10 COMPOSITE CONSTRUCTION

This section gives recommendations for the interactive behaviour between structural steel and concrete designed to utilize the best load-resisting characteristics of each material. Detailed design considerations and design methods of the following composite members are provided:

- Composite beams with either a solid slab or a composite slab using profiled steel sheets.
- Composite slabs with profiled steel sheets of either trapezoidal or re-entrant cross-sections.
- Composite columns with fully encased H sections, partially encased H sections and infilled rectangular and circular hollow sections.

Recommendation on the use of shear connectors is also given. However, the design of composite joints is not covered, and reference to specialist design recommendation shall be made.

10.1 MATERIALS

10.1.1 Structural steel

Structural steel shall comply with clause 3.1 with proper allowances on strength, resistance to brittle fracture, ductility and weldability.

10.1.2 Concrete

Normal weight concrete shall comply with the recommendations given in HKCC. The nominal maximum size of aggregate shall not exceed 20 mm. In the absence of other information, the wet and the dry densities of reinforced concrete shall be taken as 2450 kg/m^3 and 2350 kg/m^3 respectively, and the grade specified shall be in the range of C25 to C60. For concrete grades above C60, a performance-based approach based on test and analytical methods should be used to justify the composite behaviour. According to the HKCC, the short-term elastic modulus, E_{cm} (kN/mm^2), of the normal weight concrete is given by:

$$E_{cm} = 3.46\sqrt{f_{cu}} + 3.21 \qquad (10.1)$$

where

f_{cu} is the cube compressive strength of concrete (N/mm^2).

Table 10.1 - Compressive strength and short-term elastic modulus for various concrete grades

Concrete Grade	Cube compressive strength, f_{cu} (N/mm^2)	Elastic modulus, E_{cm} (kN/mm^2)
C25	25	20.5
C30	30	22.2
C35	35	23.7
C40	40	25.1
C45	45	26.4
C50	50	27.7
C55	55	28.9
C60	60	30.0

For design data on creep coefficient, shrinkage coefficient and coefficient of thermal expansion for concrete, refer to HKCC.

10.1.3 Reinforcement

Reinforcement shall comply with HKCC, and the characteristic strength, f_y , shall not be larger than 500 N/mm^2. The elastic modulus shall be taken as 205 kN/mm^2, i.e. same as that of structural steel sections.

Different types of reinforcement may be used in the same structural member.

10.1.4 Shear connectors

Shear connection shall be capable of transmitting longitudinal shear forces between the concrete and the steel section due to factored loads, without causing crushing or other damage to the concrete and without allowing excessive slip or separation between the concrete and the steel section.

10.1.4.1 Headed shear studs

Shear connectors commonly take the form of headed studs welded to the steel section, either directly or through profiled steel sheets. The purpose of the head of the studs is to resist any uplift component of the forces applied to the studs.

The stud material shall be mild steel with the following minimum properties (before cold drawn or cold forging):

Ultimate tensile strength, f_u:	450 N/mm^2
Elongation (on a gauge length of $5.65\sqrt{A_o}$):	15%

where

f_u is the ultimate strength of the stud material,

A_o is the original cross section area.

The minimum diameter and the minimum depth of the head of a headed stud shall be $1.5d$ and $0.4d$ respectively, where d is the nominal shank diameter of the stud.

10.1.4.2 Other types of shear connectors

Other materials may also be used for shear connectors provided that they can be demonstrated to

i) produce shear connection possessing sufficient deformation capacity as shown in clause 10.3, and
ii) prevent separation between the concrete and the steel section effectively.

10.1.5 Profiled steel sheets

10.1.5.1 Specification

The steel used to manufacture profiled steel sheets shall have a yield strength between 220 and 550 N/mm^2.

10.1.5.2 Sheet thickness

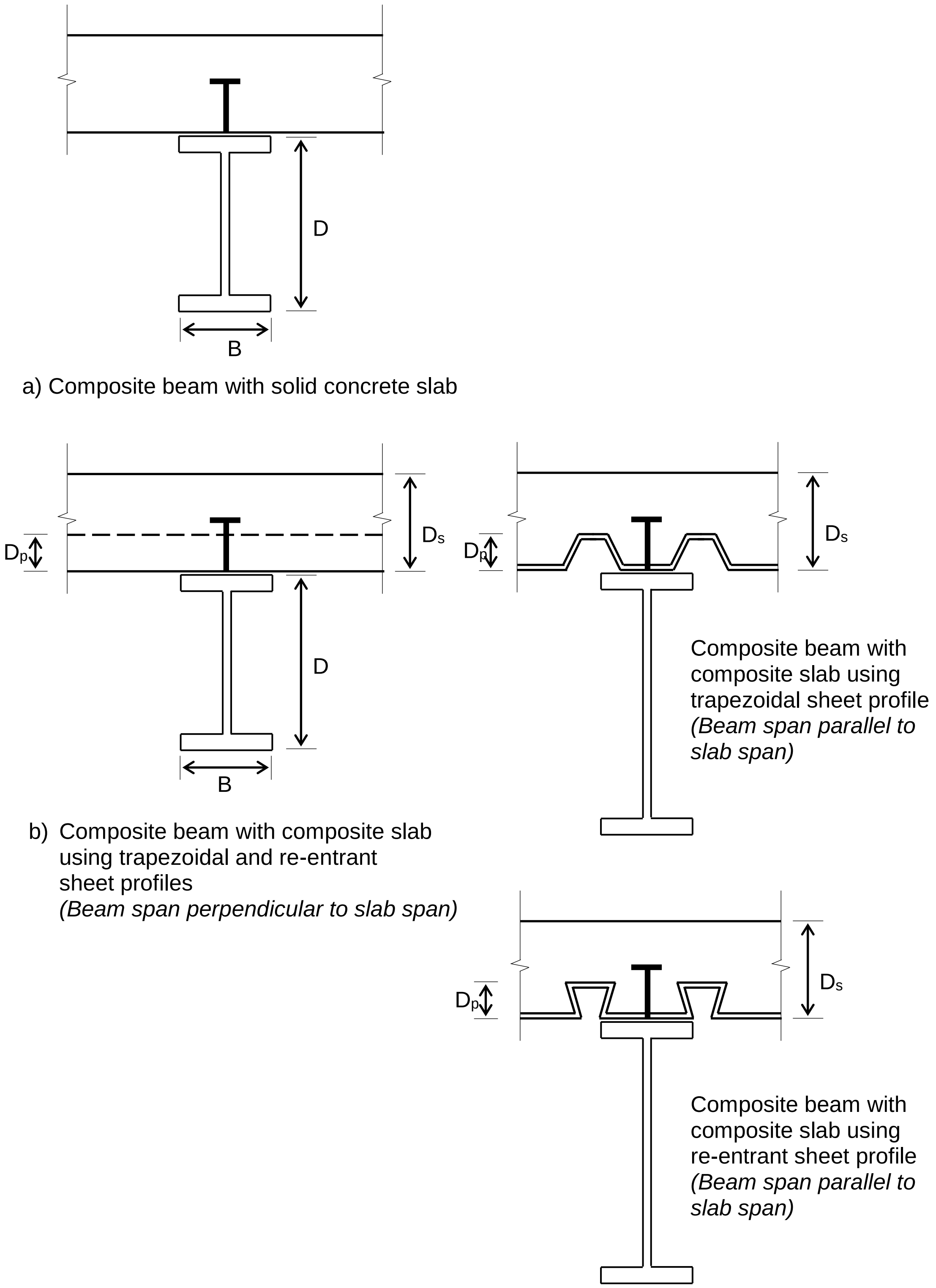
The structural thickness of the profiled steel sheets, to which the stresses and section properties apply, is the “bare metal thickness” of the sheets excluding any protective or decorative finish such as zinc coating or organic coating.

The nominal bare metal thickness of the sheets shall not normally be less than 0.70 mm except where the profiled steel sheets are used only as permanent shuttering. Thinner sheets should not be used unless proper justification on their resistance against local damage is provided.

10.2 COMPOSITE BEAMS

10.2.1 General

(1) This clause presents the design of composite beams with either solid slabs or composite slabs using profiled steel sheet.



a) Composite beam with solid concrete slab

b) Composite beam with composite slab using trapezoidal and re-entrant sheet profiles *(Beam span perpendicular to slab span)*

Composite beam with composite slab using trapezoidal sheet profile *(Beam span parallel to slab span)*

Composite beam with composite slab using re-entrant sheet profile *(Beam span parallel to slab span)*

Figure 10.1 - Typical composite beams

(2) This clause applies to composite beams using steel sections with yield strengths between 235 and 460 N/mm^2 and C25 to C60 concrete.

The steel sections in the composite beams shall be bi-symmetrical I or H sections. The flanges of the steel sections should be Class 1 plastic, Class 2 compact or Class 3 semi-compact while the webs should be at least Class 3 semi-compact.

(3) Composite beams shall be checked for

- resistance of critical cross-sections;
- resistance to lateral-torsional buckling;
- resistance to shear buckling and transverse forces on webs; and
- resistance to longitudinal shear.

(4) Critical cross-section shall include:

- sections under maximum moment;
- supports;
- sections subject to concentrated loads or reactions; and
- places where a sudden change of cross-section occurs, other than a change due to cracking of concrete.

A cross-section with a sudden change in geometry should be considered as a critical cross-section when the ratio of the greater to the lesser resistance moment is greater than 1.2.

(5) For checking resistance to longitudinal shear, a critical length consists of a length of the interface between two critical cross-sections. For this purpose, critical cross-sections also include:

- free-end of cantilever; and
- in tapering members, sections so chosen that the ratio of the greater to the lesser plastic resistance moments (under flexural bending of the same direction) for any pair of adjacent cross-sections does not exceed 1.5.

(6) The concepts "full shear connection" and "partial shear connection" are applicable only to beams in which plastic theory is used for calculating bending resistances of critical cross-sections. A span of beam, or a cantilever, has full shear connection when increase in the number of shear connectors does not increase the design bending resistance of the member. Otherwise, the shear connection is partial.

(7) Refer to clause 3.1.2 for the design strength of the structural steel section, p_y. The design strengths of the concrete, f_{cd} , and the steel reinforcement, f_{sd} , are given as follows:

$$f_{cd} = f_{cu} / \gamma_c \qquad \gamma_c = 1.5 \tag{10.2}$$

$$f_{sd} = f_y / \gamma_s \qquad \gamma_s = 1.15 \tag{10.3}$$

where

f_{cu} is the cube compressive strength of concrete;

f_y is the characteristic strength of steel reinforcement; and

γ_c, γ_s are the partial safety factors of concrete and steel reinforcement respectively.

(8) In unpropped construction, composite beams with Class 1 plastic or Class 2 compact compression steel flanges throughout shall be designed assuming that at the ultimate limit state the whole of the loading acts on the composite beams, provided that the longitudinal shear is calculated accordingly.

Where propped construction is used, all composite beams shall be designed assuming that at the ultimate limit state the whole of the loading acts on the composite beams.

10.2.2 Analysis of internal forces and moments

10.2.2.1 Simply supported or cantilever beams

Elastic analysis shall be used to evaluate both the shear forces and the moments.

10.2.2.2 Continuous beams

The moments in continuous composite beams shall be determined using any of the following methods, provided that the beams comply with the relevant conditions.

a) Simplified method (see clause 10.2.2.3)
b) Elastic global analysis (see clause 10.2.2.4)
c) Plastic global analysis (see clause 10.2.2.5)

All composite beams are assumed to be effectively continuous at all internal supports while the supports are assumed to be simple supports. In each case, the shear forces shall be in equilibrium with the moments and the applied loads.

10.2.2.3 Simplified method

The moments in continuous composite beams shall be determined using the coefficients given in Table 10.2, provided that the following conditions are satisfied.

a) In each span, the cross section of the steel beam is uniform with equal flanges and without any haunches.
b) The same beam section is used in all spans.
c) The dominant loading is uniformly distributed.
d) The unfactored imposed load does not exceed 2.5 times the unfactored dead load.
e) No span is less than 75% of the longest.
f) End spans do not exceed 115% of the length of the adjacent span.
g) There is no cantilever.

Table 10.2 - Simplified moment coefficients

<table>
<tr><th rowspan="3">Location</th><th rowspan="3">Number of spans</th><th colspan="4">Classification of compression flange at supports</th></tr>
<tr><th rowspan="2">Class 3 Semi-compact</th><th rowspan="2">Class 2 Compact</th><th colspan="2">Class 1 Plastic</th></tr>
<tr><th>Generally</th><th>Non-reinforced</th></tr>
<tr><td>Middle of end span</td><td>2
3 or more</td><td>+0.71
+0.80</td><td>+0.71
+0.80</td><td>+0.75
+0.80</td><td>+0.79
+0.82</td></tr>
<tr><td>First internal support</td><td>2
3 or more</td><td>- 0.81
- 0.76</td><td>- 0.71
- 0.67</td><td>- 0.61
- 0.57</td><td>- 0.50
- 0.48</td></tr>
<tr><td>Middle of internal spans</td><td>3
4 or more</td><td>+0.51
+0.65</td><td>+0.52
+0.65</td><td>+0.56
+0.65</td><td>+0.63
+0.67</td></tr>
<tr><td>Internal supports except the first</td><td>4 or more</td><td>- 0.67</td><td>- 0.58</td><td>- 0.50</td><td>- 0.42</td></tr>
</table>

Note:
1) The coefficients shall be multiplied by the free bending moment $WL/8$, where W is the total factored load on the span L.
2) Where the spans in each side of a support differ, the mean of the values of $WL/8$ for the two adjacent spans shall be used to calculate the support moment.
3) The values of the coefficients already allow for both pattern loads and possible moment redistribution. No further redistribution shall be carried out when using this method.
4) For the use of non-reinforced Class 1 plastic sections, refer to clause 10.2.2.6.

10.2.2.4 Elastic global analysis

(1) Elastic global analysis of continuous beams shall be carried out using the section properties of the gross uncracked section described in clause 10.2.5.3(2) throughout. The resulting negative moment at any support shall be reduced (except adjacent to cantilevers) by an amount not exceeding the appropriate maximum percentage given in Table 10.3. Corresponding increases shall then be made to the positive moments in the adjacent spans to maintain equilibrium with the applied loads. The shear forces shall also be adjusted, if necessary, to maintain equilibrium.

For beams with unequal spans with over 15% different in lengths, concrete cracking in internal supports with short spans should be properly allowed and designed for.

(2) Alternatively, provided that the ratios of shorter to longer span are smaller than 0.6, an elastic global analysis shall be carried out assuming that for a length of 15% of the span on each side of internal supports, the section properties are those of the cracked section under negative moments (see clause 10.2.5.3(4)). Elsewhere the section properties of the gross uncracked section are used.

The resulting moments shall be adjusted by an amount not exceeding the appropriate maximum percentage given in Table 10.3. It is also permissible to iteratively adjust the length of span which is assumed to be cracked on each side of an internal support, to correspond to the points of contraflexure determined from the redistributed moment diagram.

Table 10.3 - Maximum redistribution of support moments for elastic global analysis

Elastic global analysis using	Classification of compression flange at supports			
	Class 3 Semi-compact	Class 2 Compact	Class 1 Plastic	
			Generally	Non-reinforced
Gross uncracked section	20%	30%	40%	50%
Cracked section	10%	20%	30%	40%

Note: "Non-reinforced" sections are defined in clause 10.2.2.6.

Imposed loads shall be arranged in the most unfavourable realistic pattern for each case. Dead load γ_f factors shall not be varied when considering such pattern loading, i.e. either 1.0 or 1.4 for all spans.

For continuous beams subject to uniformly distributed imposed load, only the following arrangements of imposed load shall be considered.

a) Alternate spans loaded
b) Two adjacent spans loaded

10.2.2.5 *Plastic global analysis*

Plastic global analysis shall be used to determine the moments in continuous beams with non-reinforced Class 1 plastic sections (see clause 10.2.2.6) at internal supports and with Class 1 plastic sections at mid-span, provided that conditions a) to d) given in clause 10.2.2.3 for the simplified method are satisfied.

Alternatively, a plastic global analysis shall be used to determine the moments in continuous beams subject to the following conditions:

a) Adjacent spans do not differ by more than 33% of the larger span.
b) End spans do not exceed 115% of the length of the adjacent span.
c) In any span in which more than half the total factored load on a span is concentrated within a length of one-fifth of the span, the cross section at each positive moment plastic hinge location is such that the plastic neutral axis lies within 0.15(D + D_s) below the top of the concrete flange, where D_s is the depth of the concrete flange.
   Note: This condition need not be satisfied where it can be shown that the hinge will be the last to form in that span.
d) At plastic hinge locations, both the compression flange and the web are Class 1 plastic.

10.2.2.6 *Non-reinforced Class 1 plastic sections*

The recommendations given for non-reinforced Class 1 plastic sections apply exclusively to cross sections with only nominal tension reinforcement in negative moment regions. The nominal tension reinforcement should be neglected when calculating the plastic moment capacity (see clause 10.2.5.1). The classification of both the web and the compression flange shall be Class 1 plastic, in accordance with clause 10.2.4.

10.2.3 Establishment of composite cross-sections

10.2.3.1 Effective span

(1) The effective span of a beam, L, shall be taken as the distance between the centres of the supports, but not greater than the clear distance between the supports plus the depth of the steel member.

(2) The effective length of a cantilever shall be taken from the centre of the support, but not greater than the projecting length from the face of the support plus half the depth of the steel member.

10.2.3.2 Effective section

The moment capacity of a composite beam shall be based on the following effective cross section:

(1) The total effective breadth B_e of the concrete flange (see clause 10.2.3.3) shall be used.

(2) The effective section of a composite slab which spans onto a beam with its ribs running perpendicular to the beam shall be taken as the concrete above the top of the ribs only. The concrete within the depth of the ribs shall be neglected.

The effective section of a composite slab with its ribs running parallel to the beam shall be taken as the full cross section of the concrete.

The effective section of a composite slab with its ribs running at an angle θ to the beam shall be taken as the full area of the concrete above the top of the ribs plus $cos^2\theta$ times the area of the concrete within the depth of the ribs. Alternatively, for simplicity, the concrete within the depth of the ribs should conservatively be neglected.

Concrete in tension shall be neglected, and profiled steel sheets shall not be included in the calculation of the effective section.

Reinforcement in compression shall be neglected, unless it is properly restrained by links in accordance with HKCC. Moreover, all welded mesh reinforcement and any bar reinforcement which is less than 10 mm in diameter shall be treated as nominal reinforcement and shall not be included in the calculation of the effective section.

10.2.3.3 Effective breadth of concrete flange

(1) Allowance shall be made for the in-plane shear flexibility (shear lag) of a concrete flange by using an effective breadth.

(2) The total effective breadth B_e of concrete flange acting compositely with a steel beam shall be taken as the sum of the effective breadths b_e of the portions of flange each side of the centreline of the steel beam.

In the absence of any more accurate determination, the effective breadth of each portion shall be taken as follows:

- For a slab spanning perpendicular to the beam,
  $b_e = L_z / 8 \;\; \leq \; b$ (10.4)
- For a slab spanning parallel to the beam,
  $b_e = L_z / 8 \;\; \leq \; 0.8b$ (10.5)
  where L_z is the distance between points of zero moment.
  However, if a separate allowance is made for the co-existing effects of slab bending,
  $b_e = L_z / 8 \;\; \leq \; b$ (10.6)
- For a simply supported beam, L_z is equal to the effective span L (see clause 10.2.3.1)
- For a continuous beam, L_z should be obtained from Figure 10.2.

(3) The actual breadth b of each portion shall be taken as half the distance to the adjacent beam, measured to the centreline of the web, except that at a free edge the actual breadth is the distance from the beam to the free edge; refer to Figure 10.2.

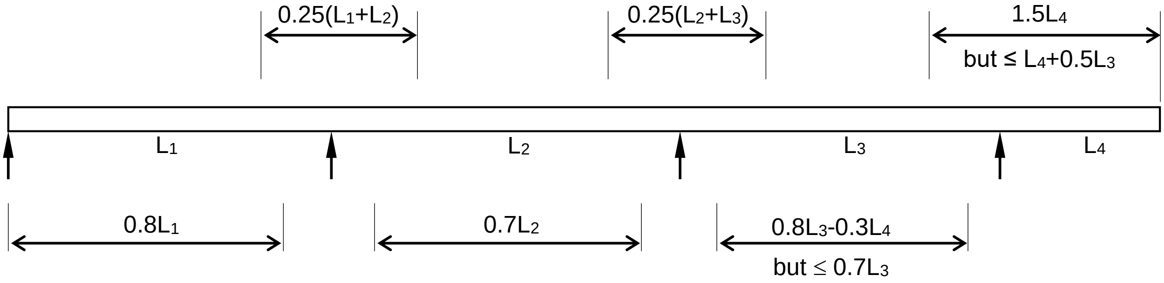


Figure 10.2 - Value of L_z for continuous beams

10.2.3.4 Modular ratio

(1) The elastic section properties of a composite cross-section shall be expressed in terms of an equivalent steel section by dividing the contributions of the concrete flange by the effective modular ratio, α_e .

In the absence of detailed information about the effective modular ratio, α_e , the following expression shall be adopted:

$$\alpha_e = \alpha_s + \rho_L (\alpha_L - \alpha_s) \quad (10.7)$$

where

α_L is the modular ratio for long-term loading;
α_s is the modular ratio for short-term loading;
ρ_L is the proportion of the total loading which is long term.

