CHAPTER I

DESIGN OF COMPOSITE MEMBERS

This chapter addresses *composite* members 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 *steel headed stud anchors, concrete-encased,* and *concrete filled beams*, constructed with or without temporary shores, are included.

The chapter is organized as follows:

I1. 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.

1. Concrete and Steel Reinforcement

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 *applicable building code*. Additionally, the provisions in ACI 318 shall apply with the following exceptions and limitations:

(1) ACI 318 Sections 7.8.2 and 10.13, and Chapter 21 shall be excluded in their entirety.
(2) Concrete and steel reinforcement material limitations shall be as specified in Section I1.3.
(3) *Transverse reinforcement* limitations shall be as specified in Section I2.1a(2), in addition to those specified in ACI 318.
(4) The minimum longitudinal reinforcing ratio for *encased composite members* shall be as specified in Section I2.1a(3).

Concrete and steel reinforcement components designed in accordance with ACI 318 shall be based on a level of loading corresponding to *LRFD load combinations*.

> **User Note:** It is the intent of the Specification that the concrete and reinforcing steel portions of composite concrete members be detailed utilizing the noncomposite provisions of ACI 318 as modified by the Specification. All requirements specific to composite members are covered in the Specification.
>
> Note that the design basis for ACI 318 is strength design. Designers using ASD for steel must be conscious of the different *load factors*.

2. Nominal Strength of Composite Sections

The *nominal strength* of composite sections shall be determined in accordance with the *plastic stress distribution method* or the *strain compatibility method* as defined in this section.

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

Local buckling effects shall be considered for *filled composite members* as defined in Section I1.4. Local buckling effects need not be considered for *encased composite members*.

2a. 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 due to axial force and/or flexure have reached a stress of $0.85f'_c$. For round *HSS* filled with concrete, a stress of $0.95f'_c$ is permitted to be used for concrete components in compression due to axial force and/or flexure to account for the effects of concrete confinement.

2b. 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 in./in. (mm/mm). The stress-strain relationships for steel and concrete shall be obtained from tests or from published results for similar materials.

> **User Note**: The strain compatibility method should be used to determine *nominal strength* for irregular sections and for cases where the steel does not exhibit elasto-plastic behavior. General guidelines for the strain compatibility method for encased members subjected to axial *load*, flexure or both are given in AISC Design Guide 6 and ACI 318.

3. Material Limitations

For concrete, *structural steel*, and steel reinforcing bars in composite systems, the following limitations shall be met, unless justified by testing or analysis:

(1) For the determination of the *available strength*, concrete shall have a compressive strength, f'_c, of not less than 3 ksi (21 MPa) nor more than 10 ksi (70 MPa) for normal weight concrete and not less than 3 ksi (21 MPa) nor more than 6 ksi (42 MPa) for *lightweight concrete.*

> **User Note:** Higher strength concrete material properties may be used for *stiffness* calculations but may not be relied upon for strength calculations unless justified by testing or analysis.

(2) The *specified minimum yield stress* of structural steel and reinforcing bars used in calculating the strength of composite members shall not exceed 75 ksi (525 MPa).

4. Classification of Filled Composite Sections for Local Buckling

For compression, filled composite sections are classified as compact, noncompact or slender. For a section to qualify as compact, the maximum width-to-thickness ratio of its compression steel elements shall not exceed the limiting width-to-thickness ratio, λ_p, from Table I1.1a. If the maximum width-to-thickness ratio of one or more steel compression elements exceeds λ_p, but does not exceed λ_r from Table I1.1a, the filled composite section is noncompact. If the maximum width-to-thickness ratio of any compression steel element exceeds λ_r, the section is slender. The maximum permitted width-to-thickness ratio shall be as specified in the table.

For flexure, filled composite sections are classified as compact, noncompact or slender. For a section to qualify as compact, the maximum width-to-thickness ratio of its compression steel elements shall not exceed the limiting width-to-thickness ratio, λ_p, from Table I1.1b. If the maximum width-to-thickness ratio of one or more steel compression elements exceeds λ_p, but does not exceed λ_r from Table I1.1b, the section is noncompact. If the width-to-thickness ratio of any steel element exceeds λ_r, the section is slender. The maximum permitted width-to-thickness ratio shall be as specified in the table.

Refer to Table B4.1a and Table B4.1b for definitions of width (b and D) and thickness (t) for rectangular and round *HSS* sections.

> **User Note**: All current ASTM A500 Grade B square HSS sections are compact according to the limits of Table I1.1a and Table I1.1b except HSS7×7×1/8, HSS8×8×1/8, HSS9×9×1/8 and HSS12×12×3/16 which are noncompact for both axial compression and flexure.
>
> All current ASTM A500 Grade B round HSS sections are compact according to the limits of Table I1.1a and Table I1.1b for both axial compression and flexure with the exception of HSS16.0×0.25, which is noncompact for flexure.

