_c E_c} \quad 13.3.8$$

where

$t_f$  = flange thickness of channel shear connector, mm

$t_w$  = web thickness of channel shear connector, mm

$L_c$  = length of channel shear connector, mm

The strength of the channel shear connector shall be developed by welding the channel to the beam flange for a force equal to  $Q_n$ , considering eccentricity on the connector.

#### (5) Required Number of Shear Connectors

The number of shear connectors required between the section of maximum bending moment, positive or negative, and the adjacent section of zero moment shall be equal to the *horizontal shear force* as determined in Sections 10.9.3.2d(1) and 10.9.3.2d(2) divided by the nominal strength of one shear connector as determined from Section 10.9.3.2d(3) or Section 10.9.3.2d(4).

#### (6) Shear Connector Placement and Spacing

Shear connectors required on each side of the point of maximum bending moment, positive or negative, shall be distributed uniformly between that point and the adjacent points of zero moment, unless otherwise specified. However, the number of shear connectors placed between any *concentrated load* and the nearest point of zero moment shall be sufficient to develop the maximum moment required at the *concentrated load* point.

Shear connectors shall have at least 25 mm of lateral concrete cover, except for connectors installed in the ribs of *formed steel decks*. The diameter of studs shall not be greater than 2.5 times the thickness of the flange to which they are welded, unless located over the web. The minimum center-to-center spacing of stud connectors shall be six diameters along the longitudinal axis of the supporting *composite beam* and four diameters transverse to the longitudinal axis of the supporting composite beam, except that within the ribs of formed steel decks oriented perpendicular to the steel beam the minimum center-to-center spacing shall be four diameters in any direction. The maximum center-to-center spacing of shear connectors shall not exceed eight times the total slab thickness nor 900 mm.

### 13.3.3 Slab Reinforcement

#### 13.3.3.1 General

Slabs shall be adequately reinforced to support all loads and to control both cracking transverse to the composite beam span and longitudinal cracking over the steel section. Reinforcement shall not be less than that required by the specified fire-resistance design of the assembly.

#### 13.3.3.2 Parallel reinforcement

Reinforcement parallel to the span of the beam in regions of negative bending moment of the composite beam shall be anchored by embedment in concrete that is in compression. The reinforcement of slabs that are to be continuous over the end support of steel sections or joists fitted with flexible end connections shall be given special attention. Reinforcement at the ends of beams supporting ribbed slabs perpendicular to the beam shall be not less than two 15M bars or equivalent.

#### 13.3.3.3 Transverse reinforcement-concrete slab on metal deck

Unless it is known from experience that longitudinal cracking caused by composite action directly over the steel section is unlikely, additional transverse reinforcement or other effective means shall be provided. Such additional reinforcement shall be placed in the lower part of the slab and anchored so as to develop the yield strength of the reinforcement. The area of such reinforcement shall be not less than 0.002 times the concrete area being reinforced and shall be uniformly distributed.

#### 13.3.3.4 Transverse reinforcement- Ribbed slabs

- a) Where the ribs are parallel to the beam span, the area of transverse reinforcement shall be not less than 0.002 times the concrete cover slab area being reinforced and shall be uniformly distributed.
- b) Where the ribs are perpendicular to the beam span, the area of transverse reinforcement shall be not less than 0.001 times the concrete cover slab area being reinforced and shall be uniformly distributed.

### 13.3.4 Flexural Strength of Concrete-Encased and Filled Members

The *nominal flexural strength* of concrete-encased and filled members shall be determined using one of the following methods:

- a) The superposition of elastic *stresses* on the composite section, considering the effects of shoring, for the *limit state of yielding (yield moment)*, where

$$\phi_b = 0.90 \text{ (LRFD)} \quad \Omega_b = 1.67 \text{ (ASD)}$$

- b) The plastic stress distribution on the steel section alone, for the *limit state of yielding (plastic moment)*, where

$$\phi_b = 0.90 \text{ (LRFD)} \quad \Omega_b = 1.67 \text{ (ASD)}$$

- c) If *shear connectors* are provided and the concrete meets the requirements of Section 10.9.1.2, the nominal flexural strength shall be computed based upon the plastic stress distribution on the composite section or from the strain compatibility method, where

$$\phi_b = 0.85 \text{ (LRFD)} \quad \Omega_b = 1.76 \text{ (ASD)}$$

### 13.3.5 Combined Axial Force and Flexure

The interaction between axial forces and flexure in composite members shall account for stability as required by Chapter C. The *design compressive strength*,  $\phi_c P_n$ , and *allowable compressive strength*,  $P_n / \Omega_c$  and the *design flexural strength*,  $\phi_b M_n$  and *allowable flexural strength*,  $M_n / \Omega_b$ , are determined as follows:

$$\phi_c = 0.75 \text{ (LRFD)} \quad \Omega_c = 2.00 \text{ (ASD)}$$

$$\phi_b = 0.90 \text{ (LRFD)} \quad \Omega_b = 1.67 \text{ (ASD)}$$

(1) The *nominal strength* of the cross section of a *composite* member subjected to combined axial compression and flexure shall be determined using either the *plastic stress distribution method* or the *strain-compatibility method*.

(2) To account for the influence of length effects on the axial strength of the member, the nominal axial strength of the member shall be determined by Section 10.9 with  $P_o$  taken as the nominal axial strength of the cross section determined in Section 10.9.4(13) above.

### 13.3.6 Special Cases

When *composite* construction does not conform to the requirements of Section 13.2 and Section 13.3, the strength of *shear connectors* and details of construction shall be established by testing.

## 13.4 Composite Connections

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This Section is applicable to connections in buildings that utilize composite or dual steel and concrete systems. Composite connections shall be demonstrated to have Design Strength, ductility and toughness that is comparable to that exhibited by similar structural steel or reinforced concrete connections that meet the requirements in Part 6 Chapter 10 and Chapter 5, respectively. Methods for calculating the connection strength shall meet the requirements in this Section.

### 13.4.1 General

Connections shall have adequate deformation capacity to resist the critical Required Strengths at the Design Story Drift. Additionally, connections that are required for the lateral stability of the building under seismic forces shall meet the requirements in Section 13.5 based upon the specific system in which the connection is used. When the Required Strength is based upon nominal material strengths and nominal member dimensions, the determination of the required connection strength shall account for any effects that result from the increase in the actual Nominal Strength of the connected member.

### 13.4.2 Nominal Strength of Connections

The Nominal Strength of connections in composite Structural Systems shall be determined on the basis of rational models that satisfy both equilibrium of internal forces and the strength limitation of component materials and elements based upon potential limit states. Unless the connection strength is determined by analysis and testing, the models used for analysis of connections shall meet the following requirements:

(1) When required, force shall be transferred between structural steel and reinforced concrete through direct bearing of headed shear studs or suitable alternative devices, by other mechanical means, by shear friction with the necessary clamping force provided by reinforcement normal to the plane of shear transfer, or by a combination of these means. Any potential bond strength between structural steel and reinforced concrete shall be ignored for the purpose of the connection force transfer mechanism.

(2) The nominal bearing and shear-friction strengths shall meet the requirements in Part 6 Chapter 6 and 10, except that the strength reduction (resistance) factors shall be as given in Part 6 Chapter 6. Unless a higher strength is substantiated by cyclic testing, the nominal bearing and shear-friction strengths shall be reduced by 25 percent for the composite seismic systems.

(3) The Design Strengths of structural steel components in composite connections, as determined in Section 13.2 and Section 13.3 and the LRFD Specification, shall equal or exceed the Required Strengths. Structural steel elements that are encased in confined reinforced concrete are permitted to be considered to be braced against out of plane buckling. Face Bearing Plates consisting of stiffeners between the flanges of steel beams are required when beams are embedded in reinforced concrete columns or walls.

(4) The nominal shear strength of reinforced-concrete-encased steel Panel Zones in beam-to-column connections shall be calculated as the sum of the Nominal Strengths of the structural steel and confined reinforced concrete shear elements as determined in Part 6 Chapter 10 and Part 6 Chapter 5, respectively. The strength reduction (resistance) factors for reinforced concrete shall be as given in Part 6 Chapter 6.

(5) Reinforcement shall be provided to resist all tensile forces in reinforced concrete components of the connections. Additionally, the concrete shall be confined with transverse reinforcement. All reinforcement shall be fully developed in tension or compression, as appropriate, beyond the point at which it is no longer required to resist the forces. Development lengths shall be determined in accordance with Part 6 Chapter 6. Connections shall meet the following additional requirements:

(a) When the slab transfers horizontal diaphragm forces, the slab reinforcement shall be designed and anchored to carry the in-plane tensile forces at all critical sections in the slab, including connections to collector beams, columns, braces and walls.