The values of short-term and long-term modular ratios in Table 10.4 shall be used for all grades of concrete.

Table 10.4 - Modular ratios for normal weight concrete

Modular ratio for short-term loading α_s	**Modular ratio for long-term loading α_L**
8	22

Refer to clause 3.1 of HKCC for detailed information on the long-term deformation of concrete due to creep and drying shrinkage.

(2) Storage loads and loads which are permanent in nature shall be taken as long term. Imposed roof loads, wind loads and snow loads shall be treated as short term.

(3) For the purpose of determining the modular ratio, all spans are assumed to be fully loaded.

(4) In the absence of detailed information about the nature of imposed loads, imposed loads on floors shall be assumed to be two-third short-term and one-third long-term in general.

10.2.4 Classification of composite cross-sections

(1) General
The capacities of composite cross sections shall be limited by local buckling in the steel web or in the steel compression flange. In the absence of a more refined calculation, the design method given in clause 10.2.6 shall be adopted.

In calculations for the construction stage of a composite beam based on the plain steel section, the classification of cross sections shall be in accordance with section 7.

(2) Classification limits

To classify a composite cross section, the position of the neutral axis shall be based on the effective cross section determined in accordance with clause 10.2.3.2.

When the concrete flange is in tension, the flange reinforcement should be included in the calculation of the effective section if utilized in the design of the member.

Classification of composite cross sections shall be determined according to Tables 10.5 and 10.6.

Table 10.5 - Limiting width to thickness ratios for flanges and webs in composite sections

(Elements which exceed these limits are to be taken as Class 4 slender)

Type of element		Class of section		
		Class 1 Plastic	**Class 2 Compact**	**Class 3 Semi-compact**
Outstand element of compression flange, b / T	Rolled section	9 ε	10 ε	15 ε
	Welded section	8 ε	9 ε	13 ε
Web, d / t	With neutral axis at mid-depth	80 ε	100 ε	120 ε
	Generally	$\frac{64\varepsilon}{1+r}$	$\frac{76\varepsilon}{1+r}$	See Table 10.6

Table 10.6 - Limiting width to thickness ratios for semi-compact webs

(Elements which exceed these limits are to be taken as Class 4 slender)

Web, generally	**Class 3 Semi-compact**	
r ≥ 0.66	$\frac{114\varepsilon}{1+2r}$	for rolled sections
	$\left(\frac{41}{r}-13\right)\varepsilon$	for welded sections
0.66 > r ≥ 0	$\frac{114\varepsilon}{1+2r}$	
0 > r	$\frac{114\varepsilon(1+r)}{\frac{3}{2}(1+2r)}$	

Notes:

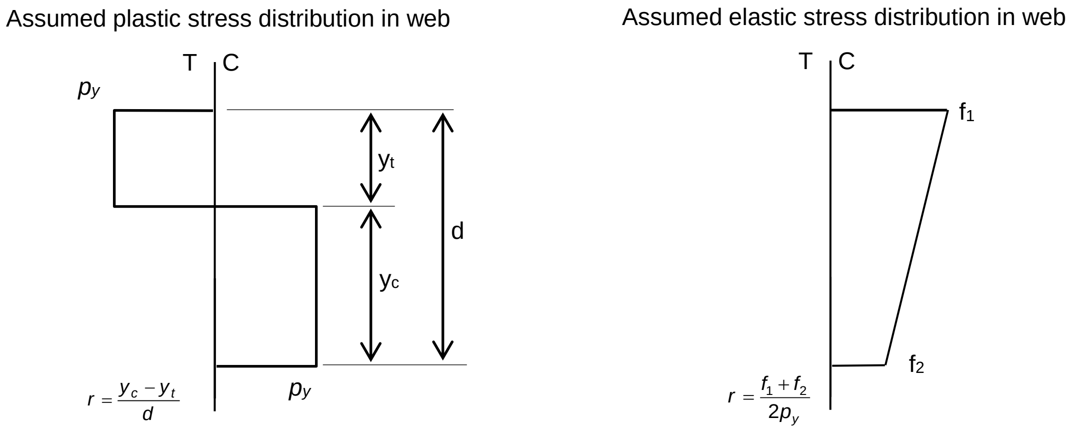
1) These ratios apply to composite sections. During construction, the classification in section 7 applies.

2) Check webs for shear buckling with section 8 when d / t ≥ 63 ε.

3) The values in this table do not apply to T sections.

4) $\varepsilon = \sqrt{275/p_y}$

5) r is the ratio of the mean longitudinal stress in the web to the design strength p_y, compressive stresses being taken as positive and tensile stresses as negative (see Figure 10.3).



a) Section with Class 1 plastic or Class 2 compact compression flange

b) Section with Class 3 semi-compact compression flange

Figure 10.3 - Determination of web stress ratio *r*

The value of r is given by:

$$r = -\frac{F_c}{R_w} \quad \text{but} \quad r \geq -1 \qquad \text{for positive moment} \qquad (10.8a)$$

$$= -\frac{R_r}{R_w} \quad \text{but} \quad r \leq -1 \qquad \text{for negative moment} \qquad (10.8b)$$

where

F_c is the compressive force in the concrete flange;

R_r is the resistance of the reinforcement equal to f_{sd} A_r;

R_w is the resistance of the clear web depth equal to $d\ t\ p_y$;

In the case of partial shear connection, reference shall also be made to clause 10.3.3.2(2).

(3) Enhancement due to attachments

a) A steel compression flange restrained by effective attachment to a solid concrete flange by shear connectors in accordance with clause 10.3 shall be considered as Class 1 plastic.

b) Where a steel compression flange is restrained by effective attachment by shear connectors in accordance with clause 10.3 to a composite slab in which either:

- the ribs run at an angle of at least 45° to the axis of the beam; or
- the breadth b_r (as defined in clause 10.3, measured perpendicular to the axis of the rib) of the rib located directly over the beam is not less than half the breadth of the beam flange;

then the classification of the compression flange shall be considered as:

- Class 1 plastic if its classification in accordance with Table 10.5 is Class 2 compact; or
- Class 2 compact if its classification in accordance with Table 10.5 is Class 3 semi-compact.

10.2.5 Section capacities and properties of composite cross-sections

(1) The moment capacities of composite cross-sections shall be determined by rigid-plastic theory only where the effective composite cross-sections is Class 1 plastic or Class 2 compact, and where no pre-stressing by tendons is used.

The rigid-plastic theory is also applicable to a composite cross-section with a steel section of Class 1 or Class 2 flanges but a Class 3 semi-compact web provided that the depth of web taken as effective in resisting compression is reduced according to clause 10.2.5.2.

(2) Elastic analysis and non-linear theory for moment capacities shall be applied to cross-sections of any class.

(3) For elastic analysis and non-linear theory, it shall be assumed that the composite cross-section remains plane after bending if the shear connection and the transverse reinforcement are designed in accordance with clause 10.3, considering appropriate distributions of design longitudinal shear forces.

(4) The tensile strength of concrete shall be neglected.

(5) Where the steel section of a composite member is curved in plan, the effects of curvature should be taken into account.

(6) For composite cross-sections using steel sections with yield strengths larger than 420 N/mm^2 but less than 460 N/mm^2, where the distance y_{pl} between the plastic neutral axis and the extreme fibre of the concrete flange in compression exceeds 15% of the overall depth h of the composite cross-section (i.e. depth of steel beam plus depth of concrete flange), the design resistance moment M_c should be taken as $\beta\, M_c$ where β is a reduction factor given by

$$\beta = 1 \quad \text{when} \quad y_{pl} / h \leq 0.15 \qquad (10.9a)$$
$$= 0.85 \quad \text{when} \quad y_{pl} / h = 0.4 \qquad (10.9b)$$

Linear interpolation is permitted for β when y_{pl} / h is within the range of 0.15 to 0.4.

(7) Plastic moment resistance of composite cross-section with partial shear connection.

a) In regions of sagging bending, partial shear connection in accordance with clause 10.3 shall be used in composite beams for buildings.

b) Unless otherwise verified, the plastic moment capacity in hogging bending shall be determined in accordance with clause 10.2.5.1, and full shear connection should be provided to ensure yielding of reinforcement in tension, i.e. partial shear connection is not permitted.

c) Where ductile shear connectors are used, the moment capacity of the critical cross-section of the composite beam, M_c , shall be calculated by means of rigid plastic theory in accordance with clause 10.2.5.1, except that a reduced value of the compressive force in the concrete flange R_q should be used in place of R_c when $R_c < R_s$, or in place of R_s when $R_c > R_s$, where

R_c = Resistance of concrete flange

$$= 0.45\, f_{cu}\, B_e\, (D_s – D_p) \qquad (10.10)$$

R_s = Resistance of steel beam

$$= A\, p_y \qquad (10.11)$$

R_q = Resistance of shear connection

$$= N\, P \qquad (10.12)$$

where

N is the number of shear connectors from the point of zero moment to the point of maximum moment.

P = P_p is the design resistance of shear connectors given in clause 10.3.2

The degree of shear connection, k_{sc} , is given by:

$$k_{sc} = \frac{R_q}{R_s} \quad \text{when} \quad R_s < R_c \quad \text{or} \qquad (10.13a)$$

$$= \frac{R_q}{R_c} \quad \text{when} \quad R_s > R_c \qquad (10.13b)$$

The location of the plastic neutral axis in the slab shall be determined with R_q. It should be noted that there is a second plastic neutral axis within the steel section, which should be used for the classification of the steel web.

d) The plastic moment capacity of the composite cross-section with partial shear connection, M_{co} , may be determined conservatively as follows:

$$M_{co} = M_s + k_{sc} (M_c - M_s) \quad (10.14)$$

where

M_c = moment capacity of composite section
M_s = moment capacity of steel section
k_{sc} = degree of shear connection

e) For composite beams under sagging moment, the following limit on the minimum degree of shear connection applies, but not less than 0.4:

For span L_e up to 25 m,

$$k_{sc} \geq 1 - \left(\frac{355}{p_y}\right)(0.75 - 0.03 L_e) \quad (10.15a)$$

For L_e exceed 25 m,

$$k_{sc} = 1 \quad (10.15b)$$

where L_e is the distance in sagging moment between points of zero moment in meters.

10.2.5.1 *Plastic moment capacity*

The plastic moment capacity of a composite cross section with Class 1 or Class 2 flanges and webs shall be calculated on the following basis.

(1) Concrete is stressed to a uniform compression of 0.45 f_{cu} over the full depth of concrete on the compression side of the plastic neutral axis.

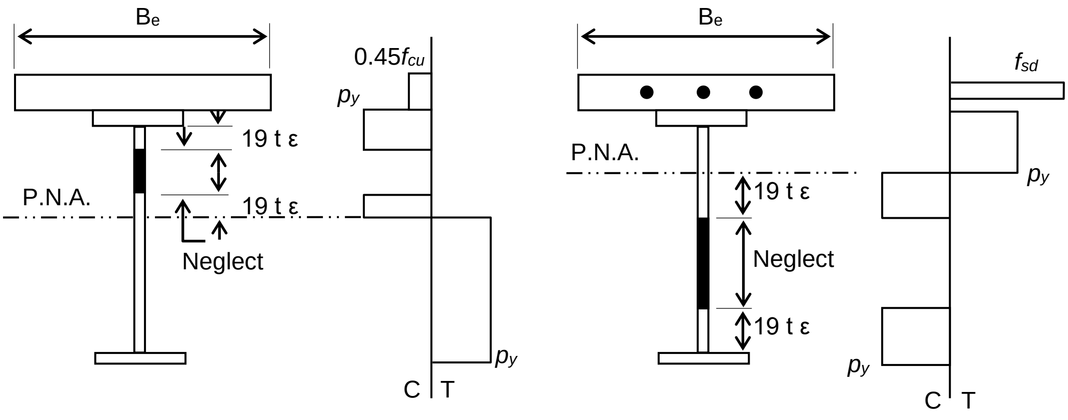
(2) The structural steel section is stressed to its design strength p_y either in tension or in compression.

(3) Longitudinal reinforcement is stressed to its design strength f_{sd} where it is in tension.

10.2.5.2 *Reduced plastic moment capacity*

The reduced plastic moment capacity of a composite beam with Class 1 or Class 2 steel compression flanges but with a Class 3 semi-compact steel web shall be determined using the reduced section shown in Figure 10.4.

The depth of steel web taken as effective in resisting compression shall be limited to 19 t ε adjacent to the compression steel flange and 19 t ε adjacent to the plastic neutral axis. The remainder of the steel web on the compression side of the plastic neutral axis shall be neglected.



a) Positive moments b) Negative moments

NOTE: P.N.A. denotes plastic neutral axis of effective section

Figure 10.4 - Reduced section

10.2.5.3 Second moment of area and elastic section modulus

(1) For composite beams, three possible values of the second moment of area should be distinguished:

I_g for the gross section, i.e. uncracked section;
I_p for the cracked section value under positive moments; and
I_n for the cracked section value under negative moments.

The appropriate value of the second moment of area should be used as follows:

a) I_g for elastic global analysis with gross uncracked section method (see clause 10.2.2.4(1));
b) I_g and I_n for elastic global analysis with cracked section method (see clause 10.2.2.4(2));
c) I_g, I_p or I_n for elastic section modulus (see clause 10.2.5.3(5)), as appropriate;
d) I_g for deflection calculations (see clause 10.2.7.1).

(2) Gross uncracked section, I_g
The gross value of the second moment area of the uncracked composite section I_g shall be calculated using the mid-span effective breadth of the concrete flange with concrete flange uncracked but unreinforced. The full area of concrete within the effective breadth of the concrete flange shall be included in the effective section.

Alternatively, the concrete within the depth of the ribs shall conservatively be neglected for simplicity. Any concrete beam casing shall be neglected.

(3) Cracked section under positive moments, I_p
For positive moments, the second moment of area of the cracked composite section shall be calculated using the mid-span effective breadth of the concrete flange but neglecting any concrete in tension.

(4) Cracked section under negative moments, I_n
For negative moments, the second moment of area of the cracked composite section shall be calculated using a section comprising the steel section together with the effectively anchored reinforcement located within the effective breadth of the concrete flange at the support.

(5) Elastic section modulus
In determining stresses at the serviceability limit state, the elastic section modulus of a composite section shall be determined from the appropriate value of the second moment of area as follows:

a) For positive moments:
I_g if the elastic neutral axis is located within the steel section;
I_p if the elastic neutral axis is located within the concrete flange.

b) For negative moments:
I_n .

10.2.6 Ultimate limit state design

10.2.6.1 Moment capacities

(1) Simply supported beams
The moment capacity of simply supported composite beams with steel sections of Class 1 plastic or Class 2 compact compression flanges subject to positive moment shall be taken as the plastic moment capacities of the composite sections, provided that the webs are not Class 4 slender. If the web is Class 1 plastic or Class 2 compact, the plastic moment capacity in clause 10.2.5.1 should be used. If the web is Class 3 semi-compact, the reduced plastic moment capacity in clause 10.2.5.2 should be used.

(2) Cantilevers
The moment capacity of cantilever composite beams shall be based on the steel sections together with any effectively anchored tension reinforcement within the effective breadth of the concrete flanges. However, tension reinforcement which

is provided to reinforce the slabs for moments due to loading acting directly on them should not be included.

When the compression flange is Class 1 plastic or Class 2 compact, the moment capacity should be taken as the plastic moment capacity, provided that the web is not Class 4 slender. If the web is Class 1 plastic or Class 2 compact, the plastic moment capacity in clause 10.2.5.1 should be used. If the web is Class 3 semi-compact, the reduced plastic moment capacity in clause 10.2.5.2 should be used.

(3) Continuous beams
The positive moment capacity of continuous beams shall be determined as for simply supported beams (see clause 10.2.6.1(1)) and the negative moment capacities shall be determined as for cantilevers (see clause 10.2.6.1(2)).

10.2.6.2 Shear capacities

The steel beams shall be conservatively designed in accordance with Section 8 to resist the whole of the vertical shear force.

10.2.6.3 Combined bending and shear

The reduction of moment capacity due to high shear force shall be determined as follows:

$$M_{cv} = M_c - \rho\,(M_c - M_f) \quad \text{when} \quad V \geq 0.5\,V_c \tag{10.16}$$

where

M_{cv} is the reduced plastic moment capacity of the composite cross-section under high shear force;

M_c is the plastic moment capacity of the composite cross-section;

M_f is the plastic moment capacity of that part of the section remaining after deduction of the shear area A_v defined in section 8;

$$\rho = \left(\frac{2V}{V_c} - 1\right)^2$$

V is the shear force;

V_c is the lesser of the shear capacity and the shear buckling resistance, both determined from section 8.

For a composite cross-section with a web of Class 3 semi-compact, M_c is the reduced plastic moment capacity to clause 10.2.5.2.

Alternatively, the influence of transverse shear force shall be taken into account through a reduced design strength $(1 - \rho)\,p_y$ in the shear area A_v of the steel section to clause 8.2.1.

In all cases, the shear force F_v should not exceed the shear capacity of the steel section determined according to clause 8.2.1.

10.2.6.4 Stability of compression flanges

(1) The stability of the bottom flanges in continuous beams shall be checked for each span in turn.

The span being checked shall be assumed to be loaded with factored dead load only and the negative moments at each internal support shall be assumed to be equal to the relevant moment capacity M_c (elastic, plastic or reduced plastic) applicable for design of the composite cross-section at the support (see Figure 10.5).

However, the support moments shall be taken as more than those obtained from an elastic analysis (using the properties of the gross uncracked section) without redistribution.

(2) To prevent lateral-torsional buckling, the compression flanges should be laterally restrained as recommended in section 8. When checking the lateral stability of the bottom flanges in negative moment regions, the methods given in clause 8.3 should be used.

Alternatively, other methods that include allowances for the torsional restraint provided by the concrete slab should also be used.

(3) At plastic hinge locations, other than the last hinge to form in each span, the recommendations for plastic hinge locations given in section 8 shall be followed.

Where the reduction of negative moments as described in elastic global analysis using the gross section method exceeds 30% at the supports of beams of uniform section (or 20% when using the cracked section method), the points of support shall be treated as plastic hinge locations.

In beams of varying section, the locations of the potential negative moment plastic hinges, implied by the redistribution of support moments, should be identified. When the reduction of negative moments at such locations exceeds 30% in elastic global analysis using the gross section method (or 20% when using the cracked section method), they should be treated as active plastic hinge locations.

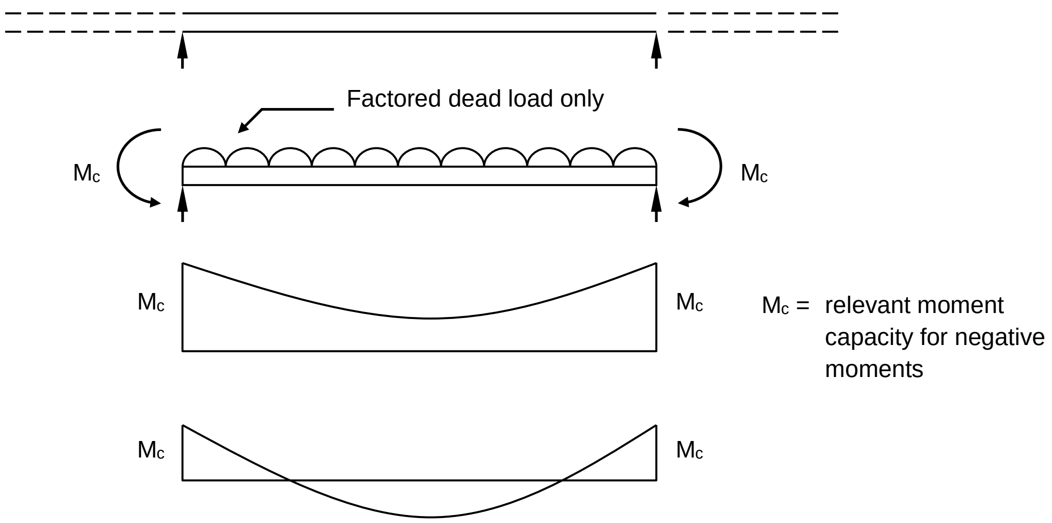


Figure 10.5 - Lateral buckling of bottom flanges in continuous composite beams

10.2.7 Serviceability limit state design

10.2.7.1 Deflection

(1) General
The deflection under serviceability loads of a building or part shall not impair the strength or efficiency of the structure or its components or cause damage to the finishing. Deflections shall be determined under serviceability loads.

(2) Deflection of a composite beam at the construction stage due to the dead load of the concrete flange and the steel beam should be limited when unpropped construction is used. Precambering in the steel beam may be adopted to reduce the deflection of the composite beam under the self-weights of the steel beam and the concrete flange.

(3) For unpropped construction, the imposed load deflection shall be based on the properties of the composite cross-section but the dead load deflection, due to the self-weights of the steel beam and the concrete flange, shall be based on the properties of the steel beam.

(4) For propped construction, all deflections shall be based on the properties of the composite cross-section. When calculating deflections, the behaviour of composite beams shall be taken as linear elastic, except for the redistribution of moments recommended in clause 10.2.7.1(6) and the increased deflections for partial shear connection recommended in clause 10.2.7.1(6e).