TABLE I1.1A
Limiting Width-to-Thickness Ratios for Compression Steel Elements in Composite Members Subject to Axial Compression For Use with Section I2.2

Description of Element	Width-to-Thickness Ratio	λ_p Compact/ Noncompact	λ_r Noncompact/ Slender	Maximum Permitted
Walls of Rectangular HSS and Boxes of Uniform Thickness	b/t	$2.26\sqrt{\frac{E}{F_y}}$	$3.00\sqrt{\frac{E}{F_y}}$	$5.00\sqrt{\frac{E}{F_y}}$
Round HSS	D/t	$\frac{0.15E}{F_y}$	$\frac{0.19E}{F_y}$	$\frac{0.31E}{F_y}$

TABLE I1.1B
Limiting Width-to-Thickness Ratios for Compression Steel Elements in Composite Members Subject to Flexure For Use with Section I3.4

Description of Element	Width-to-Thickness Ratio	λ_p Compact/ Noncompact	λ_r Noncompact/ Slender	Maximum Permitted
Flanges of Rectangular HSS and Boxes of Uniform Thickness	b/t	$2.26\sqrt{\frac{E}{F_y}}$	$3.00\sqrt{\frac{E}{F_y}}$	$5.00\sqrt{\frac{E}{F_y}}$
Webs of Rectangular HSS and Boxes of Uniform Thickness	h/t	$3.00\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$
Round HSS	D/t	$\frac{0.09E}{F_y}$	$\frac{0.31E}{F_y}$	$\frac{0.31E}{F_y}$

I2. AXIAL FORCE

This section applies to two types of *composite* members subject to axial force: *encased composite members* and *filled composite members.*

1. Encased Composite Members

1a. Limitations

For *encased composite members*, the following limitations shall be met:

(1) The cross-sectional area of the steel core shall comprise at least 1% of the total composite cross section.

(2) Concrete encasement of the steel core shall be reinforced with continuous longitudinal bars and lateral ties or spirals.

Where lateral ties are used, a minimum of either a No. 3 (10 mm) bar spaced at a maximum of 12 in. (305 mm) on center, or a No. 4 (13 mm) bar or larger spaced at a maximum of 16 in. (406 mm) on center shall be used. Deformed wire or welded wire reinforcement of equivalent area are permitted.

Maximum spacing of lateral ties shall not exceed 0.5 times the least *column* dimension.

(3) The minimum reinforcement ratio for continuous longitudinal reinforcing, ρ_{sr}, shall be 0.004, where ρ_{sr} is given by:

$$\rho_{sr} = \frac{A_{sr}}{A_g} \tag{I2-1}$$

where

A_g = gross area of composite member, in.^{2} (mm^2)

A_{sr} = area of continuous reinforcing bars, in.^{2} (mm^2)

User Note: Refer to Sections 7.10 and 10.9.3 of ACI 318 for additional tie and spiral reinforcing provisions.

1b. Compressive Strength

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

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

(a) When $\dfrac{P_{no}}{P_e} \leq 2.25$

$$P_n = P_{no}\left[0.658^{\frac{P_{no}}{P_e}}\right] \tag{I2-2}$$

(b) When $\dfrac{P_{no}}{P_e} > 2.25$

$$P_n = 0.877P_e \tag{I2-3}$$

where

P_{no} = $F_y A_s + F_{ysr} A_{sr} + 0.85 f_c' A_c$ (I2-4)

P_e = elastic critical *buckling load* determined in accordance with Chapter C or Appendix 7, kips (N)

= $\pi^2(EI_{eff})/(KL)^2$ (I2-5)

A_c = area of concrete, in.^{2} (mm^2)

A_s = area of the steel section, in.^{2} (mm^2)

E_c = modulus of elasticity of concrete

= $w_c^{1.5}\sqrt{f_c'}$, ksi $\left(0.043w_c^{1.5}\sqrt{f_c'},\ \text{MPa}\right)$

EI_{eff} = effective *stiffness* of composite section, kip-in.^{2} (N-mm^2)

= $E_s I_s + 0.5E_s I_{sr} + C_1 E_c I_c$ (I2-6)

C_1 = coefficient for calculation of effective rigidity of an encased composite compression member

$$= 0.1 + 2\left(\frac{A_s}{A_c + A_s}\right) \le 0.3 \tag{I2-7}$$

E_s = modulus of elasticity of steel

= 29,000 ksi (200 000 MPa)

F_y = *specified minimum yield stress* of steel section, ksi (MPa)

F_{ysr} = specified minimum yield stress of reinforcing bars, ksi (MPa)

I_c = moment of inertia of the concrete section about the elastic neutral axis of the composite section, in.^{4} (mm^4)

I_s = moment of inertia of steel shape about the elastic neutral axis of the composite section, in.^{4} (mm^4)

I_{sr} = moment of inertia of reinforcing bars about the elastic neutral axis of the composite section, in.^{4} (mm^4)

K = *effective length factor*

L = laterally *unbraced length* of the member, in. (mm)

f_c' = specified compressive strength of concrete, ksi (MPa)

w_c = weight of concrete per unit volume ($90 \le w_c \le 155$ lbs/ft^3 or $1500 \le w_c \le 2500$ kg/m^3)

The *available compressive strength* need not be less than that specified for the bare steel member as required by Chapter E.

1c. Tensile Strength

The *available tensile strength* of axially loaded *encased composite members* shall be determined for the limit state of *yielding* as follows:

$$P_n = F_y A_s + F_{ysr} A_{sr} \tag{I2-8}$$

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

1d. Load Transfer

Load transfer requirements for *encased composite members* shall be determined in accordance with Section I6.

