PRELIMINARY DRAFT

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 6 Steel and Composite Construction: 6B Composite Construction in Structural Steel and Concrete

BUREAU OF INDIAN STANDARDS

CONTENTS

National Building Code Sectional Committee, CED 46

FOREWORD

This Code (Part 6/Subsection 6B) covers the composite construction consists of the use of prefabricated steel structural units like steel beams and steel open web joists (trusses) in combination with concrete elements and often with steel reinforcements.

The design and construction should ensure monolithic action between the structural steel and concrete components so that they act as a single structural unit.

This Section was first published in 1970 and was subsequently revised in 1983 and 2005. In the 2005 version, the detailed design provisions were not included as IS 800 : 1984 'Code of practice for general construction in steel (*second revision*)' was under revision at that time. This Section, therefore, then referred to the 1984 version of IS 800 with the remarks that the latest version would prevail, as and when published. It, however, covered the philosophical aspects of limit state design which was being introduced in the revision of IS 800. IS 800 was subsequently revised and published as IS 800 : 2007 and hence, became the prevailing version for use under this Section. In the 2016 version of the NBC, the Section 6 of Part 6 of the Code brought in line with the revised version of IS 800:2007 and its Amendment No. 1 and to take care of further improvements and incorporations required.

In this revision, the Section has been subdivided into the following subsections:

6A Steel
6B Composite Construction

In this Subsection (Part 6/Subsection 6B), the provisions have been brought in line with IS 11384 : 2022 'Composite Construction in Structural Steel and Concrete - Code of Practice (*first revision*)'. Since composite construction in steel and Concrete has come a long way after that in India, the revision of this Code has become necessary. This was a major revision of IS 11834 and includes provisions for the design of most of the members and components of composite construction, based on the Limit States Method. This Subsection is restricted to the design of steel-concrete composite components and systems used in buildings. In this revision, the following major modifications have been effected:

a) The provisions conform to the limit state design philosophy, which is in line with IS 456 'Code of practice for plain and reinforced concrete' and IS 800 'Code of practice for general construction in structural steel'.

b) Provisions for the design of beam, slab, and columns of composite construction have been added.

c) Two types of composite column constructions are covered; namely, the Concrete encased steel columns (both fully encased and partially encased) and the concrete-filled steel columns have been considered.

d) The following types of composite slabs are presented:

1) Profiled sheeting, serving as form work for the reinforced concrete slab,
2) The embossed profile sheeting acting as form work and also as a tension reinforcement acting along with in-situ Concrete with or without shear connectors.

e) Improved provisions for the design of shear connectors and their testing methods.

f) Revised limit state of serviceability is also included to check for deflection, vibration and fire performance of the steel-concrete composite components.

g) Additional specifications on the use of light gauge steel and lightweight concrete (structural) are included.

Though the common methods of designs have been covered in this Code, special systems of design and construction, not covered by this code, may be permitted on production of satisfactory evidence regarding their design adequacy and safety based on specialist literature or by analysis, test, or both.

All requirements of IS 800, in so far as they apply, shall be deemed to form a part of this Code, except where otherwise laid down in this subsection.

Composite construction may be used in beams, columns as well as slabs in structures. Because of the special nature of bridge structures, where dynamic loadings are expected, this Code is restricted to buildings.

For the purpose of deciding whether a particular requirement of this Subsection is complied with the final value, observed or calculated, expressing the result of a test or analysis shall be rounded off in accordance with IS 2 : 2022 'Rules for rounding off numerical values (*second revision*)'. The number of significant places retained in the rounded-off value should be the same as that of the specified value in this subsection.

NATIONAL BUILDING CODE OF INDIA

PART 6 STRUCTURAL DESIGN

Section 6 Steel and Composite
6B Composite Construction in Structural Steel and Concrete

1 SCOPE

1.1 This Code (Part 6/Subsection 6B) deals with the design and construction of composite structures made up of structural steel and cast-in-situ/precast concrete, joined together to act integrally.

1.2 This Code is applicable to simply supported as well as continuous beams slabs and supporting column systems. The design provisions in this code are based on the limit states method of design.

2 REFERENCES

The standards listed ~~in Annex A~~ at the end of this Subsection under the title contain provisions that, through reference in this text, constitute the provisions of this Subsection. At the time of publication, the editions indicated were valid. All standards are subject to revision, and parties to agreements based on this code are encouraged to investigate the possibility of applying the most recent editions of these standards.

This clause 2 is intentionally retained in this Subsection as in the base standard namely IS 11384 for the convenience of users to enable parallel referencing of clause numbers in this Code and in IS 11384

3 TERMINOLOGY

For the purpose of this Subsection, the definitions given in [6-6B(1)] and the following shall apply.

3.1 Accidental Load — The load is not normally expected in design life but has a major impact if it ever occurs, such as ramming of vehicles against columns or any other member of the frame like bracings, blast loading, etc.

3.2 Accompanying Load — Live (imposed) load acting along with leading imposed load but causing lower action and/or deflections.

3.3 Action — The primary cause for stress or deformations in a structure, such as dead, live, wind, seismic, or temperature loads.

3.4 Built-up Section — A member fabricated by inter-connecting more than one element to form a compound section acting as a single member.

3.5 Camber — Intentionally introduced pre-curving (generally upwards) in a system, member or any portion of a member with respect to its chord. These are generally introduced to compensate for deflections at a specific level of loads.

3.6 Composite Action — Integral action of primary supporting steel member and supported concrete deck with or without limited slip at their interface to ensure greater strength and rigidity. In composite columns, it is the integral action between steel and its encasement or infill concrete. The shear transfer is to be ensured through the use of mechanical devices known as shear connectors in composite beams and columns, as required (*see* Fig. 8).

3.7 Design Load/ Factored Load — A load value obtained by multiplying the characteristic load with the partial safety factor for loads.

3.8 Design Service Life — The time period during which the structure or its components should satisfy the design objectives and functions.

3.9 Detail Category — Designation given to a particular detail to indicate the S-N curve to be used in fatigue assessment.

3.10 Differential Shrinkages — It is entirely due to shrinkage of concrete from the time composite action comes into effect. When the coefficient of thermal expansion varies significantly between steel and concrete (concrete with limestone or granite aggregate), it also includes the difference in thermal strain between the steel and concrete. Differential shrinkage may lead to an increase in stresses and is more pronounced in continuous beams.

3.11 Fatigue — Damage caused by repeated fluctuations of stress, leading to progressive cracking of a structural element.

3.12 Fatigue Limit State — The state of failure through fatigue damage due to repeated application of loads.

3.13 Fatigue Strength — Stress range that can be endured by a category of detail, depending upon the number of cycles.

3.14 Flexible Shear Connector — It resists shear forces by bending, tension or shearing in the root at the connection point of the steel member. It is subjected to plastic deformations as it reaches the ultimate strength, and the failure mode is ductile. It consists of studs, channels, etc. (Fig.8) welded to steel members to develop composite action with the concrete.

3.15 Initial Dead Load — The combination of weight of steel structure and the portion of the concrete deck that is supported by the steel structure alone before the development of full composite action with concrete reaching 75 percent of its 28 days strength.

3.16 Limit State — The load state beyond which the structure is incapable of performing its desired function.

3.17 Loads — Applied forces as per loads indicated in relevant standards like [6-6B(2)] for dead, live, wind, snow loads, etc.; and [6-6B(3)] for seismic loads that the structure is subjected to during its lifetime.

3.18 Partial Safety Factor for Loads – The factors multiplied by the loads or their combinations to obtain design loads while checking performance under various limit states.

3.19 m-k Factors — Physically, in a composite slab, 'm' is a broad measure of the mechanical interlock between the embossed profiled decking sheet and the reinforced cement concrete on top of the deck, and 'k' represents the frictional resistance between the two elements that is, steel and concrete (*see* Fig. 38).

3.20 Rigid Shear Connectors — These are shear hoops welded to bars, angles, horseshoes or tees welded to the flange of the fabricated steel units, as shown in Fig. 8(b) and (c). They resist shear forces by concrete bearing on the vertical face and shearing. They exhibit negligible deformation under shear transfer (that is, brittle failure). These are not usually recommended for ultimate limit state design.

3.21 Service Limit — The loading state beyond which the structure or its components becomes incapable of performing its intended function due to excessive deformation, or deflection, or vibration.

3.22 Serviceability Loads — The loads on the structure against which the serviceability of the structure needs to be checked.

3.23 Shear Connectors — These are the mechanical attachments to steel members to transfer interface shear between steel and concrete to develop composite action and are composed of flexible shear connectors [Fig. 8(a), (c)], rigid shear connectors [Fig. 8(b)], etc.

3.24 S-N Curve — Curve, defining the relationship between the numbers of stress cycles to failure N_{sc} at a constant stress range S_c, during fatigue loading on parts of a structure.

3.25 Strength Factors — The factors by which the specified strength is divided to obtain design strength; while assessing the safety under limit states of strength.

3.26 Stress Range — Algebraic difference between two extremes of stresses in a cycle of loading at a location in a member.

3.27 Superimposed Dead Load — The dead loads added subsequent to concrete hardening of concrete that is resisted by composite action.

3.28 Transient Load — The loads that are assumed to be varying over a short time interval like live load, loads with dynamic effect, temperature effects, wind loads on the structure, earthquake loads, accident loads, etc.

3.29 Ultimate Limit State — The state at which the structure fails and loses its integrity leading to its collapse.

4 SYMBOLS

For the purpose of this Subsection, the following letter symbols (other than those used for load categorization) shall have the meaning indicated against each; and where other symbols are used, they are explained at the appropriate place:

a	Span length of the slab transverse to the steel beams
A_s	Area of structural steel cross section
A_{sl}	Area of structural steel cross section in tension
A_c	Gross area of concrete
A_{co}	Equivalent area of the cracked composite beam section
A_{ec}	Area of concrete effective in compression
A_p	Effective cross-sectional area of profile steel sheeting
A_{st}	Area of steel reinforcements
A_f	Area of each flange of steel section
A_e	Effective cross sectional area
A_{sv}	Area of shear resistance
B	Centre to centre distance between beams and is equal to the transverse span of inner beam
b_c	Width of concrete encasement
b_p	Distance between the centres of adjacent ribs or one wave length of a profile deck
b_e	Effective width of the flange between pair of bolts
b_{eff}	Effective width of concrete flange
b_f	Width of the structural steel flange
D	Overall depth of girder/diameter of the steel cross section.
d	Depth of the web, Nominal diameter of bolts/rivets/studs.
d_2	Twice the clear distance from the compression flange of angles, plates, or tongue plates to the neutral axis
d_c	Vertical distance between the centroid of concrete slab and centroid of steel beam
d_s	Overall depth of the concrete slab
d_o	Nominal diameter of the pipe column or the dimensions of the column in the direction of depth of the base plate
d_p	Depth of profiled sheet deck
d_{sp}	Overall depth of the profiled sheet deck and concrete slab
e	Distance between the C.G. axis of the profiled steel sheeting and the extreme tension fiber of the composite slab
e_p	Distance between the plastic neutral axis and the extreme tension fiber of the composite slab
$E_{c,l}$	Modulus of elasticity of lightweight concrete
E_s	Modulus of elasticity of structural steel
E_{cm}	Secant Modulus of elasticity of concrete
E_{st}	Modulus of elasticity of steel reinforcements
F_f	Axial capacity of a single flange
F_{sr}	Axial capacity of reinforcing steel
F_{cc}	Compressive force in the concrete above the neutral axis
F_{sc}	Compressive force in the steel section above the neutral axis
F_{st}	Tensile force in the steel section below the neutral axis
F_w	Design capacity of the web in bearing
f	Actual normal stress range for the detail category
f_c	Actual axial compressive stress in concrete at service load
$f_{ctm,l}$	Tensile strength of lightweight concrete
f_{ck}	Characteristic compressive cube strength of concrete at 28 days
$f_{ctk(0.05)}$	Characteristic axial tensile strength of concrete based on 5% fractile
f_{ctm}	Mean axial tensile strength of concrete
f_f	Fatigue stress range corresponding to 5×10^6 cycles of loading

f_{fd}	Design fatigue normal stress range
f_{feq}	Equivalent constant amplitude stress
$f_{f\,max}$	Highest normal stress range
f_{fn}	Normal fatigue stress range
f_o	Proof stress
f_{yk}	Characteristic yield strength of reinforcement
f_{yf}, f_{yw}	Yield strength of flange and web, respectively
f_u	Characteristic ultimate tensile stress
f_{up}	Characteristic ultimate tensile stress of the profiled deck
f_y	Characteristic yield stress of structural steel
f_{yp}	Characteristic yield stress of profiled deck
h	Depth of the section / total depth of steel beam
H	Distance between top of concrete and bottom of bottom flange of steel beam
h_s	Distance between centroids of top and bottom flanges /nominal height of stud
h_y	Distance between shear centre of the two flanges of the cross section
I	Moment of inertia of the member about an axis perpendicular to the plane of the frame
I_c	Moment of inertia of concrete (assumed uncracked) about the axis of bending for column
I_{co}	Moment of inertia of the composite section
$I_{fc,y}$	Moment of Inertia of the compression flange about the minor axis of the steel beam
$I_{ft,y}$	Moment of Inertia of the tension flange about the minor axis of the steel beam
I_s	Moment of inertia of the steel section about the axis of bending for column
I_{st}	Moment of inertia of reinforcement about the axis of bending for column
I_z	Moment of inertia about the major axis
I_{ys}, I_{zs}	Second moment of area of steel section about the minor and major axis respectively
I_{ybf}	Moment of inertia of the bottom flange about the minor axis of the steel section
I_{zco}	Second moment of the equivalent area of the cracked composite beam section for major axis bending
k_1	Flexural stiffness of the steel web per unit length along the beam
k_2	Flexural stiffness of the cracked concrete or composite slab transverse to the spanning direction of steel beam
KL	Effective length of the member
KL/r	Appropriate, effective slenderness ratio of the section
KL/r_y	Effective slenderness ratio of the section about the minor axis
KL/r_z	Effective slenderness ratio of the section about the major axis
L	Actual span of composite beams
L_{bs}	Minimum bearing lengths of steel decking on the support
L_{bc}	Composite slab including the cast in place concrete
L_c	Effective span of the cantilever for overhang
L_o	Length between points of zero moments (inflection) in the span
L_s	Shear span
L_x	Distance of the maximum moment cross section under consideration to

	the support
M	Bending moment
M_v	Reduced bending moment due to effect of shear force
M_{cr}	Elastic critical moment corresponding to lateral torsional buckling
M_d	Design bending strength/design bending resistance under only bending moment
M'_d	Design bending resistance under combined bending and compression
M_{dp}	Design bending resistance of profiled steel sheeting
M_e	Elastic moment capacity of the section
M_f	Design plastic resistance of the flange alone for steel section
M_p	Plastic moment capacity of the section
M_y	Factored applied moments about the minor axis of the cross section
M_z	Factored applied moments about the major axis of the cross section
m	Modular ratio
m_{dl}	Modular ratio (long term)
m_{ll}	Modular ratio (short term)
N_{SC}	Number of stress cycles
P	Design axial force
P_{cr}	Elastic critical buckling load
P_p	Plastic resistance of encased steel column section or concrete filled rectangular or square column section
R_h	Flange stress reduction factor for hybrid section
r	Appropriate radius of gyration
r_y	Radius of gyration about the minor axis
r_z	Radius of gyration about the major axis
S	Spacing
S_l	Spacing of shear connectors for longitudinal shear due to flexural force
S_r	Spacing of shear connectors due to bending moment
t	Thickness of element/angle, time in minutes
t_f	Thickness of flange of steel section
t_p	Thickness of profiled deck sheet
t_w	Thickness of web of steel section,
V, V_v, V_L	Factored applied shear force
V_d	Design shear resistance
V_{dvd}	Vertical shear resistance
V_p	Plastic shear resistance under pure shear
V_{vd}	Design vertical shear resistance
W	Total equivalent load
X	Distance from the centre line of edge beam to the edge of slab
x_e	Depth of elastic neutral axis of the composite section from the centroid of steel section
x_u	Depth of neutral axis at limit state of flexure from top of the concrete
Z_e	Elastic section modulus
Z_p	Plastic section modulus
Z_{pc}, Z_{pcn}	Plastic section modulus of concrete about its own centroid and about the neutral axis of the composite section, respectively
Z_{pr}, Z_{prn}	Plastic section modulus of reinforcement about its own centroid and about the neutral axis of the composite section, respectively
Z_{ps}, Z_{psn}	Plastic section modulus of structural steel section about its own centroid and about the neutral axis of the composite section, respectively

y_g	Distance between the point of application of the load and shear centre of the cross section
α	Imperfection factor
α_c	Strength coefficient of concrete
δ	Steel contribution ratio
χ	Stress reduction factor due to buckling under compression
χ_m	Stress reduction factor, χ, at f_{ym}
	Stress reduction factor to account for lateral torsional buckling of a beam
γ	Unit weight of steel
γ_c	Partial safety factor for material (concrete)
γ_f	Partial safety factor for load
γ_{m0}	Partial safety factor against yield stress and buckling (structural steel)
γ_{m1}	Partial safety factor against ultimate stress (structural steel)
γ_{mf}	Partial safety factor for the strength of bolt
γ_{fft}	Partial safety factor for fatigue load
γ_{mft}	Partial safety factor for fatigue strength
γ_{mv}	Partial safety factor against shear failure
γ_{mw}	Partial safety factor for the strength of the weld
γ_k	Partial safety factor for material (reinforcements)
ε	Yield stress ratio, $(250/f_y)^{1/2}$
λ, λ_r	Non dimensional slenderness ratio = $\sqrt{f_y (KL/r)^2/\pi^2 E} = \sqrt{f_y/f_{cc}} = \sqrt{P_y/P_{cc}}$
λ_e	Equivalent slenderness ratio
μ_s	Poisson's ratio of structural steel
μ_c	Correction factor
η_1, η_2	Strength coefficients
η_{10}, η_{20}	Strength coefficients
τ	Actual shear stress range for the detail category
τ_f	Fatigue shear stress range
τ_{fd}	Design fatigue shear stress range
$\tau_{f,max}$	Highest shear stress range
τ_{fn}	Fatigue shear stress range at N_{SC} cycle for the detail category

5 MATERIALS AND WORKMANSHIP

5.1 The main materials for composite construction are structural steel, reinforcing steel, and concrete. The materials and workmanship of structural steel shall generally comply with specifications laid down in [6-6B(5)], and that of reinforced concrete shall comply with specifications laid down in [6-6B(4)]. However, the general properties and specifications of materials for composite construction are detailed as given in **5.2** to **5.4**.

5.2 Structural Steel

All the structural steel used in general construction, coming under the purview of this code, shall before fabrication conform to [6-6B(6)].

Structural steel other than those complying with [6-6B(6)] may also be used provided that the limiting stresses and other design provisions are suitably modified and the steel is also suitable for the type of fabrication adopted.

Steel that is not supported by mill test results may be used only in unimportant members and details, where their properties such as ductility and weldability would not affect the performance requirements of the members and the structure as a whole. However, such steels may be used in the structural system after confirming their quality is as per [6-6B(6)], by carrying out appropriate tests in accordance with the method specified in [6-6B(7)].

5.2.1 *Properties of Structural Steel*

The following physical properties shall be assumed for all grades of steel for design purposes:

Unit mass of steel	= 7850 kg/m^3
Young's modulus (modulus of elasticity)	= 2.0 x 10^5 MPa
Shear modulus	= 0.769 x 10^5 MPa
Poisson's ratio	= 0.30
Coefficient of thermal expansion	= 0.000012 / °C

The mechanical properties of steel like yield stress, ultimate stress and, elongation shall be as per values indicated in [6-6B(5)].

5.2.2 *Specifications of Structural Steel*

Unless otherwise permitted herein, structural steel used shall, before fabrication, comply with the requirements of the Indian Standards or their latest revisions [6-6B(8)].

The use of structural steel not covered by the above standards may be permitted with the specific approval of the competent authority.

5.2.3 *Other Ste*els

Except where permitted with the specific approval of the authority, steels for machined parts and for uses in features other than structural members or elements shall comply with the relevant Indian Standards [6-6B(9)], as appropriate.

5.2.4 *Castings and Forgings*

Steel casting and forgings shall comply with the requirements of the Indian Standards [6-6B(10)] as appropriate.

5.2.5 *Fasteners*

Bolts, nuts, washers and rivets shall comply with the following or relevant Indian Standards [6-6B(11)], as appropriate.

5.2.6 *Welding Consumables and Practices*

Welding consumables and practices shall comply with the following Indian standards [6-6B(12)], as appropriate:

5.2.7 *Wire Ropes and Cables*

These shall conform to the relevant Indian standards [6-6B(13)] except where the use of other types is specifically permitted by the authority.

5.3 Concrete

5.3.1 All structural reinforced concrete shall be of minimum grade M20 and shall be in accordance with material specification and workmanship as stipulated in [6-6B(4)]. The strengths shall be specified in terms of the characteristic compressive strengths of cubes, f_{ck}, measured at 28 days. The design provisions in this Subsection are applicable for concrete strength between M20 to M75. Specialist literature shall be adopted in composite design, while using concrete strength outside this range.

5.3.2 Concrete grade shall be designated based on its characteristic strength. The three main categories of concrete strength grade are given below, and the recommended design properties of concrete are correlated to 28-day characteristic compressive strength unless specified otherwise. The mechanical properties of concrete, namely, tensile strength and modulus of elasticity, shall be determined as per [6-6B(4)]. Additional data is given in Annex F.

a) *Ordinary Concrete* – Concrete grades up to M20 are included in this type. It could be prepared by a nominal mix proportioned by the weight of its main ingredients.

b) *Standard Concrete* – This type comprises of concrete grades from M25 to M50. It is made based on a design mix proportioned by the weight of its main ingredients, along with chemical admixtures to achieve certain target values.

c) *High strength concrete* – Concrete grades from M60 to M80 are included in this type. Even though usage of high strength concrete is allowed in composite construction, capacity equations specified in this provision shall not be directly used in the design. Specialist literature and experimental results are required for using this concrete.