(5) Simply supported beams
Deflections of simply supported composite beams shall be calculated using the properties of the gross uncracked section described in clause 10.2.5.3(2).

(6) Continuous beams

For continuous beams, the imposed load deflections shall allow for the effects of pattern loading. Where design at the ultimate limit state is based on plastic global analysis or on an analysis involving significant redistribution of support moments, the effects of shakedown on deflections should also be included in the imposed load deflections.

As an alternative to rigorous analysis, the following methods should be used to allow for the effects of pattern loading and shakedown by modifying the initial support moments.

a) Allowance for pattern loading

The initial moments at each support shall be determined for the case of unfactored imposed load on all spans. Reductions shall then be made to these initial support moments (except adjacent to cantilevers) to allow for pattern loading, as follows:

- for normal loading type: 30%
- for storage loading type: 50%

b) Allowance for shakedown effects

Allowance shall be made for the effects of shakedown if the beam has been designed for the ultimate limit state using:

- plastic global analysis (see clause 10.2.2.5);
- elastic global analysis, using the properties of the gross uncracked section (see clause 10.2.2.4(1)) with redistribution exceeding 40%;
- elastic global analysis, using the properties of the cracked section (see clause 10.2.2.4(2)) with redistribution exceeding 20%.

The support moments shall be determined, without any redistribution, for the following combination of unfactored loads:

- for normal loading type: dead load plus 80% of imposed load;
- for storage loading type: dead load plus 100% of imposed load.

Where these support moments exceed the plastic moment capacity of the section for negative moments, the excess moments should be taken as the moments due to shakedown.

The deflections produced by these shakedown moments shall be added to the imposed load deflections. This shall be done by further reducing the calculated support moments due to imposed loading, by values equal to the shakedown moments, in addition to the reductions for the effects of pattern loading.

c) Calculation of moments

The support moments shall be based on an analysis using the properties of the gross uncracked section throughout.

Alternatively, provided the conditions given in clause 10.2.2.3 for the simplified method are satisfied, the support moments should be taken as follows:

- two-span beam: $WL/8$;
- first support in a multi-span beam: $WL/10$;
- other internal supports: $WL/14$.

In these expressions, W is the appropriate unfactored load on the span L. Where the spans in each side of a support differ, the mean of the values of WL for the two adjacent spans should be used.

d) Calculation of deflections

The imposed load deflection in each span shall be based on the loads applied to the span and the support moments for that span, modified as recommended to allow for pattern loading and shakedown effects.

Provided that the steel beam is of uniform section without any haunches, the properties of the gross uncracked composite section should be used throughout.

The dead load deflections shall be based on an elastic analysis of the beam. For unpropped construction, the properties of the steel beam should be used. For propped construction, the properties of the gross uncracked composite section should be used.

For continuous beams under uniform load or symmetric point loads, the deflection δ_c at mid-span is given by:

$$\delta_c = \delta_o \left[1 - 0.6 \frac{M_1 + M_2}{M_o} \right] \tag{10.17}$$

where

δ_o is the deflection of a simply supported beam for the same loading;

M_o is the maximum moment in the simply supported beam;

M_1 and M_2 are the moments at the adjacent supports (modified as appropriate).

e) Partial shear connection

The increased deflection under serviceability loads arising from partial shear connection is given by:

- For propped construction

$$\delta = \delta_c + 0.5 \left(1 - k_{sc}\right)\left(\delta_s - \delta_c\right) \tag{10.18a}$$

- For unpropped construction

$$\delta = \delta_c + 0.3 \left(1 - k_{sc}\right)\left(\delta_s - \delta_c\right) \tag{10.18b}$$

where

k_{sc} is the degree of shear connection defined in clause 10.2.5 (7c);

δ_s is the deflection for the steel beam acting alone;

δ_c is the deflection of a composite beam with full shear connection for the same loading.

For continuous beams, the same formulae apply, but δ_s and δ_c refer to the deflection of the continuous beam, δ_c being calculated as recommended in clause 10.2.7.1(6d).

10.2.7.2 Irreversible deformation and check against stresses

(1) To prevent gross deformations under normal service conditions, irreversible deformations should be avoided in simply supported beams, cantilevers, and in the mid-span regions of continuous beams.

The stresses based on the elastic properties of the section shall be calculated under serviceability loading. It is not necessary to modify the elastic section modulus to take into account partial shear connection at the serviceability limit state.

(2) In simply supported beams and cantilevers and the mid-span regions of continuous beams, the stresses in the extreme fibre of the steel beam shall not exceed the design strength p_y and the stress in the concrete flange shall not exceed 0.5 f_{cu}.

(3) It is not necessary to limit the stresses over the supports of continuous beams, provided that the recommendations given in clause 10.2.7.1(6) are followed.

It should be noted that it is normally not necessary to calculate stresses in unpropped composite beams with symmetric cross sections whenever the unfactored imposed load is larger than the unfactored dead load.

10.2.7.3 *Cracking*

(1) Where it is required to limit the crack width, reference shall be made to HKCC. Where environmental conditions will not give rise to corrosion, such as in heated office buildings, it is not normally necessary to check crack widths in the design of composite beams, even where the composite beams are designed as simply supported, provided that the concrete flange slab is reinforced as recommended to reduce concrete cracking to HKCC.

(2) In cases of exposure to adverse environmental conditions (such as floors in car-parking structures or roofs generally) additional reinforcement in the concrete flange over the beam supports shall be provided to control cracking and the relevant clauses in HKCC shall be referred to. To avoid visible cracks where hard finishes are used, the use of crack control joints in the finishes should be considered.

(3) In general, nominal reinforcement with a cross sectional area of 0.2% of the cross section of the concrete slab over the profiled steel sheets shall be provided in unpropped composite slab construction.

In propped composite slab construction or situations where cracking may be visually important, the nominal reinforcement should be increased to 0.4% of the corresponding cross sectional area of the concrete slab.

10.2.7.4 *Vibration*

Where vibration may cause discomfort to the occupants of a building or damage to its contents, the response of long-span composite floors should be considered and complied with clause 5.4.

10.3 SHEAR CONNECTION

10.3.1 General

The shear connection shall be capable of transmitting the longitudinal shear force between the concrete and the steel section due to factored loads, without causing crushing or other damage to the concrete and without allowing excessive slip or separation between the concrete and the steel section.

Shear connectors shall be provided along the entire length of composite beams to transmit the longitudinal shear force between the concrete slab and the steel beam. Moreover, transverse reinforcements shall also be provided along the entire length of composite beams to prevent longitudinal shear failure and splitting of concrete slab due to concentrated forces applied by the shear connectors.

10.3.2 Design resistance of shear connectors

10.3.2.1 General

In a solid slab, the design resistance of shear connectors against longitudinal shear is given by:

a) For positive moments, $P_p = 0.8\ P_k$ (10.19a)

b) For negative moments, $P_n = 0.6\ P_k$ (10.19b)

where

P_k is the characteristic resistance of the shear connector.

The characteristic resistance for a headed stud shall be obtained by reference to clause 10.3.2.2. As values are not at present given in this Code for types of shear connector other than headed studs, the characteristic resistances of other types of shear connector shall be determined from push out tests.

10.3.2.2 Headed shear studs in solid slabs

The characteristic resistance P_k of a headed shear stud with the dimensions and properties given in clause 10.1.4.1 embedded in a solid slab of normal weight concrete shall be taken from Table 10.7.

Table 10.7 - Characteristic resistance P_k of headed shear studs in normal weight concrete

Characteristic resistance of headed shear studs P_k (kN)									
Dimensions of headed shear stud		Cube compressive strength of concrete, f_{cu} (N/mm^2)							
Nominal shank diameter (mm)	Minimum as-welded height (mm)	C25	C30	C35	C40	C45	C50	C55	C60
25	100	116.1	133.1	147.6	162.4	176.7	176.7	176.7	176.7
22	88	89.9	102.4	114.3	125.8	136.8	136.8	136.8	136.8
19	76	67.1	76.3	85.3	93.8	102.1	102.1	102.1	102.1
16	64	47.5	54.2	60.5	66.5	72.4	72.4	72.4	72.4

Note: For cube compressive strength of concrete greater than 60 N/mm^2, the values of P_k should be taken as those with f_{cu} and E_{cm} limiting to those of concrete grade C60.

The characteristic resistance of headed shear studs, P_k , is given by:

$$P_k = 0.29\, d^2\, \alpha\, \sqrt{0.8 f_{cu}\ E_{cm}} \quad \leq \quad 0.8 f_u \left(\frac{\pi d^2}{4}\right) \qquad (10.20)$$

where

d = diameter of headed shear studs, 16 mm ≤ d ≤ 25 mm

$$\alpha = 0.2\left(\frac{h}{d}+1\right) \quad \text{for} \quad 3 \leq \frac{h}{d} \leq 4$$

$$= 1 \quad \text{for} \quad \frac{h}{d} > 4$$

h = overall height of headed shear studs;
f_{cu} = cube compressive strength of concrete;
E_{cm} = elastic modulus of concrete from clause 10.1.2 according to HKCC;
f_u = ultimate tensile strength of stud material before cold-drawn (or cold-forging) which is taken conservatively as 450 N/mm^2 according to clause 10.1.4.1.

10.3.2.3 *Headed shear studs with profiled steel sheets*

(1) General

The recommendations of this clause apply to headed shear studs with the dimensions and properties given in clause 10.1.4.1, embedded in slabs comprising profiled steel sheets and concrete. These recommendations apply only when all the following conditions are satisfied.

a) The overall depth of the profiled steel sheet is not less than 35 mm nor greater than 80 mm.
b) The mean width of the troughs of the profiled steel sheet is not less than 50 mm.
c) The height of the headed shear studs is at least 35 mm greater than the overall depth of the profiled steel sheet.

(2) Ribs perpendicular to the beam

The capacity of headed shear studs in composite slabs with the ribs running perpendicular to the beam shall be taken as their capacity in a solid slab (see clause 10.3.2.1) multiplied by the shape correction factor k given by the following expressions:

for 1 stud per rib: $$k = 0.7\frac{b_r}{D_p}\left(\frac{h}{D_p}-1\right) \leq 1.0 \qquad (10.21)$$

for 2 studs or more per rib: $$k = 0.5\frac{b_r}{D_p}\left(\frac{h}{D_p}-1\right) \leq 0.8 \qquad (10.22)$$

where

b_r is the average width of the concrete rib for trapezoidal profiles or the minimum width for re-entrant profile as defined in clause 10.3.2.3 (6);
D_p is the overall depth of the profiled steel sheet;
h is the overall height of the headed shear stud, and it should not be taken as more than $2D_p$ or D_p + 75 mm, whichever is less, although studs of greater height may be used.

(3) Ribs parallel to the beam

The capacity of headed shear studs in composite slabs with the ribs running parallel to the beam shall be taken as their capacity in a solid slab (see clause 10.3.2.1) multiplied by the shape correction factor k given by the following expression:

$$k = 0.6\frac{b_r}{D_p}\left(\frac{h}{D_p}-1\right) \leq 1.0 \qquad (10.23)$$

where

b_r, D_p and h are as in clause 10.3.2.3.

Where there is more than one longitudinal line of studs in a concrete rib, the mean width b_a of the trough of the profiled steel sheet should be at least 50 mm greater than the transverse spacing of the lines of studs.

Optionally, the trough of the profiled steel sheet may be split longitudinally and separated to form a wider concrete rib over the flange of the steel beam and, in this case, b_r should also be increased accordingly in the expression for k given above.

(4) Ribs running at an angle to the beam

Where the ribs run at an angle θ to the beam, the shape correction factor k should be determined from the expression:

$$k = k_1 \sin^2\theta + k_2 \cos^2\theta \qquad (10.24)$$

where

k_1 *is the value of k from clause 10.3.2.3 (2);*
k_2 *is the value of k from clause 10.3.2.3 (3).*
Alternatively, the smaller value of k_1 and k_2 shall be used.

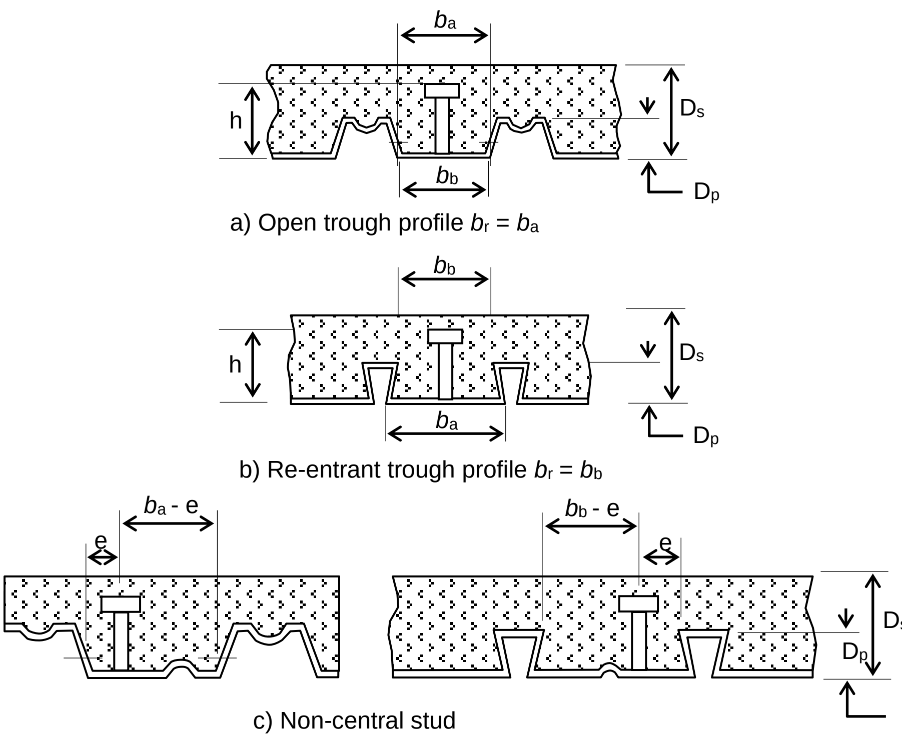
(5) Upper limits for the shape correction factor k
In the absence of suitable test data, the shape correction factor k should be taken as the appropriate values given in Table 10.8.

Table 10.8 - Upper limit for *k*

Condition	Number of headed shear studs per rib	Thickness of profiled steel sheet (mm)	Upper limit for k
'*Welded-through*' profiled steel sheet with $d \leq 20$	1	$\leq$ 1.0 $>$ 1.0	0.85 1.0
	2	$\leq$ 1.0 $>$ 1.0	0.7 0.8
Profiled steel sheet with pre-formed holes using d = 19 and 22 only	1	0.75 to 1.5	0.75
	2	0.75 to 1.5	0.60

Note: d denotes diameter of headed shear studs (mm).

(6) Trough width
Provided that the headed shear studs are located centrally in the rib, b_r shall be taken as the mean width of the trough b_a for trapezoidal profiled steel sheet, but equal to the minimum width of the trough b_b for re-entrant profiled steel sheet (see Figure 10.6).



a) Open trough profile $b_r = b_a$

b) Re-entrant trough profile $b_r = b_b$

c) Non-central stud

Figure 10.6 - Breadth of concrete rib, b_r

Where it is necessary for the headed shear studs to be located noncentrally in the rib, the studs should preferably be placed in the favourable location such that the zone of concrete in compression in front of the stud is maximized. Where it is necessary for the headed shear studs to be placed in the unfavourable location, b_r should be reduced to 2e, where e is the distance to the nearer side of the rib (see Figure 10.6). The distance e shall be not less than 25 mm.

Where the headed shear studs are placed in pairs but in an off-set pattern alternately on the favourable and unfavourable sides (subject to the minimum spacings given in clause 10.3.4.1), b_r should be determined as for centrally located studs.

10.3.3 Provision of shear connectors

10.3.3.1 Full shear connection

(1) Positive moments

For full shear connection, the number of shear connectors N_p, required to develop the positive moment capacity of the section, i.e. the number of shear connectors required to be provided along each side of the point of maximum moment (see clause 10.3.3.3), is given by:

$$N_p P_p \geq F_p \tag{10.25}$$

where

P_p is the design resistance of the shear connector in positive moment regions obtained from clause 10.3.2.1;

F_p is the longitudinal compressive force in the concrete slab at the point of maximum positive moment. It is equal to the lesser of the resistances of steel section and the concrete flange when the design is based on the plastic moment capacity of the composite section, or it is determined from the calculated stresses in the concrete slab when the design is based on the elastic moment capacity of the composite section.

(2) Negative moments

For full shear connection, the number of shear connectors N_n, required to develop the negative moment capacity of the section, i.e. the number of shear connectors required to be provided along each side of the point of maximum moment (see clause 10.3.3.3), is given by:

$$N_n P_n \geq F_n \tag{10.26}$$

where

P_n is the design resistance of the shear connector in negative moment regions obtained from clause 10.3.2.1;

F_n is the resistance of longitudinal reinforcement located within the effective cross section.

The number of shear connectors provided to develop the negative moment capacity shall not be reduced below N_n. This also applies where the elastic moment capacity is used.

10.3.3.2 Partial shear connection

(1) Conditions

This method should be used only when the shear connectors are headed shear studs with the dimensions and properties given in clause 10.1.4.1 or other types of shear connectors which have at least the same deformation capacity as headed shear studs. The spacing of the shear connectors should satisfy the recommendations given in clause 10.3.3.3.

(2) Number of shear connectors

Where the maximum positive moment in a span is less than the plastic moment capacity of the composite section, calculated on the basis given in clause 10.2 as appropriate, the actual number of shear connectors N_a shall be reduced below N_p, the number required for full shear connection, as given in clause 10.3.3.1.

No reduction shall be made in the number of shear connectors N_n required for full shear connection for negative moments.

The reduced plastic moment capacity of the composite section shall be calculated assuming a reduced value of the compressive force F_c in the concrete flange equal to the resistance of the shear connection R_q as follows:

$$N_a P_p = R_q = F_c \qquad (10.27)$$

The classification of the web (see clause 10.2) should be based on the value of the ratio r determined assuming that the compressive force F_c in the concrete flange is equal to R_q.

10.3.3.3 Spacing of shear connectors

(1) Conditions

The total number of shear connectors between a point of maximum positive moment and each adjacent support shall not be less than the sum of N_p and N_n, obtained from clause 10.3.3.1.

All the shear connectors should be spaced uniformly along the beam, provided that the recommendations on spacing in clauses 10.3.3.3(2) to 10.3.3.3(6) are satisfied. Where variation of the spacing is necessary for any reason, the shear connectors should be spaced uniformly within two or more zones, changing at intermediate points which comply with clause 10.3.3.3(5).

In continuous beams, the shear connectors should be spaced more closely in negative moment regions, where this is necessary, to suit the curtailment of tension reinforcement. In cantilevers, the spacing of the shear connectors should be based on the curtailment of the tension reinforcement.

(2) Additional checks

Additional checks on the adequacy of the shear connection as recommended in clause 10.3.3.3(5) shall be made at intermediate points where any of the following apply.

a) A heavy concentrated load occurs within a positive moment region.
b) A sudden change of cross section occurs.
c) The member is tapered (see clause 10.3.3.3(3)).
d) The concrete flange is unusually large (see clause 10.3.3.3(4)).

In case a), a concentrated load should be considered as “heavy” if its free moment M_o exceeds 10% of the positive moment capacity of the composite section. The free moment M_o is the maximum moment in a simply supported beam of the same span due to the concentrated load acting alone.

(3) Tapered members

In members which reduce in depth towards their supports, additional checks as recommended in clause 10.3.3.3(5) shall be made at a series of intermediate points, selected such that the ratio of the greater to the lesser moment capacity for any pair of adjacent intermediate points does not exceed 2.5.

(4) Large concrete flanges

If the concrete flange is so large that the plastic moment capacity of the composite section exceeds 2.5 times the plastic moment capacity of the steel member alone, additional checks as recommended in clause 10.3.3.3(5) shall be made at intermediate points approximately mid-way between points of maximum positive moment and each adjacent support.

(5) Adequacy of shear connection at intermediate points

The adequacy of the shear connection shall be checked at all intermediate points where the spacing of shear connectors changes (see clause 10.3.3.3(1)) and at the intermediate points described in clause 10.3.3.3(2).

The total number of shear connectors between any such intermediate point and the adjacent support shall not be less than N_i determined from the following expressions:

For positive moments:

$$N_i = \frac{M - M_s}{M_c - M_s} N_p + N_n \qquad \text{but } N_i \geq N_n \qquad (10.28)$$

For negative moments:

$$N_i \quad = \quad \frac{M_c - M}{M_c - M_s} N_n \qquad \text{but} \quad N_i \;\le\; N_n \qquad (10.29)$$

where

M is the moment at the intermediate point;

M_c is the positive or negative moment capacity of the composite section, as appropriate;

M_s is the moment capacity of the steel member.

Alternatively, for positive moments, the adequacy of the shear connection shall be demonstrated by checking the plastic moment capacity at the intermediate point, assuming that the compressive force F_c in the concrete flange is equal to (N_a - N_n) P_p where N_a is the actual number of shear connectors between the intermediate point and the adjacent support. In this check, the classification of the web (see clause 10.2) should also be based on the value of the ratio r determined using the above value of F_c.