1e. Detailing Requirements

Clear spacing between the steel core and longitudinal reinforcing shall be a minimum of 1.5 reinforcing bar diameters, but not less than 1.5 in. (38 mm).

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.

2. Filled Composite Members

2a. Limitations

For *filled composite members*, the cross-sectional area of the steel section shall comprise at least 1% of the total composite cross section.

Filled composite members shall be classified for *local buckling* according to Section I1.4.

2b. Compressive Strength

The *available compressive strength* of axially loaded doubly symmetric *filled composite members* shall be determined for the limit state of *flexural buckling* in accordance with Section I2.1b with the following modifications:

(a) For *compact sections*

$$P_{no} = P_p \tag{I2-9a}$$

where

$$P_p = F_y A_s + C_2 f_c' \left(A_c + A_{sr} \frac{E_s}{E_c} \right) \tag{I2-9b}$$

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

(b) For *noncompact sections*

$$P_{no} = P_p - \frac{P_p - P_y}{\left(\lambda_r - \lambda_p\right)^2}\left(\lambda - \lambda_p\right)^2 \tag{I2-9c}$$

where

λ, λ_p and λ_r are slenderness ratios determined from Table I1.1a

P_p is determined from Equation I2-9b

$$P_y = F_y A_s + 0.7 f_c' \left(A_c + A_{sr} \frac{E_s}{E_c} \right) \tag{I2-9d}$$

(c) For slender sections

$$P_{no} = F_{cr} A_s + 0.7 f_c' \left(A_c + A_{sr} \frac{E_s}{E_c} \right) \tag{I2-9e}$$

where

(i) For rectangular filled sections

$$F_{cr} = \frac{9E_s}{\left(\frac{b}{t}\right)^2} \tag{I2-10}$$

(ii) For round filled sections

$$F_{cr} = \frac{0.72F_y}{\left(\left(\frac{D}{t}\right)\frac{F_y}{E_s}\right)^{0.2}} \tag{I2-11}$$

The effective stiffness of the composite section, EI_{eff}, for all sections shall be:

$$EI_{eff} = E_sI_s + E_sI_{sr} + C_3E_cI_c \tag{I2-12}$$

where

C_3 = coefficient for calculation of effective rigidity of filled composite compression member

$$= 0.6 + 2\left[\frac{A_s}{A_c + A_s}\right] \leq 0.9 \tag{I2-13}$$

The available compressive *strength* need not be less than specified for the bare steel member as required by Chapter E.

2c. Tensile Strength

The *available tensile strength* of axially loaded *filled composite members* shall be determined for the limit state of *yielding* as follows:

$$P_n = A_sF_y + A_{sr}F_{ysr} \tag{I2-14}$$

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

2d. Load Transfer

Load transfer requirements for *filled composite members* shall be determined in accordance with Section I6.

I3. FLEXURE

This section applies to three types of *composite* members subject to flexure: *composite beams* with *steel anchors* consisting of steel headed stud anchors or steel channel anchors, *encased composite members*, and *filled composite members*.

1. General

1a. Effective Width

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

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

1b. 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% of its specified strength f_c'. The *available flexural strength* of the steel section shall be determined in accordance with Chapter F.

2. Composite Beams With Steel Headed Stud or Steel Channel Anchors

2a. Positive Flexural Strength

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

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

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

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

> **User Note:** All current ASTM A6 W, S and HP shapes satisfy the limit given in Section I3.2a(a) for $F_y \leq 50$ ksi (345 MPa).

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

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

2b. Negative Flexural Strength

The *available negative flexural strength* shall be determined for the steel section alone, in accordance with the requirements of Chapter F.

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

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

provided that the following limitations are met:

(1) The steel *beam* is *compact* and is adequately braced in accordance with Chapter F.
(2) Steel headed stud or steel channel anchors connect the slab to the steel beam in the negative moment region.
(3) The slab reinforcement parallel to the steel beam, within the *effective width* of the slab, is *properly developed*.

2c. 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 Sections I3.2a and I3.2b, with the following requirements:

(1) The *nominal rib height* shall not be greater than 3 in. (75 mm). The average width of concrete rib or haunch, w_r, shall be not less than 2 in. (50 mm), but shall not be taken in calculations as more than the minimum clear width near the top of the steel deck.

(2) The concrete slab shall be connected to the steel beam with welded steel headed stud anchors, $^3/_4$ in. (19 mm) or less in diameter (AWS D1.1/D1.1M). Steel headed stud anchors shall be welded either through the deck or directly to the steel cross section. Steel headed stud anchors, after installation, shall extend not less than $1^1/_2$ in. (38 mm) above the top of the steel deck and there shall be at least $^1/_2$ in. (13 mm) of specified concrete cover above the top of the steel headed stud anchors.

(3) The slab thickness above the steel deck shall be not less than 2 in. (50 mm).

(4) Steel deck shall be anchored to all supporting members at a spacing not to exceed 18 in. (460 mm). Such anchorage shall be provided by steel headed stud anchors, a combination of steel headed stud anchors and arc spot (puddle) welds, or other devices specified by the contract documents.

(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 is permitted to be included in determining composite section properties and shall be included in calculating A_c.