(b) For connections between structural steel or Composite Beams and reinforced concrete or Reinforced-Concrete-Encased Composite Columns, transverse hoop reinforcement shall be provided in the connection region to meet the requirements in Chapter 6 of Part 6 except for the following modifications:

(i) Structural steel sections framing into the connections are considered to provide confinement over a width equal to that of face bearing stiffener plates welded to the beams between the flanges.

(ii) Lap splices are permitted for perimeter ties when confinement of the splice is provided by Face Bearing Plates or other means that prevents spalling of the concrete cover.

(c) The longitudinal bar sizes and layout in reinforced concrete and Composite Columns shall be detailed to minimize slippage of the bars through the beam-to-column connection due to high force transfer associated with the change in column moments over the height of the connection.

## 13.5 Seismic Provisions for Composite Structural Systems

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These Provisions are intended for the design and construction of composite structural steel and reinforced concrete members and connections in the Seismic Load Resisting Systems in buildings for which the design forces resulting from earthquake motions have been determined on the basis of various levels of energy dissipation in the inelastic range of response.

### 13.5.1 Scope

Provisions shall be applied in conjunction with the AISC Load and Resistance Factor Design (LRFD) Specification for Structural Steel Buildings, hereinafter referred to as the LRFD Specification. All members and connections in the Seismic Load Resisting System shall have a Design Strength as required in the LRFD Specification and shall meet the requirements in these Provisions. The applicable requirements in Part 6 Chapter 10 shall be used for the design of structural steel components in composite systems. Reinforced-concrete members subjected to seismic forces shall meet the requirements in Chapter 5 and 10 of Part 6 except as modified in these provisions. When the design is based upon elastic analysis, the stiffness properties of the component members of composite systems shall reflect their condition at the onset of significant yielding of the building.

### 13.5.2 Seismic Design Categories

The Required Strength and other seismic provisions for Seismic Design Categories, Seismic Use Groups or Seismic Zones and the limitations on height and irregularity shall be as stipulated in the Part 6 Chapter 10.

### 13.5.3 Loads, Load Combinations, and Nominal Strengths

The loads and load combinations shall be as stipulated by the Applicable Building Code. Where Amplified Seismic Loads are required by these provisions, the horizontal earthquake load  $E$  (as defined in Part 6 Chapter 10) shall be multiplied by the over strength factor  $\Omega_o$  prescribed by the Part 6 Chapter 10.

### 13.5.4 Materials

#### 13.5.4.1 Structural Steel

Structural steel used in composite Seismic Load Resisting Systems shall meet the requirements in Section 10.20 of Part 6 in addition Section 13.1 of Part 6. The structural steels that are explicitly permitted for use in seismic design have been selected based upon their inelastic properties and weld ability. In general, they meet the following characteristics: (1) a ratio of yield stress to tensile stress not greater than 0.85; (2) a pronounced stress-strain plateau at the yield stress; (3) a large inelastic strain capability (for example, tensile elongation of 20 percent or greater in a 2-in. (50 mm) gage length); and (4) good weldability. Other steels should not be used without evidence that the above criteria are met.

#### 13.5.4.2 Concrete and Steel Reinforcement

Concrete and steel reinforcement used in composite Seismic Load Resisting Systems shall meet the requirements in Part 6 Chapter 5, and the following requirements:

(1) The specified minimum compressive strength of concrete in composite members shall equal or exceed 2.5 ksi (17 MPa).

(2) For the purposes of determining the Nominal Strength of composite members,  $f'_c$  shall not be taken as greater than 10 ksi (69 MPa) for normal-weight concrete nor 4 ksi (28 MPa) for lightweight concrete.

Concrete and steel reinforcement used in the composite Seismic Load Resisting Systems described shall also meet the requirements in Part 6 Chapter 6.

### 13.5.5 Composite Members

#### 13.5.5.1 Composite Floor and Roof Slabs

The design of composite floor and roof slabs shall meet the requirements of ASCE 3-91. Composite slab diaphragms shall meet the requirements in this Section.

Details shall be designed to transfer forces between the diaphragm and Boundary Members, Collector Elements, and elements of the horizontal framing system.

The nominal shear strength of composite diaphragms and concrete-filled steel deck diaphragms shall be taken as the nominal shear strength of the reinforced concrete above the top of the steel deck ribs in accordance with Part 6 Chapter 5. Alternatively, the composite diaphragm design shear strength shall be determined by in-plane shear tests of concrete-filled diaphragms.