5.3.3 *Lightweight Concrete*

Lightweight concrete may be used in composite construction, and the design provisions in this code shall be used within a strength range of M20 to M60. The mean tensile strength $f_{ctm,l}$ and modulus of elasticity $E_{cm,l}$ of lightweight concrete is to be calculated as modifications over the provisions in [6-6B(4)], as follows:

$$f_{\mathrm{ctm,l}} = \eta_{\mathrm{l}} f_{\mathrm{ctm}}$$

$$E_{\mathrm{cm,l}} = \eta_{\mathrm{E}} E_{\mathrm{cm}}$$

The factors η_l and η_E are determined using the following equation:

$$\eta_{\mathrm{l}} = 0.4 + \frac{0.6\,\rho}{2200}$$

$$\eta_{\mathrm{E}} = \left(\frac{\rho}{2200}\right)^2$$

Here, ρ is the upper limit of the oven-dry density of the relevant class of lightweight concrete, as given in Table 1.

Table 1 Density Classes and Corresponding Design Densities, ρ for Lightweight Concrete
(*Clause* 5.3.3)

Sl. No.	**Density Class, ρ**		**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
(1)	(2)		(3)	(4)	(5)	(6)	(7)	(8)
i)	Density (kg/m^3)		801-1000	1001-1200	1201-1400	1401-1600	1601-1800	1801-2000
ii)	Density (kg/m^3)	Plain concrete	1050	1250	1450	1650	1850	2050
		Reinforced concrete	1150	1350	1550	1750	1950	2150

5.3.4 Design of concrete components of a composite structure against creep shrinkage and temperature stress may be done as indicated in [6-6B(4)] or as per specialist literature.

5.4 Reinforcement Steel

5.4.1 Reinforcement steel shall consist of hot rolled, thermo-mechanically treated, or heat-treated rods, de-coiled rods, or cold-worked steel of various grades as given in Table 2. The grade designations and strength properties are given in Table 3.

Table 2 Grades of Reinforcing Steel
(*Clause* 5.4)

Sl No.	**Types of Steel**	**Grade / Designation**	**Relevant Standard**
(1)	(2)	(3)	(4)
i)	Mild steel (MS)	Grade - I	[6-6B(14)]
ii)	High yield strength deformed (HYSD)	Fe 415, Fe 415S, Fe 415D , Fe 500, Fe 500S, Fe 500D, Fe 550, Fe 550D, Fe 600	[6-6B(15)]

5.4.2 The minimum strength of reinforcing steel as specified in [6-6B(4)] is either the yield stress in case of mild steel or 0.2 percent proof strength in case of high yield strength steel and it is notionally taken as the characteristic strength of reinforcement f_{yk} .

5.4.3 The steel may be coated or galvanized to improve its corrosion resistance. The following corrosion resistive steel may be used as reinforcements:

a) *Galvanized Reinforcements* — The strength, elongation and bond properties are not adversely affected by galvanizing.

b) *Epoxy-coated Reinforcements* — These are reinforcements conforming to [6-

6B(15)] coated by fusion bonding epoxy conforming to [6-6B(16)]. The bond of coated reinforcements is lowered by up to 20 percent compared to un-coated reinforcements. The lap length and anchorage length shall be increased by 25 percent while using these steel bars.

c) *Stainless Steel Reinforcements* — Properties of stainless steel reinforcement shall not be inferior to carbon steel reinforcement of corresponding strength and class. These are reinforcements conforming to [6-6B(17)].

Table 3 Strength of Reinforcing Steel
(*Clause* 5.4.1)

Sl No.	**Type of Steel**	**Grade / Designation**	**Minimum Yield Stress/0.2 Percent Proof Stress,**	**Minimum Tensile Strength, as Percent of the Actual 0.2 Percent Proof Stress/Yield stress but not less than**	**Minimum Percent Elongation**
(1)	(2)	(3)	(4)	(5)	(6)
i)	Mild steel	Grade - I	Bars up to and including 20 mm dia = 250 MPa	410 MPa	23.0
			Bar dia. 20 mm ≤ 50 mm = 240 MPa	410 MPa	23.0
ii)	High yield strength deformed steel (HYSD)	Fe 415	415 MPa	110 Percent (≥ 485 MPa)	14.5
		Fe 415D/S		112 Percent (≥ 500 MPa)	18.0
		Fe 500	500 MPa	108 Percent (≥ 545 MPa)	12.0
		Fe 500D/S		110 Percent (≥ 565 MPa)	16.0
		Fe 550	550 MPa	110 Percent (≥ 585 MPa)	10.0
		Fe 550 D/S		108 Percent (≥ 600 MPa)	14.5
		Fe 600	600 MPa	106 Percent (≥ 600 MPa)	10.0

NOTES
1) Elongation on a gauge length of 5.65√A, where A is the cross-sectional area of the test piece, when tested in accordance with [6-6B(7)].
2) For Seismic Zone III, IV and V; HYSD steel bars having a minimum elongation of 14.5 percent and conforming to other requirements of [6-6B(15)] shall be used.
3) For Seismic Zone III, IV and V; Structural steel maximum yield strength shall not exceed the specified minimum value by more than 20 percent and conforming to other requirements of [6-6B(15)] shall be used.

6 STRUCTURAL ANALYSIS

This section is applicable to composite structures in which most of the members are composite. Where the structural behaviour is essentially that of reinforced or prestressed concrete, with only a few composite members, analysis provisions of [6-6B(4)] or [6-6B(18)] respectively, shall be used. Similarly, where the structural behaviour is essentially that of structural steel, with only a few composite members, analysis provisions of [6-6B(5)] shall be used.

6.1 Modelling and Basic Assumptions

6.1.1 Local buckling of plate elements of steel section and section classification

The types of elements that are encountered in steel sections are listed below:

a) Internal elements are elements attached along both longitudinal edges to other elements or to longitudinal stiffeners connected at suitable intervals to transverse stiffeners, for example, web of I-section and flanges and web of box section.
b) Outside elements or outstands are elements attached along only one of the longitudinal edges to an adjacent element, the other edge being free to displace out of plane for example, flange overhang of an I-section, the stem of T-section and legs of an angle section.
c) Tapered elements may be treated as a flat elements having an average thickness as defined in [6-6B(19)].

6.1.2 The section strength at the ultimate limit state should be considered based on their ability to resist local buckling before full plastic strength is developed. In this respect, the structural steel sections may be classified as given in [6-6B(5)]. Guidelines for sectional classification of composite sections with and without concrete encasements are given below:

a) *Composite sections*

   1) Steel compression elements (plates) in a composite section should be classified according to the least favourable class of steel elements in compression.
   2) The class of a composite section depends on the direction of the bending moment at that section.
   3) A steel compression element restrained by a reinforced concrete element through shear connectors may be placed in a more favourable class, after ensuring its improved local buckling resistance due to the above connection.
   4) Plastic stress distribution over the cross section should be used for section classifications 1 and 2. In classification 3, the elastic stress distribution should be used taking into account the sequence of construction and the effects of creep and shrinkage.
   5) For classification, design values of the strength of materials should be taken. Concrete in tension should be neglected. The stress distribution should be established for the gross cross-section of the steel web and the effective concrete flanges.
   6) The tension capacity of steel reinforcements satisfying the ductility requirements specified in **5.4**, only can be used in evaluating the strength in the hogging moment regions of the composite members. When the design resisting moment of the full or partial composite members is evaluated based on either simple rectangular stress block or parabolic stress block, the minimum area of steel reinforcement, A_{st}, used within the effective width of the concrete flange should satisfy the following:

$$A_{st} \geq A_c\ \alpha\ \frac{1}{\varepsilon^2}\frac{f_{ctm}}{f_{yk}}\sqrt{k_c}$$

where

$\varepsilon = \sqrt{250/f_y}$

A_c = Effective area of the concrete flange

f_y = Nominal value of yield strength of structural steel, in MPa

f_{yk} = Characteristic yield strength of tension reinforcement, in MPa

f_{ctm} = Mean tensile strength of concrete, in MPa (Annex F)

$k_c = \frac{1}{(1+d_s/(2\,z_0))} + 0.3 \leq 1.0$

α = 1.06 for Compact section, 1.17 for plastic section

d_s = thickness of concrete slab excluding haunch if any

z_0 = vertical distance between the centroid of the uncracked concrete slab and the centroid of the uncracked effective composite section using the modular ratio m for short-term loading.

7) The welded mesh should not be included in the effective section unless it has sufficient ductility before fracture when embedded in concrete.
8) Account should be taken of the class of steel section at every stage of construction in the global elastic analysis.

b) *Composite sections without concrete encasements*

1) A steel compression flange that is restrained from buckling by effective attachment to a concrete flange by shear connectors may be assumed to be in class 1 if the maximum spacing of the connectors is in accordance with **11.6.3**
2) Other steel flanges and webs in compression in the composite beams should be classified on the basis of width to thickness ratios and susceptibility to local buckling of steel only unless they are also restrained by concrete as in columns. Accordingly, sections are categorized into three groups as indicated in [6-6B(5)].
3) In beams, cross-sections with webs in Class 3 and flanges in Class 1 or 2 may be treated as an effective cross-section in Class 2 with an effective web in accordance with Fig. 1. The proportion of the web in compression should be replaced by a part of $20\varepsilon\, t_w$ adjacent to the compression flange, with another part of $20\varepsilon\, t_w$ adjacent to the plastic neutral axis of the effective cross-section where $\varepsilon = \sqrt{250/f_y}$ and t_w is the thickness of the web.

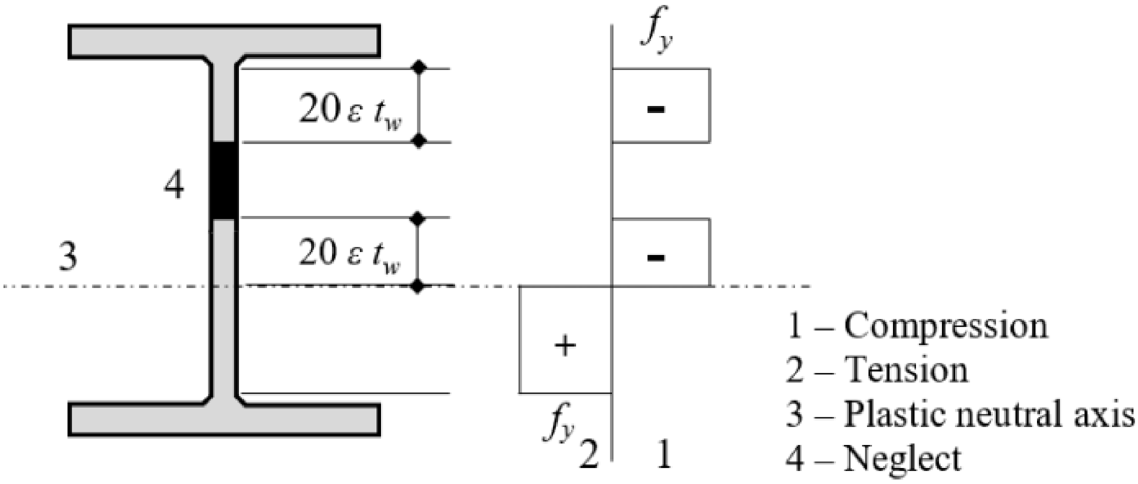


FIG. 1 EFFECTIVE CLASS 2 WEB IN BENDING

c) *Composite section with concrete encasements*

1) A steel outstand flange shall be classified as per Table 4.
2) In partially concrete-encased I-section (Fig.2), wherein the concrete is effectively attached to the web with stud or any reinforcement and covers on each side of the at least 80 percent of the flange overhang, the limiting overhang width of the flange to thickness ratio, b/t_f, for section classification may be taken as given in Table 4, where $\varepsilon = \sqrt{250/f_y}$.

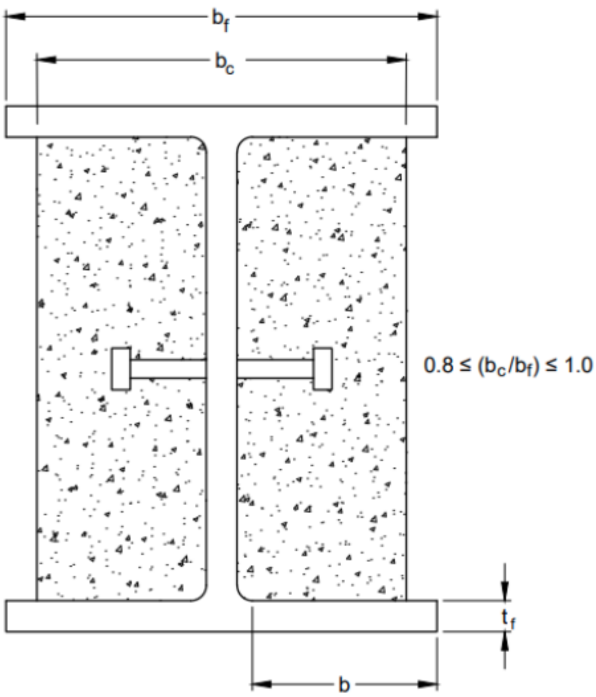


FIG. 2 PARTIALLY ENCASED I-SECTION

Table 4 Encased I Section classification

[*Clause* 6.1.2(c)]

Sl No.	Section	Flange overhang to thickness ratio (b/t_f)		
		Class 1 Plastic	Class 2 Compact	Class 3 Semi-compact
(1)	(2)	(3)	(4)	(5)
(i)	Rolled Section	9.4ε	10.5ε	15.7ε
(ii)	Welded Section	8.4ε	9.4ε	13.6ε

6.1.3 Joints

Joints in a composite structure may be modelled as:

a) Hinged, wherein the connection transfers only axial force and shear force, and negligible moment may be assumed to be transferred.
b) Rigid, wherein relative deformation between the members being joined is negligible due to the rigid nature of the joint, and the compatibility of deformation of the adjacent members at the joint may be assumed in the analysis.
c) Semi-rigid, wherein the joint deformation under the force transferred across the joint should be modelled.

The effects of the deformation behaviour of the joints on the internal force resultants and deformations in the members and structure may be generally neglected. However, where such effects may be significant (such as in semi-rigid connections), they shall be considered as discussed in [6-6B(5)].

6.1.4 Foundation Support

Normally the support from the foundation may be assumed as hinged or rigid, depending on the soil condition and foundation type. Effects of support deformation may be considered, where significant, as given in [6-6B(5)].

6.1.5 Deformation of the Structure

6.1.5.1 If under the external actions, the deformations are small, then the analysis can be done by studying the equilibrium of the undeformed structure. Such an analysis is referred to as first-order analysis or linear elastic analysis.

6.1.5.2 If under the external actions, the deformations are large enough to significantly affect or modify the structural behaviour, then the analysis should be performed considering the equilibrium of the structure under deformed configuration. Such an analysis is referred to as second-order analysis.

6.1.5.3 First-order analysis is adequate if the increase in the internal forces or moments due to actions on the deformation from the first-order analysis is less than 10 percent. This may be assumed to be satisfied if the ratio of the P_{cr}/P_d is greater than or equal to 10, where P_{cr} is the load corresponding to elastic instability of the member or structure as a whole and P_d, is the factored design compressive load. In evaluating the elastic critical load, P_{cr}, effect of cracking and creep of concrete, deformation of the joint shall be considered.

6.1.6 *Imperfections*

Effects of imperfections in the member (local imperfections) and imperfections in the overall geometry of the structure should be considered if they are significant. The first order analysis does not account for these, and hence the design method used should consider these effects. The second-order analysis should include the imperfections in the modelling, if the ratio of the factored design loads to the elastic critical load, $P_d/P_{cr} < 4.0$. If the second-order analysis does not include the imperfection effects (when $P_d/P_{cr} \geq 4.0$), the design method used should consider the imperfection effects. If the second-order analysis includes imperfection modelling, then the design method shall not include the effects of imperfections.

6.1.7 *Shear Lag and Effective Width of Flanges*

The effectiveness of steel and concrete flanges is reduced due to shear deformation in their plane either through rigorous analysis or by using effective width of the concrete as given in section **8.3.3** and that of steel flanges as given in [6-6B(5)].

6.1.8 *Creep, Shrinkage and Temperature*

The effect of creep of concrete on the internal forces can be accounted by using effective modulus of elasticity of concrete, as recommended in [6-6B(4)], by considering the creep and the age of concrete at the time of loading. The shrinkage strain may be taken as recommended in [6-6B(4)] or as per specialized literature.

When permanent loads are imposed in several stages, one mean value of time may be used for calculating creep coefficient. Such a mean value may also be used for evaluating prestressing deformations if the age of the relevant portion of the concrete at the time of prestressing is more than 14 days.

If a prefabricated and prestressed concrete slab is made composite with structural steel subsequently, the creep and shrinkage values only after the composite action becomes effective are to be taken.

The effects of creep, shrinkage and differential temperature, in terms of internal forces, moments and deformation indeterminate structure and indeterminate structures, where compatibility is not enforced, are referred to as 'primary effects'. In indeterminate structures, wherein the compatibility of deformation is ensured in the analysis, the additional effects of enforcing compatibility are referred to as 'secondary effects'.

The effects of creep, shrinkage, and temperature are normally neglected in analysis for the ultimate limit state other than fatigue for composite members, in which steel sections used should meet either plastic or compact classification.

The characteristic values of indirect actions due to the controlled imposed deformations (e.g. jacking of supports) may be calculated using the characteristic or nominal values of material properties in prestressed indeterminate composite beams and slabs unless a more accurate method is followed.

6.1.9 *Cracking of Concrete*

The effect of cracking of concrete in tension zones, as obtained from elastic analysis of the uncracked concrete model, may be considered by using effective stiffness of cracked concrete as given in [6-6B(4)].

6.1.10 *Sequence of Construction*

Depending upon the type of construction (propped/unpropped, prestressed, precast, etc.) the model for different stages of loading may differ. For the resultant internal forces, stresses and deflection at service load considering the effects of the staged construction may be obtained by superposing the analysis results for different loads obtained from different appropriate models for the stage of construction.

The effect of the sequence of construction may be neglected in structures made of plastic and compact steel sections and the model corresponding to the final stage of construction may be utilized for the ultimate limit state, other than fatigue.

6.2 Methods of Global Analysis

Different methods of global analysis, as discussed in the following sections, are permitted. The design and member strength check methods are chosen to be consistent with assumptions in analysis and actual global/member behaviour. Linear elastic first order method is most commonly used, but the second order effects and member imperfection effects etc., have to be taken care of in the design and member check stages.

6.2.1 Linear Elastic Analysis

6.2.1.1 Effect of cracking of concrete in tension, creep, shrinkage, temperature, prestressing and sequence of construction should be considered in the linear elastic analysis, as discussed in **6.1** and **7**. Forces and moments from the linear elastic analysis may be used even when the design is based on the ultimate limit state. In the case of continuous structures with hogging moments over supports, evaluation of the effective section at supports may be necessary, as mentioned in **8.3.3**. Appropriate load combinations with corresponding load factors are to be used to find out the maximum design values of moments and shears.

The stability of a structure as a whole against overturning shall be ensured under the limit state as per provisions of [6-6B(4)]. The foundation components of the structure shall also be safe against sliding under adverse conditions of the applied characteristic loads. The following factor of safety shall be ensured:

a) *Overturning* – The stability of a structure as a whole against overturning shall be ensured so that the restoring moment shall not be less than the sum of 1.2 times the maximum overturning moment due to the characteristic dead load and 1.4 times the maximum overturning moment due to the characteristic imposed loads. In cases where dead load provides the restoring moment, only 0.9 times the characteristic dead load as per [6-6B(4)] shall be considered. Restoring moments due to imposed loads shall be ignored.

b) *Sliding* – The structure shall have a factor against sliding of not less than 1.4 under the most adverse combination of the applied characteristic forces. In this case also, only 0.9 times the characteristic dead load shall be taken into account if it assists in resisting sliding.

c) *Instability* – Linear elastic instability analysis of member and the global system is possible, wherein first order assumptions and the resulting eigenvalue problem leads to eigenvalues as the instability load,

6.2.1.2 *Redistribution of moments from first order analysis*

Bending moments obtained from the linear elastic analysis of indeterminate slabs, beams and frames in composite structures in buildings may be redistributed to a limited extent as given below while ensuring the equilibrium is still satisfied:

a) concrete members are subjected primarily to flexure in accordance with [6-6B(4)];
b) steel members satisfy the plastic section requirements of [6-6B(5)];
c) composite members have partial or full shear connection as given in **11**;
d) redistributed internal forces and moments should satisfy equilibrium;
e) inelastic behaviour of the material, and local and lateral-torsional buckling should allow such a redistribution;
f) at least one end of the beam should be connected by a rigid or full-strength joint to the adjacent member;
g) each span is of uniform depth.

Table 5 gives the maximum percentage reduction in the hogging bending moments obtained from linear elastic global analysis, in composite beams, unless it is verified that the plastic rotation capacity permits a higher value.

Table 5 Limits to Percentage Redistribution of Hogging Bending Moments Obtained from Linear Elastic Analysis

(Clause 6.2.1.2)

Classification of the cross-section in the hogging moment region	**Plastic**	**Compact**	**Semi-compact**	**Slender**
(1)	(2)	(3)	(4)	(5)
For un-cracked section analysis moment	40	30	20	10
For cracked section analysis moment	25	15	10	0

NOTES

1) For grades of steel having yield strength greater than 360 MPa, redistribution is applicable to beams with plastic or compact cross-sections. In the redistribution, the reduction in hogging moment should not exceed the values for compact sections given in Table 5, unless it is demonstrated that rotation capacity permits a higher value.
2) The limits in Table 5 for semi-compact and slender sections relate to bending moments to be resisted by composite members and moments to be resisted by bare steel section should not be redistributed.