(6) Curtailment of reinforcement

Where tension reinforcement is used in negative moment regions, every bar should extend beyond the point at which it is no longer required to assist in resisting the negative moment, by a distance not less than 12 times the bar size. In addition the lengths of the bars shall comply with the recommendations given in HKCC for anchorage of bars in a tension zone.

The longest bars shall extend beyond the zone containing the N_n shear connectors required to transfer the longitudinal force F_n, by a distance not less than the longitudinal spacing of the shear connectors. If necessary, the lengths of the bars should be increased to achieve this. Alternatively, the shear connectors should be spaced more closely in this region to avoid increasing the lengths of the bars.

10.3.4 Detailing of shear connectors

10.3.4.1 General

(1) Maximum spacing

The longitudinal spacing of shear connectors shall not normally exceed 600 mm or $4D_s$, whichever is less, where D_s is the overall depth of the slab.

Shear connectors may be arranged in groups, with a mean spacing as above and a maximum spacing of $8D_s$, provided that due account is taken of the resulting non-uniform flow of longitudinal shear and of the greater possibility of vertical separation between the concrete flange and the steel beam. Where the stability of either the steel beam or the concrete flange depends on the shear connectors, the maximum spacing should be limited accordingly, and appropriate resistance to uplift should be provided.

(2) Edge distance

The clear distance between a shear connector and the edge of the steel flange shall not be less than 20 mm (see Figure 10.7(a)).

(3) Minimum spacing

The minimum centre-to-centre spacing of stud shear connectors is $5d$ along the beam and $4d$ between adjacent studs, where d is the nominal shank diameter. Where rows of studs are staggered, the minimum transverse spacing of longitudinal lines of studs is $3d$.

(4) Maximum diameter

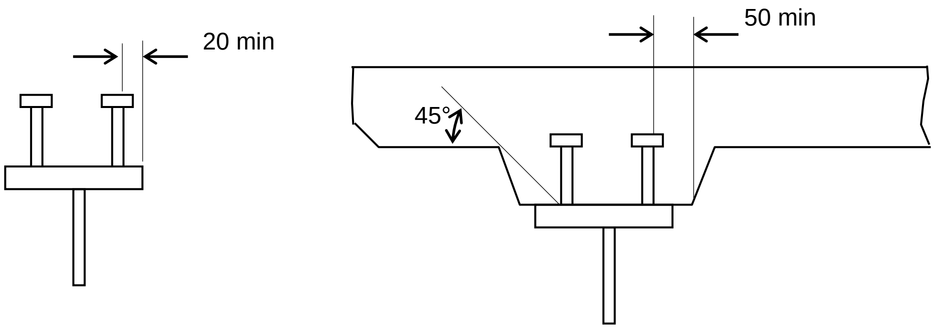
Unless located directly over the web, the nominal diameter of a stud shear connector should not exceed 2.5 times the thickness of the flange to which it is welded.

10.3.4.2 *Other types of shear connectors*

The dimensional details and the minimum spacing of other types of shear connector shall be within the ranges demonstrated as satisfactory by push out tests.

10.3.4.3 *Haunches*

Except where profiled steel sheets are used, the sides of a concrete haunch between the steel beam and the soffit of the slab should lie outside a line drawn at 45° from the outside edge of the shear connectors, and the concrete cover to the shear connectors should not be less than 50 mm (see Figure 10.7(b)).



(a) Minimum edge distance

(b) Concrete haunch

All dimensions shown are in mm

Figure 10.7 - Minimum dimensions

10.3.5 Transverse reinforcement

10.3.5.1 *General*

Transverse reinforcement refers to the reinforcement in the concrete flange running transverse to the span of the beam. Where profiled steel sheets are used they may also act as transverse reinforcement (see clause 10.3.5.4).

Sufficient transverse reinforcement should be used to enable the concrete flange to resist the longitudinal shear force transmitted by the shear connectors, both immediately adjacent to the shear connectors and elsewhere within its effective breadth.

10.3.5.2 *Longitudinal shear in the slab*

The total longitudinal shear force per unit length v to be resisted at any point in the span of the beam shall be determined from the spacing of the shear connectors by the following expression:

$$v = \frac{N P}{s} \qquad (10.30)$$

where

N is the number of shear connectors in a group;

s is the longitudinal spacing centre-to-centre of groups of shear connectors;

P is either P_p or P_n for shear connectors resisting positive or negative moments respectively (see clause 10.3.2.1).

For positive moments, the shear force on any particular surface of potential shear failure should be determined taking account of the proportion of the effective breadth of the concrete flange lying beyond the surface under consideration.

For negative moments, the shear force on any particular surface of potential shear failure should be determined taking account of the arrangement of the effective longitudinal reinforcement.

10.3.5.3 *Resistance of concrete*

For any surface of potential shear failure in the concrete flange, the longitudinal shear force per unit length shall not exceed the shear resistance v_r given by the following relationship:

$$v_r = 0.7\, A_{sv}\, f_y + 0.03\, \eta\, A_{cv}\, f_{cu} + v_p \qquad (10.31a)$$

$$\leq 0.8\, \eta\, A_{cv} \sqrt{f_{cu}} + v_p \qquad (10.31b)$$

where

f_{cu} is the cube compressive strength of the concrete in N/mm^2, but not more than 40 N/mm^2 when concrete of higher strengths is used;

η = 1.0 for normal weight concrete;

A_{cv} is the mean cross-sectional area, per unit length of the beam, of the concrete shear surface under consideration;

A_{sv} is the cross-sectional area per unit length of the beam, of the combined top and bottom reinforcement crossing the shear surface (see clause Figure 10.8);

v_p is the contribution of the profiled steel sheets, if applicable (see clause 10.3.5.4).

Only reinforcement which is fully anchored should be included in A_{sv}. Where U-bars are used, they should be looped around the shear connectors.

The length of the shear surface *b-b* shown in Figure 10.8 should be taken as

i) 2*h* plus the head diameter — for a single row of headed shear studs or staggered headed shear studs, or

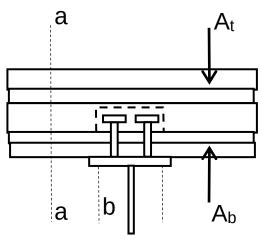
ii) $2h + s_t$ plus the head diameter — for headed shear stubs in pairs.

where

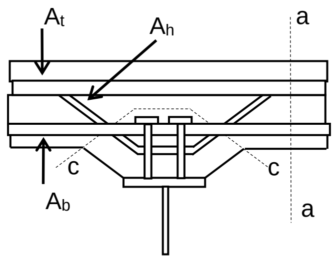
h is the height of the studs; and

s_t is the transverse spacing centre-to-centre of the studs.

Where a profiled steel sheet is used, it is not necessary to consider shear surfaces of type *b–b*, provided that the capacities of the studs are determined using the appropriate reduction factor *k* as recommended in clause 10.3.2.3.

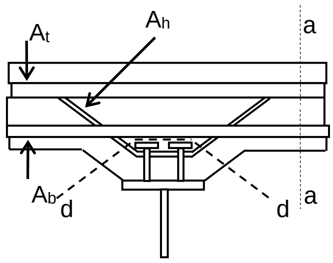


(a) Solid slabs

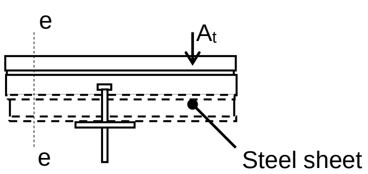


(b) Solid slabs with haunch

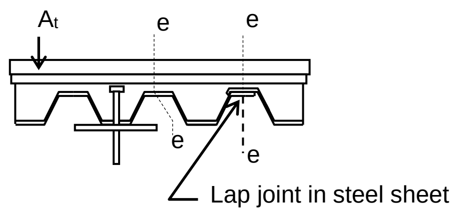
Surface	A_{sv}
a-a	$(A_b + A_t)$
b-b	$2A_b$
c-c	$2(A_b + A_h)$
d-d	$2A_h$
e-e	A_t



(c) Solid slab with deep haunch



(d) Composite slab with the sheeting spanning perpendicular to the beam



(e) Composite slab with the sheeting spanning parallel to the beam

Figure 10.8 - Transverse shear surfaces

10.3.5.4 *Contribution of profiled steel sheet*

Profiled steel sheet is assumed to contribute to the transverse reinforcement provided that it is either continuous across the top flange of the steel beam or welded to the steel beam by headed shear studs.

The resistance of the concrete flange v_r given in clause 10.3.5.3 should be modified to allow for profiled steel sheets as follows:

a) Where the profiled steel sheets are continuous across the top flange of the steel beam, the contribution of profiled steel sheet v_p which is defined as the resistance per unit length of the beam for each intersection of the shear surface by the profiled steel sheet, with ribs running perpendicular to the span of the beam is given by:

$$v_p = t_p\, p_y \qquad (10.32)$$

where

t_p is the thickness of the profiled steel sheet;

p_y is the design strength of the profiled steel sheet; refer to clauses 3.8.1 and 11.2.2.

b) Where the profiled steel sheet is discontinuous across the top flange of the steel beam, and headed shear studs are welded to the steel beam directly through the profiled steel sheets, the contribution of the profiled steel sheets v_p should be determined from the relationship:

$$v_p = (N/s)(n\, d\, t_p\, p_y) \leq t_p\, p_y \qquad (10.33)$$

where

d is the diameter of the headed shear stud;

N and s are as given in clause 10.3.5.2;

n is taken as 4 unless a higher value is justified by tests.

In the case of a beam with separate spans of profiled steel sheets on each side, the studs should be staggered or arranged in pairs, so that each span of the profiled steel sheets is properly anchored.

c) The area of concrete shear surface A_{cv} should be determined with full consideration on the orientation of the ribs as follows:

Where the ribs run perpendicular to the span of the beam, the concrete within the depth of the ribs should be included in the value of A_{cv}.

Where the ribs of the profiled steel sheets run parallel to the span of the beam, the potential shear failure surfaces at lap joints between the sheets should also be checked.

Where the ribs of the profiled steel sheets run at an angle θ to the span of the beam, the effective resistance is given by:

$$v_r = v_1 \sin^2\theta + v_2 \cos^2\theta \qquad (10.34)$$

where

v_1 is the value of v_r for ribs perpendicular to the span;

v_2 is the value of v_r for ribs parallel to the span.

10.3.5.5 *Longitudinal splitting*

To prevent longitudinal splitting of the concrete flange caused by the shear connectors, the following recommendations should be applied in all composite beams where the distance from the edge of the concrete flange to the centreline of the nearest row of shear connectors is less than 300 mm.

a) Transverse reinforcement shall be supplied by U-bars passing around the shear connectors. These U-bars shall be located at least 15 mm below the top of the shear connectors (see Figure 10.9).

b) Where headed studs are used as shear connectors, the distance from the edge of the concrete flange to the centre of the nearest stud should not be less than 6d, where d is the nominal diameter of the stud, and the U-bars should not be less than 0.5d in diameter and detailed as shown in Figure 10.9.

c) The nominal bottom cover to the U-bars should be the minimum permitted by the design requirements for the concrete flange.

In addition, the recommendations given in clause 10.3.5.3 should be met.

Note: These conditions apply to edge beams and also to beams adjacent to large slab openings.

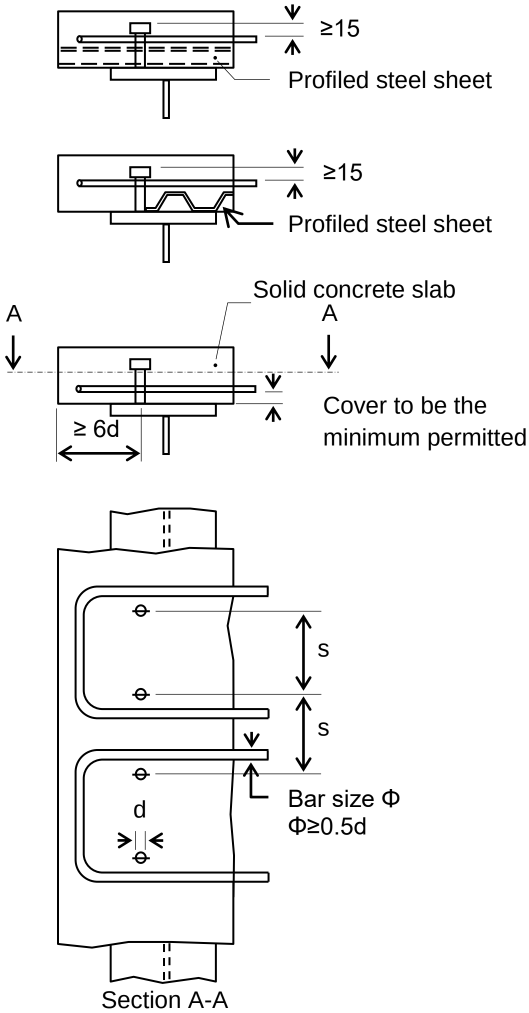


Figure 10.9 - Edge beam details

10.4 COMPOSITE SLABS WITH PROFILED STEEL SHEETS

10.4.1 General

(1) This clause applies to the design composite slabs with profiled steel sheets in building construction where in-situ concrete is placed on profiled steel sheets, and they form a composite element after hardening of the concrete.

(2) This clause applies to composite slabs with profiled steel sheets at yield strengths equal to or less than 550 N/mm^2 with a minimum bare metal thickness of 0.7 mm, and normal weight concrete of C25 to C45. It covers slabs spanning only in the direction of span of the profiled steel sheets.

(3) At construction stage, profiled steel sheets shall be checked for

- bending capacity;
- shear capacity;
- web crushing resistance;
- combined bending and web crushing;
- combined bending and shear; and
- deflection.

(4) At composite stage, composite slabs with profiled steel sheets shall be checked for

- resistance to bending moment;
- resistance to vertical shear;
- resistance to shear-bond failure between profiled steel sheets and concrete slabs in the absence of any chemical bond at the interface; and
- deflection.

It is essential to perform full-scale dynamics and static tests to demonstrate structural adequacy against shear-bond failure between the concrete and the profiled steel sheets. Refer to clause 16.4 for details of the test set-up, the testing procedures and the interpretation of the test results.

Moreover, full-scale structural fire tests are also required to demonstrate structural adequacy in load carrying capacity, insulation and integrity in fire limit state against specific fire resistant periods. Refer to clause 12.2 for fire resistance derived from standard fire tests. Alternatively, fire resistance derived from performance-based design method as described in clause 12.4 shall be adopted with proper justification.

(5) For the design of composite steel beams with a composite slab as the concrete flange, reference should be made to clause 10.2. Diaphragm action produced by the capacity of the composite slab (or of the profiled steel sheets at the construction stage) to resist distortion in its own plane is not within the scope of the Code.

10.4.2 Form of construction

(1) Composite slabs with profiled steel sheets act as composite elements to resist both dead and imposed loads during composite stage. In general, they are constructed without propping.

(2) Composite action shall be obtained in one of the following ways:

a) by mechanical interlock;
b) by friction induced by the profile shape;
c) by a combination of end anchorages with either a) or b).

Any bonding or adhesion of a chemical nature should be neglected in design.

(3) Steel reinforcements should be provided to resist negative hogging moments and act as secondary and nominal reinforcements wherever necessary (see clause 10.4.6). However, steel reinforcements should not be used to resist positive sagging moments in combination with profiled steel sheets, unless the moment capacity has been determined by testing (see clause 10.4.3.2(2)).

(4) Alternatively, the profiled steel sheets are designed to act only as permanent formwork which support the following types of loading during construction:

- self-weight of profiled steel sheets and wet concrete;
- construction loads; and
- storage loads.

In general, they are constructed without propping. Tensile reinforcement should be provided and the slab should be designed as a reinforced concrete slab as recommended in HKCC, without relying on composite action with the profiled sheets.

(5) Where service ducts are formed in the slab, due allowance should be made for the resulting reduction in load carrying capacity.

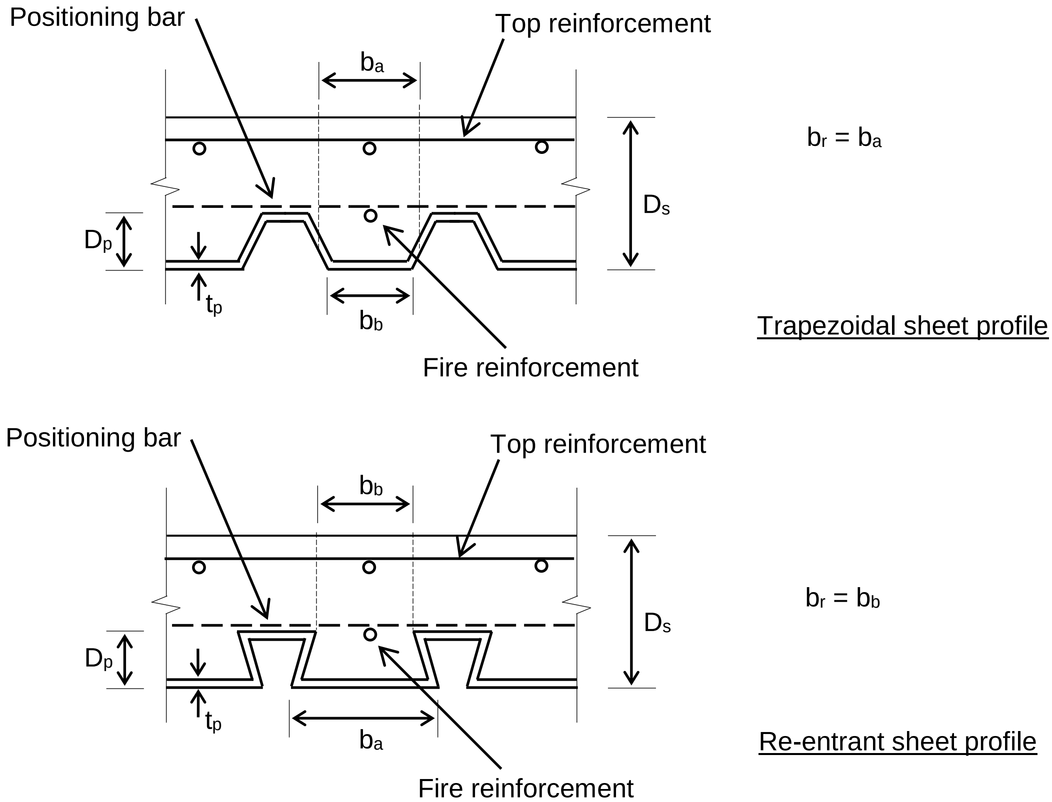


Figure 10.10 - Typical profiled steel sheets

10.4.3 Limit state design

10.4.3.1 General principles

Composite slabs should be designed by considering the limit states at which they would become unfit for their intended use.

It is essential to perform full-scale dynamics and static tests to demonstrate structural adequacy against shear-bond failure between the concrete and the profiled steel sheets. Refer to clause 16.4 for details of the test set-up, the testing procedures and the interpretation of the test results.

Moreover, full-scale structural fire tests are also required to demonstrate structural adequacy in load carrying capacity, insulation and integrity in fire limit state against specific fire resistant periods. Refer to clause 12.2 for fire resistance derived from standard fire tests. Alternatively, fire resistance derived from performance-based design method as described in clause 12.4 shall be adopted with proper justification.

Appropriate safety factors shall be applied for the ultimate, the fire and the serviceability limit states.

10.4.3.2 *Design methods*

(1) General

The following methods may be used for the design of composite slabs:

a) composite design in which the concrete and the profiled steel sheets are assumed to act compositely to support loads (see clause 10.4.5);
b) design by testing (see clause 10.4.3.2(2)); or
c) design as a reinforced concrete slab as recommended in HKCC, neglecting any contribution from the profiled steel sheets.

In all cases, the profiled steel sheets shall be designed for use as permanent formwork during construction (see clause 10.4.4).

(2) Testing

a) Specific tests

Where testing is used as an alternative to calculation methods of design, the load carrying capacity of a composite slab may be determined directly from the results of specific tests as recommended in clause 16.4.2.

b) Parametric tests

In the calculation method for composite design given in clause 10.4.5, the shear-bond capacity shall be determined using the empirical parameters obtained from the results of parametric tests as recommended in clause 16.4.3.

10.4.3.3 *Ultimate limit states*

(1) Strength of materials

In the design of the profiled steel sheets before composite action with the concrete slab is developed, the design strength of the profiled steel sheets, p_y , should be taken as the yield strength of the steel materials divided by the appropriate material factor.

In the design of composite slabs, the design strength, p_y , of the profiled steel sheets should also be taken as the yield strength of the steel materials divided by the appropriate material factor.

Refer to clauses 3.8.1 and 11.2.2 for details.

The design strengths of the concrete, f_{cd} , and the steel reinforcement, f_{sd} , are given as follows:

$$f_{cd} = f_{cu} / \gamma_c \qquad \gamma_c = 1.5 \tag{10.35a}$$

$$f_{sd} = f_y / \gamma_s \qquad \gamma_s = 1.15 \tag{10.35b}$$

where

f_{cu}	is the cube compressive strength of concrete;
f_y	is the characteristic strength of steel reinforcement; and
γ_c, γ_s	are the partial safety factors of concrete and steel reinforcement respectively.

All the properties of concrete and reinforcement shall follow the recommendations of HKCC.