### 13.5.5.2 Composite Beams

Composite Beams shall meet the requirements in Section 13.3. Composite Beams that are part of C-SMF shall also meet the following requirements:

(1) The distance from the maximum concrete compression fiber to the plastic neutral axis shall not exceed:

$$\frac{Y_{con} + d_b}{1 + \left( \frac{1700F_y}{E_s} \right)} \quad 13.5.1$$

where

$Y_{con}$  = distance from the top of the steel beam to the top of concrete, mm

$d_b$  = depth of the steel beam, mm

$F_y$  = specified minimum yield strength of the steel beam, MPa

$E_s$  = modulus of elasticity of the steel beam, MPa

(2) Beam flanges shall meet the requirements in Part 6 Section 10.20.9.4.2, except when fully reinforced-concrete-encased compression elements have a reinforced concrete cover of at least 2 in. (50 mm) and confinement is provided by hoop reinforcement in regions where plastic hinges are expected to occur under seismic deformations. Hoop reinforcement shall meet the requirements in Part 6 Chapter 6.

### 13.5.5.3 Reinforced Concrete Encased Composite Columns

This Section is applicable to columns that: (1) consist of reinforced-concrete encased structural steel sections with a structural steel area that comprises at least 4 percent of the total composite-column cross-section; and (2) meet the additional limitations in Section 13.2.2.1. Such columns shall meet the requirements in Section 13.2.2, except as modified in this Section. Additional requirements, as specified for intermediate and special seismic systems in Sections 13.5.5.3.2 and 13.5.5.3.3, shall apply as required.

Columns that consist of reinforced-concrete-encased structural steel sections with a structural steel area that comprises less than 4 percent of the total composite column cross-section shall meet the requirements for reinforced concrete columns in Part 6 Chapter 5 except as modified for:

(1) The steel shape shear connectors in Section 13.5.4.3.1 (2).

(2) The contribution of the reinforced-concrete-encased structural steel section to the strength of the column as provided in Part 6 Chapter 6.

(3) The seismic requirements for reinforced concrete columns as specified in the description of the composite seismic systems in Sections 13.5.5.3.1 through 13.5.5.3.3.

#### 13.5.5.3.1 Ordinary Seismic System Requirements

The following requirements for Reinforced-Concrete-Encased Composite Columns are applicable to all composite systems:

(1) The nominal shear strength of the column shall be determined as the nominal shear strength of the structural shape plus the nominal shear strength that is provided by the tie reinforcement in the reinforced-concrete encasement. The nominal shear strength of the structural steel section shall be determined in accordance with Section 10.20 of Chapter 6. The nominal shear strength of the tie reinforcement shall be determined in accordance with Part 6 Chapter 5. In Part 6 Chapter 5, the dimension  $b_w$  shall equal the width of the concrete cross-section minus the width of the structural shape measured perpendicular to the direction of shear. The nominal shear strength shall be multiplied by  $\phi_v$  equal to 0.75 to determine the design shear strength.

(2) Composite Columns that are designed to share the applied loads between the structural steel section and reinforced concrete shall have shear connectors that meet the following requirements:

(a) If an external member is framed directly to the structural steel section to transfer a vertical reaction  $V_u$ , shear connectors shall be provided to transfer the force  $V_u(1 - A_s F_y / P_n)$  between the structural steel section and the reinforced concrete, where  $A_s$  is the area of the structural steel section,  $F_y$  is the specified minimum yield strength of the structural steel section, and  $P_n$  is the nominal compressive strength of the Composite Column.

(b) If an external member is framed directly to the reinforced concrete to transfer a vertical reaction  $V_u$ , shear connectors shall be provided to transfer the force  $V_u A_s F_y / P_n$  between the structural steel section and the reinforced concrete, where  $A_s$ ,  $F_y$  and  $P_n$  are as defined above.

(c) The maximum spacing of shear connectors shall be 16 in. (406 mm) with attachment along the outside flange faces of the embedded shape.

(3) The maximum spacing of transverse ties shall be the least of the following:

(a) one-half the least dimension of the section

(b) 16 longitudinal bar diameters

(c) 48 tie diameters

Transverse ties shall be located vertically within one-half the tie spacing above the top of the footing or lowest beam or slab in any story and shall be spaced as provided herein within one-half the tie spacing below the lowest beam or slab framing into the column.

Transverse bars shall have a diameter that is not less than one-fiftieth of greatest side dimension of the composite member, except that ties shall not be smaller than No. 3 bars and need not be larger than No. 5 bars. Alternatively, welded wire fabric of equivalent area is permitted as transverse reinforcement except when prohibited for intermediate and special systems.