6.2.2 *Non-linear global analysis*

The non-linear analysis may be performed as specified in [6-6B(5)]. The non-linear behaviour of the shear connection shall be taken into account. Effects of deformed geometry of the structure, imperfections, joint and foundations, creep, shrinkage, temperature, stages of loading shall be taken into account as discussed in **6.1**.

6.2.3 *Rigid Plastic Analysis*

Rigid plastic global analysis may be used in composite structures to obtain ultimate limit state design values other than fatigue if the conditions given below are satisfied. Second-order effects, imperfections, creep, shrinkage, temperature effects, etc. need not be considered in the rigid plastic analysis. The effect of alternating plasticity due to variable loading need not be normally considered in buildings.

a) All the members and joints of the frame are either composite or structural steel.
b) Material and cross-section should satisfy requirements of plastic analysis as given in [6-6B(5)];
c) Joints are able to sustain the plastic resisting moments over a large plastic hinge rotation;
d) Critical plastic hinge location shall satisfy the following requirements:
    1) Cross-section of the structural steel section shall be symmetric about the plane of bending
    2) restraints are provided to prevent lateral torsional buckling
    3) lateral restraint is provided to the compression flange at all critical plastic hinge sections under any loading
    4) The rotation capacity of the section, considering any axial compression at the section, should be adequate to permit the plastic hinge rotation to form the plastic collapse mechanism.
    5) When rotation requirements are not calculated and ensured, all effective sections at potential plastic hinge locations should be of plastic classification.
e) The plastic hinge rotation capacity in composite sections may be assumed to be adequate where:
    1) the yield strength of structural steel does not exceed 450 MPa.
    2) the concrete slab is of normal density concrete, having a characteristic strength within the range of 25 MPa to 50 MPa.
    3) contribution of reinforced concrete encasement in compression is neglected in calculating design moment resistance.
    4) all effective steel sections at potential plastic hinge rotation should be plastic, and all the other sections should be plastic or compact.
    5) all beam connections to adjacent beams/columns should have adequate plastic rotation capacity or should have a design resisting moment of at least 1.2 times the design plastic resistance of the adjacent beam.
    6) structural steel compression flange at plastic hinge location should be laterally restrained.
    7) adjacent span lengths do not differ by more than 50 percent of the shorter span length.
    8) the end span length is not more than 115 percent of the adjacent span length.
    9) at plastic sections with the concrete slab is under compression, not more than 15 percent of the total depth of the section shall be under compression at the ultimate moment unless the plastic hinge at the section is the last to form (not requiring much plastic hinge rotation).
    10) Plastic hinges shall not be located in composite columns unless plastic rotation capacity can be verified.
    11) The lateral support to the compression flange shall be located within half the depth of the steel section from the plastic hinge section.
    12) Sections away from plastic hinge regions should satisfy at least compact section requirements.

7 LIMIT STATE DESIGN

7.1 General

In the limit state design method, the structure shall be designed to withstand safely all loads likely to act on it throughout its life. The objective of the design is to arrive at a structure that will remain fit for use during its life with acceptable target reliability. In other words, the probability of a limit state being reached during its lifetime should be very low. The acceptable limit for the safety and serviceability requirements before failure occurs is called a limit state. In general, the structure shall be designed on the basis of the most critical limit state and shall be checked for other limit states.

For satisfactory functioning of a structure during its design life conditions, stipulations and directives laid down in [6-6B(1)] shall be adequately satisfied for all steel-concrete composite structures.

7.1.1 *Effect of Construction Sequence*

The linear elastic method is valid for the analysis of the structure after considering load history, sequence of concrete casting and development of composite strength. In the case of propped construction, most of the initial dead load is resisted through the beam-prop system and the steel beam remains basically unstressed at this stage. In the case of un-propped construction, the steel beams alone have to carry the initial dead load and construction loads. Consequently, stresses and deflections at this stage shall not exceed specified design limits. The necessary distinction has to be made in the analysis about the stage of loading and the effectiveness of the system resisting the load. In the ultimate limit state, however, this distinction is not necessary while checking for flexural resistance. For the design of steel components and concrete deck, stipulations of [6-6B(1)] and this Subsection shall be applied.

7.2.1 The sequence of construction should be considered as an integral part of the design process, for example, when calculating the stresses under serviceability limit criteria. The sequence and method of construction need to be mentioned in the drawings. Additional information such as the position of construction joints, if any, may also be included in the drawings.

Where the composite section is loaded before the concrete has attained its 28-days characteristic cube strength, the elastic properties and limiting compressive strength of the concrete, and the nominal strengths of shear connectors should be based upon the cube strength of the concrete at the time of loading. Where a partially cast slab is assumed to act compositely, the shear connection should be designed for this condition as well as for the final condition. In calculating deflections, consideration should be given to the sequence of construction and, where appropriate, proper account should be taken of the pre-camber of steel section, the deflections of the steel section due to loads applied to it prior to the development of composite action and of partial composite action where deck slabs are cast in stages. In execution, the rate and sequence of concreting should be required to be such that partly matured concrete is not damaged as a result of limited composite action occurring from the deformation of the steel beams under subsequent concreting operations.

7.2 Limit States

A composite structure or part of it is considered unfit for use when it exceeds a particular state called the limit state, beyond which it infringes on one of the criteria governing its performance or use. The limit states can be classified into the following categories:

a) Ultimate Limit State

b) Serviceability Limit State

c) Fatigue Limit State

7.2.1 *Ultimate Limit State*

It is the state when under the worst combination of factored loads the structure or its components either reach design strength or becomes unstable. Both stability and strength need to be checked under the ultimate limit state.

In steel-concrete composite structures used in buildings or general constructions, the significant ultimate limit states to be considered are as follows:

a) Collapse due to flexural, shear or bearing failure of one or more critical sections or components,
b) Collapse due to horizontal shear failure at the interface between the steel beam and the concrete slab or composite slab system involving concrete slab and embossed profiled sheets,
c) Collapse due to the vertical separation of the concrete/composite slab from the steel beams, and
d) Collapse due to shear failure between steel and concrete components of the composite column or due to buckling of both fully/partially concrete encased steel columns or concrete filled hollow sections used as columns.

7.2.2 *Serviceability Limit States*

It is the state at which any of the following conditions occur during the loads encountered under construction and service:

a) Stress in structural steel has reached the prescribed limit.
b) Stress in concrete has reached the prescribed limit.
c) Deflection of a structure or its component reaches the prescribed limit.
d) Concrete crack width reaches the prescribed limit.
e) Slip at the interface between steel and concrete exceeds the prescribed limit.
f) Vibration becomes excessive, especially at overhangs.
g) Excessive corrosion affecting the durability of the structure.
h) Unacceptable effects due to fire.

7.2.3 *Fatigue Limit State*

It is the state at which stress range due to the application of live loads reaches the prescribed limit, corresponding to the number of load cycles and detail configuration.

7.3 Design Philosophy

For ensuring the design objectives, the design should be based on the characteristic values for material strengths (resistance) and applied loads (actions), which take into account the probability of variations in the material strengths and in the loads to be supported. The characteristic values should be based on statistical data, if available. Where such data is not available, they should be based on experiments. The design values are derived from the characteristic values through the use of partial safety factors, both for material strengths and for loads. In the absence of special considerations, these factors should have the values given in this section according to the material, the type of load and the limit state being considered. The reliability of the design is achieved by ensuring that:

$$\text{Design Action (Load)} \leq \text{Design Strength (Resistance)}$$

Design action refers to the external actions or load which act on the structure and the design resistance refers to the maximum resistance the structure and its components provides to resist the actions without causing failure of the structure and its components or causing hindrances to the smooth operation of the structure for which it is intended.

7.4 Design Actions (Loads)

7.4.1 Actions (loads) are classified into three main categories:

a) *Permanent or dead loads* — Loads due to self-weight of structural and non-structural components, fittings, ancillaries, and fixed equipment etc. Dead loads shall be calculated on the basis of unit weights which shall be established taking into consideration the materials specified for construction. Alternatively, the dead loads may be calculated on the basis of nominal dimensions and unit weights of materials given in [6-6B(20)].

b) *Variable or live loads*— Construction and service stage loads such as imposed (live) loads (for example, crane loads, impact, etc.), wind loads, snow loads and other loads shall be assumed in accordance with [6-6B(2)] and the earthquake forces shall be calculated in accordance with [6-6B(21)].

c) *Accidental loads* — Accidental loads are actions or loads expected due to explosions and the impact of vehicles, etc. The characteristic values of accidental loads generally correspond to the value specified by relevant code, standard or client. Design for the accidental load is generally not required in every building unless it is required by the client or approving authority, in which case, general recommendations given in [6-6B(5)] or specialist literature shall be followed.

7.4.2 Other than the actions due to the externally applied loads as given in **7.4.1**, if the effects of shrinkage, creep and temperature are liable to affect material safety and serviceability of the structure, these shall be accounted for in the design calculations {*see* [6-6B(22)]}.

7.4.3 *Load Combination*

The different combinations of loads considered shall be as given in [6-6B(22)]. For each combination, different partial safety factors for loads, γ_f are assigned to different loads to account for:

a) Possibility of unfavourable deviation of the load from the characteristic value,
b) Possibility of inaccurate assessment of the load,
c) Uncertainty in the assessment of effects of the load, and
d) Uncertainty in the assessment of the limit states being considered.

The loads or load effects shall be multiplied by the relevant γ_f factors, given in Table 6, to calculate the design loads or design load effects.

Table 6 Partial Safety Factor for Loads, γ_f for Limit States
(*Clause* 7.4.3)

Combination	Limit State of Strength					Limit state of serviceability			
	DL	**LL** [2]		**WL/ EL**	**AL**	**DL**	**LL** [2]		**WL /EL**
		Leading	Accompanying				Leading	Accompanying	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
DL+LL+CL	1.5	1.5	1.05	–	–	1.0	1.0	1.0	–
DL+LL+CL+ WL/EL	1.2 1.2	1.2 1.2	1.05 0.53	0.6 1.2	–	 1.0	 0.8	 0.8	 0.8
DL+WL/EL	1.5 (0.9) [1]	–	–	1.5	–	1.0	–	–	1.0
DL+ER	1.2 (0.9) [1]	1.2	–	–	–	–	–	–	–
DL+LL+AL	1.0	0.35	0.35	–	1.0	–	–	–	–

1) This value is to be considered when the dead load contribution to stability against overturning is critical or the dead load causes a reduction in stress due to other loads.

2) When the action of different live loads is simultaneously considered, the leading live load shall be considered to be the one causing the higher load effects in the member/section.

NOTE – The effects of actions (loads) in terms of stresses or stress resultants may be obtained from an appropriate method of analysis.

Abbreviations: DL= Dead Load, LL= Imposed Load (Live Loads), WL= Wind Load, CL= Crane Load (Vertical / horizontal), AL=Accidental Load, ER= Erection Load, EL= Earthquake Load.

7.5 Material Strength and Partial Safety Factor

The design strength, S_d of a structural component, is obtained as given below from nominal ultimate strength, S_u, and partial safety factors for material strength, γ_m.

$$S_d = S_u / \gamma_m$$

where the partial safety factor for materials, γ_m, (*see* Table 7), account for:

a) Possibility of unfavorable deviation of material strength from the characteristic value,
b) Possibility of unfavorable variation of member sizes,
c) Possibility of unfavorable reduction in member strength due to fabrication and tolerances, and
d) Uncertainty in the calculation of the strength of the members.

Table 7 Partial Safety Factor for Material γ_m
(*Clause* 7.5)

Sl No.	Material	Item	Partial Safety Factor γ_m	
			Strength Limit State	Serviceability Limit State
(1)	(2)	(3)	(4)	(5)
i)	Structural Steel, and steel sheeting	Resistance against yield stress, γ_{m0}	1.10	1.00
		Resistance of member to buckling, γ_{m0}	1.10	1.00
		Resistance against ultimate stress, γ_{m1}	1.25	1.00
		For Accidental Load Combinations, γ_{m0}, γ_{m1}	1.00	1.00
ii)	Reinforcement	Reinforcement resistance against yield stress, γ_k	1.15	1.00
iii)	Shear connectors	Yield stress, γ_{mv}	1.25	1.00
iv)	Connections	Bolts-friction type, γ_{mf}	1.25	1.00
		Bolts-bearing type, γ_{mb}	1.25	1.00
		Rivets, γ_{mr}	1.25	1.00
		Welds for shop fabrication, γ_{mw}	1.25	1.00
		Welds for site fabrication, γ_{mw}	1.50	1.00
v)	Concrete	For basic and seismic load combinations, γ_c	1.50	1.00
		For accidental load combinations, γ_c	1.20	1.00

8 DESIGN FOR ULTIMATE LIMIT STATES

8.1 Composite Beams

Composite beams shall be checked for the following:

a) resistance to flexure;
b) resistance to lateral-torsional buckling;
c) resistance to web shear buckling;
d) resistance to longitudinal shear (between steel and concrete components).

8.1.1 *Assumptions*

Design for the limit state of collapse in flexure shall be based on the assumptions given below:

1) Plane sections normal to the neutral axis remain plane and normal after bending;
2) The maximum strain in concrete at the outermost compression fiber at collapse is taken as 0.0035 in bending as per [6-6B(4)];
3) The tensile strength of the concrete is ignored;
4) The stress-strain curve for the reinforcing steel shall be assumed to be the same as given in [6-6B(4)];
5) The properties of structural steel shall be taken as given in [6-6B(5)].
6) The stress-strain curve for concrete may be taken to be the same as given in [6-6B(4)]. The corresponding parabolic stress block may be used to evaluate the design bending resistance of composite beams and slabs. The bending resistance of a composite beam for the value of concrete stress using an equivalent rectangular stress block, which simplifies the design resistance equation, may be used as an alternative method. Both methods and the design resistance equations are given in Annex B.

NOTE – Critical cross-sections include the following:

a) the sections where the bending moment is maximum;
b) the section at the faces of the support;
c) sections adjacent to concentrated force or reactions;
d) locations where a sudden change of cross-section occurs; (ratio of greater to the lesser resisting moment at adjacent sections exceeds 1.2) (changes due to cracking of concrete not to be included).

8.1.2 *Ultimate Bending Resistance*

For determining the position of the plastic neutral axis and the ultimate moment of resistance of composite beams, the guidelines given in Annex B may be used.

8.1.2.1 *General*

Composite constructions of types: (a) Top flange of steel beam supporting the concrete slab (Fig. 3) and (b) composite slab resting on structural steel beam (Fig. 4) are included in this design philosophy. A typical composite beam arrangement is shown in Fig. 3, where cast in place or precast RC slab is directly resting on the steel beam. Fig. 4 depict a composite slab consisting of RC and profiled deck sheet (with or without embossments) and supported on a structural steel beam, parallel and perpendicular to the steel beam, respectively. The neutral axis may be in the concrete slab, in the top flange of the steel section, or in the web of the steel section.

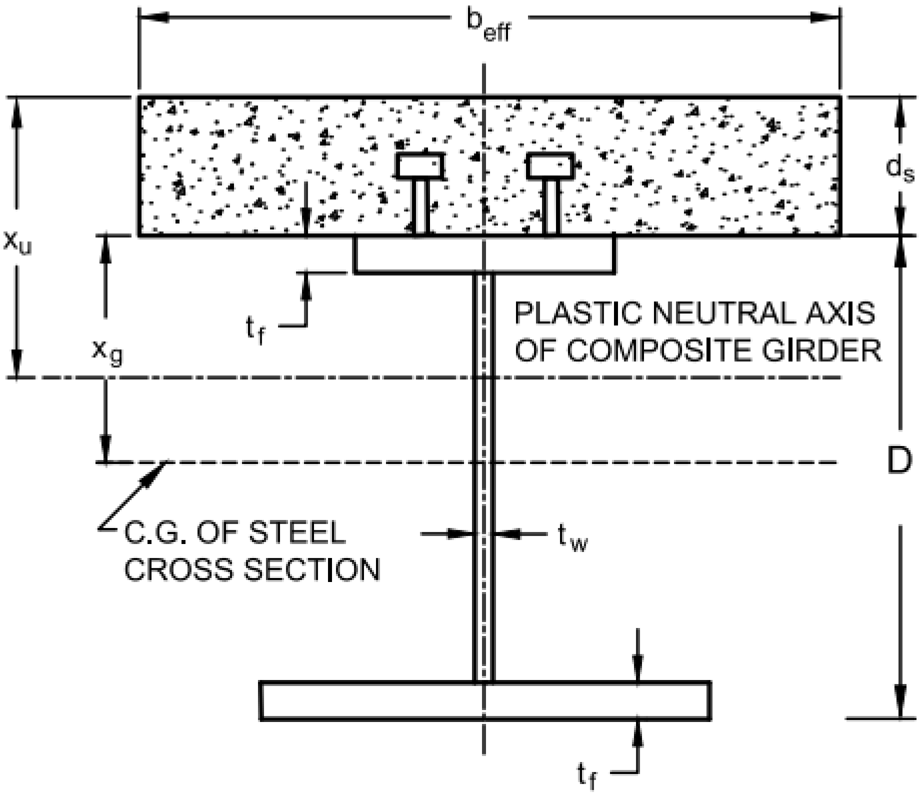


FIG. 3 TYPICAL COMPOSITE BEAMS WITHOUT PROFILED DECKING SHEET

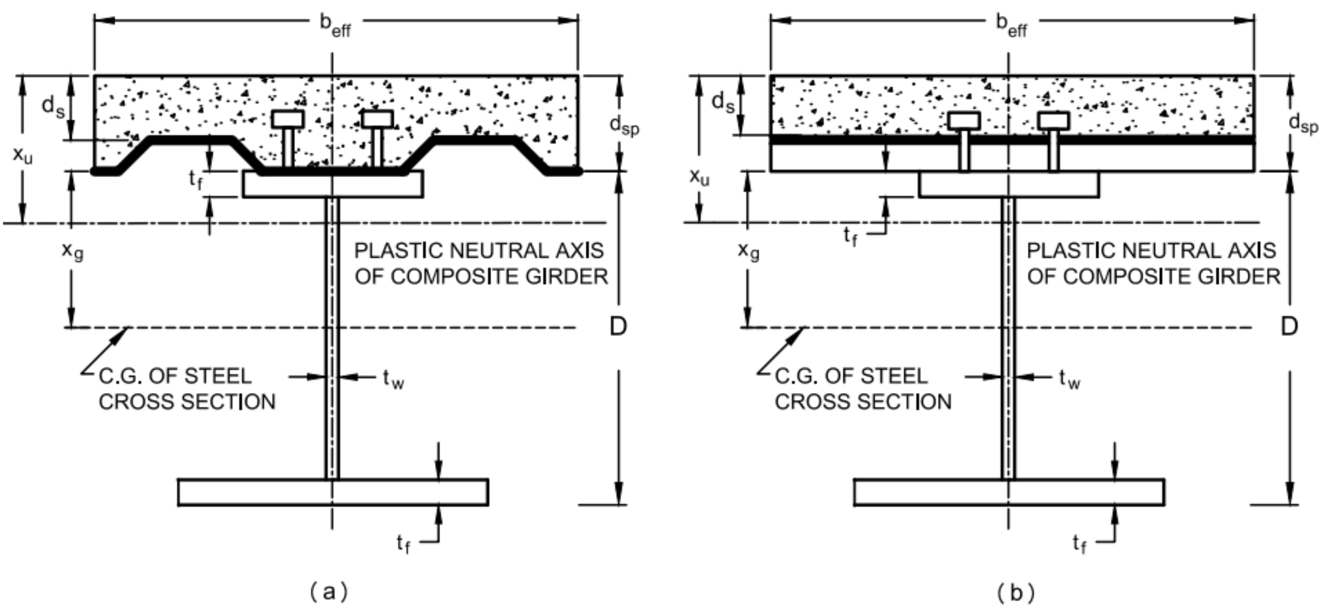


FIG. 4 TYPICAL COMPOSITE BEAMS WITH PROFILED DECKING SHEET (A) RIBS PARALLEL TO BEAMS (B) RIBS PERPENDICULAR TO BEAM

Bending moments and shears due to the application of factored loads may be analyzed in indeterminate structures by elastic theory assuming the concrete in the slab as un-cracked and unreinforced.

Hogging moments over internal supports as calculated above should be checked against section resistance assuming steel beam acting integrally with concrete (considering un-cracked and unreinforced). If the flexural tensile stress in concrete thus calculated exceeds the tensile strength of concrete, $f_{ctk,\,0.05}$ as in Annex F then a new analysis, neglecting concrete but including reinforcements over the effective width of the slab (*see* **8.3.3**) and over 15 percent of the span on each side of the support should be done to calculate the required design resistance, provided adjacent spans do not differ appreciably.

The redistribution of elastic analysis moment of indeterminate structure, as recommended in **6.2.1.2**, is allowed to arrive at the design moments in the case of plastic beams. Design for plastic analysis moments (as given in **6.2.3**) is permitted in the case of beams classified as plastic.

The bottom flange of the beam in the hogging moment zone should be adequately braced against lateral buckling. Otherwise, lateral buckling resistance has to be evaluated as recommended in **B-4**

8.2 Precast Slab on Steel Beam

The use of precast slab, both full depth and partial depth, is allowed for composite construction as one of the components of composite beams. Precast slabs shall be erected and connected to the steel section so as to ensure composite action along with the steel beam.