(2) Nominal minimum slab thickness for fire resistance

In the absence of any other information, the nominal minimum slab thickness of concrete for composite slabs with both trapezoidal and re-entrant profiled steel sheets should comply with Tables 10.9 and 10.10.

10.4.3.4 *Serviceability limit states*

(1) Serviceability loads

Generally, the serviceability loads shall be taken as the unfactored values. When considering dead load plus imposed load plus wind load, only 80 % of the imposed load and the wind load need be considered. Construction loads shall not be included in the serviceability loads.

(2) Deflections
Deflections under serviceability loads should not impair the strength or the use of the structure nor to cause any damage to finishes. The recommendations given in clause 10.4.4.4 should be followed for profiled steel sheets at the construction stage while those given in clause 10.4.5.4 should be followed for the deflection of composite slabs.

10.4.3.5 *Durability*

(1) Corrosion protection of profiled steel sheets
The exposed surface at the underside of profiled steel sheets shall be adequately protected against relevant environmental conditions, including those arising during site storage and erection.

(2) Concrete durability
For durability of concrete in composite slabs, the relevant recommendations in HKCC shall be followed.

Table 10.9 - Nominal minimum slab thickness of concrete – trapezoidal sheets

Fire resistance period (hours)	Nominal minimum insulation thickness of normal weight concrete above trapezoidal sheets excluding non-combustible screeds
0.5	60
1	70
1.5	80
2	95
3	115
4	130

Table 10.10 - Nominal minimum slab thickness of concrete – re-entrant sheets

Fire resistance period (hours)	Nominal minimum insulation thickness of normal weight concrete (equals to overall slab thickness)
0.5	90
1	90
1.5	110
2	125
3	150
4	170

10.4.4 Design of profiled steel sheets in construction stage

10.4.4.1 *General*

The design of profiled steel sheets before composite action is developed should follow the recommendations given in this sub-clause.

The cross sectional properties of profiled steel sheets should be evaluated according to the recommendations given in section 11. Embossments and indentations designed to provide composite action should be ignored when calculating the cross-sectional properties of profiled steel sheets.

Alternatively the load-carrying capacity of the profiled steel sheets shall be determined by testing.

10.4.4.2 Loads and span arrangement

(1) For design purposes, the loads carried by the profiled steel sheets include:

- the self-weight of profiled steel sheets, wet concrete and reinforcements;
- the construction loads (see clause 10.4.4.2(2));
- the storage loads;
- the effects of any temporary propping used at this stage, and
- wind forces where necessary.

(2) The following loads during construction shall also be considered:

a) Basic construction loads

In general purpose working areas, the basic construction load of the profiled steel sheets should be taken as not less than 1.5 kN/m^2. For spans of less than 3 m, the basic construction load should be increased to not less than $4.5/L_p$ kN/m^2, where L_p is the effective span of the profiled steel sheets in metres. Construction loads with a partial load factor of 1.6 should be considered in addition to the self-weights of profiled steel sheets and wet concrete, both with a partial load factor of 1.4.

In order to determine the most critical combination of loaded spans in a continuous sheet, an end span should be taken as fully loaded while the adjacent span should be taken as either

- loaded with the self-weight of the wet concrete slab plus a construction load of one-third of the basic construction load, or
- unloaded apart from the self-weight of the profiled steel sheets

whichever is the more critical for positive and negative moments in the sheet (see Figure 10.11).

It should be noted that the load factor for the self-weight of concrete may be reduced in construction sites where concreting sequences are carefully planned and properly performed.

b) Storage loads

In general, a minimum storage load of 3.0 kN/m^2 should be considered to be acting onto the profiled steel sheets together with the self-weight of profiled steel sheets. However, it is not necessary to consider storage loads together with construction load and self-weight of wet concrete.

c) Additional self-weight of concrete slab

The self-weight of the finished slab should be increased if necessary to allow for the additional concrete placed as a result of the "ponding" deflection of the profiled steel sheets (see clause 10.4.4.4(2)) for use in clause 10.2 and in the design of the supporting structure.

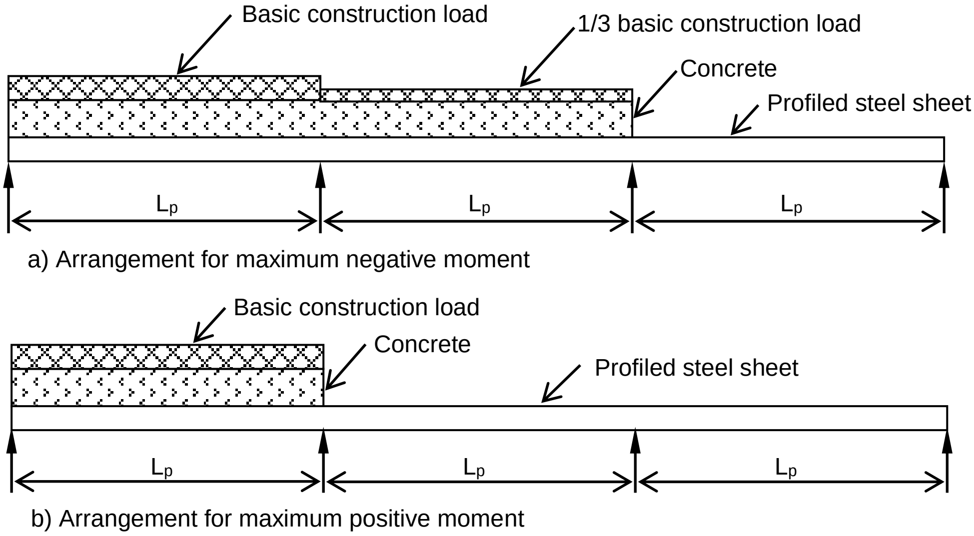


Figure 10.11 - Arrangement of construction loads

10.4.4.3 Ultimate limit states

In general, the load carrying capacities of profiled steel sheets shall be determined as recommended in section 11 by:

a) calculation;
b) testing; or
c) a hybrid design method with calculation and testing.

The internal forces and moments of the profiled steel sheets are generally obtained from linear elastic analysis.

The section capacities shall be calculated with the following considerations:

- local buckling in flange elements under compression; and
- local buckling in web element under bending.

The following checks on the profiled steel sheets shall be carried out:

- bending capacity;
- shear capacity;
- web crushing resistance;
- combined bending and web crushing;
- combined bending and shear; and
- deflection.

10.4.4.4 Serviceability limit states

(1) The deflection of profiled steel sheets should be calculated as recommended in section 11 using the serviceability loads (see clause 10.4.3.4(1)) for the construction stage, comprising the self weight of the profiled steel sheets and the wet concrete only.

(2) The deflection, Δ, should not normally exceed the following:

a) when $\Delta \leq D_s / 10$

$$\Delta \leq L_p / 180 \text{ (but } \leq 20 \text{ mm)} \quad (10.36)$$

where L_p is the effective span of the profiled steel sheet.

b) when $\Delta > D_s / 10$, the effect of ponding should be taken into account, i.e. the self weight of additional concrete due to the deflection of profiled steel sheets should be included in the deflection calculation. This may be evaluated by assuming that the nominal thickness of the concrete is increased by 0.7 Δ over the entire span;

$$\Delta \leq L_p / 130 \text{ (but } \leq 30\text{mm)} \quad (10.37)$$

Also refer to clause 5.2.

(3) In calculating the deflection of profiled steel sheets, the second moment of area of the profiled sheets at serviceability limit state, I_{ser}, may be taken as:

$$I_{ser} = \frac{1}{4}(2I_{xg} + I_{xr,s} + I_{xr,h}) \geq 0.8 I_{xg} \quad (10.38)$$

where I_{xg} = second moment of area of the gross section;
$I_{xr,s}$ = second moment of area of the effective section under sagging moment due to serviceability load; and
$I_{xr,h}$ = second moment of area of the effective section under hogging moment due to serviceability load.

(4) These limits should be increased only where it is shown that larger deflections will not impair the strength or efficiency of the slab. These limits should be reduced, if necessary, where soffit deflection is considered important, e.g. for service requirements or aesthetics.

10.4.5 Design of composite slabs in composite stage

10.4.5.1 General

(1) Composite slabs shall be designed as either:

a) simply supported, with nominal reinforcement over intermediate supports; or
b) continuous, with full continuity reinforcement over intermediate supports in accordance with HKCC.

Where slabs or portions of slabs span onto supports in the transverse direction, this aspect of the design should be in accordance with HKCC.

(2) Composite slabs are normally designed as simply supported, with nominal steel mesh reinforcement over supports.

(3) For composite slabs designed as continuous slabs subjected to uniformly distributed imposed load, only the following arrangements of imposed load need to be considered.

a) alternate spans loaded; and
b) two adjacent spans loaded.

For dead load, the same value of the partial safety factor for loads γ_f should be applied on all spans.

If the effects of cracking of concrete are neglected in the analysis for ultimate limit states, the bending moments at internal supports may optionally be reduced by up to 30%, and corresponding increases made to the sagging bending moments in the adjacent spans.

Plastic analysis without any direct check on rotation capacity may be used for the ultimate limit state if reinforcing steel with sufficient ductility is used and the span is not greater than 3.0 m.

(4) Analysis of internal forces and moments

The following methods of analysis may be used for ultimate limit states:

- linear elastic analysis with or without re-distribution;
- rigid plastic global analysis provided that it is shown that sections where plastic rotations are required have sufficient rotation capacity;
- elastic-plastic analysis, taking into account the non-linear material properties.

For serviceability limit states, linear elastic analysis methods shall be used.

(5) Concentrated loads and reactions

Punching shear should also be checked where concentrated loads and reactions are applied to the slab (see clause 10.4.5.3(5)).

(6) Effects of holes and ducts

Where holes or ducts interrupt the continuity of a composite slab, the region affected should be designed as reinforced concrete and reference should be made to HKCC.

10.4.5.2 Design considerations

(1) The capacity of the composite slab shall be sufficient to resist the factored loads for the ultimate limit state, and all the critical sections indicated in Figure 10.12 shall be considered appropriately:

a) Flexural failure at section 1-1
This criterion is represented by the moment capacity of the composite slab, based on full shear connection at the interface between the concrete and the profiled steel sheets (see clause 10.4.5.3(1)).

b) Longitudinal slip at section 2-2
This criterion is represented by the shear-bond capacity along the interface between the concrete and the profiled steel sheets. In this case, the capacity of the composite slab is governed by the shear connection at section 2-2 (see clause 10.4.5.3(2)).

c) Vertical shear failure at section 3-3
This criterion is represented by the vertical shear capacity of the composite slab (see clause 10.4.5.3(4)). In general, vertical shear failure is rarely critical.

(2) The composite slab should be designed assuming all the loading acts on the composite slab.

(3) Where composite slabs are designed as continuous with full continuity reinforcement over internal supports in accordance with HKCC, the resistance to shear-bond failure contributed by the adjacent spans should be allowed for by basing the value of shear span L_v for use as described in clause 10.4.5.3(2)b on an equivalent simple span between points of contraflexure when checking the shear-bond capacity of an internal span. However, for end spans, the value of L_v should be based on the full end span length.

(4) As an alternative to the design procedure given in clause 10.4.5.2(1), the relevant design criterion and capacity for a particular arrangement of profiled steel sheets and concrete slabs may be determined by testing.

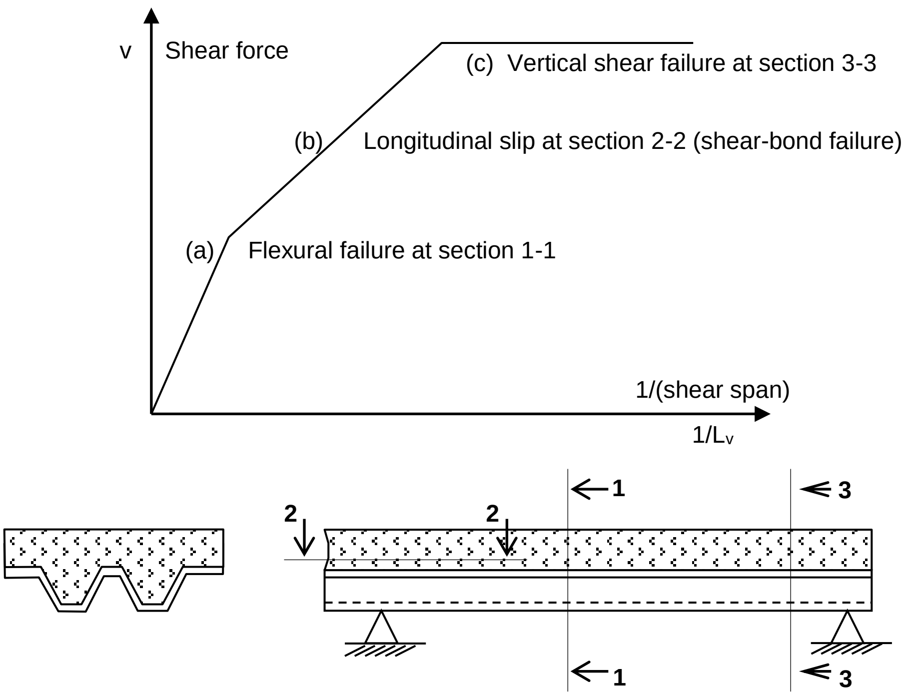


Figure 10.12 - Mode of failure of a composite slab

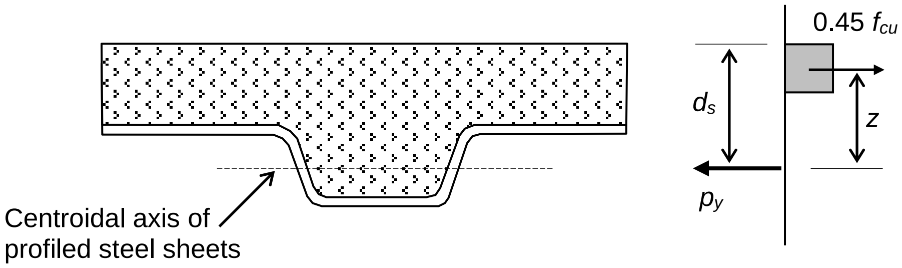
10.4.5.3 Ultimate limit states

(1) Moment capacity

The moment capacity at full shear connection shall be treated as an upper bound to the capacity of a composite slab. The moment capacity of a composite slab shall be calculated as for reinforced concrete, with the profiled steel sheets acting as tensile reinforcement.

The moment capacity in positive moment regions shall be determined assuming rectangular stress blocks for both concrete and profiled steel sheets. The design strengths shall be taken as $0.45f_{cu}$ for the concrete and p_y for the profiled steel sheeting (see Figure 10.13). The lever arm z should not exceed $0.95d_s$ and the depth of the stress block for the concrete should not exceed $0.45d_s$. Tension reinforcement in positive moment regions shall be neglected, unless the moment capacity is determined by testing.

The moment capacity in negative moment regions shall be determined as recommended in HKCC. In determining the negative moment capacity, the profiled steel sheets should be neglected.



where

d_s is the effective depth of slab to the centroid of the profiled steel sheets
f_{cu} is the cube compressive strength of concrete
p_y is the design strength of the profiled steel sheets
z is the lever arm

Figure 10.13 - Stress blocks of moment capacity

(2) Longitudinal shear capacity for slabs without end anchorage

a) Shear-bond capacity V_s

When the load carrying capacity of a composite slab is governed by shear bond, it should be expressed in terms of the vertical shear capacity at the supports.

Generally the shear-bond capacity V_s (in N) shall be calculated using

$$V_s = \frac{B_s d_s}{1.25}\left[\frac{m_r A_p}{B_s L_v} + k_r \sqrt{f_{cu}}\right] \quad (10.39)$$

where

- A_p is the cross-sectional area of the profiled steel sheeting (in mm²);
- B_s is the width of the composite slab (in mm);
- d_s is the effective depth of slab to the centroid of the profiled steel sheets (in mm);
- f_{cu} is the cube compressive strength of concrete (in N/mm²);
- k_r is an empirical parameter (in $\sqrt{N/mm^2}$);
- L_v is the shear span of the composite slab (in mm), determined in accordance with clause 10.4.5.3(2)b; and
- m_r is an empirical parameter (in N/mm²).

The factor of 1.25 is an additional partial safety factor γ_m, for the shear-bond capacity according to the sudden failure behaviour of the slab.

The empirical parameters m_r and k_r in this formula shall be obtained from parametric tests for the particular profiled sheet as recommended in section 16.4.3. In using this formula the value of A_p should not be taken as more than 10% greater than that of the profiled steel sheets used in the tests and the value of f_{cu} should not be taken as more than $1.1f_{cm}$ where f_{cm} is the value used in clause 16.4.3 to determine m_r and k_r

When the value of k_r obtained from the tests is negative, the nominal strength grade of the concrete used in this formula should be not less than the nominal strength grade of the concrete used in the tests.

As an alternative to calculation of the shear-bond capacity, the load carrying capacity of the composite slab can be determined directly by means of specific tests according to clause 16.4.2.

Where it is necessary to use end anchors to increase the resistance to longitudinal shear above that provided by the shear-bond capacity V_s, reference should be made to clause 10.4.5.3(3).

b) Shear span L_v

The shear span L_v for a simply supported composite slab with a span of L_s shall be taken as:

- 0.25 L_s for a uniformly distributed load applied to the entire span;
- the distance between the applied load and the nearest support for two equal and symmetrically place loads.

The shear span L_v for a continuous composite slab shall be taken as:

- 0.8 L for internal spans; or
- 0.9 L for external span

based on an equivalent iso-static span for the determination of the resistance.

For other loading arrangements, including partial distributed loads and asymmetrical point load systems, the shear span L_v should be determined on the basis of appropriate tests or by approximate calculations where the shear span may be taken as the maximum moment divided by the greater support reaction.

(3) Longitudinal shear capacity for slabs with end anchorage

End anchorage may be provided by headed shear studs welded to supporting steel beams by the technique of through-the-sheet welding with an end distance, measured to the centre line of the studs, of not less than 1.7 times the stud diameter, or by other suitable ductile shear connectors. Provided that not more than one shear connector is used in each rib of the profiled steel sheets, the shear capacity per unit width should be determined from

$$\overline{V}_a = \frac{NP_a(d_s - \frac{y_c}{2})}{L_v} \qquad (10.40)$$

where

N is the number of shear connectors attached to the end of each span of sheets, per unit length of supporting beam;

d_s is the effective depth of the slab to the centroid of the profiled steel sheeting;

y_c is the depth of concrete in compression at mid-span (for simplicity y_c may conservatively be taken as 20 mm);

L_v is the shear span (for a uniformly loaded slab L_v is span/4); and

P_a is the end anchorage capacity per shear connector.

For the conditions defined above, the end anchorage capacity should be obtained from

$$P_a = 4\ d\ t_p\ p_y \qquad (10.41)$$

where

t_p is the thickness of profiled steel sheet,

p_y is the design strength of profiled steel sheet.

Where end anchorage is used in conjunction with the shear bond between the concrete and the profiled steel sheets, the combined resistance to longitudinal shear should be limited as follows:

$$\overline{V}_c = \overline{V}_s + 0.5\overline{V}_a \quad \text{but} \quad \overline{V}_c \le 1.5\overline{V}_s \qquad (10.42)$$

where

$\overline{V}_c$ is the total longitudinal shear capacity per unit width of slab; and

$\overline{V}_s$ is the shear bond capacity per unit width.

(4) Vertical shear resistance

The vertical shear capacity V_v of a composite slab, over a width equal to the distance between centres of ribs, should be determined from the following:

a) for open trough profile sheets: $V_v = b_a\ d_s\ v_c$ (10.43a)

b) for re-entrant trough profile sheets: $V_v = b_b\ d_s\ v_c$ (10.43b)

where

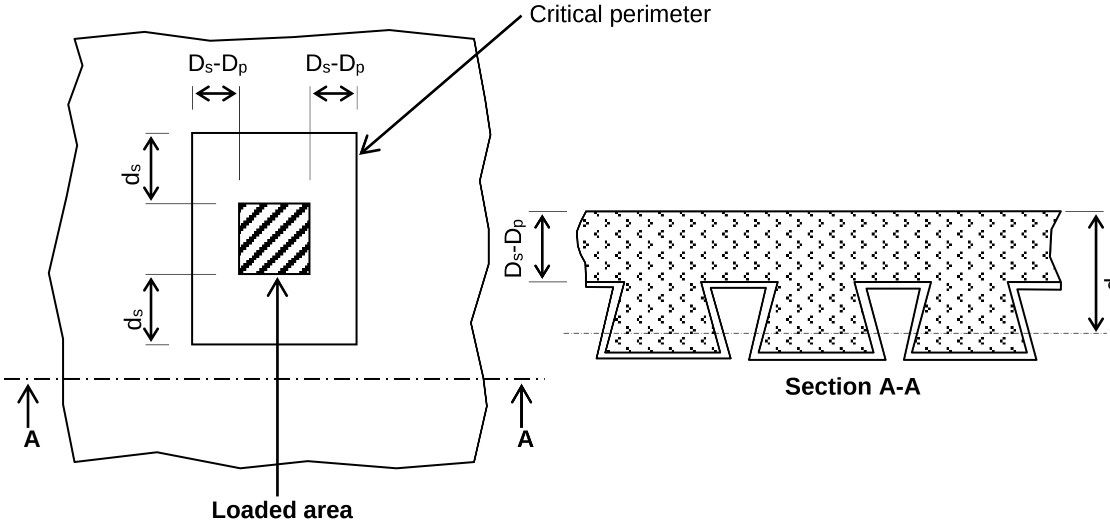
b_a is the mean width of a trough of an open profile (see Figure 10.10);

b_b is the minimum width of a trough of a re-entrant profile (see Figure 10.10);

d_s	is the effective depth of the slab to the centroid of the sheet (see Figure 10.10); and
v_c	is the design concrete shear stress from HKCC taking A_s as A_p, d as d_s and b as B_s.