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

When the nominal depth of steel deck is $1^1/_2$ in. (38 mm) or greater, the average width, w_r, of the supported haunch or rib shall be not less than 2 in. (50 mm) for the first steel headed stud anchor in the transverse row plus four stud diameters for each additional steel headed stud anchor.

2d. Load Transfer Between Steel Beam and Concrete Slab

(1) Load Transfer for Positive Flexural Strength

The entire *horizontal shear* at the interface between the steel *beam* and the concrete slab shall be assumed to be transferred by steel headed stud or steel channel anchors, except for *concrete-encased beams* as defined in Section I3.3. For composite action with concrete subject to flexural compression, the nominal shear force between the steel beam and the concrete slab transferred by *steel anchors*, V', between the point of maximum positive moment and the point of zero moment shall be determined as the lowest value in accordance with the *limit* *states* of *concrete crushing, tensile yielding* of the steel section, or the shear strength of the steel anchors:

(a) Concrete crushing

$$V' = 0.85 f_c' A_c \tag{I3-1a}$$

(b) Tensile yielding of the steel section

$$V' = F_y A_s \tag{I3-1b}$$

(c) Shear strength of steel headed stud or steel channel anchors

$$V' = \Sigma Q_n \tag{I3-1c}$$

where

A_c = area of concrete slab within *effective width*, in.^{2} (mm^2)
A_s = area of steel cross section, in.^{2} (mm^2)
ΣQ_n = sum of *nominal shear strengths* of steel headed stud or steel channel anchors between the point of maximum positive moment and the point of zero moment, kips (N)

(2) Load Transfer for Negative Flexural Strength

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 between the point of maximum negative moment and the point of zero moment shall be determined as the lower value in accordance with the following limit states:

(a) For the limit state of tensile yielding of the slab reinforcement

$$V' = F_{ysr} A_{sr} \tag{I3-2a}$$

where

A_{sr} = area of adequately developed longitudinal reinforcing steel within the effective width of the concrete slab, in.^{2} (mm^2)
F_{ysr} = *specified minimum yield stress* of the reinforcing steel, ksi (MPa)

(b) For the limit state of shear strength of steel headed stud or steel channel anchors

$$V' = \Sigma Q_n \tag{I3-2b}$$

3. Encased Composite Members

The *available flexural strength* of concrete-encased members shall be determined as follows:

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

The nominal flexural strength, M_n, 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*).

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

(c) The plastic stress distribution on the composite section or the strain-compatibility method, for the limit state of yielding (plastic moment) on the composite section. For concrete-encased members, *steel anchors* shall be provided.

4. Filled Composite Members

4a. Limitations

Filled composite sections shall be classified for *local buckling* according to Section I1.4.

4b. Flexural Strength

The *available flexural strength* of *filled composite members* shall be determined as follows:

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

The nominal flexural strength, M_n, shall be determined as follows:

(a) For *compact sections*

$$M_n = M_p \tag{I3-3a}$$

where

M_p = moment corresponding to plastic *stress* distribution over the composite cross section, kip-in. (N-mm)

(b) For *noncompact sections*

$$M_n = M_p - \left(M_p - M_y\right)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \tag{I3-3b}$$

where

λ, λ_p and λ_r are slenderness ratios determined from Table I1.1b.

M_y = *yield moment* corresponding to yielding of the tension flange and first yield of the compression flange, kip-in. (N-mm). The capacity at first yield shall be calculated assuming a linear elastic stress distribution with the maximum concrete compressive stress limited to $0.7f_c'$ and the maximum steel stress limited to F_y.

(c) For slender sections, M_n, shall be determined as the first yield moment. The compression flange stress shall be limited to the *local buckling* stress, F_{cr}, determined using Equation I2-10 or I2-11. The concrete stress distribution shall be linear elastic with the maximum compressive stress limited to $0.70f'_c$.

I4. SHEAR

1. Filled and Encased Composite Members

The *design shear strength,* $\phi_v V_n$, and *allowable shear strength,* V_n/Ω_v, shall be determined based on one of the following:

(a) The *available shear strength* of the steel section alone as specified in Chapter G
(b) The available shear strength of the reinforced concrete portion (concrete plus steel reinforcement) alone as defined by ACI 318 with

$$\phi_v = 0.75 \text{ (LRFD)} \qquad \Omega_v = 2.00 \text{ (ASD)}$$

(c) The *nominal shear strength* of the steel section as defined in Chapter G plus the nominal strength of the reinforcing steel as defined by ACI 318 with a combined resistance or *safety factor* of

$$\phi_v = 0.75 \text{ (LRFD)} \qquad \Omega_v = 2.00 \text{ (ASD)}$$

2. Composite Beams With Formed Steel Deck

The available shear strength of *composite beams* with steel headed stud or steel channel anchors shall be determined based upon the properties of the steel section alone in accordance with Chapter G.

I5. COMBINED FLEXURE AND AXIAL FORCE

The interaction between flexure and axial forces in composite members shall account for *stability* as required by Chapter C. The *available compressive strength* and the *available flexural strength* shall be determined as defined in Sections I2 and I3, respectively. 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 in accordance with Section I2.

For *encased composite members* and for *filled composite members* with *compact sections*, the interaction between axial force and flexure shall be based on the interaction equations of Section H1.1 or one of the methods as defined in Section I1.2.

For filled composite members with noncompact or slender sections, the interaction between axial forces and flexure shall be based on the interaction equations of Section H1.1.

