17.5.3 Transverse Reinforcement, Solid Slabs

Unless it is known from experience that longitudinal cracking caused by composite action directly over the steel section or joist 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.

17.5.4 Transverse Reinforcement, Ribbed Slabs

17.5.4.1

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.

17.5.4.2

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

17.6 Interconnection

17.6.1

Except as permitted by Clauses 17.6.2 and 17.6.4, interconnection between steel sections, trusses, or joists and slabs or steel decks with cover slabs shall be attained by the use of shear connectors as prescribed in Clause 17.7.

17.6.2

Unpainted steel sections, trusses, or joists that support slabs and are totally encased in concrete do not require interconnection by means of shear connectors provided that

(a) a minimum of 50 mm of concrete covers all portions of the steel section, truss, or joist except as noted in Item (c);

(b) the cover in Item (a) is reinforced to prevent spalling; and

(c) the top of the steel section, truss, or joist is at least 40 mm below the top and 50 mm above the bottom of the slab.

17.6.3

Studs may be welded through a maximum of two steel sheets in contact, each not more than 1.71 mm in overall thickness including coatings (1.52 mm in nominal base steel thickness plus zinc coating not greater than nominal 275 g/m^2). Otherwise, holes for placing studs shall be made through the sheets as necessary. Welded studs shall meet the requirements of CSA Standard W59.

17.6.4

Other methods of interconnection that have been adequately demonstrated by test and verified by analysis may be used to effect the transfer of forces between the steel section, truss, or joist and the slab or steel deck with cover slab. In such cases the design of the composite member shall conform to the design of a similar member employing shear connectors, insofar as practicable.

17.6.5

The diameter of a welded stud shall not exceed 2.5 times the thickness of the part to which it is welded, unless test data satisfactory to the designer are provided to establish the capacity of the stud as a shear connector.

17.7 Shear Connectors

17.7.1 General

The resistance factor, ϕ_{sc}, to be used with the shear resistances given in this Clause shall be taken as 0.80. The factored shear resistance, q_r, of other shear connectors shall be established by tests acceptable to the designer.

17.7.2 End-Welded Studs

End-welded studs shall be headed or hooked with $h/d \geq 4$. The projection of a stud in a ribbed slab, based on its length prior to welding, shall be at least two stud diameters above the top surface of the steel deck.

17.7.2.1

In solid slabs,

$$q_{rs} = 0.50\phi_{sc} A_{sc}\sqrt{f_c' E_c} \leq \phi_{sc}A_{sc}F_u$$

where F_u for commonly available studs is 415 MPa and q_{rs} is in newtons.

17.7.2.2

In ribbed slabs with ribs parallel to the beam,

(a) when $w_d/h_d \geq 1.50$

$q_{rr} = q_{rs}$

(b) when $w_d/h_d < 1.50$

$q_{rr} = [0.6 (w_d/h_d)(h/h_d - 1)]q_{rs} \leq 1.0\, q_{rs}$

17.7.2.3

In ribbed slabs with ribs perpendicular to the beam,

(a) when h_d = 75 mm

$$q_{rr} = 0.35\phi_{sc}\rho A_p \sqrt{f_c'} \leq q_{rs}$$

(b) when h_d = 38 mm

$$q_{rr} = 0.61\, \phi_{sc}\, \rho\, A_p \sqrt{f_c'} \leq q_{rs}$$

where

A_p is the concrete pull-out area taking the deck profile and stud burn-off into account. For a single stud, the apex of the pyramidal pull-out area, with four sides sloping at 45°, is taken as the centre of the top surface of the head of the stud. For a pair of studs, the pull-out area has a ridge extending from stud to stud.

ρ = 1.0 for normal-density concrete (2150 to 2500 kg/m^3)

= 0.85 for semi-low-density concrete (1850 to 2150 kg/m^3)

17.7.2.4

The longitudinal spacing of stud connectors in both solid slabs and in ribbed slabs when ribs of formed steel deck are parallel to the beam shall be not less than six stud diameters. The maximum spacing of studs shall not exceed 1000 mm. See also Clause 17.8.

The transverse spacing of stud connectors shall not be less than four stud diameters.

17.7.3 Channel Connectors

In solid slabs of normal-density concrete with $f_c' \geq 20$ MPa and a density of at least 2300 kg/m^3,

$$q_{rs} = 36.5\, \phi_{sc}(t + 0.5w)L_c\sqrt{f_c'}$$

17.8 Ties

Mechanical ties shall be provided between the steel section, truss, or joist and the slab or steel deck to prevent separation. Shear connectors may serve as mechanical ties if suitably proportioned. The maximum spacing of ties shall not exceed 1000 mm, and the average spacing in a span shall not exceed 600 mm or be greater than that required to achieve any specified fire-resistance rating of the composite assembly.