8.2.1 *Full Depth Precast Slab*

Full-depth precast concrete deck panels may be used for new construction as well as for the replacement of deteriorated concrete decks on existing steel beams so as to obtain composite action. This shall be ensured by proper shear connection during the erection of these precast panels. The typical requirements for these types of beams are as given below:

a) Panels shall either span the full width of the concreting deck or shall be in lengths that span between two or more parallel beams. The minimum thickness of the slab shall be 150 mm.
b) The panels shall be connected to the beams using shear connectors in pockets, which consist of mechanical connectors, such as shear studs encapsulated in non-shrinking grouted pockets. These connections cause the panels to develop composite action with the beams.
c) The contact between the precast panels at their longitudinal edge should ensure transfer of compression between the panels, necessary for composite action.

8.2.2 *Partial Depth Precast Slab*

Partial depth precast concrete deck panels are generally thin RCC/prestressed concrete panels that span between beams and also serve as forms/shuttering for the cast-in-place concrete deck. The typical geometrical parameters that govern the use of these panels as a composite unit for the floor system are as given below:

a) The minimum thickness of the precast panels shall be 75 mm.
b) As a composite floor system, the cast in-situ concrete and the partial-depth panels

together create the total thickness of the slab, with the panel reinforcing steel serving as the positive moment reinforcement in the bending direction of the combined slab.

c) Dimensions of the precast panels shall be chosen from consideration of easy handling, ease of lifting by cranes and for catering to the construction loads, including load of wet cast in-situ concrete.

d) Partial-depth panels must be capable of developing sufficient composite action with the cast *in-situ* concrete to be an effective floor system.

e) To ensure full bond between the cast *in-situ* concrete and precast panels, it is recommended that the top surface of the precast panel is intentionally roughened while casting it and is cleaned by removing the laitance or other contaminants on the surface and other measures may be taken, before the placement of the cast *in-situ* concrete so as to ensure a good bond between precast and cast in place concrete.

f) After the precast panels are in place, the top layer of the reinforcing steel shall be placed over it, and the cast-in-situ concrete shall be placed on top of the panels.

8.2.3 *General Design Principles*

The design of precast slabs is based on the following principles:

a) The precast slab together with any *in-situ* concrete (for partial depth slab) should be designed as continuous in both the longitudinal and the transverse direction.

b) The joints between slabs should be designed to transmit membrane forces as well as bending moment and shear forces.

c) The effective width of precast slab in the composite beam action shall be calculated as per **8.3.3**.

d) The design principles of composite beams involving either full depth or partial depth precast slabs are similar to standard composite decks using cast *in-situ* reinforced concrete.

e) Vertical shear check of the composite beam shall be done as per **8.3.6**.

f) For serviceability limit states, guidelines given in **7.2.2** and **9** shall be followed.

8.2.4 *Joints between Steel Beam and Precast Concrete Slab*

a) Where precast slabs are supported on steel beams without bedding, the influence of the vertical tolerances of the bearing surfaces shall be considered.

b) The shear transfer between steel flange and precast concrete through mechanical shear connector shall be designed as per **11** with the following precautions:

   1) If shear connectors are welded to the steel beam projecting into the recesses within the slabs or joints between slabs, which are filled with concrete after erection, the detailing and the properties of the concrete should be such that it can be cast properly.

   2) The minimum infill around the shear connectors should be at least 25 mm.

   3) If shear connectors are arranged in groups, sufficient reinforcement should be provided near each group to prevent premature local failure in either the precast or the *in-situ* concrete.

4) Special provision for protection against corrosion shall be adopted, wherein the steel flange under precast slabs without bedding should have the same corrosion protection as the rest of the steelwork in addition to the top coating provided after erection.
5) Bedding with the purpose of protecting against corrosion may be designed to be non-load bearing.

8.2.5 *Joints between Precast members*

The critical sections of members close to joints should be designed to resist the most adverse combination of shear, axial force and bending caused by the ultimate vertical and horizontal forces. When the design of the precast members is based on the assumption that the joint between them is not capable of transmitting bending moment (*see* **8.3.4**), suitable precautions should be taken to ensure that if any crack developed is not excessive, and reduce the shear or axial force resistance of the member and should not be aesthetically or functionally objectionable.

Where space is left between two or more precast units, to be filled later with *in-situ* concrete or non-shrink grout, the gap should be large enough for easy placement and adequate compaction of the filling material to fill the gap completely.

8.2.6 *Structural Connection at Joints*

When designing and detailing the connections across joints between precast members, the overall stability of the structure, including its stability during construction, shall be considered. A typical joint connection is shown in Fig. 5.

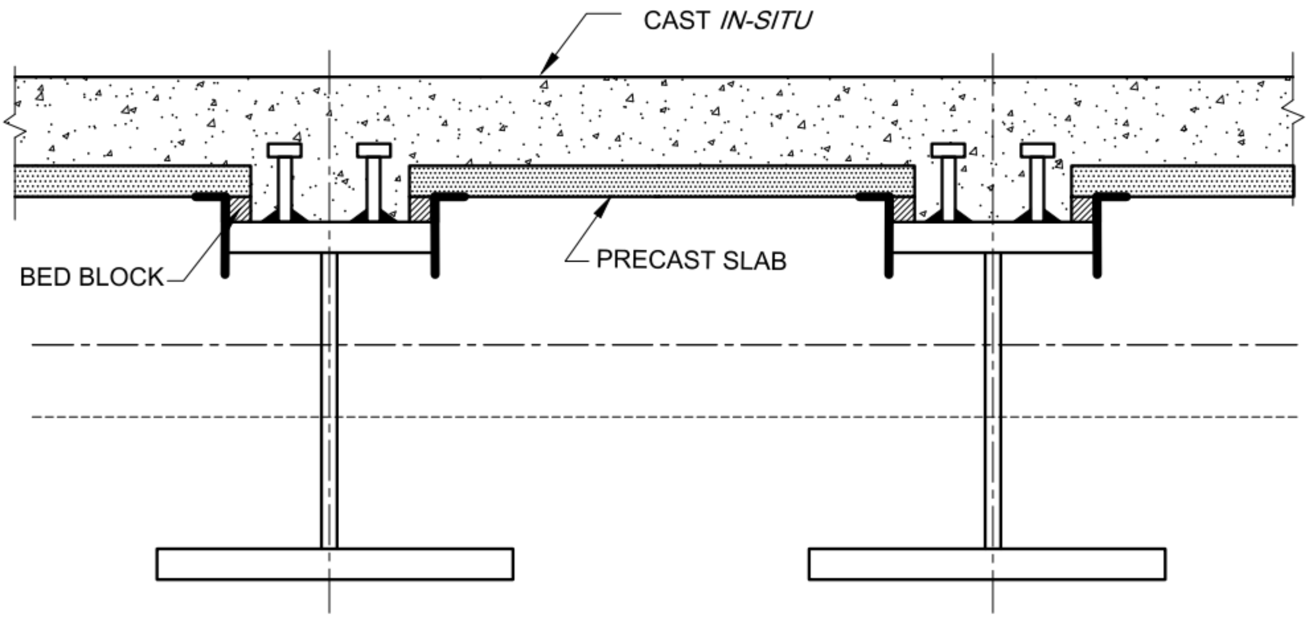


FIG. 5 TYPICAL JOINT CONNECTION FOR PARTIAL DEPTH PRECAST SLAB

8.3 Design Method

Connections should, where possible, be designed in accordance with the generally accepted methods applicable to reinforced concrete or structural steel.

8.3.1 *General Considerations in Design Details*

In addition to ultimate resistance requirements, the following should be considered:

a) *Protection* — Connections should be designed so that the standard of protection against weather and corrosion required for the rest of the structure is sufficiently maintained.
b) *Appearance* — Where connections are to be exposed, they should be designed to achieve and maintain the quality of appearance required for the rest of the structure.
c) *Manufacture, assembly, and erection* — Methods of manufacture and erection should be considered during design. Care should be taken during erection and the following precautions are mandatory:

    1) Where projecting bars or sections are required, they should be kept to a minimum and made as simple as possible. The lengths of such projections should be not more than necessary.
    2) Fixing devices should be located in concrete sections of adequate strength.
    3) The practicability of both casting and assembly should be considered.
    4) Most connections require the introduction of suitable jointing material. Sufficient space should be allowed in the design for such material to ensure that the proper filling of the joint is possible.

8.3.2 *Reinforcement Continuity at Joint*

Where continuity of reinforcement is required through the connection, the joining method used should be such that the assumptions made in analyzing the structure and critical sections are realized. The standard methods applicable for achieving continuity of reinforcements are lapping and butt welding of bars.

8.3.3 *Effective Width of Concrete Slab*

(a) For resistance calculation, the width of the slab, which is effective as the compression flange of the composite beam on each side of the steel beam, b_{eff}, shall be as per the equation given below:

$$b_{\mathrm{eff}} = b_{\mathrm{o}} + (b_{\mathrm{e1}} + b_{\mathrm{e2}}) = b_{\mathrm{o}} + \frac{L_{\mathrm{e}}}{4} \leq b_{\mathrm{o}} + (b_1 + b_2)$$

where

b_{o} = centre to centre between the outer row of shear connectors.

$b_{\mathrm{e1}}, b_{\mathrm{e2}}$ = are the value of the effective width of the concrete flange on each side of the web and taken as L_{e} / 8 but not greater than the geometric width b_1 and b_2, respectively as shown in Fig.6.

= $L_{\mathrm{e}}/8 \leq b_1$ and b_2 *(respectively)*

L_e = the effective equivalent span length of the corresponding composite beam,
= centre-to-centre distance between the supports for simply supported beam.
= as shown in Fig. 7 in a continuously supported beam

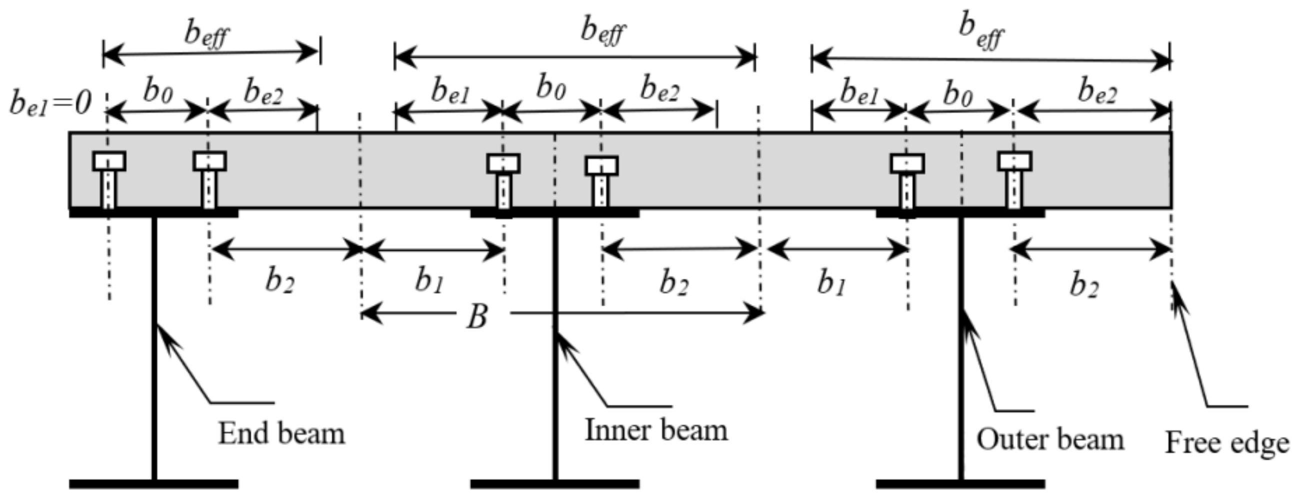


FIG. 6 EFFECTIVE WIDTHS FOR CONCRETE FLANGE OF COMPOSITE BEAMS

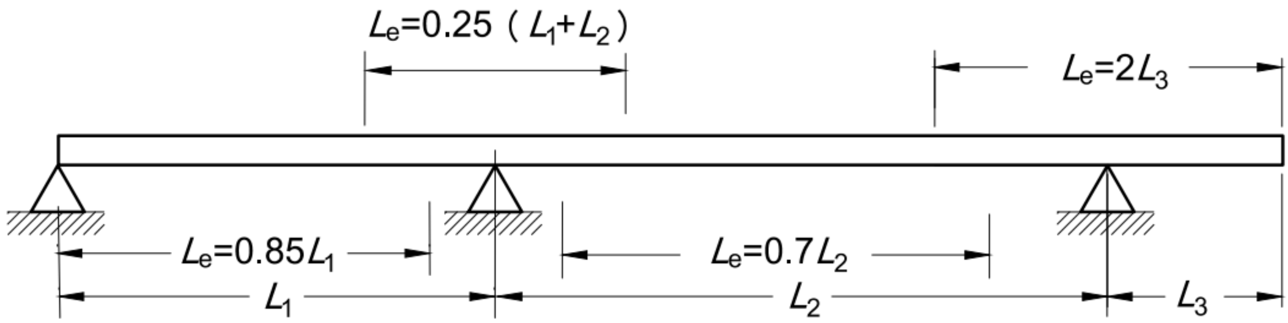


FIG. 7 EQUIVALENT SPANS FOR BEAM CONTINUITY

Reinforcements placed parallel to the steel beam within the effective width of the concrete slab should be considered for hogging moment resistance calculation of the composite beam at the continuous support.

(b) The effective width at an end beam may be determined as:

$$b_{eff} = b_0 + \beta_{e1}\, b_{e1} + \beta_{e2}\, b_{e2}$$

where

β_{e1} = (0.55 + 0.025 (L_e/b_{e1})

β_{e2} = (0.55 + 0.025 (L_e/b_{e2})

L_e = Equivalent effective end span

b_{e1} = 0 as shown in Fig.6

8.3.4 *Effective Cross Section for Strength Calculation*

In calculating the strength of the cross section of the composite beams, the following should be considered:

a) For Sagging Moment – Concrete in the effective width and structural steel beam to be included but not the steel reinforcements in the effective width of concrete.
b) For Hogging Moment – Concrete to be neglected but longitudinal steel reinforcement along the beam length within the effective width of the concrete in tension and structural steel beam to be included.

8.3.5 *Design of Structure for Bending Moment*

8.3.5.1 The factored design flexural strength, M_d, in a beam and the external action, M, shall satisfy

$$M \leq M_d$$

where,

$M_d = M_n / \gamma_{m0}$ = design flexural resistance calculated as given below.

γ_{m0} = partial safety factor against flexural failure (Table 7).

M_n = nominal ultimate flexural resistance

8.3.5.2 Considering local buckling, sections are to be analyzed as plastic, compact or semi-compact with the following additional consideration.

a) Load history and development in composite action are to be taken into consideration with appropriate values of the modular ratio, m at each stage, and stresses and deflections are to be the summation of values over successive stages.
b) Determination of bending resistance for a beam before composite action has set in or during the construction stage shall be done as per [6-6B(5)].
c) The structural steel reinforcement and concrete fully interact with full shear transfer between the concrete and structural steel under composite action.
d) The effective width of concrete in composite action may be as mentioned in **8.3.3**.
e) For calculating the bending resistance of plastic, compact and semi-compact composite beams, the procedure presented in Annex B may be followed.
f) The partial shear connection may be used in plastic and compact sections either for attaining economy without losing much in the moment capacity of the composite section or under conditions where the number of shear connectors required for full shear interactions cannot be provided. Requirements on the method of evaluating the bending resistance of partial shear connection are presented in **B-1.2**.

The bending moment and shear force distribution in continuous beams for secondary, as well as primary moment resistant frames/beams have to be determined by structural analysis.

8.3.5.3 *Design of structure for effect of lateral buckling on moment resistance*

Lateral Buckling may govern the design under the following conditions:

a) At the construction stage, in the top flange closer to mid-span in both simply supported and continuous beams.
b) At the construction and composite stage, in the bottom flange closer to support in continuous beams.

At the construction stage, the effect of lateral buckling on the bottom flange in a continuous beam shall be taken care of by considering cantilever action up to the point of inflection from the support.

If required, suitable horizontal bracings or members may be provided at the bottom flange to reduce the effective length of the compression flange near support. For beams that are provided with such bracings or members giving effective lateral restraint to the compression flange at intervals along the span, the effective lateral restraint shall be capable of resisting a force equal to 2.5 percent of the maximum force in the compression flange taken as divided equally between the numbers of points at which the restraint in bracing members occur. In beams supporting composite slabs, wherein the sheeting is welded to the top flange before concreting is done, the lateral restraint of the deck sheet may be considered.

The effective laterally unsupported length of compression flange of steel sections in sagging and hogging moment regions may be taken as given in [6-6B(5)]. In sagging moment segments of composite beams, the concrete slab may be assumed to provide lateral support, provided an adequate shear connection between the slab and the beam is provided. In the hogging moment regions of composite beams, the effective length considering the rotational restraint provided by the slab to the top tension flange may be taken into account as given in **B-2** of Annex B.

8.3.5.4 *Simplified Design of Continuous Beams in Buildings without Lateral Bracings*

Continuous composite beams (or composite beams within a frame) with plastic, compact, or semi-compact sections may be designed without any additional lateral bracings to the compression flange when the following conditions are satisfied:

a) The concrete slab on top should be adequately attached with shear connectors to the structural steel section and to at least on another parallel member top flange to facilitate inverted U-frame action that rotationally restrains bottom flange lateral buckling at hogging moment sections.
b) Adjacent span lengths of continuous beams should not differ by more than 20 percent of the shorter span length. Cantilever span length should not exceed 15 percent of the adjacent span length.
c) The loading on each span should be uniformly distributed, with the permanent design loads exceeding 40 percent of the total design load.
d) At each support of steel member, the bottom flange should be laterally restrained with a web bearing stiffener at that location.
e) The maximum depth of beam sections should not exceed the value given in Table 8.

Table 8 Maximum Depth of Steel Beams in Composite Construction (In mm)
[*Clause* 8.4.5.4 (e)]

Sl No.	**Type of Member**	**Nominal Yield Strength limit of Structural Steel (MPa)**							
		250		300		350		>400	
		Un-cased	En-cased	Un-cased	En-cased	Un-cased	En-cased	Un-cased	En-cased
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
i)	I section	800	1000	700	900	650	850	500	650
ii)	Parallel Flange	600	800	550	750	400	600	270	430

8.3.6 Design of Structural Steel Web against Vertical Shear Force

8.3.6.1 The factored design shear force, V_d, in a composite beam due to external action, V, shall satisfy:

$$V \leq V_d$$

where,

V_d = design shear resistance calculated as given below.

$$V_d = V_n / \gamma_{m0}$$

γ_{m0} = partial safety factor against shear failure (Table 7)

V_n = nominal ultimate shear resistance

The vertical shear force is assumed to be resisted by the web of the steel section only unless the value for contribution from the reinforced concrete part of the beam has been established. The nominal shear resistance, V_n may be governed by plastic shear resistance or strength of the web or governed by shear buckling as given below:

a) *Plastic shear resistance*

The nominal plastic shear resistance of composite beams under pure shear shall be calculated as indicated in [6-6B(5)], disregarding the contribution of the concrete slab. The shear resistance for I-sections, channels both for major axis bending and minor axis bending as well as for rectangular and circular hollow sections of uniform thickness shall be as per [6-6B(5)].

b) *Shear Buckling Resistance*

Resistance to shear buckling shall be verified as specified in [6-6B(5)]. The nominal shear resistance, V_n, of webs with or without intermediate stiffeners as governed by buckling may be evaluated as detailed in [6-6B(5)] either using the simple post-critical method or tension field method. The contribution of encasing concrete to shear resistance may be considered and that of shear reinforcement can be considered provided they consist of closed-loop stirrups. The total shear may be shared between the structural steel section and encasing concrete in the ratio of their contribution to plastic moment capacity.

8.3.6.2 *Reduction in bending resistance under high shear force*

If V is less than $0.6V_d$, reduction in the plastic bending resistance of the section need not be considered. When $V > 0.6V_d$, the bending resistance is reduced as the contribution of the web to bending gets diminished. The reduced bending resistances of the following sections are given below:

a) *Plastic or compact section*

$$M_{\mathrm{dv}} = M_{\mathrm{d}} - \beta(M_{\mathrm{d}} - M_{\mathrm{fd}}) \leq (1.2\, z_{\mathrm{e}} f_{\mathrm{y}})/\gamma_{\mathrm{m0}}$$

where,

$\beta = (2V/V_d - 1)^2$

M_d = plastic design moment of the whole section disregarding high shear force effect considering web buckling effects

V = factored applied shear force.

V_d = design shear resistance as governed by web yielding or web buckling

M_{fd} = Plastic design resistance of the area of the cross section excluding the shear area, considering partial safety factor γ_{m0}

b) *Semi-compact Section*

$$M_{dv} = Z_e f_y / \gamma_{mo}$$

where

Z_e = elastic section modulus of the whole section

8.3.7 *Hybrid Sections*

Use of hybrid steel sections consisting of different grade steel elements is permitted. The plastic and compact section design strength is obtained by using the appropriate value of yield stress in the different elements of the hybrid section. The design strength of the semi-compact section is evaluated with necessary adjustment (reduction) in stresses of the flange element in the cross-section with higher yield stress by the reduction factor, R_h. The R_h may be determined using the procedure as presented in **B-3** of Annex B.

8.3.8 *Partially Encased Sections*

This section is applicable for partially encased composite plastic or compact beams, provided the web depth to thickness ratio, d/t_w, is less than or equal to 117ε.

The full shear connection between the structural steel section and concrete encasing the web shall be ensured. The steel reinforcement in the compression zone and concrete in the tension zone may be disregarded.

The partial shear connection may be used in the partially encased composite section, provided the requirements of **8.3.5.2** are satisfied.