> **User Note:** Methods for determining the capacity of composite *beam-columns* are discussed in the Commentary.

I6. LOAD TRANSFER

1. General Requirements

When external forces are applied to an axially loaded encased or *filled composite member*, the introduction of force to the member and the transfer of longitudinal shears within the member shall be assessed in accordance with the requirements for force allocation presented in this section.

The *design strength*, ϕR_n, or the *allowable strength*, R_n/Ω, of the applicable force transfer mechanisms as determined in accordance with Section I6.3 shall equal or exceed the required longitudinal shear force to be transferred, V_r', as determined in accordance with Section I6.2.

2. Force Allocation

Force allocation shall be determined based upon the distribution of external force in accordance with the following requirements:

> **User Note:** *Bearing* strength provisions for externally applied forces are provided in Section J8. For *filled composite members*, the term $\sqrt{A_2/A_1}$ in Equation J8-2 may be taken equal to 2.0 due to confinement effects.

2a. External Force Applied to Steel Section

When the entire external *force* is applied directly to the steel section, the force required to be transferred to the concrete, V_r', shall be determined as follows:

$$V_r' = P_r\,(1 - F_y A_s/P_{no}) \tag{I6-1}$$

where

P_{no} = nominal axial compressive strength without consideration of *length effects*, determined by Equation I2-4 for *encased composite members*, and Equation I2-9a for *filled composite members*, kips (N)

P_r = required external force applied to the composite member, kips (N)

2b. External Force Applied to Concrete

When the entire external force is applied directly to the concrete encasement or concrete fill, the force required to be transferred to the steel, V_r', shall be determined as follows:

$$V_r' = P_r\,(F_y A_s/P_{no}) \tag{I6-2}$$

where

P_{no} = nominal axial compressive strength without consideration of *length effects*, determined by Equation I2-4 for *encased composite members*, and Equation I2-9a for *filled composite members*, kips (N)

P_r = required external force applied to the composite member, kips (N)

2c. External Force Applied Concurrently to Steel and Concrete

When the external force is applied concurrently to the steel section and concrete encasement or concrete fill, V_r' shall be determined as the force required to establish equilibrium of the cross section.

User Note: The Commentary provides an acceptable method of determining the longitudinal shear force required for equilibrium of the cross section.

3. Force Transfer Mechanisms

The *nominal strength*, R_n, of the force transfer mechanisms of *direct bond interaction*, shear connection, and direct *bearing* shall be determined in accordance with this section. Use of the force transfer *mechanism* providing the largest nominal strength is permitted. Force transfer mechanisms shall not be superimposed.

The force transfer mechanism of direct bond interaction shall not be used for *encased composite members*.

3a. Direct Bearing

Where force is transferred in an encased or *filled composite member* by direct *bearing* from internal bearing mechanisms, the *available bearing strength* of the concrete for the *limit state* of *concrete crushing* shall be determined as follows:

$$R_n = 1.7 f_c' A_1 \tag{I6-3}$$

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

where

A_1 = loaded area of concrete, in.^{2} (mm^2)

User Note: An example of force transfer via an internal bearing mechanism is the use of internal steel plates within a filled composite member.

3b. Shear Connection

Where force is transferred in an encased or *filled composite member* by shear connection, the *available shear strength* of steel headed stud or steel channel anchors shall be determined as follows:

$$R_c = \Sigma Q_{cv} \tag{I6-4}$$

where

ΣQ_{cv} = sum of *available shear strengths*, ϕQ_{nv} or Q_{nv}/Ω as appropriate, of steel headed stud or steel channel anchors, determined in accordance with Section I8.3a or Section I8.3d, respectively, placed within the *load introduction length* as defined in Section I6.4, kips (N)

3c. Direct Bond Interaction

Where force is transferred in a *filled composite member* by *direct bond interaction*, the *available bond strength* between the steel and concrete shall be determined as follows:

$$\phi = 0.45 \text{ (LRFD)} \qquad \Omega = 3.33 \text{ (ASD)}$$

(a) For rectangular steel sections filled with concrete:

$$R_n = B^2 C_{in} F_{in} \tag{I6-5}$$

(b) For round steel sections filled with concrete:

$$R_n = 0.25\pi D^2 C_{in} F_{in} \tag{I6-6}$$

where

C_{in} = 2 if the filled composite member extends to one side of the point of force transfer
= 4 if the filled composite member extends on both sides of the point of force transfer

R_n = nominal bond strength, kips (N)

F_{in} = nominal bond *stress*= 0.06 ksi (0.40 MPa)

B = overall width of rectangular steel section along face transferring *load*, in. (mm)

D = outside diameter of round *HSS*, in. (mm)

4. Detailing Requirements

4a. Encased Composite Members

Steel anchors utilized to transfer longitudinal shear shall be distributed within the *load introduction length*, which shall not exceed a distance of two times the minimum transverse dimension of the *encased composite member* above and below the *load* transfer region. Anchors utilized to transfer longitudinal shear shall be placed on at least two faces of the steel shape in a generally symmetric configuration about the steel shape axes.

Steel anchor spacing, both within and outside of the load introduction length, shall conform to Section I8.3e.