17.9 Design of Composite Beams with Shear Connectors

17.9.1

The composite beam shall consist of steel section, truss or joist, shear connectors, ties, and slab or steel deck with cover slab.

17.9.2

The properties of the composite section shall be calculated neglecting any concrete area that is in tension within the maximum effective area (equal to effective width times effective thickness). If a steel truss or joist is used, the area of its top chord shall be neglected in determining the properties of the composite section and only Clause 17.9.3(a) is applicable.

17.9.3

The factored moment resistance, M_{rc}, of the composite section with the slab or cover slab in compression shall be calculated as follows, where $\phi = 0.90$ and ϕ_c, the resistance factor for concrete, = 0.60:

(a) **Case 1** — Full shear connection and plastic neutral axis in the slab; that is, $Q_r \geq \phi A_s F_y$ and $\phi A_s F_y \leq 0.85\ \phi_c\ bt,\ f_c'$ where Q_r equals the sum of the factored resistances of all shear connectors between points of maximum and zero moment

$$M_{rc} = T_r e' = \phi A_s F_y e'$$

where e' is the lever arm and is calculated using

$$a = \frac{\phi A_s F_y}{0.85\phi_c b f_c'}$$

(b) Case 2 — Full shear connection and plastic neutral axis in the steel section; that is,

$$Q_r \geq 0.85_c btf_c' \text{and} 0.85\phi_c btf_c' \prec \phi A_s F_y$$

$$M_{rc} = C_r e + C_r' e'$$

$$C_r' = 0.85\phi_c bft_c'$$

$$C_r = \frac{\phi A_s F_y - C_r'}{2}$$

(c) **Case 3 —** Partial shear connection; that is, $Q_r < 0.85\phi_c\ btf_c'$ and $< \phi A_s F_y$

$$M_{rc} = C_r e + C_r' e'$$

$$C_r' = Q_r$$

$$C_r = \frac{\phi A_s F_y - C_r'}{2}$$

where e' is the lever arm and is calculated using

$$a = \frac{C_r'}{0.85\phi_c b f_c'}$$

17.9.4
No composite action shall be assumed in calculating flexural strength when Q_r is less than 0.4 times the lesser of $0.85\phi_c btf_c^{\prime}$ and $\phi A_s F_y$. No composite action shall be assumed in calculating deflections when Q_r is less than 0.25 times the lesser of $0.85\phi_c btf_c^{\prime}$ and $\phi A_s F_y$.

17.9.5
For full shear connection, the total horizontal shear, V_h, at the junction of the steel section, truss, or joist and the concrete slab or steel deck, to be resisted by shear connectors distributed between the point of maximum bending moment and each adjacent point of zero moment shall be

$$V_h = \phi A_s F_y$$

$$V_h = 0.85\phi_c btf_c^{\prime}$$

for Cases 1 and 2 as defined in Clause 17.9.3(a) and (b), respectively, and $Q_r \geq V_h$.

17.9.6
For partial shear connection the total horizontal shear, V_h, as defined in Clause 17.9.3(c) shall be

$$V_h = Q_r$$

17.9.7
Composite beams employing steel sections and concrete slabs may be designed as continuous members. The factored moment resistance of the composite section, with the concrete slab in the tension area of the composite section, shall be the factored moment resistance of the steel section alone, except that when sufficient shear connectors are placed in the negative moment region, suitably anchored concrete slab reinforcement parallel to the steel sections and within the design effective width of the concrete slab may be included in calculating the properties of the composite section. The total horizontal shear, V_h, to be resisted by shear connectors between the point of maximum negative bending moment and each adjacent point of zero moment shall be taken as $\phi A_r F_{yr}$.

17.9.8
The number of shear connectors to be located on each side of the point of maximum bending moment (positive or negative, as applicable), distributed between that point and the adjacent point of zero moment, shall be not less than

$$n = \frac{V_h}{q_r}$$

Shear connectors may be spaced uniformly, except that in a region of positive bending the number of shear connectors, n′, required between any concentrated load applied in that region and the nearest point of zero moment shall be not less than

$$n^{\prime} = n\left(\frac{M_{f1} - M_r}{M_f - M_r}\right)$$

where

M_{f1} = positive bending moment under factored load at concentrated load point
M_r = factored moment resistance of the steel section alone
M_f = maximum positive bending moment under factored load