9 DESIGN FOR SERVICEABILITY LIMIT STATES

9.1 General

9.1.1 Serviceability limit states are related to the criteria governing normal use. Serviceability limit state is the limit state beyond which the serviceability criteria specified below are no longer met:

a) Stress and deflection limit,
b) Vibration limit,
c) Durability consideration, and
d) Fire resistance

9.1.2 Linear elastic analysis is used for finding out design moments and stresses under various load combinations and load factors, as mentioned in Table 7, for serviceability limit states. Concrete is assumed as unreinforced and un-cracked for the analysis.

9.1.3 *Method of Construction*

The stress and strain at serviceability limit state depend on whether the steel beam is propped or un-propped during construction, as given below:

a) *Un-propped construction* – In un-propped construction, the steel beam has to carry the construction load, including shuttering, wet concrete and its own weight until the concrete hardens.

b) *Propped construction* – In propped construction, both the dead and live load are resisted by the composite section. When props are used, they should be kept in place until the *in-situ* concrete has attained a strength equal to approximately twice the stress to which the concrete may be subjected to upon removal of props.

c) *Non-composite Construction* - In non-composite construction, the concrete slab is designed according to [6-6B(4)] and the steel deck sheet act just as a permanent shuttering.

The difference in the method (a) and (b) of construction does not, however, affect the ultimate limit load, wherein the total load, including the transient loads, shall be resisted by the composite section.

9.2 Negative Moments

Negative moments over intermediate supports may be adjusted as mentioned in **8.1.2.1**

9.3 Stresses and Deflections

For calculating stresses at service load and deflection, the value of the modular ratio, *m* shall be taken as,

$m = \frac{E_s}{E_{cm}} \geq 7.5$ For short-term effect or loading.

$m = \frac{E_s}{K_c E_{cm}} \geq 15.0$ For permanent or long-term loads.

where

K_c = *Creep factor = 0.5*

E_s = Modulus of elasticity for steel = 2.0 x 10^5 N/mm ^{2}

E_{cm} = Modulus of elasticity of cast *in-situ* concrete ([6-6B(4)])

f_{ck} = characteristic cube compressive strength of concrete in N/mm^2

E_{ci} = Modulus of elasticity of cast *in-situ* concrete at *i* days (*i* < 28 days)

The equivalent area of concrete slab at any stage shall be determined by dividing the effective width of the concrete slab by the modular ratio as given below.

$$m = \frac{E_s}{E_{ci}}$$

Final stresses and deflection are to be worked out separately at each stage of load history with relevant modular ratios and section modulus as discussed above and then added together.

9.3.1 *Limiting Stresses for Serviceability*

Limiting stresses for different stages of construction are as indicated below:

a) Concrete – The limiting compressive stress in concrete should not exceed one-third of the characteristic strength of concrete.
b) Reinforcement Steel – The limiting tensile stress in steel reinforcement should not exceed f_{yk}/γ_k
c) Structural Steel –The limiting stress in steel beam considering the different stages of construction should not exceed f_y / γ_{mo}

Where bearing stress is combined with tensile or compressive stress, bending and shear stresses under the most unfavourable conditions of loading, the equivalent stress obtained from the following equation shall not exceed $0.9f_y$:

$$f_{ec} = \sqrt{f_{bc}^2 + f_p^2 + f_{bc}.f_p + 3\tau_b^2}$$

and

$$f_{et} = \sqrt{f_{bt}^2 + f_p^2 + f_{bt}.f_p + 3\tau_b^2}$$

where,

f_{ec} and f_{et} = equivalent compressive and tensile stress in steel section;
f_{bc} and f_{bt} = actual compressive and tensile stress in steel section;
f_p = actual bearing stress in steel section; and
τ_b = actual shear stress in steel section.

The value of bending stresses f_{bc} about each axis, to be used in the above formula, shall be individually lesser than the values of the maximum allowable stresses in bending about the corresponding axis.

9.3.2 *Limiting Deflection and Camber*

9.3.2.1 *Deflection limit*

The deflection under serviceability loads of a building or a building component should not impair the strength of the structure or components or cause damage to finishing. Deflections are to be checked for the most adverse but realistic combination of service loads and their arrangements by elastic analysis using a load factor given Table 7. The deflection of a member shall be calculated without considering the impact factor or dynamic effect of the loads on deflection. [6-6B(5)] gives recommended limits of deflections for certain structural members and systems. Circumstances may arise where greater or lesser values would be more appropriate depending upon the nature of the material in element to be supported (vulnerable to cracking or not) and intended use of the structure, as required by the client.

9.3.2.2 *Provision of camber*

Where the deflection due to the combination of dead load and live load is likely to be excessive, consideration should be given to pre-camber the steel beams. The values of desired camber shall be specified in the design drawing. Generally, for spans greater than 25 m, camber approximately equal to the deflection due to dead loads plus half the live load may be used.

9.4 Vibration

Suitable provisions in the design shall be made for the dynamic effects of live loads, impact loads and vibration due to machinery operating loads. In severe cases, the possibility of resonance, fatigue or unacceptable vibrations shall be investigated. Unusually flexible structures (generally the height to the effective width of lateral load resistance system exceeding 5:1) shall be investigated for lateral vibration under dynamic wind loads. Structures subjected to a large number of cycles of loading shall be designed against fatigue failure in accordance with [6-6B(5)]. The floor vibration effect shall be considered using specialist literature.

9.5 Durability

9.5.1 Several factors that affect the durability of the buildings under conditions relevant to their intended life are listed below:

a) The environment,
b) The degree of exposure,
c) The shape of the member and the structural detail,
d) The protective measure, and
e) Ease of maintenance.

The durability of the structural steel component of a composite section shall be ensured by recommendations as given in [6-6B(5)]. Specialist literature may be referred to for more detailed additional information in design for durability. For concrete, the durability shall be ensured by following the recommendations as given in [6-6B(4)].

9.5.2 *Profiled Steel Sheeting for Composite Slabs*

The durability criteria for profiled steel sheets of composite deck slab shall be met by the following procedures:

a) The exposed surfaces of the steel sheeting shall be adequately protected to resist particular atmospheric conditions.
b) A zinc coating of a total mass 275 g/m^2 (including both sides) is sufficient for internal floors in a non-aggressive environment, but the specification may be varied depending on service conditions

9.6 Fire Resistance

Fire resistance of a steel component of a composite member is a function of its mass, its geometry, and the actions to which it is subjected, its structural support conditions, fire protection measures adopted and the fire to which it is exposed. Design provisions to the resistance of fire for concrete shall be as per guidelines given in [6-6B(4)]. For the design of structural steel components for fire resistance, [6-6B(5)] shall be referred. Specialist literature may be referred to for more detailed information on the design for fire resistance of steel/composite structures. The aspect of fire resistance is given in **15.**

9.7 Control of Cracking in Concrete and Crack Width Calculation

Minimum reinforcements in terms of diameter and spacing required for crack control at the top of concrete as per [6-6B(4)] are to be provided in composite beams at the zone of the negative moment to prevent cracks adversely affecting the appearance and durability of the structure. Crack width calculation as well as limiting crack width as given in [6-6B(4)] may be followed, subject to the discretion of engineers. The crack width in concrete shall be restricted to values as indicated in [6-6B(4)].

10 DESIGN FOR FATIGUE LIMIT

This section applies to the design of structures and structural elements subject to loading which could lead to fatigue failure. The following effects are not considered in the section:

a) Corrosion fatigue,
b) Low cycle (high stress) fatigue,
c) Thermal fatigue,
d) Stress corrosion cracking,
e) Effects of high temperature (> 150^{o} C), and
f) Effects of low temperature (< below transition temperature).

The fatigue design of various components of composite structures like members, welded joints, bolts, shear lugs etc. shall be carried out as per the specifications laid down in [6-6B(5)]. Fatigue provisions in the design of shear connectors are discussed in **11.3**.

11 SHEAR CONNECTORS

The shear connectors shall fulfill the dual purpose of transferring shear force between concrete and structural steel as well as anchoring the two components relative to each other with minimum slip to ensure full or partial composite action as per the design requirement. The dimensional details of shear shall be as given in Fig. 8.

11.1 Longitudinal Shear in Beams and Slabs

Longitudinal shear load on shear connectors in a composite section, irrespective of boundary conditions of the members, is to be calculated for service and fatigue limit states on the basis of elastic theory. Appropriate sectional properties based on effective widths and modular ratios as per the load history and development of composite action shall be considered for the design of the section for resistance against longitudinal shear between steel and concrete.

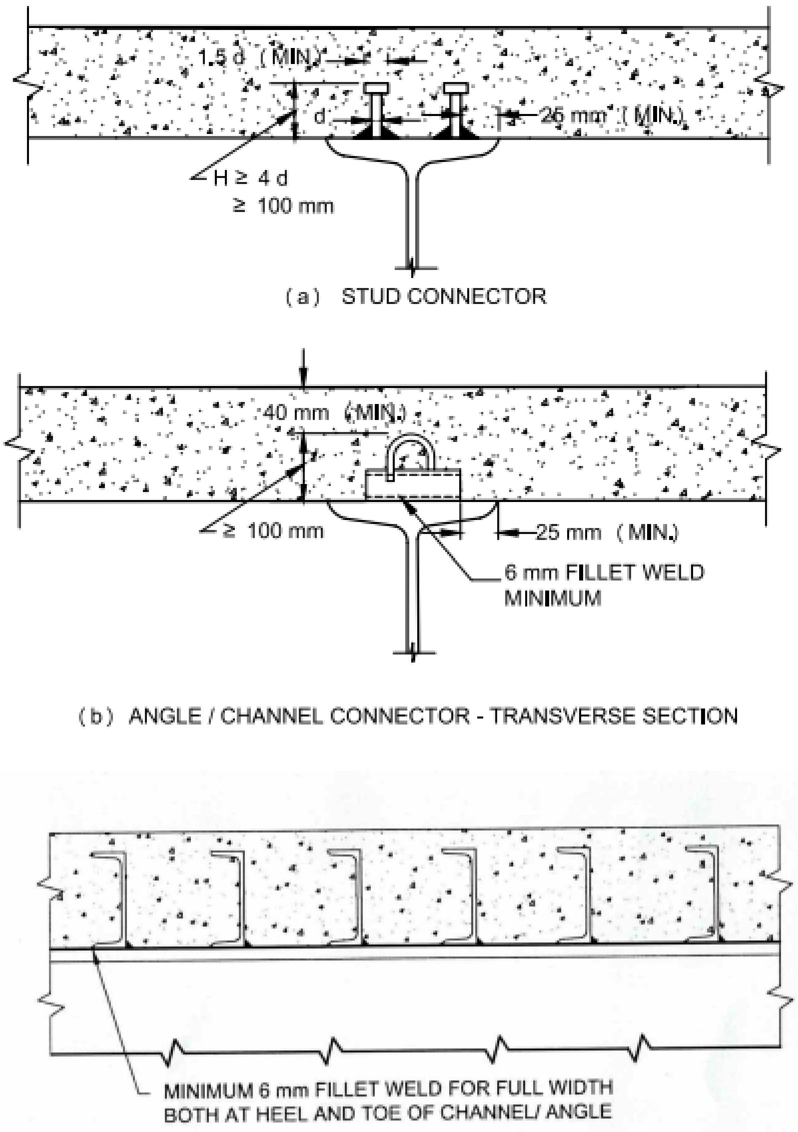


Fig. 8 DETAILS OF SHEAR CONNECTORS

NOTES

1 The diameter of the stud connector welded to the flange plate shall not exceed two and half times the flange plate thickness.

2 The height of the stud connectors shall not be less than four times their diameter nor 100 mm.

3 The diameter of the head of the stud shall not be less than one and a half times the diameter of the stud and the depth shall not be less than 0.4d.

4 The size of the fillet weld joining other types of connectors to the flange plate shall not exceed half the thickness of the flange plate.

5 Channel and angle connectors shall have at least 6 mm fillet welds placed along the heel and toe of the channels/angles. The clear distance between the edge of the flange and the edge of the shear connectors shall not be less than 25 mm.

6 The overall height of a connector, including any hoop, which is an integral part of the connector, shall be at least 100 mm with a clear cover of 25 mm.

11.2 Design Strength of Shear Connectors

Shear connectors shall be checked for adequacy against failure in both ultimate limit states and fatigue limit states. The strength of shear connectors against failure under ultimate limit states and fatigue limit states shall be considered as per **11.2.1**, **11.2.2** and **11.3**, respectively.

11.2.1 *Design Strength of Shear Connectors*

Design static strengths of flexible shear connectors, mainly stud connectors and channel connectors can be determined by the following equations:

a) *Stud Connectors*

The design resistance, Q_d of stud shears connectors shall be as given below:

$$Q_d = \frac{0.8 f_{us}.\pi.d^2/4}{\gamma_{mv}} \leq \frac{0.26\,\alpha\,d^2\sqrt{f_{ck}.E_{cm}}}{\gamma_{mv}}$$

Where,

$\alpha = 0.2\left\{\frac{h_s}{d}+1\right\}$ for $3 \leq \frac{h_s}{d} \leq 4$ and $\alpha = 1.0$ for $\frac{h_s}{d} > 4$

Q_d = design strength of stud in, Newton (N)

γ_{mv} = partial safety factor for stud connector = 1.25

d = diameter of the shank of the stud, in mm (16 mm ≤ d ≤ 25 mm)

f_{us} = ultimate tensile strength of the stud material ≤ 500 N/mm^{2};

f_{ck} = Characteristic compressive strength of concrete of density not less than 1 750 kg/m^3

h_s = nominal height of stud in mm; and

E_{cm} = Secant modulus of elasticity of concrete (*see also* Annex F)

b) *Channel Connectors*

Assuming that the web of the channel is placed vertical and the shear applied is nominally perpendicular to the web, the design resistance of a channel connector shall be determined as given below:

$$Q_n = 45(t_f + 0.5t_w)L\sqrt{f_{ck}}$$

$$Q_d = Q_n/\gamma_{mv}$$

where

Q_d = Design strength of channel in Newton (N)

L = Length of the channel in mm

t_f = Thickness of flange in mm

t_w = Thickness of web in mm

f_{ck} = Characteristic compressive strength of concrete in MPa

γ_{mv} = partial safety factor for channel shear connector = 1.25

While using the channel shear connectors, the following recommendations need to be followed:

1) The height *h* of the channel should not exceed 20 times the channel web thickness or 150 mm, whichever is less.

2) The width *b* of the channel should not exceed 300 mm.
3) The underside of the top flange of the channel should not be less than 30 mm clear above the bottom reinforcement.
4) The size of the fillet weld connecting the channel to the flange plate should not exceed half the flange plate thickness.

11.2.2 The design strengths of some standard shear connectors are given in Table 9. The following points have to be considered while using Table 9:

a) f_{ck} is the specified characteristic cube strength at 28 days.

b) Connector strengths for concrete of intermediate grade may be obtained by linear interpolation.

c) For channels of lengths different from those quoted above, the capacities are proportional to the lengths greater than 150 mm.

d) For rolled angle and tee shear connectors, the values given for channel connectors are applicable, provided the height is at least equal to that of the channel.

e) For stud connectors of overall height greater than 100 mm, the design static strength should be taken as the values given in Table 9 for 100 mm high connectors.

f) The above provisions of stud connectors are not applicable to composite slab using a profiled deck. The strength of the shear connector in such cases is given in **11.2.3**.

g) The number of shear connectors given by the above Table 9 shall be distributed in the zone between the maximum and the zero moment sections. The number of connectors has to meet both ultimate strength consideration as well as fatigue consideration.

h) In order to avoid undesirable slip, the maximum interface shear per unit length due to superimposed dead load and live load under service conditions at any point in the beam should satisfy **11.3.1.**

Table 9 Ultimate Static Design Strengths of Shear Connectors (Q_n for Different Concrete Strengths)

(*Clause* 11.2.2)

Sl No.	**Type of Connector**	**Material**	**Size**		**Ultimate static strength in kN per connector**			
			Nominal diameter (mm)	Nominal height (mm)	For concrete strengths, f_{ck} (MPa)			
					25	30	40	50
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)

i)	Stud Connectors	Material with a characteristic yield strength of 385 MPa, minimum elongation of 18% and a characteristic tensile strength of 495 MPa	25	100	25	30	40	50
			22	100	79	91	113	133
			20	100	65	75	93	110
			20	75	62	71	88	105
			16	75	42	48	59	70
ii)	Channels 150mm long (min)	As per [6-6B(6)] (E250 A/BR)	ISMC 125		244	259	285	307
			ISMC 100		206	219	241	260
			ISMC 75		166	176	194	209

11.2.3 *Design Resistance of Studs Used with Profiled Steel Sheeting*

11.2.3.1 Sheeting with ribs parallel to the supporting beams

The studs are located within a region of concrete that has the shape of a haunch (Fig.9). When the sheeting is continuous across the beam, the width of the haunch b_0 is equal to the width of the trough, as given in Fig. 10. Where the sheeting is not continuous, b_0 is as given in Fig. 9. The depth of the haunch should be taken as d_p, the overall depth of the sheeting excluding embossments.

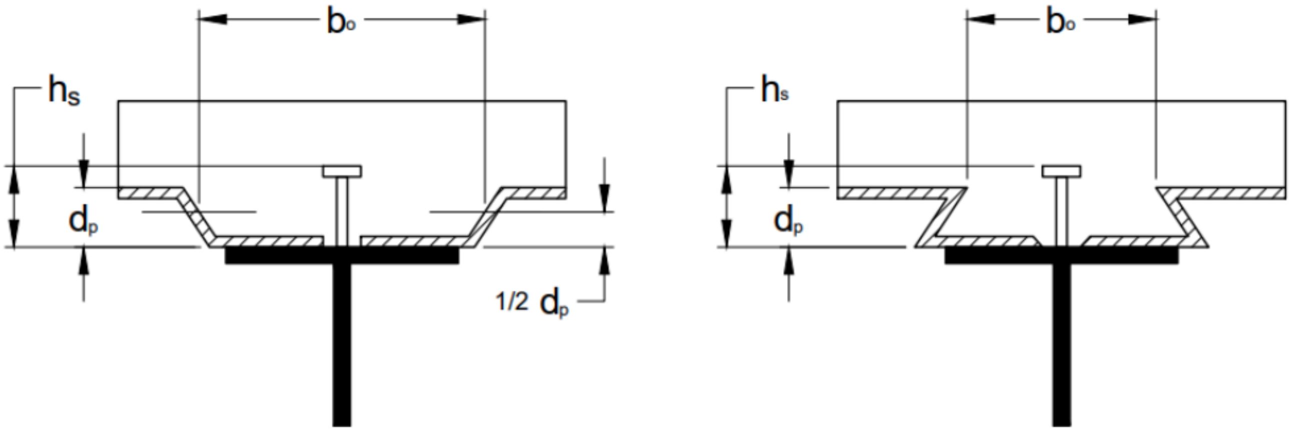


FIG. 9 BEAM WITH PROFILED DECKING SHEET RIBS PARALLEL TO THE BEAM

The design resistance of the shear connector should be taken as the resistance in a solid slab, multiplied by the reduction factor k_p given by the following expression:

$$k_{\mathrm{p}} = 0.6\frac{b_0}{d_{\mathrm{p}}}\left(\frac{h_{\mathrm{s}}}{d_{\mathrm{p}}} - 1\right) \leq 1.0$$

Where h_s is the overall height of the stud, but not greater than d_p+75 mm.

Where the sheeting is not continuous across the beam, as shown in Fig. 9, the side of the haunch and its reinforcement shoud be taken as the depth of the profile d_p excluding the embossments.

11.2.3.2 Sheeting with ribs transverse to the supporting beams

Provided that the conditions given in **11.2.2** are satisfied, the design shear resistance should be taken as the resistance in a solid slab, calculated as given by **11.2.1** (except that f_{us} should not be taken as greater than 450 N/mm^2) multiplied by the reduction factor k_t given by:

$$k_{\mathrm{t}} = \frac{0.7}{\sqrt{n_{\mathrm{r}}}} \frac{b_0}{d_{\mathrm{p}}} \left(\frac{h_{\mathrm{s}}}{d_{\mathrm{p}}} - 1 \right) \leq 1.0$$

where

n_r is the number of stud connectors in one rib at the beam intersection, not to exceed two in calculation of the reduction factor k_t and of the longitudinal shear resistance of the connection.

The factor k_t should not be taken greater than the appropriate value k_{tmax} given in Table 10. The values for k_t are applicable provided that: the studs are placed in ribs with a height d_p not greater than 85 mm and a width b_0 not less than d_p and for through deck welding, the diameter of the studs is not greater than 20mm, or for holes provided in the sheeting, the diameter of the studs is not greater than 22mm.