4b. Filled Composite Members

Where required, steel anchors transferring the required longitudinal shear force shall be distributed within the *load introduction length*, which shall not exceed a distance of two times the minimum transverse dimension of a rectangular steel member or two times the diameter of a round steel member both above and below the *load* transfer region. Steel anchor spacing within the load introduction length shall conform to Section I8.3e.

I7. COMPOSITE DIAPHRAGMS AND COLLECTOR BEAMS

Composite slab diaphragms and *collector beams* shall be designed and detailed to transfer *loads* between the diaphragm, the diaphragm's boundary members and collector elements, and elements of the lateral force resisting system.

> **User Note:** Design guidelines for composite diaphragms and collector beams can be found in the Commentary.

I8. STEEL ANCHORS

1. General

The diameter of a steel headed stud anchor shall not be greater than 2.5 times the thickness of the base metal to which it is welded, unless it is welded to a flange directly over a web.

Section I8.2 applies to a *composite* flexural member where *steel anchors* are embedded in a solid concrete slab or in a slab cast on *formed steel* deck. Section I8.3 applies to all other cases.

2. Steel Anchors in Composite Beams

The length of steel headed stud anchors shall not be less than four stud diameters from the base of the steel headed stud anchor to the top of the stud head after installation.

2a. Strength of Steel Headed Stud Anchors

The *nominal shear strength* of one steel headed stud anchor embedded in a solid concrete slab or in a composite slab with decking shall be determined as follows:

$$Q_n = 0.5 A_{sa}\sqrt{f_c' E_c} \le R_g R_p A_{sa} F_u \quad \text{(I8-1)}$$

where

A_{sa} = cross-sectional area of steel headed stud anchor, in.^{2} (mm^2)

E_c = modulus of elasticity of concrete

$= w_c^{1.5}\sqrt{f_c'}$, ksi $\left(0.043 w_c^{1.5}\sqrt{f_c'}\text{, MPa}\right)$

F_u = *specified minimum tensile strength* of a steel headed stud anchor, ksi (MPa)

R_g = 1.0 for:

(a) one steel headed stud anchor welded in a steel deck rib with the deck oriented perpendicular to the steel shape;

(b) any number of steel headed stud anchors welded in a row directly to the steel shape;

(c) any number of steel headed stud anchors 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 ≥ 1.5

= 0.85 for:

(a) two steel headed stud anchors welded in a steel deck rib with the deck oriented perpendicular to the steel shape;

(b) one steel headed stud anchor 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 steel headed stud anchors welded in a steel deck rib with the deck oriented perpendicular to the steel shape

R_p = 0.75 for:

(a) steel headed stud anchors welded directly to the steel shape;

(b) steel headed stud anchors welded in a composite slab with the deck oriented perpendicular to the *beam* and $e_{mid\text{-}ht} \geq 2$ in. (50 mm);

(c) steel headed stud anchors 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 steel headed stud anchors welded in a composite slab with deck oriented perpendicular to the beam and $e_{mid\text{-}ht} < 2$ in. (50 mm)

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

User Note: The table below presents values for R_g and R_p for several cases. Capacities for steel headed stud anchors can be found in the Manual.

Condition	R_g	R_p
No decking	1.0	0.75
Decking oriented parallel to the steel shape		
$\frac{w_r}{h_r} \geq 1.5$	1.0	0.75
$\frac{w_r}{h_r} < 1.5$	0.85**	0.75
Decking oriented perpendicular to the steel shape Number of steel headed stud anchors occupying the same decking rib		
1	1.0	0.6+
2	0.85	0.6+
3 or more	0.7	0.6+

h_r = nominal rib height, in. (mm)

w_r = average width of concrete rib or haunch (as defined in Section I3.2c), in. (mm)

** for a single steel headed stud anchor

+ this value may be increased to 0.75 when $e_{mid\text{-}ht} \geq 2$ in. (51 mm)

2b. Strength of Steel Channel Anchors

The nominal shear strength of one hot-rolled channel anchor embedded in a solid concrete slab shall be determined as follows:

$$Q_n = 0.3(t_f + 0.5t_w)l_a\sqrt{f_c' E_c} \qquad \text{(I8-2)}$$

where

l_a = length of channel anchor, in. (mm)
t_f = thickness of flange of channel anchor, in. (mm)
t_w = thickness of channel anchor web, in. (mm)

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

2c. Required Number of Steel Anchors

The number of anchors 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* as determined in Sections I3.2d(1) and I3.2d(2) divided by the nominal shear strength of one *steel anchor* as determined from Section I8.2a or Section I8.2b. The number of steel anchors required 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.

2d. Detailing Requirements

Steel anchors 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 specified otherwise on the contract documents.

Steel anchors shall have at least 1 in. (25 mm) of lateral concrete cover in the direction perpendicular to the shear force, except for anchors installed in the ribs of formed steel decks. The minimum distance from the center of an anchor to a free edge in the direction of the shear force shall be 8 in. (203 mm) if normal weight concrete is used and 10 in. (250 mm) if *lightweight concrete* is used. The provisions of ACI 318, Appendix D are permitted to be used in lieu of these values.

The minimum center-to-center spacing of steel headed stud anchors 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 steel anchors shall not exceed eight times the total slab thickness or 36 in. (900 mm).