(5) Punching shear resistance

The punching shear capacity V_p of a composite slab at a concentrated load shall be determined from the method given in HKCC taking d as D_s - D_p and the critical perimeter u as defined in Figure 10.14.



where

D_p	is the overall depth of profiled steel sheets;
D_s	is the overall depth of composite slab;
d_s	is the effective depth of the centroid of the profiled steel sheets,

Figure 10.14 - Critical perimeter for shear

10.4.5.4 *Serviceability limit states*

(1) Deflection

a) Limiting values

The deflection of composite slab shall be calculated using serviceability loads W_{ser} (see clause 10.4.3.4(1)), excluding the self-weight of the composite slab. The deflection of the profiled steel sheets due to its self weight and the self weight of wet concrete (calculated as in clause 10.4.4.4) shall not be included.

The deflection of the composite slab should not normally exceed the following:

- deflection due to the imposed load: L_s/350 or 20 mm, whichever is the lesser;
- deflection due to the total load less the deflection due to the self-weight of the slab plus, when props are used, the deflection due to prop removal: L/250.

These limits should be increased only where it is shown that larger deflections will not impair the strength or efficiency of the slab, lead to damage to the finishes or be unsightly. Also refer to clause 5.3.

b) Calculation

For uniformly distributed loading, the following approximate expressions may be used to calculate the deflection:

- for simply supported spans with nominal reinforcement over intermediate supports

$$\delta = \frac{5}{384} \frac{W_{ser} L_s{}^3}{EI_{CA}} \qquad (10.44)$$

- for end spans of continuous slabs with full continuity reinforcement over intermediate supports of approximately equal span, i.e. within 15% of the maximum span

$$\delta = \frac{1}{100} \frac{W_{ser} L_s{}^3}{EI_{CA}} \qquad (10.45)$$

- for two-span slabs with full continuity reinforcement over intermediate supports of approximately equal span, i.e. within 15% of the maximum span

$$\delta = \frac{1}{135} \frac{W_{ser} L_s{}^3}{EI_{CA}} \qquad (10.46)$$

where

E is the modulus of elasticity of the profiled steel sheets;
I_{CA} is the second moment of area of the composite slab about its centroidal axis;
L_s is the effective span of the composite slab; and
W_{ser} is the serviceability load.

The factor 1/100 is derived by dividing 5/384 (for the simply supported case) by a factor of 1.3. The factor 1.3 is a ratio obtained from the basic span / effective depth ratios for both continuous and simply supported spans. The factor 1/135 is derived by comparing two-span and three-span cases.

The value of the second moment of area of the composite slab I_{CA} about its centroidal axis (in equivalent steel units) should be taken as the average of

I_{CS} for the cracked section (i.e. the compression area of the concrete cross section combined with the profiled steel sheets on the basis of modular ratio) and
I_{GS} for the gross section (i.e. the entire concrete cross section combined with the profiled steel sheets on the basis of modular ratio).

The modular ratio shall be determined as recommended in clause 10.2.

10.4.5.5 Nominal reinforcement at intermediate supports

Where continuous composite slabs are designed as simply supported, nominal steel fabric reinforcement should be provided over intermediate supports.

For propped construction, consideration should be given to increase the area of steel reinforcement over supports as appropriate, depending on the span and the crack widths that can be tolerated.

For mild exposure conditions in accordance with HKCC, the cross-sectional area of reinforcement in the longitudinal direction should be not less than 0.2% of the cross-sectional area of the concrete above the profiled steel sheets at the support. Refer to clause 10.2.7.3 for details.

10.4.5.6 Transverse reinforcement

The cross-sectional area of transverse reinforcement near the top of the slab should not be less than 0.1% of the cross-sectional area of the concrete above the profiled steel sheets.

The minimum cross-sectional area of transverse reinforcement should be increased to 0.2% of the cross-sectional area of the concrete above the profiled steel sheets in the vicinity of concentrated loads.

10.4.5.7 Shear connection

(1) Composite steel beams

Where composite slabs with profiled steel sheets are used to form the slabs of composite steel beams, the design of the shear connection should be in accordance with clause clause 10.2.

Where headed shear studs are assumed in design to also act as end anchors (see clause 10.4.5.3(3)) in simply supported composite slabs, in addition to connecting the slab to the steel beam, the following criteria should all be satisfied:

$$F_a \le P_a \tag{10.47}$$

$$F_b \le P_b \tag{10.48}$$

$$(F_a / P_a)^2 + (F_b / P_b)^2 \le 1.1 \tag{10.49}$$

where

F_a is the end anchorage force per shear connector;

F_b is the beam longitudinal shear force per shear connector;

P_a is the end anchorage capacity per shear connector (see clause 10.4.5.3(3));

P_b is the capacity per shear connector for composite beam design in accordance with clause 10.2.

(2) Composite concrete beams

Where composite slabs with profiled steel sheets are used to form the slabs of composite concrete beams, the design of the shear connection should be in accordance with clause 10.3.

10.4.6 Detailing provisions

(1) Slab thickness

The overall depth of the composite slab D_s shall be sufficient to provide the required resistance to the effects of fire and as a minimum shall not be less than 90 mm as shown in Figure 10.15. In the absence of any other information, the thickness of concrete (D_s - D_p) above the main flat surface of the top of the ribs of the profiled steel sheets shall not be less than 50 mm subject to a concrete cover of not less than 15 mm above the top of any shear connectors.

(2) Arrangement of reinforcement

Top reinforcement, in the form of either bars or steel mesh fabric, should be provided in composite slabs as follows:

a) nominal continuity reinforcement over intermediate supports, for simple spans;

b) full continuity reinforcement over intermediate supports, for continuous spans and for cantilevers;

c) secondary transverse reinforcement to resist shrinkage and temperature stresses;

d) distribution steel, where concentrated loads are applied and around openings.

In general, bottom reinforcement is not needed.

Where necessary, top and bottom reinforcements should also be provided to increase the fire resistance of the composite slab.

(3) Size of concrete aggregate

As shown in Figure 10.15, the nominal maximum size of the concrete aggregate h_{agg} depends on the smallest dimension in the structural element within which concrete is poured and should not be larger than the least of:

a) 0.4 (D_s - D_p) (see Figure 10.15);

b) b_b / 3 (see Figure 10.15); and

c) 20 mm.

(4) Cover to reinforcement

Steel reinforcement in a slab in the form of bars or steel mesh fabric shall be positioned as follows:

a) Longitudinal reinforcement near the top of the slab should have at least 20 mm nominal cover.

b) Longitudinal reinforcement in the bottom of the slab, if needed, should be so positioned that sufficient space, not less than the nominal maximum size of the aggregate, is left between the reinforcement and the sheets to ensure proper compaction of the concrete.

c) Secondary transverse reinforcement for controlling shrinkage should be placed in the top of the slab with at least 20 mm nominal cover.

d) Fire resistance reinforcement intended to provide positive moment capacity should be placed near the bottom of the slab with not less than 20 mm between the reinforcement and the profiled steel sheets.

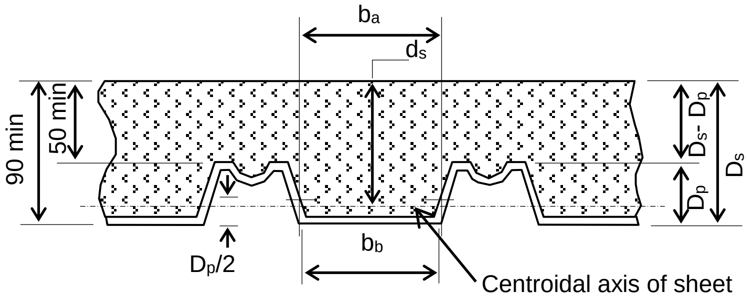
d) Transverse bars (which are not reinforcement) for positioning of longitudinal reinforcement or fire resistance reinforcement, if needed, may be placed directly on the top of the ribs of the sheets.

e) Fire resistance reinforcement for negative moment capacity should be placed near the top of the slab with at least 20 mm nominal cover.

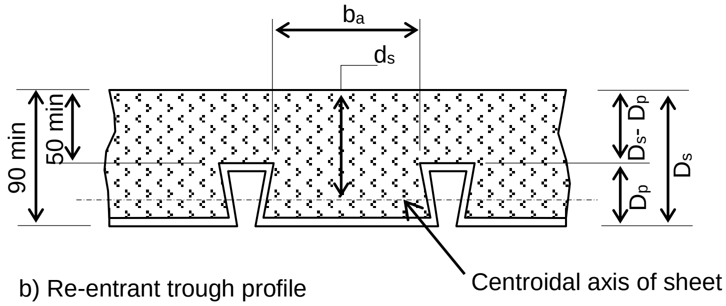
f) Distribution steel in areas of concentrated loads and around openings should be placed directly on the top of the ribs of the sheets, or not more than a nominal 20 mm above it.

The curtailment and lapping of reinforcement shall conform to HKCC. Where a single layer of reinforcement is used to fulfill more than one of the above purposes, it should satisfy all the relevant recommendations.

Note: Where a composite slab forms the concrete flange of a composite beam, clause 10.3.5 gives recommendations for transverse reinforcement of the beam, running perpendicular to the span of the beam. Such reinforcement can be either longitudinal or transverse relative to the slab.



a) Open trough profile



b) Re-entrant trough profile

All dimensions are in millimeters.

where

b_a is the mean width of the trough;

b_b is the minimum width of the trough;

D_p is the overall depth of the profiled steel sheets;

D_s is the overall depth of the composite slab;

d_s is the effective depth of slab to the centroid of the profiled steel sheets

Figure 10.15 - Sheet and slab dimensions

10.4.7 Constructional details

(1) Minimum bearing requirements

As shown in Figure 10.16, composite slabs bearing on steel or concrete should normally have an end bearing of not less than 50 mm. For composite slabs bearing on other materials, the end bearing should normally be not less than 70 mm.

For continuous slabs the minimum bearing at intermediate supports should normally be 75 mm on steel or concrete and 100 mm on other materials as shown in Figure 10.16.

Where smaller bearing lengths are adopted, account should be taken of all relevant factors such as tolerances, loading, span, height of support and provision of continuity reinforcement. In such cases, precautions should also be taken to ensure that fixings (clause 10.4.7(2)) can still be achieved without damage to the bearings, and that collapse cannot occur as a result of accidental displacement during erection.

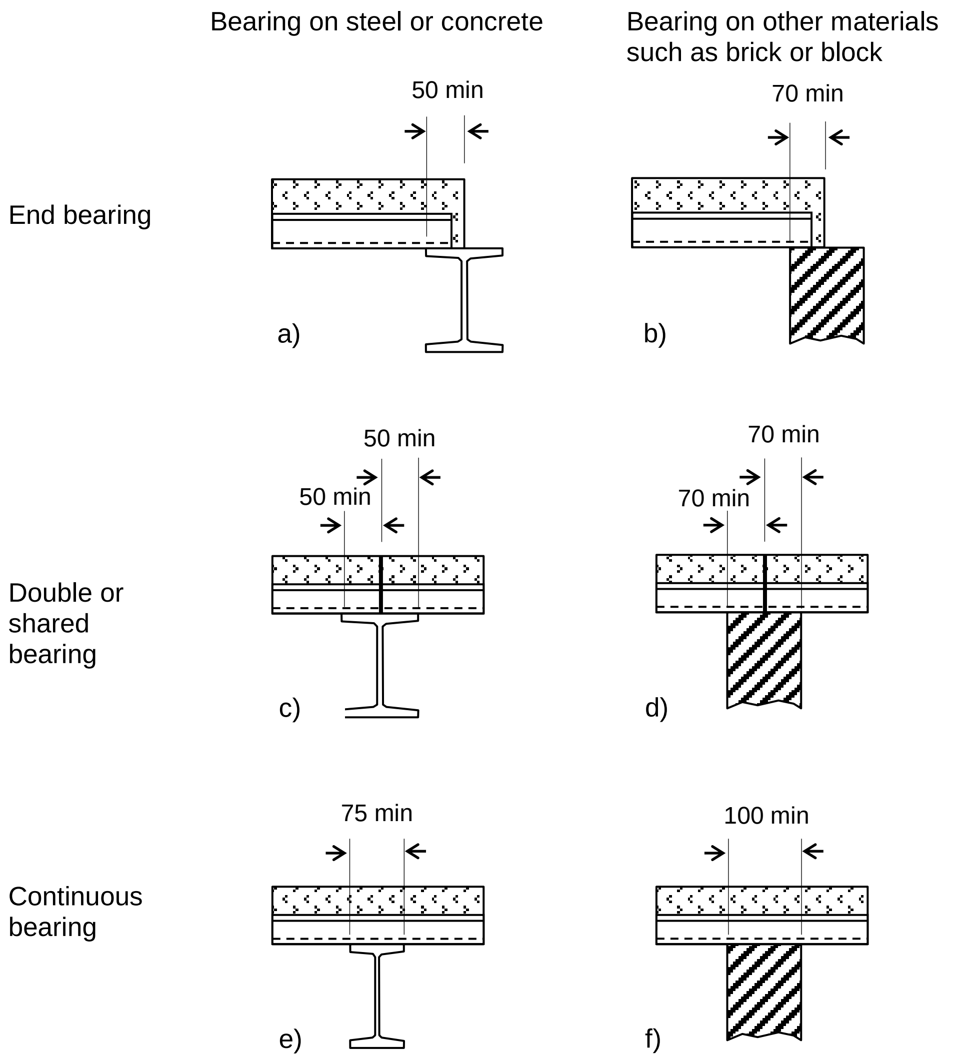


Figure 10.16 - Bearing requirements

(2) Sheet fixings

The design should incorporate provision for the profiled steel sheets to be fixed:

a) to keep them in position during construction so as to provide a subsequent safe working platform;
b) to ensure connection between the sheets and supporting beams;
c) to ensure connection between adjacent sheets where necessary;
d) to transmit horizontal forces where necessary; and
e) to prevent uplift forces displacing the sheets.

For fixing sheets to steelwork, the following types of fixing are available:

- shot fired fixings;
- self-tapping screws;
- welding;
- stud shear connectors welded through the sheeting; or
- bolting.

Due consideration should be given to any adverse effect on the supporting members. Site welding of very thin sheets should not be relied on to transfer end anchorage forces, unless the practicality and quality of the welded connections can be demonstrated by tests.

When sheets are to be attached to brickwork, blockwork, concrete or other materials where there is a danger of splitting, fixing should be by drilling and plugging or by the use of suitable proprietary fixings.

The number of fasteners should not be less than two per sheet at the ends of sheets nor less than one per sheet where the sheets are continuous. The spacing of fasteners should not be larger than 500 mm at the ends of sheets nor greater than 1000 mm where the sheets are continuous. At side laps the sheets should be fastened to each other, as necessary, to control differential deflection, except where the sides of the sheets are supported or are sufficiently interlocking.

The design of sheet fixings should be in accordance with section 11.

10.5 COMPOSITE COLUMNS

10.5.1 General

(1) This clause applies for the design of composite columns and composite compression members with fully encased H sections, partially encased H sections and infilled rectangular and circular hollow sections, see Figure 10.17.

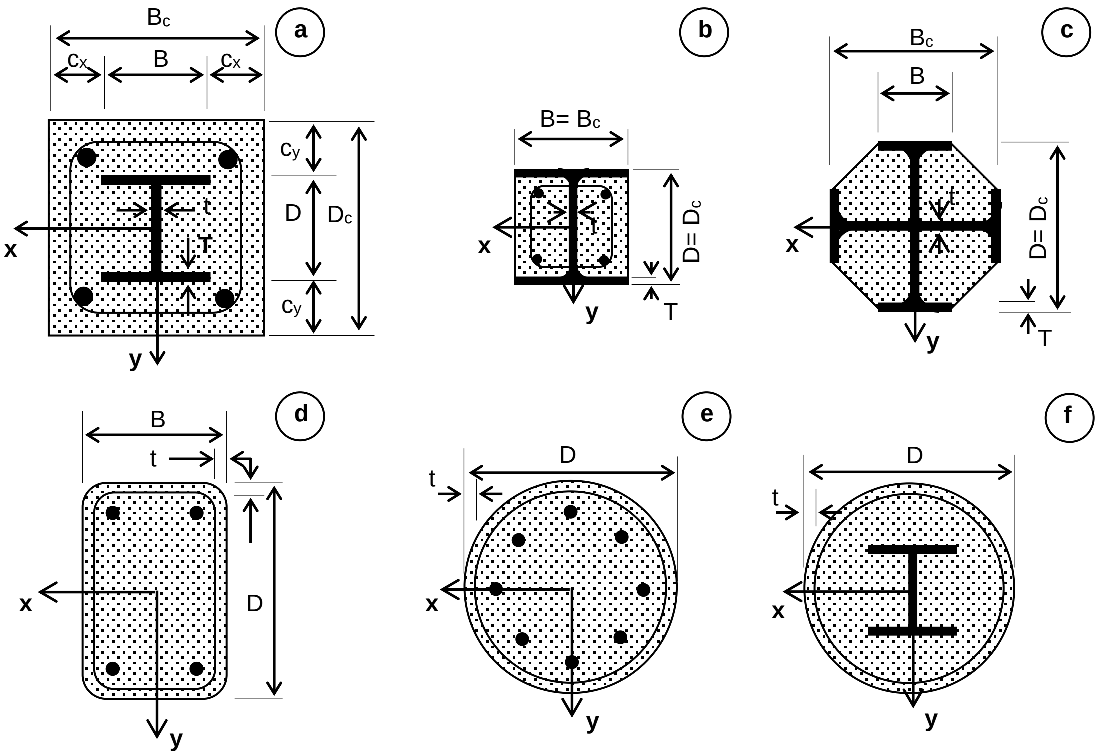


Figure 10.17 - Typical cross-sections of doubly symmetrical composite columns

(2) Composite columns or compression members of any cross-section shall be checked for:

- resistance of the member in accordance with clause 10.5.2 or 10.5.3,
- resistance to local buckling in accordance with clause 10.5.3.1(4),
- introduction of loads in accordance with clause 10.5.4.2 and
- resistance to shear between steel and concrete elements in accordance with clause 10.5.4.

(3) This clause applies to columns and compression members using steel sections with yield strengths between 235 and 460 N/mm^2, and normal weight concrete of strength classes C25 to C60.

(4) This clause applies to isolated columns and composite compression members in framed structures where the other structural members are either composite or steel members.

(5) Two methods of design are given:

- a general method in clause 10.5.2 whose scope includes members with non-symmetrical or non-uniform cross-sections over the column length and
- a simplified method in clause 10.5.3 for members of doubly symmetrical and uniform cross section over the member length.

(6) Refer to clause 3.1.2 for the design strength of the structural steel section, p_y. The design strengths of the concrete, f_{cd} , and the steel reinforcement, f_{sd} , are given as follows:

$f_{cd} = f_{cu} / \gamma_c \qquad \gamma_c = 1.5 \qquad (10.50a)$

$f_{sd} = f_y / \gamma_s \qquad \gamma_s = 1.15 \qquad (10.50b)$

where

f_{cu} is the cube compressive strength of concrete;

f_y is the characteristic strength of steel reinforcement; and

γ_c , γ_s are the partial safety factors of concrete and steel reinforcement, respectively.

10.5.2 General method of design

(1) Design for structural strength and stability shall take into the account of concrete crushing, and yielding of structural steel sections and steel reinforcement.

The design shall also ensure that instability does not occur for the most unfavourable combination of actions at the ultimate limit state and that the resistance of individual cross-sections subjected to bending, longitudinal force and shear is not exceeded.

Furthermore, second-order effects shall be incorporated including local buckling, residual stresses, geometrical imperfections, and long-term effects on concrete such as creeping and shrinkage of concrete.

Second-order effects shall also be considered in any direction in which failure might occur, if they affect the structural stability significantly. For simplification, instead of the effect of residual stresses and geometrical imperfections, equivalent initial bow imperfections (member imperfections) may be used in accordance with clause 10.5.3.3(3).

(2) Internal forces shall be determined by elastic-plastic analysis.

Plane sections may be assumed to remain plane after bending.

The influence of local buckling of the structural steel section on the resistance shall be considered in design.

(3) The following stress-strain relationships shall be used in the non-linear analysis:

- for concrete in compression as given in HKCC;
- for reinforcing steel as given in HKCC;
- for structural steel as given in section 3.

The tensile strength of concrete shall be neglected. The influence of tension stiffening of concrete between cracks on the flexural stiffness may be taken into account.

(4) The steel contribution ratio δ shall fulfill the following condition:

$0.2 \leq \delta \leq 0.9 \qquad (10.51)$

where

δ is defined in clause 10.5.3.2(2).

(5) Shrinkage and creep effects shall be considered if they are likely to reduce the structural stability significantly.