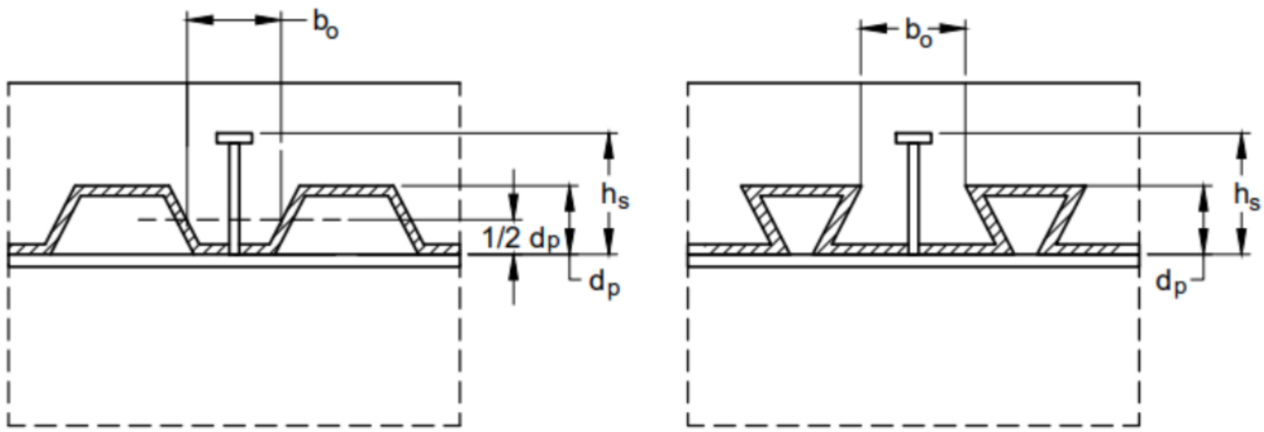


FIG.10 BEAM WITH PROFILED DECKING SHEET RIBS TRANSVERSE TO THE BEAM

Table 10 Upper Limits $K_{t,\ max}$ for the Reduction Factor K_T
(*Clause* 11.3.3.2)

SL No.	Number of stud connectors per rib	Thickness t of sheet (mm)	Studs 20 mm in diameter and welded through profiled steel sheeting	Profiled sheeting with holes and studs 19 mm or 22 mm in diameter
(1)	(2)	(3)	(4)	(5)
i)	n_r=1	≤1.0	0.85	0.75
		>1.0	1.0	0.75

ii)	n_r =2	≤1.0	0.70	0.60
		>1.0	0.8	0.60

11.3 Fatigue Strength of Shear Connectors

The fatigue shear stress range (fatigue Strength) of the shear connector shall be obtained from [6-6B(5)], corresponding to the design load life cycle, N_{SC}.

The strength shall be determined as given below:

$$\tau_f = \tau_{fn} \sqrt[5]{5 \times 10^6 / N_{SC}}$$

τ_{fn} = design normal and shear fatigue stress range respectively of the detail for 5 x10^6 cycles as given in [6-6B(5)].

τ_{fn} = 67 N/mm^2 for stud connector as per [6-6B(5)].

τ_{fn} = 59 N/mm^2 for channel connector as per [6-6B(5)] [provided that the thickness of the top flange of steel beam is greater than or equal to 12 mm and the edge distance from the end of weld to the edge of the top flange is 10 mm].

The nominal fatigue strengths of some standard shear connectors are presented in Table 11.

Table 11 Design Fatigue Strengths Q_r (in kN)
(*Clause* 11.4)

Sl No.	**Type of Connectors (Headed Stud – Diameter)**	**Connector Material**	**N = Nos. of cycles in millions**				
			0.1	**0.5**	**2**	**10**	**100**
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
i)	Headed studs φ 25	f_{ys} = 385 MPa f_{us} = 495 MPa Elongation = 18 %	72	52	40	29	18
ii)	Headed studs φ 22		56	40	31	22	14
iii)	Headed studs φ 20		46	33	25	18	12
iv)	Headed studs φ 16		29	21	16	12	7
v)	Channel 150 long for a nominal weld of 8 mm	[6-6B(6)]	108	79	60	43	27

NOTE – For intermediate stress cycles the values may be interpolated from log scales (that is, the above equation). Other connectors, if used, should have their capacities established through tests.

11.3.1 *Spacing of Shear Connectors*

The design shear action per unit length of the steel concrete interface, V_L, is given by:

$$V_{\mathrm{L}} = \sum \left[\frac{V \cdot A_{\mathrm{ec}} \cdot y}{I}\right]_{\mathrm{DL,LL}}$$

where

V_{L} = Longitudinal shear per unit length.

V = The factored vertical shear forces due to dead load and live load (includingimpact if any) separately at each state of load history.

A_{ec} = The transformed compressive area of concrete above the plastic neutral axis of the composite section with the appropriate modular ratio depending on the nature of load (short term that is, live load, or long term, that is, dead load).

y = C.G. distance of transformed concrete area from the plastic neutral axis.

I = Moment of Inertia of the composite section using an appropriate modular ratio.

DL, LL = Different load history conditions, that is, sustained load or composite action dead load, transient load, or composite action live load. These loads are to be considered with appropriate load factors at this stage.

Maximum spacing of shear connectors, S_{L1} is given as

$$S_{\mathrm{L1}} = \frac{\sum Q_{\mathrm{n}}}{V_{\mathrm{L}}}$$

Q_{n} is the ultimate static strength of one shear connector (Table 9) and the summation is over the number of shear studs in one section.

The maximum longitudinal force at the interface due to bending moment shall also be calculated over the shear span, *L,* equal to the distance from zero moment to maximum moment section and is given by the following equations:

$$H_1 = A_{\mathrm{sl}} f_{\mathrm{y}} \times 10^{-3} / \gamma_{\mathrm{m0}}$$
$$H_2 = 0.36 f_{\mathrm{ck}} A_{\mathrm{ec}} \times 10^{-3}$$

Where,

H_1, H_2 = Longitudinal interface shear force due to bending (kN);

A_{sl} = Area of tensile steel (mm^2) in the longitudinal direction;

A_{ec} = Effective area of concrete:

$b_{\mathrm{eff}}.x_{\mathrm{u}}$ (for neutral axis within the slab); and

$b_{\mathrm{eff}}.\ d_{\mathrm{s}}$ (for the neutral axis in the steel section)

Sufficient connectors should be provided to resist the longitudinal force *H*, the maximum compressive force action in the composite beam slab interface, which is the smaller of H_1 and H_2.

The maximum spacing of shear connectors, S_{L2}, is given as:

$$S_{\mathrm{L2}} = \frac{\sum Q_{\mathrm{n}}}{H} . L$$

11.3.2 *Fatigue Strength of Shear Connectors*

$$V_{\mathrm{r}} = \Sigma\left[\frac{V_{\mathrm{R}}.A_{\mathrm{ec}}.Y}{I}\right]_{\mathrm{DL}}$$

Where

V_R = The shear range in which is the difference between the maximum and minimum vertical shear envelop due to live load and impact

H = is live Live load with impact.

Spacing of shear connectors from fatigue consideration is given as:

$$S_{\mathrm{R}} = \frac{\sum Q_{\mathrm{r}}}{V_{\mathrm{r}}}$$

Q_r is the design fatigue strength of one shear connector, which is to be taken from [6-6B(5)] Table 10.

For full shear connection, the lowest spacing of S_{L1}, S_{L2}, and S_R is to be provided as the actual spacing of the shear connectors.

11.4 Partial Shear Connection

11.4.1 Partial shear connection may be used either for attaining economy without losing much in the moment capacity of the composite section or where the number of shear connectors required for full shear cannot be provided without compromising minimum spacing provisions.

Partial shear connections may be used in plastic and compact sections. The number of connectors n_p shall then be determined by a partial shear connection theory, taking into account the deformation capacity of the shear connector.

S_c degree of shear connection $= \frac{n_{\mathrm{p}}}{n_{\mathrm{f}}}$

n_p = Number of shear connectors provided for partial shear connection

n_f = Number of shear connectors required for full shear connection

M_R = Required reduced bending resistance of the section

M_d = The fully plastic moment of resistance of the composite section

M_{ps} = Plastic moment of resistance of steel section alone

$$n_{\mathrm{p}} = \frac{M_{\mathrm{R}} - M_{\mathrm{ps}}}{M_{\mathrm{d}} - M_{\mathrm{ps}}} n_{\mathrm{f}}$$

11.4.2 *Limitation on the Use of Partial Shear Connection in Beams for Buildings*

Headed studs with an overall length after welding not less than 4 times its diameter and with shank diameter not less than 16 mm and neither greater than 25 mm may be considered as ductile with the following limits for the degree of shear connection, S_c:

a) For steel sections with equal flanges:

$L_e \leq 25$: $S_c \geq 1 - 1.42\varepsilon^2 (0.75 - 0.03L_e)$; $S_c \geq 0.4$

$L_e > 25$: $S_c = 1.0$

b) For steel sections having a bottom flange with an area of three times the area of top flange:

$L_e \leq 20$: $S_c \geq 1 - 1.42\varepsilon^2 (0.30 - 0.015L_e)$; $S_c \geq 0.4$

$L_e > 20$: $S_c = 1.0$

Where, $\varepsilon = \sqrt{250/f_y}$, L_e is the distance between points of zero bending moment in the sagging bending range in metres. For typical continuous beams, L_e may be assumed as shown in Fig. 6 and Fig. 7.

c) For steel sections having a bottom flange with an area exceeding the area of the top flange but less than three times that area, the limit for S_c may be determined from expressions in (a) and (b) above by linear interpolation.

11.5 *Precautions Against Separation of Steel Beam from Concrete*

a) Where, a concrete haunch is used, between the steel flange and the soffit of the slab, top of stud and top flange of channel shear connectors shall extend up to at least 40 mm above the transverse reinforcements in the haunches, provided the reinforcements are sufficient to transfer longitudinal shear.

b) Where shear connectors are placed adjacent to the longitudinal edge of the slab, transverse reinforcement provided in accordance with **11.7** shall be fully anchored in the concrete between the edge of the slab and the adjacent row of connectors.

11.6 Detail of Haunches in Reinforced Concrete

11.6.1 Figure 11 indicates the dimension of haunches as applicable for slabs with haunches resting on steel beam. The edge of haunches shall be located outside a line drawn at 45 degrees from the outside edge of the base of the connector, as shown in Fig. 11.

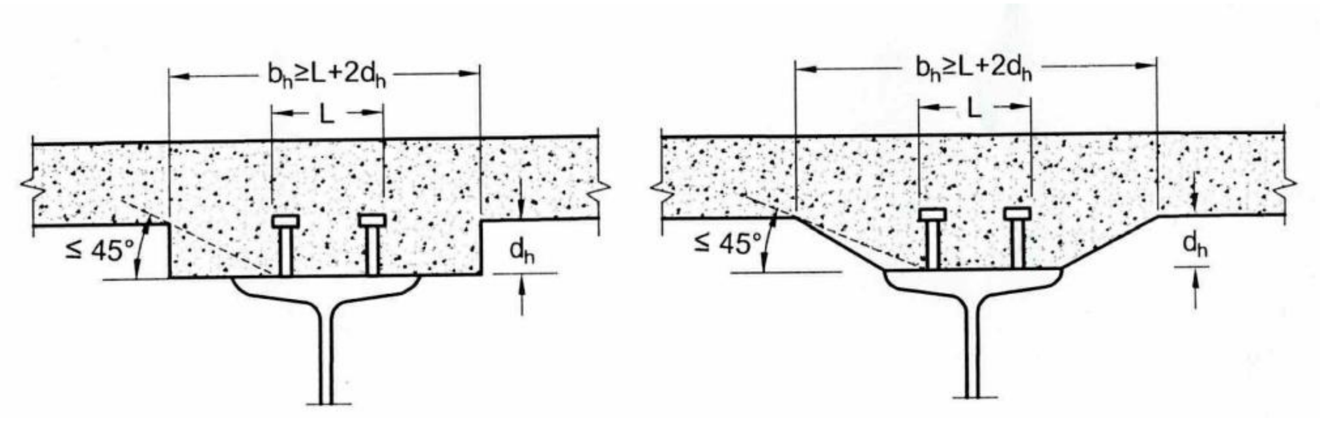


FIG. 11 DIMENSION OF HAUNCHES

11.6.2 *Clear Cover to Shear Connectors*

The clear depth of concrete cover over the top of the shear connectors shall not be less than 25 mm. The horizontal clear concrete cover to any shear connector shall not be less than 50 mm as shown in Fig.12.

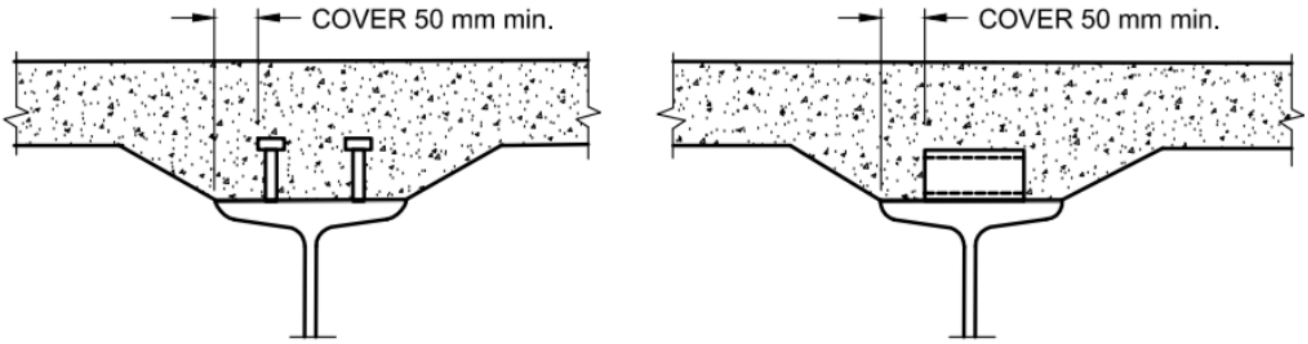


FIG. 12 CLEAR COVER TO SHEAR CONNECTORS

11.6.3 *Spacing of Shear Connectors*

Where the compression flange of steel beam, in spite of being semi-compact is assumed to be compact or plastic based on the restraint provided by shear connectors, the centre-to-centre spacing of the shear connectors in the direction of the compression should satisfy the following:

(a) Where the slab is in contact over the full length (for example, solid slab)

$S_{\mathrm{L}} < 21 \cdot t_{\mathrm{f}} \,.\, \varepsilon$

(b) Where the slab is not in contact over the full length (for example, slab with ribs transverse to the beam):

$S_{\mathrm{L}} < 14 \cdot t_{\mathrm{f}} \,.\, \varepsilon$

where

$\varepsilon = \sqrt{250/f_{\mathrm{y}}}$

t_f = thickness of the flange;

f_y = nominal yield strength of the flange in N/mm^2; and

S_L = maximum spacing of the shear connector.

In addition, the clear distance from the edge of the compression flange to the nearest line of shear connectors should not be greater than $9t_f\ \varepsilon$ or 50 mm, whichever is less.

In all cases, a shear connector shall be provided throughout the length of the beam and may be uniformly spaced between critical cross sections. The maximum longitudinal spacing of the shear connectors shall be limited to the lesser of 600 mm or 3 times the total slab thickness or four times the height of the connector (including any hoop which is an integral part of the connector), whichever is least.

Minimum spacing should be such as to allow proper concrete flow and compaction around the connectors. In stud connectors, the minimum spacing should not be less than 75 mm.

11.7 Transverse Shear Check (Requirement of Bottom Steel in Concrete Slab)

Planes, which are critical for longitudinal shear failure in the process of transfer of longitudinal shear from the beam to the slab are of four main types as shown in Fig. 13. The shear force Q in kN/m of beam is given by:

$$Q = \frac{N_{\mathrm{r}}\, Q_{\mathrm{u}}}{S_{\mathrm{L}}}$$

where

N_r = Number of shear connection at a section

Q_u = Design strength one shear connector

S_L = Longitudinal spacing of connector in meter

The shear force transferred per metre length from steel beam to concrete slab above, V_L shall satisfy both the following conditions:

a) $V_{\mathrm{L}} \leq 0.623L\sqrt{f_{\mathrm{ck}}}$, or

b) $V_{\mathrm{L}} \leq 0.232L\sqrt{f_{\mathrm{ck}}} + 0.1. A_{\mathrm{st}}. f_{\mathrm{yk}}. n$

Where,

V_L = Longitudinal shear force per unit length calculated for the ultimate limit state;
f_{ck} = Characteristic strength of concrete, in MPa;
f_{yk} = Yield stress of transverse reinforcement, in MPa;
L = Length (in mm) of possible shear planes envelop as indicated in Fig.14;
n = Number of times each lower transverse reinforcing bar is intersected by a shear surface (that is, the number of rows of shear the connector at one section of the beam). Generally, for T-beam n = 2 and for L-beam n = 1; and
A_{st} = Sectional areas (in cm^2) of transverse reinforcements per meter run of the beam.

The amount of transverse steel (cm^2/m) in the bottom of the slab shall not be less than $\frac{2.5Q}{f_{\mathrm{yk}}}$.

11.8 General Arrangements of Transverse Reinforcements

11.8.1 If the concrete by itself is insufficient to take the longitudinal shear, sufficient transverse reinforcements shall be provided to transfer longitudinal shear force from the beam to the effective width of the slab. The area of transverse reinforcement per unit length of the beam will be the sum total of all the reinforcement [A_t, A_b or A_h, as shown in Fig. 14, (a), Fig. 14(b) and Fig. 14(c)], which are intersected by the shear plane and are fully anchored on both the sides of the shear plane considered.

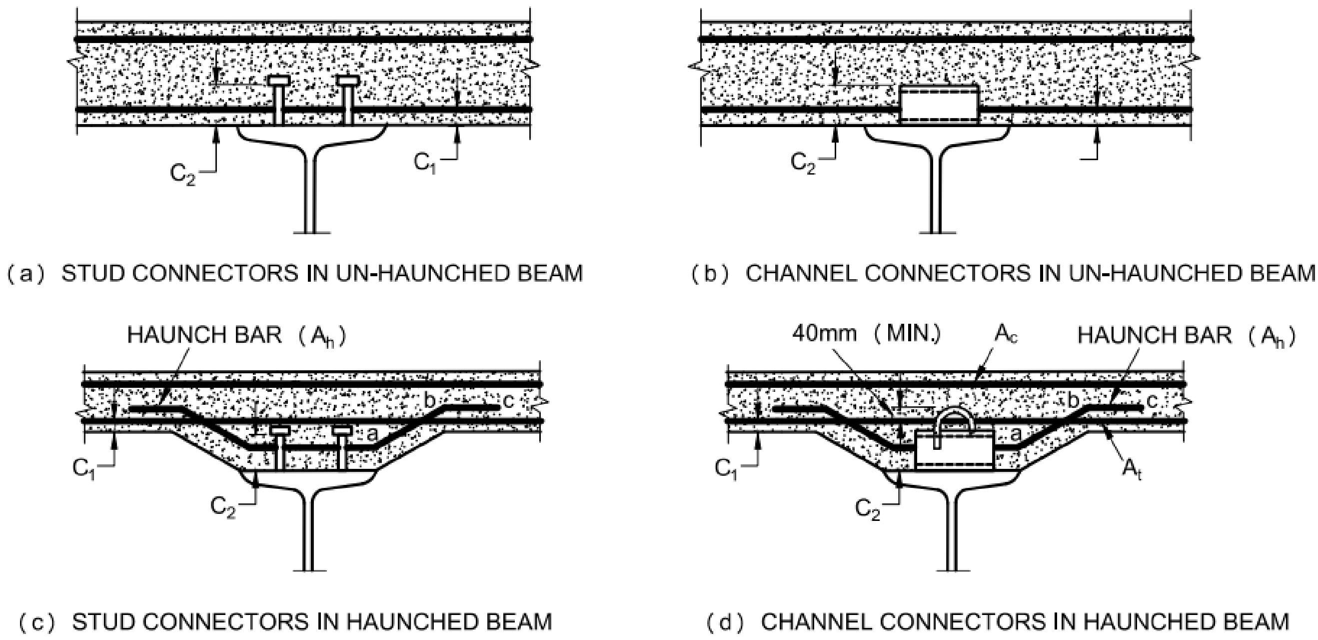
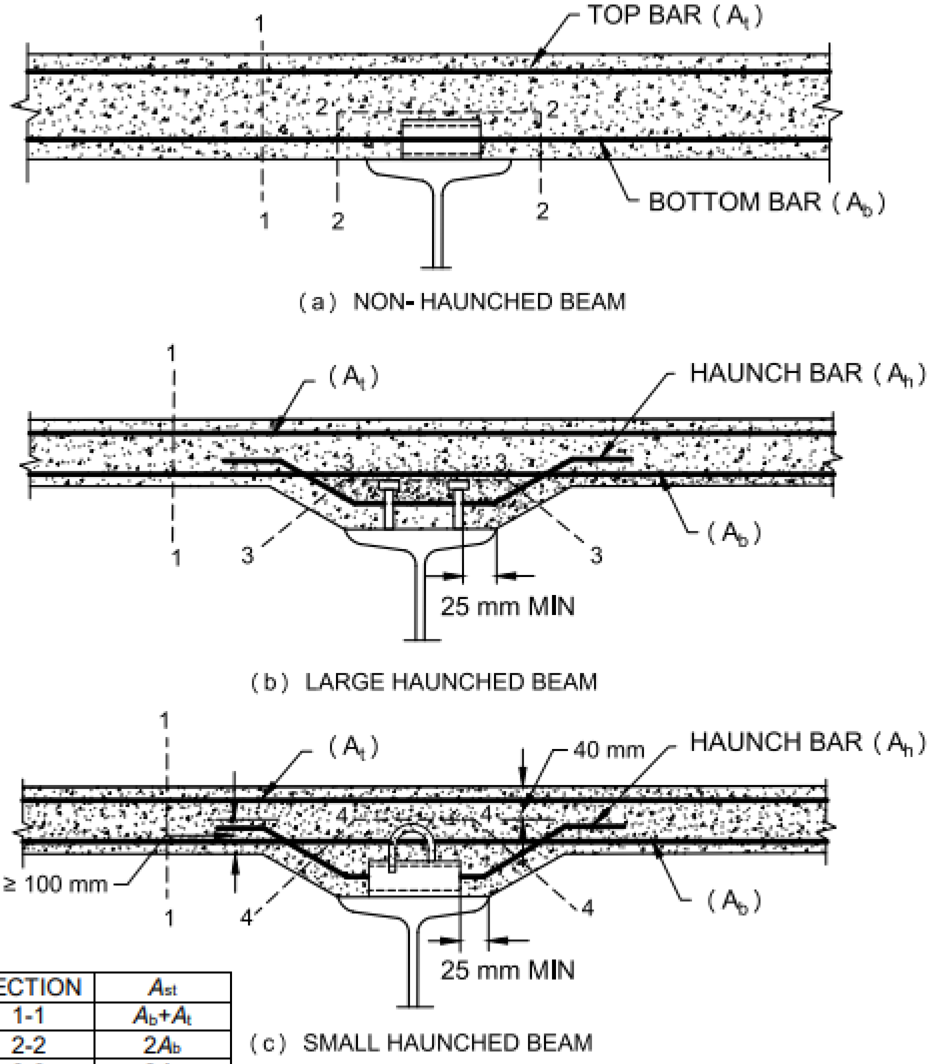


FIG. 13 ARRANGEMENT OF TRANSVERSE REINFORCEMENTS

11.8.2 Total Transverse Reinforcements

The total transverse reinforcements A_{st} per unit length of the beam in case of shear plane 1 – 1, which crosses the whole thickness of the slab, will be the sum of (A_t + A_b) [Fig. 14(a)]. Area of reinforcements A_t and A_b include those provided for flexure. The total transverse reinforcements across plane 2-2 [Fig. 14(a)] is $A_{st} = 2\ A_b$ and that across plane 3-3 [Fig. 14(b)] is $A_{st} = 2A_h$ as these planes do not cross the full thickness of the slab. In the case of shear plane 4 – 4 [Fig. 14(c)], the total transverse reinforcement is $A_{st} = 2(A_h + A_b)$. The transverse reinforcements shall be placed at locations as shown in Fig. 14. The haunch bars shall be extended beyond the junction of bottom bars by a length equal to the anchorage length.