3. Steel Anchors in Composite Components

This section shall apply to the design of cast-in-place steel headed stud anchors and steel channel anchors in *composite components*.

The provisions of the *applicable building code* or ACI 318, Appendix D may be used in lieu of the provisions in this section.

> **User Note:** The steel headed stud anchor strength provisions in this section are applicable to anchors located primarily in the *load* transfer (connection) region of composite *columns* and *beam-columns*, concrete-encased and filled composite beams, composite coupling *beams*, and composite walls, where the steel and concrete are working compositely within a member. They are not intended for hybrid construction where the steel and concrete are not working compositely, such as with embed plates.
>
> Section I8.2 specifies the strength of *steel anchors* embedded in a solid concrete slab or in a concrete slab with formed steel deck in a composite beam.
>
> *Limit states* for the steel shank of the anchor and for concrete breakout in shear are covered directly in this Section. Additionally, the spacing and dimensional limitations provided in these provisions preclude the limit states of concrete pryout for anchors loaded in shear and concrete breakout for anchors loaded in tension as defined by ACI 318, Appendix D.

For normal weight concrete: Steel headed stud anchors subjected to shear only shall not be less than five stud diameters in length from the base of the steel headed stud to the top of the stud head after installation. Steel headed stud anchors subjected to tension or interaction of shear and tension shall not be less than eight stud diameters in length from the base of the stud to the top of the stud head after installation.

For *lightweight concrete*: Steel headed stud anchors subjected to shear only shall not be less than seven stud diameters in length from the base of the steel headed stud to the top of the stud head after installation. Steel headed stud anchors subjected to tension shall not be less than ten stud diameters in length from the base of the stud to the top of the stud head after installation. The *nominal strength* of steel headed stud anchors subjected to interaction of shear and tension for lightweight concrete shall be determined as stipulated by the applicable building code or ACI 318 Appendix D.

Steel headed stud anchors subjected to tension or interaction of shear and tension shall have a diameter of the head greater than or equal to 1.6 times the diameter of the shank.

User Note: The following table presents values of minimum steel headed stud anchor *h/d* ratios for each condition covered in the Specification:

Loading Condition	Normal Weight Concrete	Lightweight Concrete
Shear	$h/d \geq 5$	$h/d \geq 7$
Tension	$h/d \geq 8$	$h/d \geq 10$
Shear and Tension	$h/d \geq 8$	N/A*

h/d = ratio of steel headed stud anchor shank length to the top of the stud head, to shank diameter

* Refer to ACI 318, Appendix D for the calculation of interaction effects of anchors embedded in lightweight concrete.

3a. Shear Strength of Steel Headed Stud Anchors in Composite Components

Where concrete breakout strength in shear is not an applicable *limit state*, the *design shear strength,* $\phi_v Q_{nv}$, and *allowable shear strength,* Q_{nv}/Ω_v, of one steel headed stud anchor shall be determined as follows:

$$Q_{nv} = F_u A_{sa} \tag{I8-3}$$

$$\phi_v = 0.65 \text{ (LRFD)} \qquad \Omega_v = 2.31 \text{ (ASD)}$$

where

Q_{nv} = nominal shear strength of steel headed stud anchor, kips (N)
A_{sa} = cross-sectional area of steel headed stud anchor, in.^{2} (mm^2)
F_u = *specified minimum tensile strength* of a steel headed stud anchor, ksi (MPa)

Where concrete breakout strength in shear is an applicable limit state, the *available shear strength* of one steel headed stud anchor shall be determined by one of the following:

(1) Where anchor reinforcement is developed in accordance with Chapter 12 of ACI 318 on both sides of the *concrete breakout surface* for the steel headed stud anchor, the minimum of the steel nominal shear strength from Equation I8-3 and the *nominal strength* of the anchor reinforcement shall be used for the nominal shear strength, Q_{nv}, of the steel headed stud anchor.
(2) As stipulated by the *applicable building code* or ACI 318, Appendix D.

> **User Note:** If concrete breakout strength in shear is an applicable limit state (for example, where the breakout prism is not restrained by an adjacent steel plate, flange or web), appropriate anchor reinforcement is required for the provisions of this Section to be used. Alternatively, the provisions of the applicable building code or ACI 318, Appendix D may be used.

3b. Tensile Strength of Steel Headed Stud Anchors in Composite Components

Where the distance from the center of an anchor to a free edge of concrete in the direction perpendicular to the height of the steel headed stud anchor is greater than or equal to 1.5 times the height of the steel headed stud anchor measured to the top of the stud head, and where the center-to-center spacing of steel headed stud anchors is greater than or equal to three times the height of the steel headed stud anchor measured to the top of the stud head, the *available tensile strength* of one steel headed stud anchor shall be determined as follows:

$$Q_{nt} = F_u A_{sa} \tag{I8-4}$$

$$\phi_t = 0.75 \text{ (LRFD)} \qquad \Omega_t = 2.00 \text{ (ASD)}$$

where

Q_{nt} = *nominal tensile strength* of steel headed stud anchor, kips (N)

Where the distance from the center of an anchor to a free edge of concrete in the direction perpendicular to the height of the steel headed stud anchor is less than 1.5 times the height of the steel headed stud anchor measured to the top of the stud head, or where the center-to-center spacing of steel headed stud anchors is less than three times the height of the steel headed stud anchor measured to the top of the stud head, the nominal tensile strength of one steel headed stud anchor shall be determined by one of the following:

(a) Where anchor reinforcement is developed in accordance with Chapter 12 of ACI 318 on both sides of the *concrete breakout surface* for the steel headed stud anchor, the minimum of the steel nominal tensile strength from Equation I8-4 and the *nominal strength* of the anchor reinforcement shall be used for the nominal tensile strength, Q_{nt}, of the steel headed stud anchor.