17.9.9 Longitudinal Shear
The longitudinal shear of composite beams with solid slabs or with cover slabs and steel deck parallel to the beam shall be taken as

$$V_u = \Sigma q_r - 0.85\phi_c f_c^{\prime} A_c - \phi A_r F_{yr}$$

where A_r is the area of longitudinal reinforcement within the concrete area, A_c

For normal-weight concrete, the factored shear resistance along any potential longitudinal shear surfaces in the concrete slab shall be taken as

$$V_r = (0.80\ \phi\ A_r\ F_{yr} + 2.76\phi_c A_{cv}) \leq 0.50\phi_c f_c^{\prime}\ A_{cv}$$

where A_r is the area of transverse reinforcement crossing shear planes, A_{cv}

17.10 Design of Composite Beams without Shear Connectors

17.10.1

Unpainted steel sections or joists supporting concrete slabs and encased in concrete in accordance with Clause 17.6.2 may be proportioned on the basis that the composite section supports the total load.

17.10.2

The properties of the composite section for determination of load carrying capacity shall be calculated by ultimate strength methods, neglecting any area of concrete in tension.

17.10.3

As an alternative method of design, encased simple-span steel sections or joists may be proportioned on the basis that the steel section, truss, or joist alone supports 0.90 times the total load.

17.11 Unshored Beams

For composite beams that are unshored during construction, the stresses in the tension flange of the steel section, truss, or joist due to the loads applied before the concrete strength reaches $0.75f_c^{\prime}$ plus the stresses at the same location due to the remaining specified loads considered to act on the composite section shall not exceed F_y.

17.12 Beams During Construction

The steel section, truss, or joist alone shall be proportioned to support all factored loads applied prior to hardening of the concrete without exceeding its calculated capacity under the conditions of lateral support or shoring, or both, to be furnished during construction.

18. Concrete-Filled Hollow Structural Sections

18.1 Scope

The provisions of Clause 18 apply to composite members consisting of steel hollow structural sections completely filled with concrete.

18.2 Application

Hollow structural sections designated as Class 1, 2, or 3 sections that are completely filled with concrete may be assumed to carry compressive load as composite columns. Class 4 hollow structural sections that are completely filled with concrete may also be designed as composite columns provided that the width-thickness ratios of the walls of rectangular sections do not exceed $1350/\sqrt{F_y}$ and the outside diameter-to-thickness ratios of circular sections do not exceed $28\ 000/F_y$.

18.3 Axial Load on Concrete

The axial load assumed to be carried by the concrete at the top level of a column shall be only that portion applied by direct bearing on the concrete. Similarly a base plate or other means shall be provided for load transfer at the bottom of a column. At intermediate floor levels, direct bearing on the concrete is not necessary.

18.4 Compressive Resistance

The factored compressive resistance of a composite column shall be taken as

$$C_{rc} = \tau C_r + \tau' C_r'$$

where

$$C_r' - 0.85\phi_c f_c' A_c \lambda_c^{-2} \left[\sqrt{1 + 0.25\lambda_c^{-4}} - 0.5\lambda_c^{-2}\right]$$

in which $\dfrac{KL}{r_c}\sqrt{\dfrac{f_c'}{\pi^2 E_c}}$

r_c = radius of gyration of the concrete area, A_c

E_c = initial elastic modulus for,concrete, considering the effects of long-term loading. For normal-weight concrete, with f_c expressed in megapascals, this may be taken as

$$(1 + S/T)\ 2500\sqrt{f_c'}$$

where

S is the short-term load and T is the total load on the column.

For all rectangular hollow structural sections and for circular hollow structural sections with a height-to-diameter ratio of 25 or greater, $\tau = \tau' = 1.0$

Otherwise $\tau = \dfrac{1}{\sqrt{1 + \rho + \rho^2}}$

and $\tau' = 1 + \left(\dfrac{25^{\rho^2}\tau}{(D/t)}\right)\left(\dfrac{F_y}{0.85\ f_{c'}}\right)$

where

ρ = 0.02 (25 – L/D)

18.5 Bending

The factored moment resistance of a composite rectangular section shall be calculated as follows, where $\phi = 0.90$ and $\phi_c = 0.60$.

$$M_{rc} = C_r e + C_r' e'$$

$$C_r' = \phi_c a(b-2t) f_c'$$

$$C_r = \frac{\phi A_s F_y - C_r'}{2}$$

$$C_r + C_r' = T_r = \phi A_{st} F_y$$

Note: *The concrete in compression is taken to have a rectangular stress block of intensity* f_c' *over a depth of a = 0.85c where c is the depth of the concrete in compression.*