SECTION	A_{st}
1-1	A_b+A_t
2-2	$2A_b$
3-3	$2A_h$
4-4	$2(A_b+A_h)$

FIG. 14 TRANSVERSE REINFORCEMENTS ALONG SHEAR PLANES

12 COMPOSITE SLABS WITH PROFILED STEEL SHEETING

12.1 General

The provisions in this section deal with composite floor slabs spanning only in the direction of ribs of the decking. The provisions are applicable in buildings, wherein the slabs are subjected to predominantly static imposed loads. The application of the provision is limited to narrowly spaced web, which is defined by the ratio of the width of the sheet rib to the rib spacing $b_r / b_p \leq 0.6$ (*see* Fig. 15).

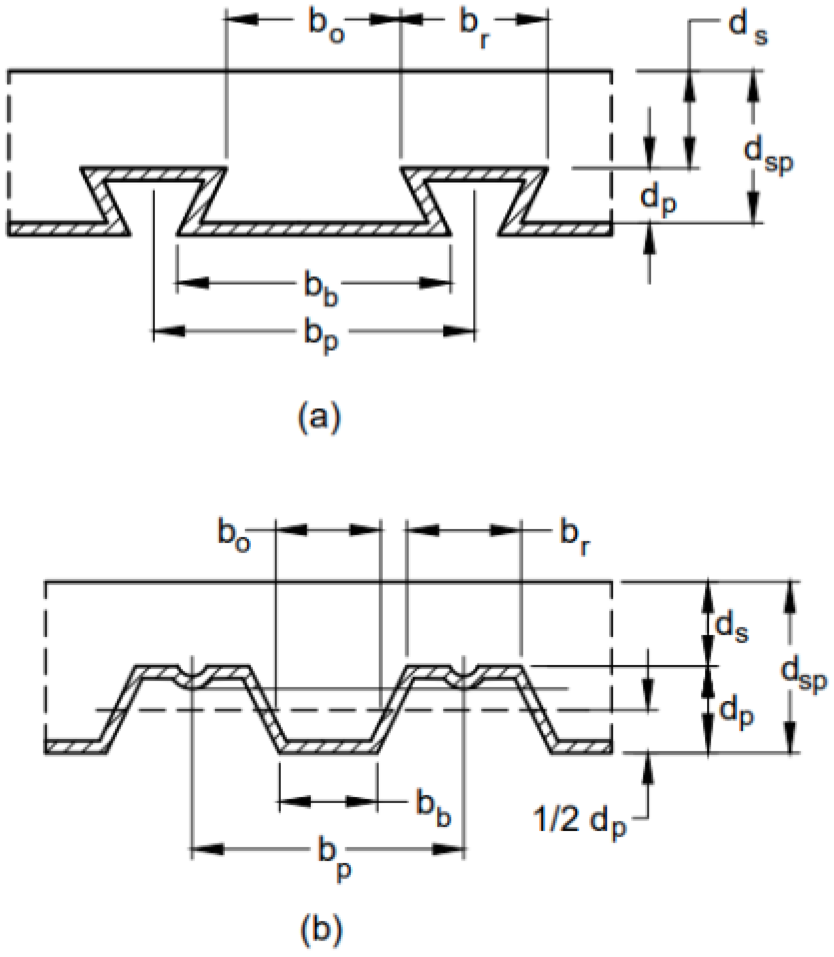


FIG. 15 PROFILE DIMENSION FOR COMPOSITE SLABS.

12.1.1 *Prequalification of Slab dimension*

The prequalification of composite and non-composite slabs are presented below.

a) Where the slab acts composite with a steel beam or used as a diaphragm,
   1) the overall depth of the slab should not be less than 90 mm; and
   2) the thickness of concrete above the main top flat surface of the sheeting ribs should not be less than 50mm.

b) Where the slab does not act composite with a steel beam or has no other stabilizing function;
   1) the overall depth of the slab should not be less than 80 mm; and
   2) the thickness of concrete above the main flat top surface of the sheeting ribs should not be less than 40mm.

12.1.2 *Bearing Length*

The recommended minimum bearing lengths of steel decking on the support (L_{bs}) and the composite slab, including the cast in place concrete (L_{bs}) shall be as given in (*see* Fig. 16):

a) For composite slab bearing on steel or concrete: L_{bc}=75 mm, L_{bs}=50 mm

b) For composite slab bearing on other material: L_{bc}=100 mm, L_{bs}= 70mm

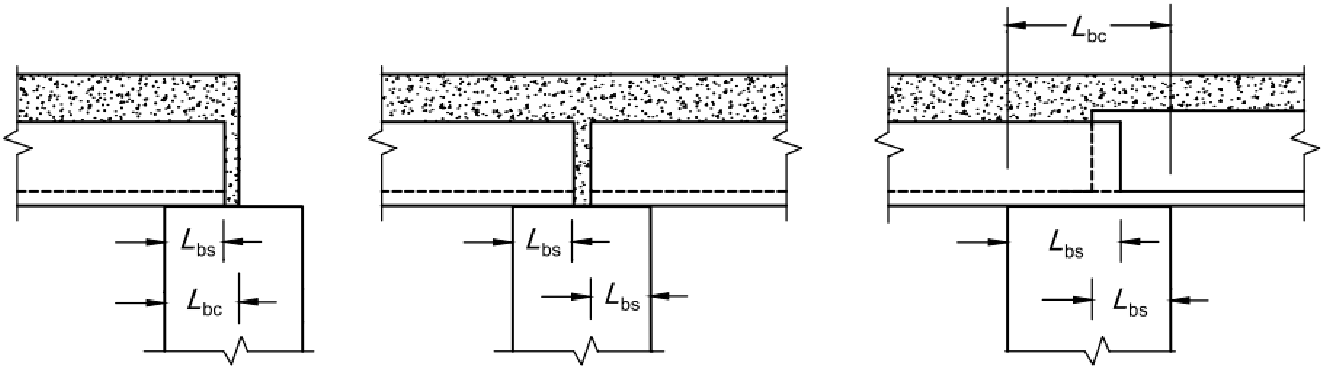


FIG. 16 MINIMUM BEARING LENGTHS

12.2.3 *Reinforcement for shrinkage and temperature stresses*

The effect of temperature stress in composite slabs in buildings is normally neglected unless required in special conditions. The effect of shrinkage is to be considered. The total shrinkage strain for design may be taken as 0.003 in the absence of test data.

12.2.4 *Minimum reinforcement*

The minimum reinforcement in either direction should be according to [6-6B(4)]. The spacing of the reinforcement bar should be in accordance with [6-6B(4)].

12.2.5 *Size of Aggregates*

In addition to the above, the largest nominal aggregate size should be according to [6-6B(4)].

12.3 Analysis for Internal Forces and Moments

12.3.1 *Analysis of Profile Steel Sheeting as Shuttering*

a) The elastic analysis shall be used where sheeting is considered. The design based on the elastic distribution of bending moment is conservative, as it does not take into account of redistribution of moments that can occur between support and mid span sections.

b) In the analysis of propped profile steel sheeting, the magnitude of bending moments and shear forces could be conservatively calculated by using the coefficients of bending moment and shear force as per [6-6B(4)].

c) More accurate analysis and design of profile steel sheeting as a shuttering should be in accordance with [6-6B(23)].

d) If the deflection at the center of the shuttering due to its own weight and the wet weight of concrete is less than (1/10) of the depth of the composite slab, the effect of 'ponding' due to shuttering deflection can be disregarded. Otherwise, that effect of additional load shall be evaluated by taking the additional uniform depth of concrete over the span as 0.7 times the central deflection.

e) The effect of ponding, especially in longer span, can be reduced by propping the shuttering during concreting until the concrete hardens. Plastic redistribution of moments in the propped shuttering is not allowed in evaluating stresses due to self-weight and wet concrete weight.

12.3.2 *Analysis of Composite Slab*

a) The application of linear methods of analysis is suitable for the serviceability limit states as well as for the ultimate limit states. Plastic methods shall be used only in the ultimate limit state.

b) The following method of analysis may be used for ultimate limit states

   1) Linear analysis without redistribution (serviceability limit state), and with redistribution (Ultimate limit state). If the effects of cracking of concrete are neglected in the analysis for ultimate limit states, the hogging bending moments at internal supports may be optionally reduced by up to 30 percent and corresponding increases are made to the sagging bending moments in the adjacent spans.
   2) Rigid-plastic global analysis may be used provided it is shown that sections where plastic rotations are required have sufficient rotation capacity. Plastic analysis without any direct check on rotation capacity may be used for the ultimate limit state if reinforcing steel satisfying the ductility requirement in **6.2.3** is used and the span length is not greater than 3.0 m. (Ultimate limit state).
   3) Elastic-plastic analysis accounting for the non-linear material behavior. (Ultimate limit states)

c) A continuous slab may be designed as a series of simply supported spans. Nominal reinforcement should be provided in accordance with [6-6B(4)], over the intermediate supports to manage crack size.

d) Concentrated loads may be assumed to be distributed over an effective width of the slab, assuming the load is distributed at an angle of 45° to the horizontal up to the top surface of the decking.

12.4 Design of Composite Slabs

Proprietary data backed by analysis and tests may be used in the design. Otherwise, the procedure given below may be used to evaluate the strength of composite slabs.

12.4.1 *Effective Span*

a) When a continuous composite slab is designed as a series of simply supported spans, for simplicity, the effective span can be taken as the lesser of the following:
   1) Distance between centres of the supports, and
   2) The clear span plus the effective depth of the slab.

b) Where the composite slab is designed as continuous, it is permitted to use an equivalent isostatic span for the determination of the resistance. The span length should be taken as:
   1) 0.8 L for internal span (effective span L) and

2) 0.9 *L* for external span (effective span *L*).

12.4.2 *Design of Profile Steel Sheeting as Shuttering During Construction*

The stress analysis of the profiled steel sheeting should be evaluated using the design equations or tests. For the ultimate limit state, the resistance of the sheet to sagging and hogging bending, together with the effects of combined bending and web crushing, are normally critical.

a) For the serviceability limit state, the limiting value of deflection $\delta_{\max}$ of steel sheeting under its own weight plus the weight of wet concrete may be considered as *L*/180 (where *L* is the effective span between supports) as per [6-6B(5)].

b) Profiled decking acting alone resists construction, dead, and live load until the concrete hardens and gains strength. 'Ponding' effect, especially in long spans due to excessive deflection of decking, has to be accounted while evaluating design forces and design strength unless sufficient propping is provided during the construction stage until the concrete hardens.

c) The longer spans will require propping to eliminate substantial deflection. Otherwise, a significant increase in concrete weight due to ponding has to be accounted

12.4.3 *Design of Composite Slab for The Ultimate States*

The bending moment and shear force distribution in continuous beams for secondary, as well as primary moment resistant frames/beams, have to be determined by structural analysis. The design values of load effects shall not exceed the design values of resistance for the relevant ultimate limit states.

12.4.3.1 *Design flexural resistance*

The design flexural resistance calculation shall take the following provisions into consideration:

a) The design Bending resistance, M_d of any cross section should be in accordance with Annex C, where the yield stress of the steel sheeting should be taken as f_{yp}.
b) The effect of local buckling of compressed parts of the sheeting should be considered by using effective widths not exceeding twice the limiting values of class-1 steel webs as discussed in [6-6B(5)].
c) For the effective area A_p of the steel sheeting, the width of embossments and indentations in the sheet should be neglected unless tests infer that a larger area is effective.
d) The sagging bending resistance of a cross-section should be calculated from the equilibrium of stress distribution as shown in **C-4**.
e) In hogging bending regions, the contribution of steel sheeting shall be taken into account only where the sheet is continuous. The redistribution of moments that can occur between support and mid-span sections shall not be considered. If the contribution of the steel sheeting is neglected, the hogging bending resistance of a cross-section should be calculated as reinforced concrete for the stress distribution, as shown in **C-4.3.**

f) In the case of partial shear connection, the sagging design bending resistance, M_d, shall be calculated as per **C-5.**

12.5 Shear Resistance of Composite Slab

12.5.1 *Punching Shear*

The punching shear resistance of a composite slab should be calculated as per as per [6-6B(4)], where the critical perimeter is as shown in Fig. 17.

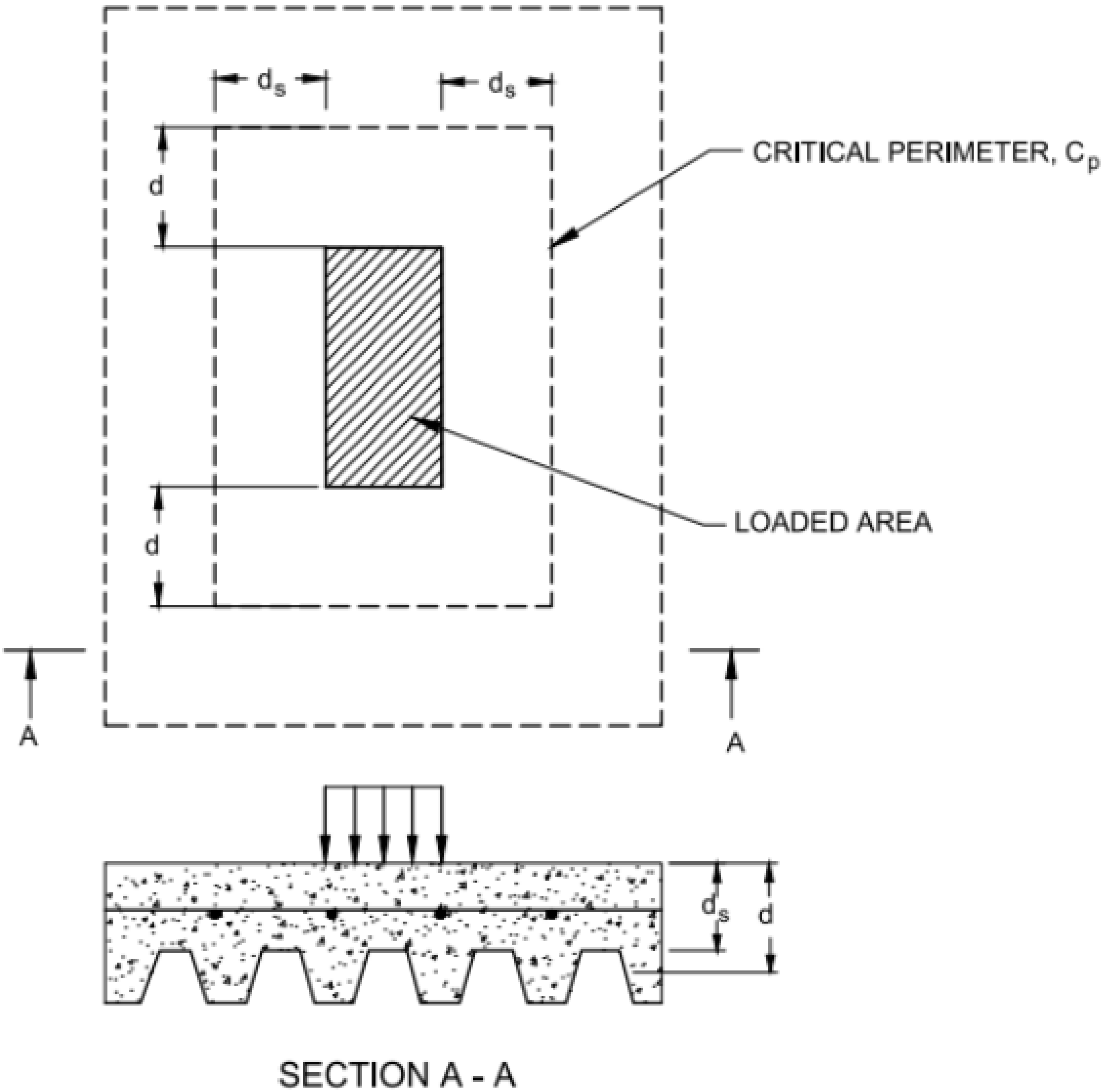


FIG. 17 PUNCHING PERIMETER IN A PROFILED COMPOSITE SLAB

12.5.2 *Resistance to Shear*

The vertical shear resistance V_{dvd} of composite slab over a width equal to the distance between centres of ribs and the effective depth, which depends on the effective depth of the cross section to the centroid of the tensile reinforcement, should be in accordance with [6-6B(4)].

The sheeting may be considered as the tensile reinforcement, provided that it is fully anchored beyond the section considered. For heavily loaded slabs, additional reinforcement may be required at the support when the profiled steel sheeting is discontinuous and has only limited anchorage.

In the case of partial shear connection, the shear capacity may be governed by the interface shear strength between steel decking and concrete, as presented in Annex C.

12.6 Serviceability Limit State

12.6.1 *Design Against Cracking*

The crack width is calculated at the top surface in the hogging moment region using standard methods prescribed for reinforced concrete in [6-6B(4)]. Crack width should not exceed 0.3 mm. The provision of 0.4 percent steel at the top of the slab will normally avoid cracking problems in propped construction. The provision of 0.2 percent of steel is normally sufficient for the same in un-propped construction.

If the environment is corrosive, it is advisable to design the slab as continuous at supports and take advantage of steel provided for negative bending moment resistance and for minimizing cracking during service loads.

12.6.2 *Deflection Limits*

[6-6B(4)] gives a stringent deflection limitation of span/350 or 20 mm whichever is less, which may be unrealistic for un-propped construction. The span to depth ratio in the range of 25 to 35 for the composite condition is recommended for the simply supported slabs and the continuous slabs. The deflection of the composite slabs is influenced by the slip between sheeting and concrete. Testing is the best method to estimate the actual deflection for the conditions adopted.

12.6.3 *Fire resistance*

The fire resistance is based on (a) thermal insulation criterion concerned with limiting the transmission of heat by conduction and (b) integrity criteria concerned with preventing the spread of flames and hot gases to nearby compartments. These are met by specifying the adequate thickness of insulation below the metal decking. When the metal decking is used as a permanent shuttering, the fire resistance of the slab shall be considered as per [6-6B(4)].

12 COMPOSITE COLUMNS

13.1 General

13.1.1 This section applies to various forms of steel-concrete composite columns, including fully or partly encased steel columns and concrete in-filled rectangular or circular steel tubes, provided:

a) The columns or compression members consist of structural steel with grade conforming to [6-6B(6)] and normal weight concrete of strength M20 to M60.
b) Shear transfer between steel-concrete interfaces is ensured basically through a bond for which calculated shear stress at interface shall be kept limited in accordance with Table 15, beyond which mechanical shear connectors are to be provided.

c) The columns or compression members are in framed structures where the other structural members are either composite or steel members.

d) The steel contribution ratio δ (Table 12) should fulfill the criteria:

$$0.2 \le \delta \le 0.9$$

e) Further,

1) The influence of local buckling of the structural steel section on the resistance of the composite section as a whole shall be considered in the design.
2) The effects of local buckling may be neglected for a steel section fully encased in accordance with **13.2**, and for other types of cross-section, the maximum width to thickness ratio given in [6-6B(5)] shall not be exceeded.

13.1.2 *Types of composite columns*

Composite columns can be of two types:

a) Encased where concrete encases the steel section (*see* Fig. 18); and
b) In-filled where concrete fills rectangular or circular steel tube (*see* Fig. 19).

13.2 Details for Composite Action

In composite columns with fully encased steel sections, concrete cover to structural steel sections shall be at least 40 mm or one-sixth of the breadth *b* of the flange, over steel section. The concrete shall be adequately held by steel reinforcements and stirrups all around. The steel section shall be unpainted to ensure bond and friction between steel and concrete, but cleaned to ensure protection against corrosion and spalling of concrete. The cover to steel reinforcement should be in accordance with [6-6B(4)].

Shear transfer between steel-concrete interfaces is ensured basically through bond for which calculated shear stress at interface shall be limited in accordance with Table 15, beyond which mechanical shear connectors are to be provided.

13.3 Members Under Axial Compression

Standard composite sections used as columns are as shown at Fig 18 and Fig 19.