For simplification, creep and shrinkage effects may be ignored if the increase in the first-order bending moments due to creep deformations and longitudinal force resulting from permanent loads is not greater than 10%.

(6) For composite compression members subjected to bending moments and normal forces resulting from independent actions, the partial safety factor γ_f for those internal forces that lead to an increase of resistance should be reduced to 80%.

10.5.3 Simplified method of design

10.5.3.1 General and scope

(1) The scope of this simplified method is limited to members of doubly symmetrical and uniform cross-section over the member length with rolled, cold-formed or welded steel sections.

However, this method is not applicable if the structural steel component consists of two or more unconnected sections. All internal forces and moments for member design against structural adequacy should be evaluated with second-order analysis.

(2) The steel contribution ratio δ shall fulfill the following condition:

$$0.2 \le \delta \le 0.9 \qquad (10.52)$$

where

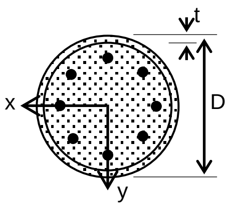
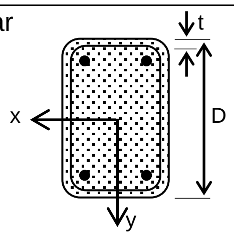
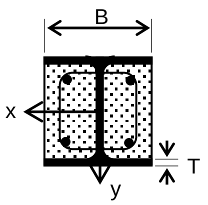
δ is defined in clause 10.5.3.2(2).

(3) The relative slenderness $\bar{\lambda}$ defined in clause 10.5.3.3 shall fulfill the following condition:

$$\bar{\lambda} \le 2.0 \qquad (10.53)$$

(4) The effect of local plate buckling in the elements of a steel section may be neglected for the steel section is fully encased in accordance with clause 10.5.5.1(2), and also for other types of cross-section provided the maximum values of Table 10.11 are not exceeded. Hence, the entire composite cross-sections are effective.

Table 10.11 - Maximum values on geometric ratios

Cross-section		Max (D/t) and max (B/T)
Infilled circular hollow sections		Max $\left(\frac{D}{t}\right) = 77 \times \left(\frac{275}{p_y}\right)$
Infilled rectangular hollow sections		Max $\left(\frac{D}{t}\right) = 48 \times \left(\sqrt{\frac{275}{p_y}}\right)$
Partially encased H-sections		Max $\left(\frac{B}{T}\right) = 41 \times \left(\sqrt{\frac{275}{p_y}}\right)$

Note: p_y in N/mm^2.

(5) The longitudinal reinforcement that may be used in calculation shall not exceed 6% of the concrete area.

(6) For a fully encased steel section, see Figure 10.17a, limits to the maximum thickness of concrete cover that may be used in calculation are:

$$max\ c_y = 0.3\,D \qquad max\ c_x = 0.4\,B \qquad (10.54)$$

(7) The depth to width ratio D_c / B_c of fully encased composite cross-sections as shown in Figure 10.17a shall be within the limits $0.2 < D_c / B_c < 5.0$.

(8) For the determination of the internal forces, the design value of effective flexural stiffness $(EI)_{e,1}$ shall be determined from the following expression:

$$(EI)_{e,1} = 0.9\,(EI + E_s I_s + 0.5\,E_{cm} I_c) \tag{10.55}$$

Long-term effects should be taken into account in accordance with clause 10.5.3.3(6).

(9) Second-order effects need not to be considered where the elastic critical buckling load is determined with the flexural stiffness $(EI)_{e,1}$ in accordance with clause 10.5.3.1(8).

(10) Within the column length, second-order effects may be allowed for by increasing the greatest first-order design bending moment M by a factor k given by:

$$k = \frac{\beta}{1 - P / P_{cp,cr}} \tag{10.56}$$

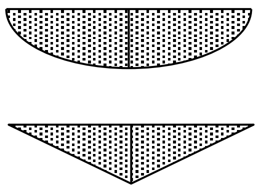
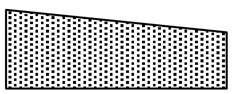
where

$P_{cp,cr}$ is the critical buckling load for the relevant axis and corresponding to the effective flexural stiffness given in clause 10.5.3.1(8), with the effective length taken as the column length;

$$= \frac{\pi^2\,(EI)_{e,1}}{L_E^{\,2}} \tag{10.57}$$

β is an equivalent moment factor given in Table 10.12.

Table 10.12 - Factors β for the determination of moments to second order theory

Moment distribution	Moment factors β	Comment
 M M	First-order bending moments from member imperfection or lateral load: $\beta = 1.0$	M is the maximum bending moment within the column length ignoring second-order effects
M  $r\,M$ $-1 \le r \le 1$	End moments: $\beta = 0.66 + 0.44r$ but $\beta \ge 0.44$	M and $r\,M$ are the end moments from first-order or second-order global analysis

(11) The influence of geometrical and structural imperfections may be taken into account by equivalent geometrical imperfections. Equivalent member imperfections for composite columns are given in Table 10.13, where L is the column length.

10.5.3.2 *Compression capacity*

(1) The compression capacity P_{cp} of a composite cross-section shall be calculated by adding the compression capacities of its components:

For fully encased and partially encased H sections:

$$P_{cp} = A\,p_y + 0.45\,A_c\,f_{cu} + A_s\,f_{sd} \tag{10.58a}$$

For infilled rectangular hollow sections:

$$P_{cp} = A\,p_y + 0.53\,A_c\,f_{cu} + A_s\,f_{sd} \tag{10.58b}$$

where

A , A_c , A_s are the areas of the steel section, the concrete and the reinforcements respectively.

(2) The steel contribution ratio δ is defined as:

$$\delta = \frac{A\ p_y}{P_{cp}} \tag{10.59}$$

where

P_{cp} is the compression capacity of the composite cross-section defined in clause 10.5.3.2(1).

(3) For infilled circular hollow sections, account may be taken of increase in strength of concrete caused by confinement provided that the relative slenderness $\bar{\lambda}$ defined in clause 10.5.3.3(3) does not exceed 0.5 and $\frac{e}{d}$ < 0.1, where e is the eccentricity of loading given by M / P and d is the external diameter of the column. The compression capacity for infilled circular hollow sections shall be calculated as follows:

$$P_{cp} = \eta_a A\,p_y + 0.53 A_c f_{cu}\left[1 + \eta_c \frac{t}{d}\frac{p_y}{0.8 f_{cu}}\right] + A_s f_{sd} \tag{10.60}$$

where

t is the wall thickness of the steel tube.

For members in combined compression and bending with 0 < $\frac{e}{d}$ < *0.1,* the values η_a and η_c are given as follows:

$$\eta_a = \eta_{ao} + (1 - \eta_{ao})(10\,\frac{e}{d}) \qquad \text{(but < 1.0)} \tag{10.61}$$

$$\eta_c = \eta_{co}\,(1 - 10\,\frac{e}{d}) \qquad \text{(but < 1.0)} \tag{10.62}$$

where

$$\eta_{ao} = 0.25\,(\,3 + 2\,\bar{\lambda}\,) \qquad \text{(but < 1.0)} \tag{10.63}$$

$$\eta_{co} = 4.9 - 18.5\,\bar{\lambda} + 17\,\bar{\lambda}^2 \qquad \text{(but} \geq 0\text{)} \tag{10.64}$$

10.5.3.3 *Column buckling*

(1) Members may be verified using second order analysis according to clause 10.5.3.5 taking into account of member imperfections.

(2) For simplification for members susceptible to axial buckling using first order analysis, the design value of the compression force P shall satisfy:

$$\frac{P}{\chi P_{cp}} \leq 1.0 \tag{10.65}$$

where

P_{cp} is the compression capacity of the composite section according to clause 10.5.3.2(1);

χ is the reduction factor for the column buckling given in clause 10.5.3.3(3) in term of the relevant relative slenderness $\bar{\lambda}$ given in clause 10.5.3.3(4).

(3) The reduction factor χ for column buckling is given by:

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \quad \text{but} \quad \chi \leq 1.0 \tag{10.66}$$

where

$$\phi = \frac{1}{2}\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right] \tag{10.67}$$

α is the imperfection factor which allows for different levels of imperfections in the columns

= 0.21 for buckling curve a

= 0.34 for buckling curve b

= 0.49 for buckling curve c

The relevant buckling curves for various cross-sections of composite columns are given in Table 10.13, and the buckling strength reduction factors of composite column are given in Table 10.14.

(4) The relative slenderness $\bar{\lambda}$ for the plane of bending being considered is given by:

$$\bar{\lambda} = \sqrt{\frac{P_{cp,k}}{P_{cp,cr}}} \tag{10.68}$$

where

$P_{cp,k}$ is the characteristic value of the compression capacity which is given by:

$$= A\,p_y + 0.68\,A_c f_{cu} + A_s f_y$$

for fully encased and partially encased H sections (10.69a)

$$= A\,p_y + 0.8\,A_c f_{cu} + A_s f_y$$

for infilled rectangular hollow sections (10.69b)

$$= \eta_a A\,p_y + 0.8 A_c f_{cu}\left[1 + \eta_c \frac{t}{d}\frac{p_y}{0.8 f_{cu}}\right] + A_s f_y$$

for infilled circular hollow sections (10.69c)

$P_{cp,cr}$ is the elastic critical buckling load for the relevant buckling mode, calculated with the effective flexural stiffness $(EI)_{e,2}$ determined in accordance with clauses 10.5.3.3(5) and 10.5.3.3(6).

$$= \frac{\pi^2\,(EI)_{e,2}}{L_E{}^2} \tag{10.70}$$

(5) For the determination of the relative slenderness $\bar{\lambda}$ and the elastic critical buckling load N_{cr}, the characteristic value of the effective flexural stiffness $(EI)_{e,2}$ of a composite column should be calculated from:

$$(EI)_{e,2} = EI + K_e\,E_{cm}\,I_c + E_s I_s \tag{10.71}$$

where

K_e is a correction factor that shall be taken as 0.6.

I, I_c, and I_s are the second moments of area of the structural steel section, the un-cracked concrete section and the reinforcement respectively for the bending plane being considered.

(6) Account should be taken to the influence of long-term effects on the effective elastic flexural stiffness. The modulus of elasticity of concrete E_{cm} should be reduced to the value E_c in accordance with the following expression:

$$E_c = E_{cm}\frac{1}{1 + (P_G / P)\varphi_t} \tag{10.72}$$

where

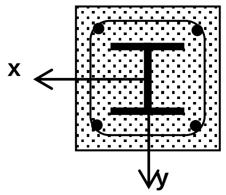
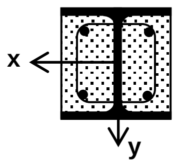
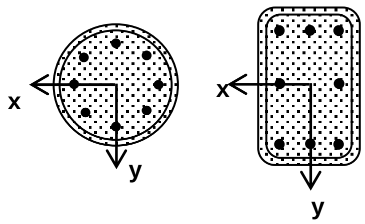
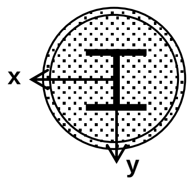
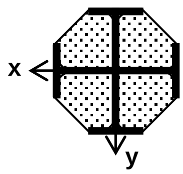
φ_t is the creep coefficient;

P is the total design normal force;

P_G is the part of this normal force that is permanent.

For the determination of the creep coefficient in accordance with HKCC, a relative humidity of 100% may be assumed for concrete infilled hollow sections.

Table 10.13 - Buckling curves and member imperfections for composite columns

Cross-section	Limits	Axis of buckling	Buckling curve	Member imperfection
Fully encased H section 		x-x	b	L/200
		y-y	c	L/150
Partially encased H section 		x-x	b	L/200
		y-y	c	L/150
Infilled circular and rectangular hollow sections 	$\rho_s \leq 3\%$	any	a	L/300
	$3\% < \rho_s \leq 6\%$	any	b	L/200
Infilled circular hollow section with additional H section 		x-x	b	L/200
		y-y	b	L/200
Partially encased H section with crossed H section 		any	b	L/200

NOTE: ρ_s is the reinforcement ratio A_s / A_c.

Table 10.14 - Buckling strength reduction factors of composite columns

	Buckling curve		
$\bar{\lambda}$	a	b	c
0.00	1.000	1.000	1.000
0.05	1.000	1.000	1.000
0.10	1.000	1.000	1.000
0.15	1.000	1.000	1.000
0.20	1.000	1.000	1.000
0.25	0.989	0.982	0.975
0.30	0.977	0.964	0.949
0.35	0.966	0.945	0.923
0.40	0.953	0.926	0.897
0.45	0.939	0.906	0.871
0.50	0.924	0.884	0.843
0.55	0.908	0.861	0.815
0.60	0.890	0.837	0.785
0.65	0.870	0.811	0.755
0.70	0.848	0.784	0.725
0.75	0.823	0.755	0.694
0.80	0.796	0.724	0.662
0.85	0.766	0.693	0.631
0.90	0.734	0.661	0.600
0.95	0.700	0.629	0.569
1.00	0.666	0.597	0.540
1.05	0.631	0.566	0.511
1.10	0.596	0.535	0.484
1.15	0.562	0.506	0.458
1.20	0.530	0.478	0.434
1.25	0.499	0.452	0.411
1.30	0.470	0.427	0.389
1.35	0.443	0.404	0.368
1.40	0.418	0.382	0.349
1.45	0.394	0.361	0.331
1.50	0.372	0.342	0.315

	Buckling curve		
$\bar{\lambda}$	a	b	c
1.55	0.352	0.324	0.299
1.60	0.333	0.308	0.284
1.65	0.316	0.292	0.271
1.70	0.299	0.278	0.258
1.75	0.284	0.265	0.246
1.80	0.270	0.252	0.235
1.85	0.257	0.240	0.224
1.90	0.245	0.229	0.214
1.95	0.234	0.219	0.205
2.00	0.223	0.209	0.196
2.05	0.213	0.200	0.188
2.10	0.204	0.192	0.180
2.15	0.195	0.184	0.173
2.20	0.187	0.176	0.166
2.25	0.179	0.169	0.160
2.30	0.172	0.163	0.154
2.35	0.165	0.157	0.148
2.40	0.159	0.151	0.143
2.45	0.152	0.145	0.137
2.50	0.147	0.140	0.132
2.55	0.141	0.135	0.128
2.60	0.136	0.130	0.123
2.65	0.131	0.125	0.119
2.70	0.127	0.121	0.115
2.75	0.122	0.117	0.111
2.80	0.118	0.113	0.108
2.85	0.114	0.109	0.104
2.90	0.111	0.106	0.101
2.95	0.107	0.103	0.098
3.00	0.104	0.099	0.095

10.5.3.4 *Moment capacity*

The moment capacity of a doubly symmetric composite cross-section may be evaluated as follows:

$$M_{cp} = p_y (S_p - S_{pn}) + 0.5\ \alpha_c\ f_{cu} (S_{pc} - S_{pcn}) + f_{sd} (S_{ps} - S_{psn}) \quad (10.73)$$

where

α_c = 0.53 for all infill hollow sections

= 0.45 for fully or partially encased H sections

S_p , S_{ps} , S_{pc} are the plastic section moduli for the steel section, the reinforcement and the concrete of the composite cross-section respectively (for the calculation of S_{pc}, the concrete is assumed to be uncracked).

S_{pn} , S_{psn} , S_{pcn} are the plastic section moduli of the corresponding components within the region of 2 d_n from the middle line of the composite cross-section.

d_n is the depth of the neutral axis from the middle line of the cross-section.

10.5.3.5 *Combined compression and uni-axial bending*

(1) For combined compression and bending based on first order analysis, both local capacity and overall stability shall be checked.

As an alternative, for composite columns subjected to combined compression and bending based on second order analysis, local capacity and overall stability of composite columns shall be checked at the same time provided that all the moments are properly evaluated to include second order moments.

(2) Local capacity check

The section capacity of a composite cross-section under combined compression and bending based on first order analysis shall be evaluated through the use of an interaction curve as shown in Figure 10.18.

Figure 10.18a shows the interaction curve of an infilled rectangular hollow section (with points A to E) while Figure 10.18b shows the same of a fully encased H section (with points A to D).

It is important to note that

- Point A marks the compression capacity of the composite cross-section:

  $P^A = P_{cp}$ (10.74a)

  $M^A = 0$ (10.74b)

- Point B corresponds to the moment capacity of the composite cross-section:

  $P^B = 0$ (10.75a)

  $M^B = M_{cp}$ (10.75b)

- At point C, the compression and the moment capacities of the composite cross-section are given as follows:

  $P^C = P_{pm} = \alpha_c A_c f_{cu}$ (10.76a)

  $M^C = M_{cp}$ (10.76b)

  The expressions are obtained by combining the stress distributions of the cross-section at points B and C; the compression area of the concrete at point B is equal to the tension area of the concrete at point C. The moment resistance at point C is equal to that at point B since the stress resultants from the additionally compressed parts nullify each other in the central region of the cross-section. However, these additionally compressed regions create an internal axial force which is equal to the plastic resistance to compression of the middle portion of the composite cross-section, P_{pm} over a depth of 2 d_n, where d_n is the plastic neutral axis distance from the centre-line of the composite cross-section.

- At point D, the plastic neutral axis coincides with the centroidal axis of the composite cross-section and the resulting axial force is half of that at point C.

  $P^D = P_{pm} / 2$ (10.77a)

  $M^D = M_{cp,max}$ (10.77b)

  In general, point D should not be included whenever both N and M cannot be guaranteed to be co-existing all the times.

- Point E is mid-way between points A and C. It is only used in composite cross-sections with concrete infilled hollow sections.

(3) The influence of transverse shear force on the compression and the moment capacities should be considered if the shear force V_1 acting on the steel section exceeds 50% of the shear capacity V_c of the steel section.

(4) For simplification, V may be assumed to act on the steel section alone, and the influence of transverse shear forces should be taken into account through a reduced design strength $(1 - \rho)\, p_y$ in the shear area A_v of the steel section to clause 8.2.1 . In all cases, the shear force V_1 shall not exceed the shear capacity of the steel section determined according to clause 8.2.1.

Alternatively, V may be distributed into V_1 acting on the steel section and V_2 acting on the reinforced concrete section. Unless a more accurate analysis is used, the shear forces acting on the steel and the reinforced concrete sections are given by:

$$V_1 = V \frac{M_s}{M_{cp}} \quad (10.78)$$

$$V_2 = V - V_1 \quad (10.79)$$

where

M_s is the moment capacity of the steel section and

M_{cp} is the moment capacity of the composite section.

The shear capacity V_c of the reinforced concrete section shall be determined in accordance with HKCC.

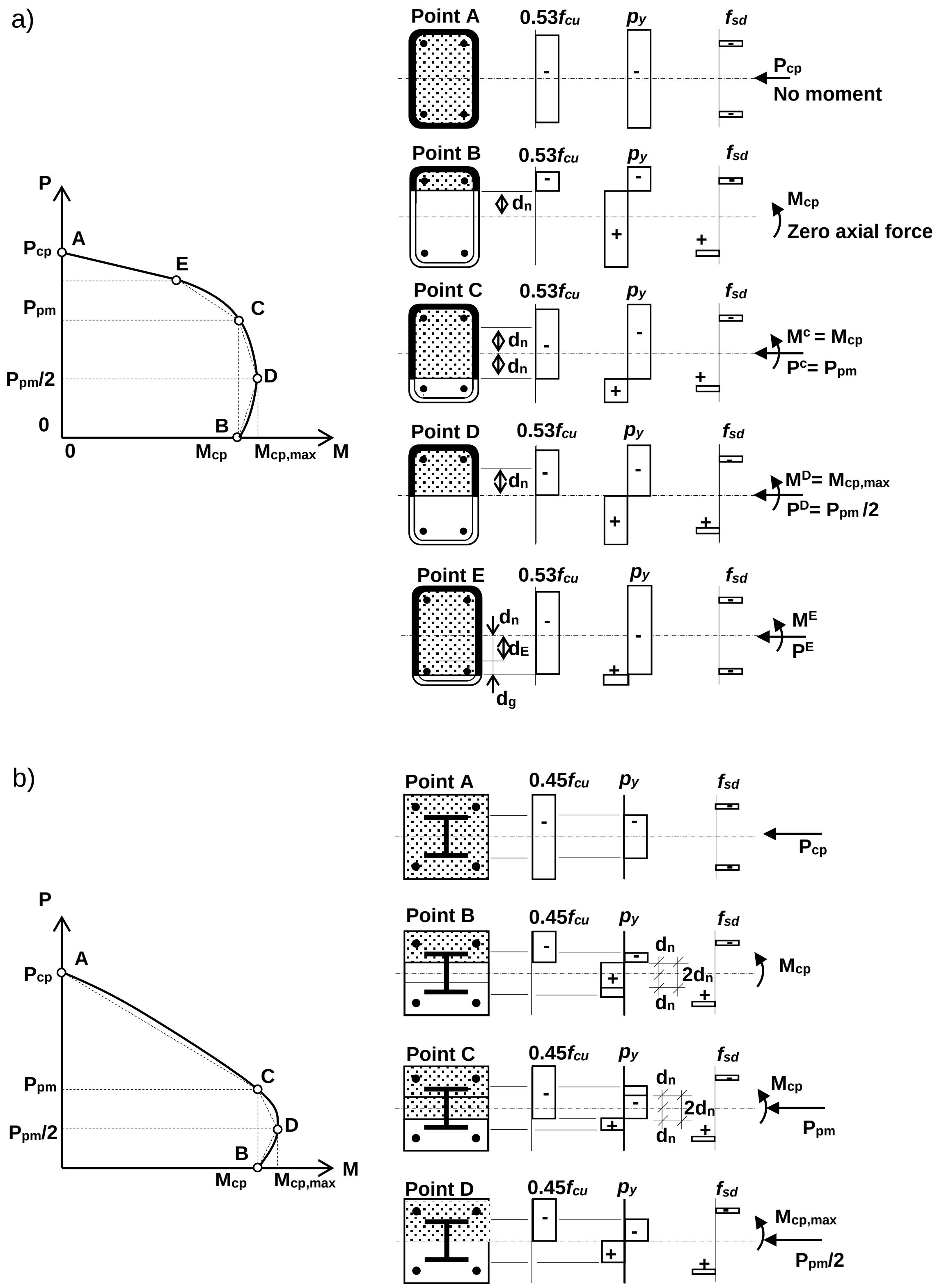


Figure 10.18 - Interaction curves and corresponding stress distributions

NOTE: In general, Point E is not needed for concrete encased I-sections subject to a moment about the major axis, or if the design axial force does not exceed P_{pm}. For concrete infilled hollow sections, the use of point E will give more economical design although much calculation effort is required. For simplicity, point E may be ignored.