(4) All Load-Carrying Reinforcement shall meet the detailing and splice requirements in Part 6 Chapter 5. Load-Carrying Reinforcement shall be provided at every corner of a rectangular cross-section. The maximum spacing of other load carrying or restraining longitudinal reinforcement shall be one-half of the least side dimension of the composite member.

(5) Splices and end bearing details for reinforced-concrete-encased structural steel sections shall meet the requirements in Chapter 5 of Part 6. If adverse behavioral effects due to the abrupt change in member stiffness and nominal tensile strength occur when reinforced-concrete encasement of a structural steel section is terminated, either at a transition to a pure reinforced concrete column or at the Column Base, they shall be considered in the design.

#### 13.5.5.3.2 Intermediate Seismic System Requirements

Reinforced-Concrete-Encased Composite Columns in intermediate seismic systems shall meet the following requirements in addition to those in Section 13.5.5.3.1:

(1) The maximum spacing of transverse bars at the top and bottom shall be the least of the following:

a) one-half the least dimension of the section

b) 8 longitudinal bar diameters

c) 24 tie bar diameters

d) 12 in. (305 mm)

These spacings shall be maintained over a vertical distance equal to the greatest of the following lengths, measured from each joint face and on both sides of any section where flexural yielding is expected to occur:

a) one-sixth the vertical clear height of the column

b) the maximum cross-sectional dimension

c) 18 in. (457 mm)

(2) Tie spacing over the remaining column length shall not exceed twice the spacing defined above.

(3) Welded wire fabric is not permitted as transverse reinforcement in intermediate seismic systems.

#### 13.5.5.3.3 Special Seismic System Requirements

Reinforced-concrete-encased columns for special seismic systems shall meet the following requirements in addition to those in Sections 13.5.4.3.2 and Sections 13.5.4.4.3:

(1) The required axial strength for Reinforced-Concrete-Encased Composite Columns and splice details shall meet the requirements in Section 13.2.



(2) Longitudinal Load-Carrying Reinforcement shall meet the requirements in Part 6 Chapter 6.  
 (3) Transverse reinforcement shall be hoop reinforcement as defined in Part 6 Chapter 6 and shall meet the following requirements:

a) The minimum area of tie reinforcement  $A_{sh}$  shall meet the following requirement:

$$A_{sh} = 0.09h_{cc}s \left( 1 - \frac{F_y A_s}{P_n} \right) \left( \frac{f'_c}{F_{yh}} \right) \quad 13.5.2$$

where

$h_{cc}$  = cross-sectional dimension of the confined core measured center-to-center of the tie reinforcement, *mm*

$s$  = spacing of transverse reinforcement measured along the longitudinal axis of the structural member, *mm*

$F_y$  = specified minimum yield strength of the structural steel core, *MPa*

$A_s$  = cross-sectional area of the structural core, *mm<sup>2</sup>*

$P_n$  = nominal axial compressive strength of the Composite Column calculated in accordance with the LRFD Specification, *N*

$f'_c$  = specified compressive strength of concrete, *MPa*

$F_{yh}$  = specified minimum yield strength of the ties, *MPa*

Equation 13.5.2 need not be satisfied if the Nominal Strength of the reinforced concrete encased structural steel section alone is greater than 1.0D + 0.5L.

- b) The maximum spacing of transverse reinforcement along the length of the column shall be the lesser of 6 longitudinal load-carrying bar diameters and 152 mm (6 in.).  
 c) When specified in Sections 13.5.5.3.3 (4), (5) or (6), the maximum spacing of transverse reinforcement shall be the lesser of one-fourth the least member dimension and 102 mm (4 in.). For this reinforcement, cross ties, legs of overlapping hoops, and other confining reinforcement shall be spaced not more than 355 mm (14 in.) on center in the transverse direction.

(4) Reinforced-Concrete-Encased Composite Columns in Braced Frames with axial compression forces that are larger than 0.2 times  $P_o$  shall have transverse reinforcement as specified in Section 13.5.5.3.3(3), over the total element length. This requirement need not be satisfied if the Nominal Strength of the reinforced-concrete-encased steel section alone is greater than 1.0D + 0.5L.

(5) Composite Columns supporting reactions from discontinued stiff members, such as walls or Braced Frames, shall have transverse reinforcement as specified in Section 13.5.5.3.3(3)(c) over the full length beneath the level at which the discontinuity occurs if the axial compression force exceeds 0.1 times  $P_o$ . Transverse reinforcement shall extend into the discontinued member for at least the length required to develop full yielding in the reinforced-concrete-encased structural steel section and longitudinal reinforcement. This requirement need not be satisfied if the Nominal Strength of the reinforced-concrete-encased structural steel section alone is greater than 1.0D + 0.5L.