18.6 Axial Compression and Bending

18.6.1 Method 1: Bending Resisted by Composite Section

Members required to resist both bending moments and axial compression shall be proportioned analogously with Clause 13.8.1 so that

$$\frac{C_f}{C_{rc}} + \frac{B\ \omega_1\ M_f}{M_{rc}\left(1 - \dfrac{C_f}{C_{ec}}\right)} \leq 1.0 \text{ and}$$

$$\frac{M_f}{M_{rc}} \leq 1.0$$

where

M_{rc} is as defined in Clause 18.5

$$B = \frac{C_{rco} - C_{rcm}}{C_{rco}}$$

C_{rco} = factored compressive resistance with $\lambda = 0$

C_{rcm} = factored compressive resistance that can coexist with M_{rc} when all of the cross-section is in compression

Conservatively, B may be taken as 1.0.

18.6.2 Method 2: Bending Assumed to be Resisted by the Steel Section Alone

For members required to resist both bending moments and axial compression, under this assumption, the steel section shall be proportioned as a beam-column in parallel with Clause 13.8.1, to carry the total bending plus axial compression equal to the difference between the total axial compression and that portion that can be sustained by the concrete:

$$M_f \leq \tau\ M_r$$

and if $C_f > \tau'\ C_r'$

$$\frac{C_f - \tau' C_r'}{\tau\ C_r} + \frac{\omega_1 M_f}{\tau\ M_r\left(1 - \dfrac{C_f - \tau'\ C_r'}{C_e}\right)} \leq 1.0$$

19. General Requirements for Built-Up Members

19.1 Members in Compression

19.1.1

All components of built-up compression members and the transverse spacing of their lines of connecting bolts or welds shall meet the requirements of Clauses 10 and 11.

19.1.2

All component parts that are in contact with one another at the ends of built-up compression members shall be connected by bolts spaced longitudinally not more than four diameters apart for a distance equal to 1.5 times the width of the member or by continuous welds having a length of not less than the width of the member.

19.1.3

Unless closer spacing is required for transfer of load or for sealing inaccessible surfaces, the longitudinal spacing in-line between intermediate bolts or the clear longitudinal spacing between Intermittent welds in built-up compression members shall not exceed the following, as applicable:

(a) 330t/, *sert*F_y, but not more than 300 mm for the outside component of the section consisting of a plate when the bolts on all gauge lines or intermittent welds along the component edges are not staggered, where t = thickness of the outside plate;

(b) 525t/, $\sqrt{F_y}$, but not more than 450 mm for the outside component of the section consisting of a plate when the bolts or intermittent welds are staggered on adjacent lines, where t = thickness of the outside plate.

19.1.4

Compression members composed of two or more rolled shapes in contact or separated from one another shall be interconnected such that the slenderness ratio of any component, based on its least radius of gyration and the distance between interconnections, shall not exceed that of the built-up member. The compressive resistance of the built-up member shall be based on:

(a) the slenderness ratio of the built-up member with respect to the appropriate axis when the buckling mode does not involve relative deformation that produces shear forces in the interconnectors;

(b) an equivalent slenderness ratio, with respect to the axis orthogonal to that in (a), when the buckling mode involves relative deformation that produces shear forces in the interconnectors, taken as

$$\rho_e = \sqrt{\rho_o 2 + \rho_i 2}$$

ρ_e = equivalent slenderness ratio of built-up member

ρ_o = slenderness ratio of built-up member acting as an integral unit

ρ_i = maximum slenderness ratio of component part of a built-up member between interconnectors;

(c) for built-up members composed of two interconnected rolled shapes, in contact or separated only by filler plates, such as back-to-back angles or channels, the maximum slenderness ratio of component parts between fasteners or welds shall be based on an effective length factor of 1.0 when the fasteners are snug-tight bolts and 0.65 when welds or pretensioned bolts are used;

(d) for built-up members composed of two interconnected rolled shapes separated by lacing or batten plates, the maximum slenderness ratio of component parts between fasteners or welds shall be based on an effective length factor of 1.0 for both snug-tight and pretensioned bolts and for welds.

19.1.5

For starred angle compression members interconnected at least at the one-third points, Clause 19.1.4 need not apply.

19.1.6

The fasteners and interconnecting parts, if any, of members defined in Clause 19.1.4(c) shall be proportioned to resist a force equal to one per cent of the total force in the built-up member.

19.1.7

The spacing requirements of Clauses 19.1.3, 19.2.3, and 19.2.4 may not always provide a continuous tight fit between components in contact. When the environment is such that corrosion could be a serious problem, the spacing of bolts or welds may need to be less than the specified maximum.

19.1.8

Open sides of compression members built up from plates or shapes shall be connected to each other by lacing, batten plates, or perforated cover plates.