(b) As stipulated by the *applicable building code* or ACI 318, Appendix D.

> **User Note:** Supplemental confining reinforcement is recommended around the anchors for steel headed stud anchors subjected to tension or interaction of shear and tension to avoid edge effects or effects from closely spaced anchors. See the Commentary and ACI 318, Section D5.2.9 for guidelines.

3c. Strength of Steel Headed Stud Anchors for Interaction of Shear and Tension in Composite Components

Where concrete breakout strength in shear is not a governing *limit state*, and where the distance from the center of an anchor to a free edge of concrete in the direction perpendicular to the height of the steel headed stud anchor is greater than or equal to 1.5 times the height of the steel headed stud anchor measured to the top of the stud head, and where the center-to-center spacing of steel headed stud anchors is greater than or equal to three times the height of the steel headed stud anchor measured to the top of the stud head, the *nominal strength* for interaction of shear and tension of one steel headed stud anchor shall be determined as follows:

$$\left[\left(\frac{Q_{rt}}{Q_{ct}}\right)^{5/3}+\left(\frac{Q_{rv}}{Q_{cv}}\right)^{5/3}\right]\leq 1.0 \qquad \text{(I8-5)}$$

where

Q_{ct} = *available tensile strength*, kips (N)
Q_{rt} = *required tensile strength*, kips (N)
Q_{cv} = *available shear strength*, kips (N)
Q_{rv} = *required shear strength*, kips (N)

For design in accordance with Section B3.3 (LRFD):

Q_{rt} = required tensile strength using *LRFD load combinations*, kips (N)
Q_{ct} = $\phi_t Q_{nt}$ = *design tensile strength*, determined in accordance with Section I8.3b, kips (N)
Q_{rv} = required shear strength using LRFD load combinations, kips (N)
Q_{cv} = $\phi_v Q_{nv}$ = *design shear strength*, determined in accordance with Section I8.3a, kips (N)
ϕ_t = *resistance factor* for tension = 0.75
ϕ_v = resistance factor for shear = 0.65

For design in accordance with Section B3.4 (ASD):

Q_{rt} = required tensile strength using *ASD load combinations*, kips (N)
Q_{ct} = $\dfrac{Q_{nt}}{\Omega_t}$ = *allowable tensile strength*, determined in accordance with Section I8.3b, kips (N)
Q_{rv} = required shear strength using ASD load combinations, kips (N)
Q_{cv} = $\dfrac{Q_{nv}}{\Omega_v}$ = *allowable shear strength*, determined in accordance with Section I8.3a, kips (N)
Ω_t = *safety factor* for tension = 2.00
Ω_v = safety factor for shear = 2.31

Where concrete breakout strength in shear is a governing limit state, or where the distance from the center of an anchor to a free edge of concrete in the direction perpendicular to the height of the steel headed stud anchor is less than 1.5 times the height of the steel headed stud anchor measured to the top of the stud head, or where the center-to-center spacing of steel headed stud anchors is less than three times the height of the steel headed stud anchor measured to the top of the stud head, the nominal strength for interaction of shear and tension of one steel headed stud anchor shall be determined by one of the following:

(a) Where anchor reinforcement is developed in accordance with Chapter 12 of ACI 318 on both sides of the *concrete breakout surface* for the steel headed stud anchor, the minimum of the steel nominal shear strength from Equation I8-3 and the nominal strength of the anchor reinforcement shall be used for the nominal shear strength, Q_{nv}, of the steel headed stud anchor, and the minimum of the steel nominal tensile strength from Equation I8-4 and the nominal strength of the anchor reinforcement shall be used for the nominal tensile strength, Q_{nt}, of the steel headed stud anchor for use in Equation I8-5.

(b) As stipulated by the *applicable building code* or ACI 318, Appendix D.

3d. Shear Strength of Steel Channel Anchors in Composite Components

The available shear strength of steel channel anchors shall be based on the provisions of Section I8.2b with the resistance factor and safety factor as specified below.

$$\phi_v = 0.75 \text{ (LRFD)} \qquad \Omega_v = 2.00 \text{ (ASD)}$$

3e. Detailing Requirements in Composite Components

Steel anchors shall have at least 1 in. (25 mm) of lateral clear concrete cover. The minimum center-to-center spacing of steel headed stud anchors shall be four diameters in any direction. The maximum center-to-center spacing of steel headed stud anchors shall not exceed 32 times the shank diameter. The maximum center-to-center spacing of steel channel anchors shall be 24 in. (600 mm).

User Note: Detailing requirements provided in this section are absolute limits. See Sections I8.3a, I8.3b and I8.3c for additional limitations required to preclude edge and group effect considerations.

I9. SPECIAL CASES

When *composite* construction does not conform to the requirements of Section I1 through Section I8, the strength of *steel anchors* and details of construction shall be established by testing.