(6) Reinforced-Concrete-Encased Composite Columns that are used in C-SMF shall meet the following requirements:

- a) Transverse reinforcement shall meet the requirements in 13.5.5.3.3(3)(c) at the top and bottom of the column over the region specified in Section 6.4b.  
 b) The strong-column/weak-beam design requirements in shall be satisfied. Column Bases shall be detailed to sustain inelastic flexural hinging.  
 c) The minimum required shear strength of the column shall meet the requirements in Part 6 Chapter 5.

(7) When the column terminates on a footing or mat foundation, the transverse reinforcement as specified in this section shall extend into the footing or mat at least 305 mm (12 in.). When the column terminates on a wall, the transverse reinforcement shall extend into the wall for at least the length required to develop full yielding in the reinforced-concrete-encased structural steel section and longitudinal reinforcement.

(8) Welded wire fabric is not permitted as transverse reinforcement for special seismic systems.

#### 13.5.5.4 Concrete Filled Composite Columns

This Section is applicable to columns that: (1) consist of concrete-filled steel rectangular or circular hollow structural sections (HSS) with a structural steel area that comprises at least 4 percent of the total composite-column cross-section; and (2) meet the additional limitations in Section 13.2. Such columns shall be designed to meet the requirements in Section 13.2, except as modified in this Section.

The design shear strength of the Composite Column shall be the design shear strength of the structural steel section alone.

In the special seismic systems described in, members and column splices for Concrete-Filled Composite Columns shall also meet the requirements in Part 6 Section 10.20.

Concrete-Filled Composite Columns used in C-SMF shall meet the following additional requirements:

- (1) The minimum required shear strength of the column shall meet the requirements in Part 6 Chapter 5.
- (2) The strong-column/weak-beam design requirements shall be met. Column Bases shall be designed to sustain inelastic flexural hinging.
- (3) The minimum wall thickness of concrete-filled rectangular HSS shall equal  $b\sqrt{F_y/(2E_s)}$  for the flat width  $b$  of each face, where  $b$  is as defined in Part 6 Chapter 10 Table 10.2.1.

### 13.5.6 Composite Steel Plate Shear Walls (C-SPW)

#### 13.5.6.1 Scope

This Section is applicable to structural walls consisting of steel plates with reinforced concrete encasement on one or both sides of the plate and structural steel or composite Boundary Members. C-SPW shall meet the requirements of this section.

#### 13.5.6.2 Wall Elements

##### 13.5.6.2.1 Nominal Shear Strength

The nominal shear strength of C-SPW with a stiffened plate conforming to Section 13.5.4.2.2 shall be determined as:

$$V_{ns} = 0.6A_{sp}F_y \quad 13.5.3$$

where

$V_{ns}$  = nominal shear strength of the steel plate, N

$A_{sp}$  = horizontal area of stiffened steel plate, mm<sup>2</sup>

$F_y$  = specified minimum yield strength of the plate, MPa

The nominal shear strength of C-SPW with a plate that does not meet the stiffening requirements in Section 13.5.4.2.2 shall be based upon the strength of the plate, excluding the strength of the reinforced concrete, and meet the requirements in the Part 6 Chapter 10, including the effects of buckling of the plate.

##### 13.5.6.2.2 Detailing Requirements

The steel plate shall be adequately stiffened by encasement or attachment to the reinforced concrete if it can be demonstrated with an elastic plate buckling analysis that the composite wall can resist a nominal shear force equal to  $V_{ns}$ . The concrete thickness shall be a minimum of 102 mm (4 in.) on each side when concrete is provided on both sides of the steel plate and 200 mm (8 in.) when concrete is provided on one side of the steel plate. Headed shear stud connectors or other mechanical connectors shall be provided to prevent local buckling and separation of the plate and reinforced concrete. Horizontal and vertical reinforcement shall be provided in the concrete encasement to meet the detailing requirements in Part 6 Chapter 5. The reinforcement ratio in both directions shall not be less than 0.0025; the maximum spacing between bars shall not exceed 455 mm (18 in.).

The steel plate shall be continuously connected on all edges to structural steel framing and Boundary Members with welds and/or slip-critical high-strength bolts to develop the nominal shear strength of the plate. The Design Strength of welded and bolted connectors shall meet the additional requirements in Part 6 Chapter 10.