19.1.9
Lacing shall provide a complete triangulated shear system and may consist of bars, rods, or shapes. Lacing shall be proportioned to resist a shear normal to the longitudinal axis of the member of not less than 2.5% of the total axial load on the member plus the shear from transverse loads, if any.

19.1.10
The slenderness ratio of lacing members shall not exceed 140. The effective length for single lacing shall be the distance between connections to the main components; for double lacing connected at the intersections, the effective length shall be 70% of that distance.

19.1.11
Lacing members shall preferably be inclined to the longitudinal axis of the built-up member at an angle of not less than 45°.

19.1.12
Lacing systems shall have diaphragms in the plane of the lacing and as near to the ends as practicable, and at intermediate points where lacing is interrupted. Such diaphragms may be plates (tie plates) or shapes.

19.1.13
End tie plates used as diaphragms shall have a length not less than the distance between the lines of bolts or welds connecting them to the main components of the member. Intermediate tie plates shall have a length of not less than one-half of that prescribed for end tie plates. The thickness of tie plates shall be at least 1/60 of the width between lines of bolts or welds connecting them to the main components, and the longitudinal spacing of the bolts or clear longitudinal spacing between welds shall not exceed 150 mm. At least three bolts shall connect the tie plate to each main component or, alternatively, a total length of weld not less than one-third the length of tie plate shall be used.

19.1.14
Shapes used as diaphragms shall be proportioned and connected to transmit from one main component to the other a longitudinal shear equal to 5% of the axial compression in the member.

19.1.15
Perforated cover plates may be used in lieu of lacing and tie plates on open sides of built-up compressive members. The net width of such plates at access holes shall be assumed to be available to resist axial load, provided that

(a) the width-thickness ratio conforms to Clause 11;

(b) the length of the access hole does not exceed twice its width;

(c) the clear distance between access holes in the direction of load is not less than the transverse distance between lines of bolts or welds connecting the perforated plate to the main components of the built-up member; and

(d) the periphery of the access hole at all points has a minimum radius of 40 mm.

19.1.16
Battens consisting of plates or shapes may be used on open sides of built-up compression members that do not carry primary bending in addition to axial load. Battens shall be provided at the ends of the member, at locations where the member is laterally supported along its length, and elsewhere as determined by Clause 19.1.4.

19.1.17

Battens shall have a length of not less than the distance between lines of bolts or welds connecting them to the main components of the member, and shall have a thickness of not less than 1/60 of this distance, if the batten consists of a flat plate. Battens and their connections shall be proportioned to resist, simultaneously, a longitudinal shear force,

$$V_f = \frac{0.025\ C_f\ d}{na}$$

and a moment,

$$M_f = \frac{0.025\ C_f\ d}{2n}$$

where
d = longitudinal centre-to-centre distance between battens, mm
a = distance between lines of bolts or welds connecting the batten to each main component, mm
n = number of parallel planes of battens

19.2 Members in Tension

19.2.1

Members in tension composed of two or more shapes, plates, or bars separated from one another by intermittent fillers shall have the components interconnected at fillers spaced so that the slenderness ratio of any component between points of interconnection shall not exceed 300.

19.2.2

Members in tension composed of two plate components in contact or a shape and a plate component in contact shall have the components interconnected so that the spacing between connecting bolts or clear spacing between welds does not exceed 36 times the thickness of the thinner plate nor 450 mm (see Clause 19.1.3).

19.2.3

Members in tension composed of two or more shapes in contact shall have the components interconnected so that the spacing between connecting bolts or the clear spacing between welds does not exceed 600 mm, except where it can be determined that a greater spacing would not affect the satisfactory performance of the member (see Clause 19.1.3).

19.2.4

Members in tension composed of two separated main components may have either perforated cover plates or tie plates on the open sides of the built-up member. Tie plates, including end tie plates, shall have a length of not less than two-thirds of the transverse distance between bolts or welds connecting them to the main components of the member and shall be spaced so that the slenderness ratio of any component between the tie plates does not exceed 300. The thickness of tie plates shall be at least 1/60 of the transverse distance between the bolts or welds connecting them to the main components and the longitudinal spacing of the bolts or welds shall not exceed 150 mm. Perforated cover plates shall comply with the requirements of Clause 19.1.15(b), (c), and (d).

19.3 Open Box-Type Beams and Grillages

Two or more rolled beams or channels used side-by-side to form a flexural member shall be connected together at intervals of not more than 1500 mm. Through-bolts and separators may be used provided that, in beams having a depth of 300 mm or more, no fewer than two bolts shall be