(5) The following expression for local capacity check of composite cross-section shall be satisfied:

$$\frac{M}{M_{cp,P}} = \frac{M}{\mu_d \, M_{cp}} \le \alpha_M \qquad (10.80)$$

where:

M is the end moment or the maximum bending moment within the column length, calculated according to clause 10.5.3.1(10) to allow for second order effects if necessary;

$M_{cp,P}$ is the moment capacity of composite cross-section taking into account the axial force P, given by $\mu_d M_{cp}$ according to the interaction curve shown in Figure 10.19;

M_{cp} is the moment capacity of composite cross-section, given by point B in Figure 10.18.

μ_d is the reduction factor for moment resistance in the presence of axial force according to the interaction curve shown in Figure 10.19.

α_M is the limiting parameter equal to 0.9 for steel sections with nominal yield strengths between 235 and 355 N/mm^2 inclusive, and 0.8 for steel sections with nominal yield strength between 420 and 460 N/mm^2.

Any value of μ_d larger than 1.0 should only be used where the bending moment M depends directly on the action of the normal force P , for example where the moment M results from an eccentricity of the normal force P. Otherwise additional verification is necessary in accordance with clause 10.5.2(6).

(6) The overall stability of a composite column under combined compression and uni-axial bending based on first order analysis shall be checked as follows:

$$M \le 0.9 \; \mu \; M_{cp} \qquad (10.81)$$

where

M is the end moment or the maximum bending moment within the column length, calculated according to clause 10.5.3.1(10) to allow for second order effects if necessary;

μ is the moment resistance ratio after allowing for axial buckling according to the interaction curve shown in Figure 10.19; and

M_{cp} is the plastic moment resistance of the composite cross-section.

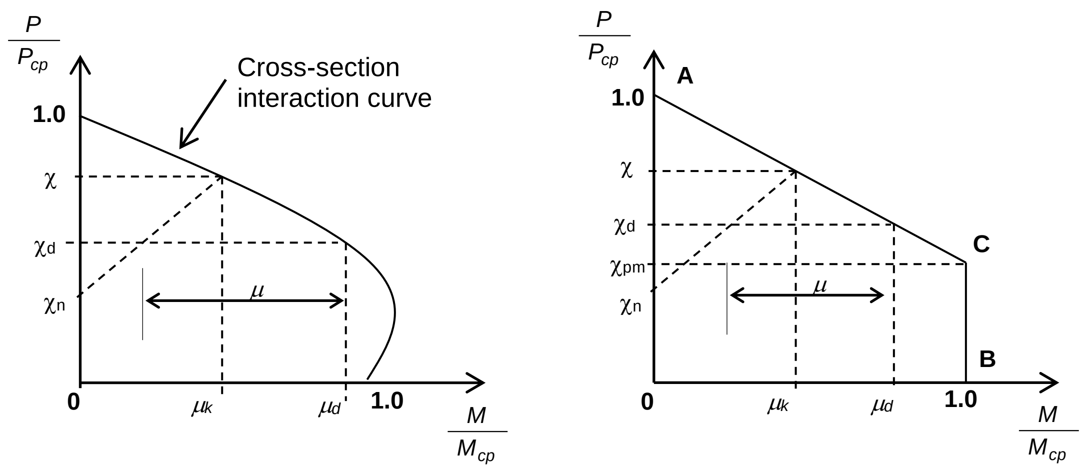


Figure 10.19 - Interaction curve for compression and uni-axial bending

(7) The moment resistance ratio μ shall be evaluated as follows:

$$\mu = \frac{(\chi - \chi_d)(1-\chi_n)}{(1-\chi_{pm})(\chi - \chi_n)} \qquad \text{when } \chi_d \geq \chi_{pm} \qquad (10.82a)$$

$$= 1 - \frac{(1-\chi)(\chi_d - \chi_n)}{(1-\chi_{pm})(\chi - \chi_n)} \qquad \text{when } \chi_d < \chi_{pm} \qquad (10.82b)$$

where

χ_{pm} is the axial resistance ratio due to the concrete given by $\dfrac{P_{pm}}{P_{cp}}$

χ_d is the design axial resistance ratio given by $\dfrac{P}{P_{cp}}$

χ is the reduction factor due to column buckling

For fully encased H sections and infilled rectangular hollow sections,

$$\chi_n = \frac{(1-r)\chi}{4} \qquad \text{for } \bar{\lambda} < 1.0 \qquad (10.83a)$$

$$= 0 \qquad \text{for } 1.0 \leq \bar{\lambda} < 2.0 \qquad (10.83b)$$

where r is the ratio of the small to the large end moment. If transverse loads occur within the column height, then r must be taken as unity and χ_n is thus equal to zero.

For infilled circular or square hollow sections

$$\chi_n = \frac{(1-r)\chi}{4} \qquad \text{for } \bar{\lambda} \leq 2.0 \qquad (10.84)$$

For infilled hollow sections, the interaction curve of A-E-C-B may be used, especially for columns under high axial load and low end moments. For better approximation, the position of point E may be chosen to be closer to point A rather than being mid-way between points A and C.

For simplicity, the expressions may be modified by taking χ_n = 0.

10.5.3.6 Combined compression and bi-axial bending

(1) For combined compression and bi-axial bending based on first order analysis, both local capacity and overall stability shall be checked.

As an alternative, for composite columns subjected to combined compression and bi-axial bending based on second order analysis, local capacity and overall stability of composite columns shall be checked at the same time provided that all the moments are properly evaluated to include second order moments.

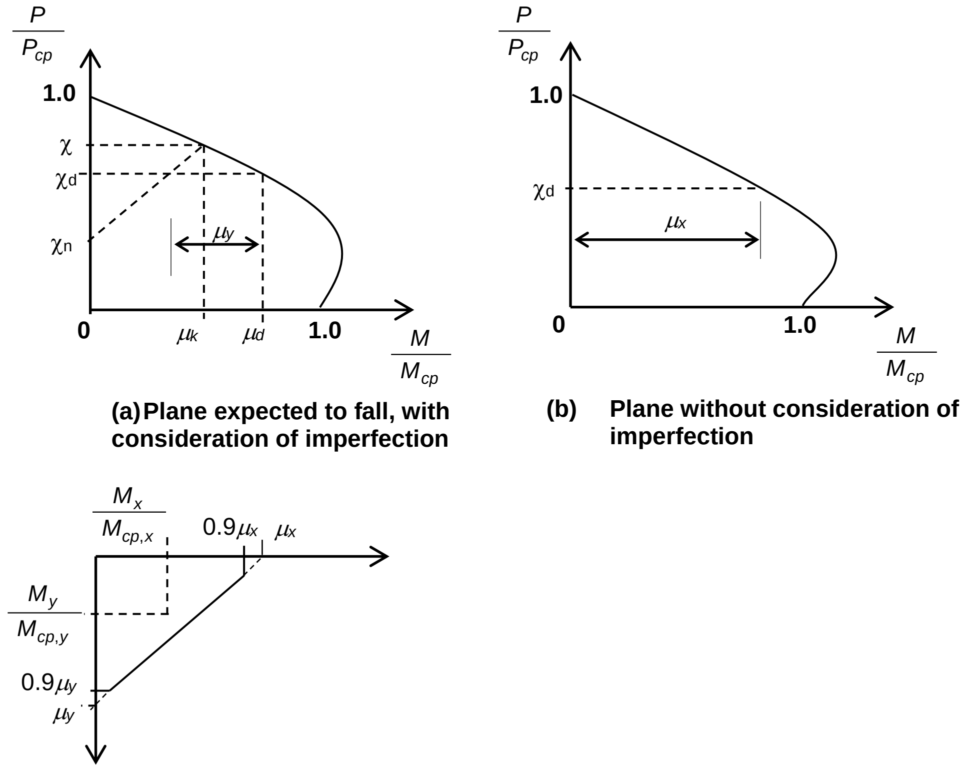
(2) For the design of a composite column under combined compression and bi-axial bending based on first order analysis, structural adequacy of the composite column should be checked as follows:

$$\frac{M_x}{\mu_x M_{cp,x}} \leq \alpha_M \qquad (10.85)$$

$$\frac{M_y}{\mu_y M_{cp,y}} \leq \alpha_M \qquad (10.86)$$

$$\frac{M_x}{\mu_x M_{cp,x}} + \frac{M_y}{\mu_y M_{cp,y}} \leq 1.0 \qquad (10.87)$$

In general, it will be obvious which of the axes is more likely to fail and the imperfections need to be considered for this direction only. If it is not evident which plane is the more critical, checks should be made for both planes.



(a) Plane expected to fall, with consideration of imperfection

(b) Plane without consideration of imperfection

(c) Moment interaction curve for bi-axial bending

Figure 10.20 - Verification for combined compression and bi-axial bending

As it is only necessary to consider the effect of geometric imperfections in the critical plane of column buckling, the moment resistance ratio μ in the other plane may be evaluated without the consideration of imperfections, which is presented as follows:

$$\mu = \frac{(1-\chi_d)}{(1-\chi_{pm})} \quad \text{when } \chi_d > \chi_{pm} \qquad (10.88a)$$

$$= 1.0 \quad \text{when } \chi_d \leq \chi_{pm} \qquad (10.88b)$$

10.5.4 Shear connection and load introduction

10.5.4.1 General

(1) Provision shall be made in regions of load introduction for internal forces and moments applied from members connected to the ends and for loads applied within the length to be distributed between the steel and concrete components, considering the shear resistance at the interface between steel and concrete. A clearly defined load path shall be provided that does not involve an amount of slip at this interface that would invalidate the assumptions made in design.

(2) Where composite columns and compression members are subjected to significant transverse shear, as for example by local transverse loads and by end moments, provision shall be made for the transfer of the corresponding longitudinal shear stress at the interface between steel and concrete.

(3) For axially loaded columns and compression members, longitudinal shear outside the areas of load introduction need not be considered.

10.5.4.2 Load introduction

(1) Shear connectors should be provided in the load introduction area and in areas with change of cross section, if the design shear strength τ_{Rd}, see clause 10.5.4.3, is exceeded at the interface between steel and concrete. The shear forces should be determined from the change of sectional forces of the steel or reinforced concrete section within the introduction length. If the loads are introduced into the concrete cross section only, the values shall be obtained from an elastic analysis when the effects from both creep and shrinkage have been allowed for properly. Otherwise, the forces at the interface should be determined by both elastic theory and plastic theory to determine the more severe case.

(2) In the absence of a more accurate method, the introduction length shall not exceed 2*d or L/3,* where *d* is the minimum transverse dimension of the column and *L* is the column length.

(3) For composite columns and compression members, no specific shear connection is needed to be provided for load introduction by endplates if the full interface between the concrete section and the endplate is permanently in compression, after taking proper allowances of both creep and shrinkage. Otherwise, the load introduction should be verified according to clause 10.5.4.2(5). For concrete infilled tubes of circular cross-section the effect caused by the confinement should be taken into account if the conditions given in clause 10.5.3.2(3) are satisfied using the values η_a and η_c for λ equal to zero.

(4) Where stud connectors are attached to the web of a fully or partially concrete encased steel I-section or a similar section, account shall be taken of the frictional forces that develop from the prevention of lateral expansion of the concrete by the adjacent steel flanges. This resistance shall be added to the calculated resistance of the shear connectors.

The additional resistance shall be assumed to be 0.5 μP_p on each flange and each horizontal row of studs, as shown in Figure 10.21, where μ is the relevant coefficient of friction. For steel sections without painting, μ shall be taken as 0.5. P_p is the resistance of a single stud in accordance with clause 10.3.2. In the absence of test information, the clear distance between the flanges should not exceed the values given in Figure 10.21.

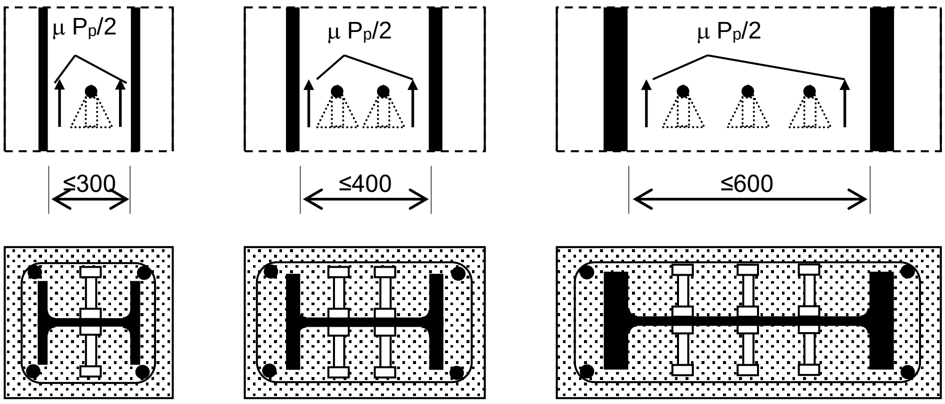


Figure 10.21 - Additional frictional forces in composite columns by use of headed studs

(5) If the cross-section is partially loaded, the loads shall be distributed with a ratio of 1:2.5 over the thickness t_e of the end plate. The concrete stresses should then be limited in the area of the effective load introduction, for concrete infilled hollow sections in accordance with clause 10.5.4.2(6) and for all other types of cross-sections in accordance with HKCC.

(6) If the concrete in an infilled circular hollow section or an infilled square hollow section is only partially loaded, for example by gusset plates or by stiffeners through the profile, the local design strength of concrete, σ_c , under the gusset plate or the stiffener resulting from the sectional forces of the concrete section shall be determined as follows:

$$\sigma_c = 0.53 f_{cu}\left[1+\eta_{cL}\,\frac{a}{t}\,\frac{p_y}{0.8 f_{cu}}\right]\sqrt{\frac{A_c}{A_1}} \le \frac{0.53 A_c f_{cu}}{A_1},\ \text{and} \le p_y \qquad (10.89)$$

where

t is the wall thickness of the steel tube;

a is the diameter of the tube or the width of the square section;

A_c is the cross sectional area of the concrete section of the column;

A_1 is the loaded area under the gusset plate;

η_{cL} = 4.9 for circular hollow sections and 3.5 for rectangular hollow sections.

The ratio A_c/A_1 shall not exceed the value 20. Welds between the gusset plate and the steel hollow sections should be designed according to section 9.

(7) For concrete infilled circular hollow sections, longitudinal reinforcement may be taken into account for the resistance of the column, even where the reinforcement is not welded to the end plates or in direct contact with the endplates provided that the gap e_g between the reinforcement and the end plate does not exceed 30 mm.

(8) Transverse reinforcement shall be designed in accordance with HKCC. In case of partially encased steel sections, concrete should be held in place by transverse reinforcement.

(9) In the case of load introduction through only the steel section or the concrete section in fully encased steel sections, the transverse reinforcement shall be designed for the longitudinal shear that results from the transmission of normal force from the parts of concrete directly connected with shear connectors into the parts of the concrete without direct shear connection.

The design and arrangement of transverse reinforcement shall be based on a truss model assuming an angle of 45° between concrete compression struts and the member axis.

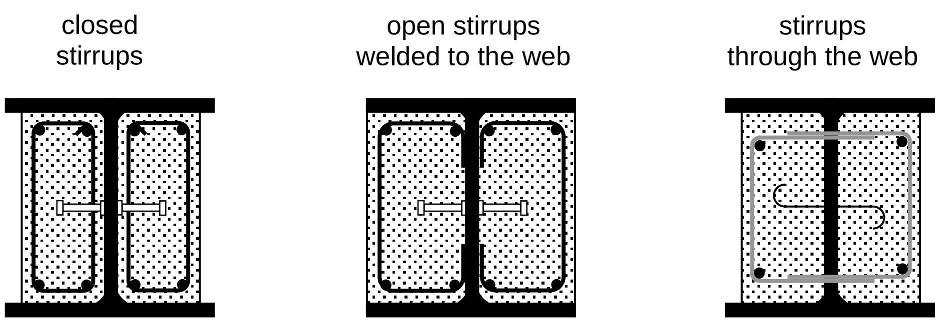


Figure 10.22 - Arrangement of stirrups

10.5.4.3 Longitudinal shear outside the areas of load introduction

(1) Outside the area of load introduction, longitudinal shear at the interface between concrete and steel shall be verified where it is caused by transverse loads and end moments. Shear connectors shall be provided, based on the distribution of the design value of longitudinal shear, where this exceeds the design shear strength τ.

(2) In absence of a more accurate method, elastic analysis shall be used to determine the longitudinal shear at the interface with full consideration of long-term effects and cracking of concrete,

(3) Provided that the surface of the steel section in contact with the concrete is unpainted and free from oil, grease and loose scale or rust, the values given in Table 10.15 should be assumed for τ.

Table 10.15 - Design shear strength τ

Type of cross section	τ (N/mm^2)
Completely concrete encased steel sections	0.30 (minimum)
Concrete infilled circular hollow sections	0.55
Concrete infilled rectangular hollow sections	0.40
Flanges of partially encased sections	0.20
Webs of partially encased sections	0.00

(4) The value of τ given in Table 10.15 for fully encased H sections applies to sections with a minimum concrete cover of 40 mm and transverse and longitudinal reinforcement in accordance with clause 10.5.5.2. For larger concrete cover and adequate reinforcement, larger values of τ should be used. Unless verified by tests, for completely encased sections, the increased value $\beta_c \tau$ given in Table 10.16 shall be used where β_c is given by:

$$\beta_c = 1 + 0.02 c_n \left[1 - \frac{c_{n,\min}}{c_n} \right] \leq 2.5 \qquad (10.90)$$

where

c_n is the nominal value of concrete cover in mm, see Figure 10.17a;

$c_{n,min}$ is the minimum concrete cover which should be taken at 40 mm.

Table 10.16 - Design shear strength τ_{Rd} for complete concrete encased steel sections

Concrete cover c_n (mm)	$\beta_c \tau$ (N/mm^2)
40	0.30
50	0.36
75	0.51
90	0.60
100	0.66
110	0.72
115 or above	0.75

(5) Unless otherwise verified by tests, shear connectors should always be provided to partially encased I-sections with transverse shear forces due to bending about the weak axis due to lateral loading or end moments,.

If transverse reinforcement is needed to provide shear resistance in additional to the shear resistance of the structural steel, then the required transverse reinforcement according to clause 10.5.3.5(4) should be welded onto the web of the steel section or should pass through the web of the steel section.

10.5.5 Detailing provisions

10.5.5.1 Concrete cover of steel profiles and reinforcement

(1) For fully encased H sections, a minimum concrete cover shall be provided to ensure the safe transmission of bond forces, the protection of the steel sections against corrosion, and the spalling of concrete.

(2) The concrete cover to a flange of a fully encased H section shall not be less than 40 mm, nor less than one-sixth of the breadth *b* of the flange.

(3) The concrete cover to reinforcement shall be in accordance with HKCC.

10.5.5.2 Longitudinal and transverse reinforcement

(1) The longitudinal reinforcement in concrete-encased columns which is allowed for in the resistance of the cross-section shall not be less than 0.3% of the cross-section of the concrete. In concrete infilled hollow sections, longitudinal reinforcement is not necessary except for fire resistance design.

(2) The transverse and longitudinal reinforcement in fully or partially encased columns shall be designed and detailed in accordance with HKCC.

(3) The clear distance between longitudinal reinforcing bars and the structural steel section may be smaller than that required by clause 10.5.5.2(2), or even zero. In this case, for bonding strength calculation, the effective perimeter c of the reinforcing bars should be taken as half or one quarter of its perimeter, depending on their positions in relation to the steel sections.

(4) For fully or partially encased members, where environmental conditions are considered as internal, and longitudinal reinforcement is neglected in design, a minimum longitudinal reinforcement of diameter 8 mm at a spacing of 250 mm and a transverse reinforcement of diameter 6 mm at a spacing of 200 mm spacing shall be provided. Alternatively welded mesh reinforcement of diameter 4 mm shall be used.