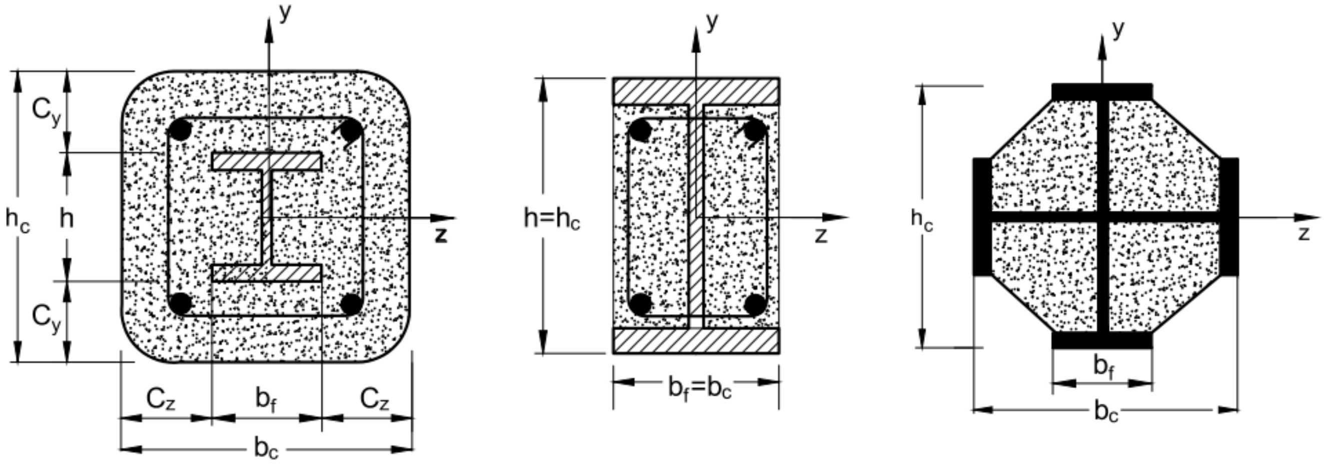


FIG. 18 FULLY AND PARTIALLY CONCRETE ENCASED COLUMNS

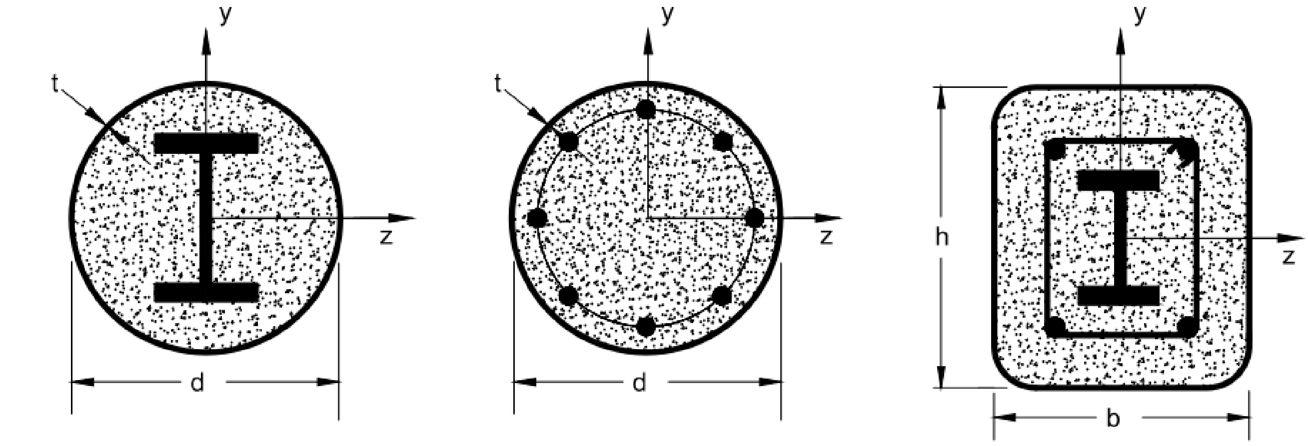


FIG. 19 CONCRETE IN-FILLED COLUMNS

13.3.1 *General Design Philosophy*

To ensure structural stability of a compression member, second-order effects like residual stresses, geometrical imperfections, local instability, cracking of concrete, creep/shrinkage of concrete, yielding of structural steel and reinforcement etc. shall be considered. While designing the compression members, including design for the above effects, the following shall be taken into account:

a) The above effects shall be considered in any direction in which failure might occur if they affect the structural stability significantly.
b) Internal forces shall be determined by any of the analysis methods discussed in **6**.
c) Full composite action between steel and concrete shall be considered up to failure.
d) Effects of creep and shrinkage 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 from permanent loads is not greater than 10 percent of first-order bending moments due to total design loads.

13.3.2 *Design Guidelines*

The simplified design provisions given in the following sections is applicable for composite members that are doubly symmetrical and uniform in cross-section throughout the length of the member. This method is not applicable to members consisting of two or more unconnected sections. Further, the composite member should conform to the following conditions.

a) The non-dimensional slenderness $\bar{\lambda}$ should not be greater than 2.

b) For a fully encased steel section (Fig. 18a), the maximum thickness of concrete cover that may be used in the strength calculation is limited to:

$$c_z\ (Max) = 0.4\ b$$
$$c_y\ (Max) = 0.3\ h$$

c) The longitudinal steel reinforcement that may be used in the calculation should not exceed 6 percent of the concrete area.

d) The ratio of the cross-section's depth h_c to width b_c, (*see* Fig. 18), should be within limits ($0.2 \leq h_c/b_c \leq 5.0$)

13.4 Local Buckling of Steel Sections

13.4.1 To prevent premature local buckling of structural steel components in partly encased steel section and concrete filled steel sections (Fig. 18 and Fig. 19), the width to thickness ratio of individual elements of the steel sections in compression must satisfy the following limits:

$\frac{d}{t} \leq 88\varepsilon^2$ for concrete-filled circular tubular sections

$\frac{h}{t} \leq 50\varepsilon$ for concrete-filled rectangular tubular sections

$b_f/t_f \leq 43\varepsilon$ for partially encased I sections

Where, $\varepsilon = \sqrt{\frac{250}{f_y}}$ and f_y is the yield strength of the steel section, in MPa

13.4.2 For fully encased steel sections, the above local buckling check is not required. However, the concrete cover to the flange of a fully encased steel section should not be less than 40 mm, nor less than one-sixth of the breadth, *b*, of the flange.

13.4.3 Design of concrete-filled rectangular tubular sections where *h/t* ratios exceed the local buckling limits for semi-compact sections should be verified by tests.

13.4.4 The cover to reinforcement should be in accordance with [6-6B(4)].

13.4.5 The steel section shall not be painted to ensure friction between steel and concrete but cleaned at the abutting surface to ensure protection against corrosion and spalling of concrete.

13.5 Design Compressive Resistance of Short Composite Columns

13.5.1 A column is termed as a 'short column' when one of the following conditions is satisfied.

(i) its non-dimensional slenderness ratio $\lambda \leq 0.2$.

(ii) If the actual design compressive force on the composite column, *P*, is less than 0.1 P_{cr}.

where

$$\lambda = \sqrt{\frac{P_{\mathrm{n}}}{P_{\mathrm{cr}}}}$$

P_{cr} = the elastic buckling load of the column $= \frac{\pi^2 (EI)_e}{L^2}$

$(EI)_e$ = effective elastic flexural stiffness of the composite column (**13.5.2**)

L = the effective length of the column, which may be conservatively taken as the overall length L for an isolated non-sway composite column

The design compressive strength P_n of short columns of plastic and compact sections may be evaluated using the design strength of the material with appropriate partial safety factors. The most common column sections are bi-symmetric. The design compressive strength of short bi-symmetric columns is evaluated using equations given in **D-2**.

NOTE — While providing spiral ties it must be designed as per of [6-6B(4)], and the ratio of the volume of helical reinforcement to the volume of the core is not less than $0.36(A_g/A_{co}-1)f_{ck}/f_{yk}$.

(Where, A_g = Gross area of section; A_{co} = Area of core of the confined core of column measured to the outer diameter of the helix)

13.5.2 *Effective Elastic Flexural Stiffness*

a) *Short Term Loading* — The effective elastic flexural stiffness, $(EI)_e$, is obtained by adding up the flexural stiffness of the individual components of the cross-section:

$$(EI)_e = E_s I_s + 0.6\, E_{cm} I_c + E_{st} I_{st}$$

where

E_s and E_{st} = modulus of elasticity of the structural steel section and the reinforcement, respectively

E_{cm} = secant modulus of the concrete

b) *Long Term Loading* — In slender columns ($\lambda > 0.2$), the effect of long-term loading should be considered.

If the eccentricity *'e'* of loading is more than twice the cross-section dimension *'D'* ($e > 2D$), the effect on the bending moment distribution caused by increased deflections due to creep and shrinkage of concrete will be very small and may be neglected. The effect of long-term loading need not be considered if the non-dimensional slenderness (λ) of the composite column is less than the limiting values given in Table 12.

Table 12 Limiting Values of λ for Long Term Loading

(*Clause* 13.5.2)

SL No.	Cross-sections	Braced Non-sway Systems	Un-braced and/or Sway System
(1)	(2)	(3)	(4)
i)	Concrete encased cross-section	0.8	0.5

ii)	Concrete filled cross-section	$\frac{0.8}{1-\delta}$	$\frac{0.5}{1-\delta}$
NOTE — δ is the steel contribution ratio defined as $\delta = \frac{A_s \cdot f_y}{P_d \gamma_{m0}}$ P_d is defined in **D-2**.			

When λ exceeds the limits prescribed above and *e/D* < 2, the effect of creep and shrinkage of concrete should be considered by adopting the modulus of elasticity of concrete E_{cs} instead of E_{cm} where E_{cs} is defined as follows:

$$E_{cs} = 0.75E_{cm}\left[1 - \frac{0.5P_{dd}}{P}\right]$$

where

P = the applied factored axial load; and
P_{dd} = the part of the factored load permanently acting on the column.

The effect of long-term loading may be ignored for concrete filled tubular sections with $\lambda \leq 2.0$ provided that $\delta > 0.6$ for braced (non-sway) columns and $\delta > 0.75$ for unbraced (sway) columns.

13.6 Long Members Subjected to Axial Compression

The non-sway composite columns need not be checked for buckling if any one of the following conditions is satisfied:

a) $P < 0.1P_{cr}$ where P is the axial force in the column, and P_{cr} is the elastic buckling load of the column, defined in **13.5.1**.
b) Non-dimensional slenderness $\lambda < 0.2$.

For columns not satisfying the above conditions, safety against buckling strength shall be checked about the corresponding axis. The following equation needs to be satisfied for buckling load:

$$P \leq \chi \cdot P_d$$

Where,

P = the applied factored axial load;
P_d = is the design plastic resistance to compression (**D-2**)

$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \lambda^2}}$ = is the stability reduction factor

$\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2]$
λ = Non-dimensional slenderness ratio.
α = A value that represents initial imperfections and residual stresses which is based on the buckling curve to be adopted, which is further dependent on the type of section and the axis of bending as given below (see Table 13):

Curve *a* – for concrete-filled tubular sections with reinforcement percentage less than 3 percent of gross cross section area and buckling about both axes.

Curve *b* – for fully or partially concrete-encased I-sections buckling about the strong axis (z-z axis) of the steel sections and for concrete-filled tubular sections with steel reinforcement percentage more than 3 percent of gross cross section area and buckling about both axes or concrete filled tubular section with the addition of I section at the core and buckling about both axes.

Curve *c* – for fully and partially concrete-encased I-sections buckling about the weak axis (*y-y* axis) of the steel sections.

Table 13 Imperfection Factor α for the Buckling Curves

(Clause 13.6)

Buckling Curve (1)	**a** (2)	**b** (3)	**c** (4)
Imperfection Factor	0.21	0.34	0.49

13.7 Members Subjected to Combined Compression and Bending

When the bending moment in the section is zero (that is, M = 0), the design compressive strength is as given in **13.5**. Similarly, the plastic moment of resistance of a column at zero compression load is as given **D-3**. The equations for evaluating the design bending resistance of symmetric column sections are given in **D-4**.

13.7.1 *Bending Resistance Under High Shear*

While determining the plastic resistance when the simultaneous shear force is high at the section as given below, the bending resistance shall be reduced as given below:

If the shear force V on the steel section exceeds 60 percent of the design shear resistance V_d of the steel (**8.3.6.1**), the influence of the transverse shear on the resistance in combined bending and compression shall be taken into account by a reduced design steel strength $(1 - \beta) f_y / \gamma_{m0}$ in the shear area A_v (β is determined as per **8.3.6.2**).

a) Unless a more accurate analysis is used, the design shear action, V_d, may be distributed into (V_s, acting on the structural steel) and (V_c, acting on the reinforced concrete section) as:

$$V_s = V \frac{M_{ds}}{M_d}$$

$$V_c = V - V_s$$

where

M_{ds} = design moment of resistance of steel section alone;
M_d = design moment of resistance of the entire composite section; and
V_d = may be assumed to be resisted by steel sections alone as a simplifying approximation.

13.7.2 *Second Order Effects on Bending Moment*

If neither of the two conditions (a) and (b) for short compression members in **13.5.1** is satisfied, then the second-order effects on first-order analysis moment shall be considered by modifying the maximum first-order bending moment (moment obtained linear elastic analysis), M_{max}, with an amplification factor k, which is defined as:

$$k = \frac{C_{\mathrm{m}}}{1 - \frac{P}{P_{\mathrm{cr}}}} \geq 1.0$$

where

P = the applied factored axial load,

P_{cr} = elastic critical load for the relevant axis and corresponding to a modified effective flexural stiffness given by $(EI)_e$ as given in **13.5.2** and the effective length of the column is taken as per [6-6B(5)].

C_m = equivalent moment factor given in Table 14 to account for non-uniform bending moment over the length of the member.

ψ = the ratio of moments at the ends of the laterally unsupported members

13.8 Members Subjected to Axial Force and Uni-axial Bending

13.8.1 While checking a section for combined axial force and bending moment, first, it should be ensured that the section is safe against an axial force acting alone, considering buckling along each principal axis. The resistance of the section shall then be checked for combined axial compression and uniaxial bending moment, as described below:
The design against combined bending and axial compression are adequate when the following condition is satisfied:

$$M'_{\mathrm{d}} = \alpha_{\mathrm{m}} \mu_{\mathrm{d}} M_{\mathrm{d}}$$

where

α_m = reduction coefficient based on steel yield strength;
= 0.9 for 250 $\geq f_y \leq$ 400 MPa
= 0.8 for $f_y >$ 400 MPa

M'_d = design bending moment, which is to be amplified to account for the second-order effects as discussed in **13.7.2**, when the column is not short.

M_d = Design bending resistance of the section about the bending axis evaluated as in **D-3.**

The moment resistance reduction ratio μ_d for a long composite column under combined compression and uniaxial bending shall be evaluated as follows:

$$\mu_d = \frac{(\chi - \chi_d)}{(1-\chi_c)\chi} \quad \text{when } \chi_d \geq \chi_c$$

and

$$\mu_d = 1 - \frac{(1-\chi)\chi_d}{(1-\chi_c)\chi} \quad \text{when } \chi_d < \chi_c$$

χ_c = Axial resistance ratio to the concrete P_c/P_d

χ_d = Design axial resistance ratio to the concrete P/P_d

χ = reduction factor due to column buckling as per [6-6B(5)].

P_c = axial compressive strength of concrete $A_c \frac{\alpha_{cc.}}{\gamma_c}.\eta.\lambda.f_{ck}$

P_d = *see* **D-2**.

13.8.2 *Members Subjected to Axial Force and Bi-axial Bending*

Members subjected to combined axial compression and biaxial bending shall satisfy the following interaction relationships equations:

$$\frac{M_z}{\mu_{dz} M_{dz}} + \frac{M_y}{\mu_{dy} M_{dy}} \leq 1$$

$$\frac{M_z}{\mu_{dz} M_{dz}} \leq \alpha_{mz}$$

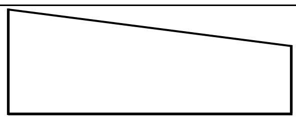
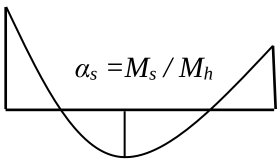
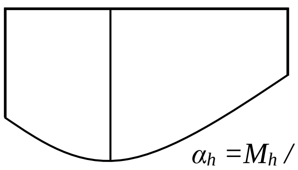
$$\frac{M_y}{\mu_{dy} M_{dy}} \leq \alpha_{my}$$

Where,

μ_{dz} and μ_{dy} = moment resistance reduction factors in z and y directions, respectively. (*see* **13.8**)

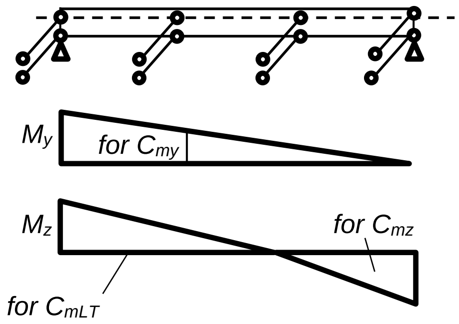
$\alpha_{mz} = \alpha_{my} = \alpha_m$ (*see* **13.8**)

Table 14 Equivalent Uniform Moment Factor
(*Clause* 13.7)

Bending Moment Diagram	**Range**		C_{my}, C_{mz}, C_{mLT}	
			Uniform Loading	Concentrated Load
(1)	(2)		(3)	(4)
	$-1 \leq \psi \leq 1$		$0.6 + 0.4\,\psi \geq 0.4$	
	$0 \leq \alpha_s \leq 1$	$-1 \leq \psi \leq 1$	$0.2 + 0.8\,\alpha_s \geq 0.4$	$0.2 + 0.8\,\alpha_s \geq 0.4$
	$-1 \leq \alpha_s \leq 0$	$0 \leq \psi \leq 1$	$0.1 - 0.8\,\alpha_s \geq 0.4$	$-0.8\,\alpha_s \geq 0.4$
		$-1 \leq \psi \leq 0$	$0.1(1-\psi) - 0.8\,\alpha_s \geq 0.4$	$0.2(-\psi) - 0.8\,\alpha_s \geq 0.4$
	$0 \leq \alpha_h \leq 1$	$-1 \leq \psi \leq 1$	$0.95 - 0.05\,\alpha_h$	$0.90 + 0.10\,\alpha_h$
	$-1 \leq \alpha_h \leq 0$	$0 \leq \psi \leq 1$	$0.95 + 0.05\,\alpha_h$	$0.90 + 0.10\,\alpha_h$
		$-1 \leq \psi \leq 0$	$0.95 + 0.05\,\alpha_h\,(1+2\,\psi)$	$0.90 + 0.1\alpha_h\,(1+2\,\psi)$
For members with sway buckling mode, the equivalent uniform moment factor $C_{my} = C_{mz} = 0.9$.				

C_{my}, C_{mz}, C_{mLT} shall be obtained according to the bending moment diagram between the relevant braced points

Moment factor	Bending axis	Points braced in direction
C_{my}	y-y	z-z
C_{mz}	z-z	y-y
C_{mLT}	z-z	*z-z*



13.9 Mechanical Shear Connection and Load Introduction

Proper sharing of loads between the steel section and concrete of composite columns should be ensured at points of load introduction due to load and moment reactions coming from members connected to the ends of the column and also for axial loads applied anywhere within the length of the column, considering the shear resistance at the interface between steel and concrete.

Where composite columns and compression members are subjected to significant transverse shear, for example, by local transverse loads and by end moments, the provision shall be made for the transfer of the corresponding local longitudinal shear stress at the interface between steel and concrete. For axially loaded columns and compression members, longitudinal shear outside the region of load introduction need not be considered.

13.9.1 *Load Introduction*

Shear connectors should be provided at regions of load introduction and at regions with change in cross section, if the design shear resistance at the concrete-steel interface, τ, (Table 15) is exceeded. 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 resulting from an elastic analysis considering creep and shrinkage should be taken into account. Otherwise, the forces at the interface should be determined by elastic theory or plastic theory to determine the more severe case. In the absence of any accurate method, the introduction length should not exceed 2*d* or *L*/3, where *d* is the minimum transverse dimension of the column and *L* is the column length.

Table 15 Design shear strength (τ)
(*Clause* 13.9.1)

Type of Cross-section	τ (N/mm²)
(1)	(2)
Completely concrete-encased steel sections	0.30
Concrete filled circular hollow sections	0.55

Concrete filled rectangular hollow sections	0.40
Flanges of partially encased sections	0.20
Webs of partially encased columns	0.00

Due to the action of creep and shrinkage, no shear connection is required for composite columns or compression members if the load application is by endplate, where the full interface between steel and concrete is permanently under compression. Otherwise, the load application/introduction should be verified as given below:

a) If the cross-section is partially loaded, as shown in Fig. 20 (a), the loads may be distributed with a ratio of 1:2.5 over the thickness $\boldsymbol{t_e}$ of the endplate. The concrete stresses should then be limited in the area of effective load introduction.

b) For concrete-filled circular hollow section or square hollow section, under partial loading as shown in Fig. 20(b), for example, by gusset plates or by stiffeners, the local design strength of concrete σ_c under the gusset or stiffener resulting from the sectional forces of the concrete section shall be determined as:

$$\sigma_c = \frac{0.8 f_{ck}}{\gamma_c}\left[1 + \eta_0 \frac{t}{d}\frac{f_y}{0.8 f_{ck}}\right]\sqrt{\frac{A_c}{A_1}} \leq \frac{0.8. A_c . f_{ck} .}{A_1 . \gamma_c}; \leq \frac{f_y}{\gamma_{m0}}$$

where,

$A_c/A_1 \leq 20$
t = is the wall thickness of the steel tube;
d = diameter of the tube or 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 (*see* Fig. 20); and
η_0 = 4.9 for circular steel tubes and 3.5 for square sections.

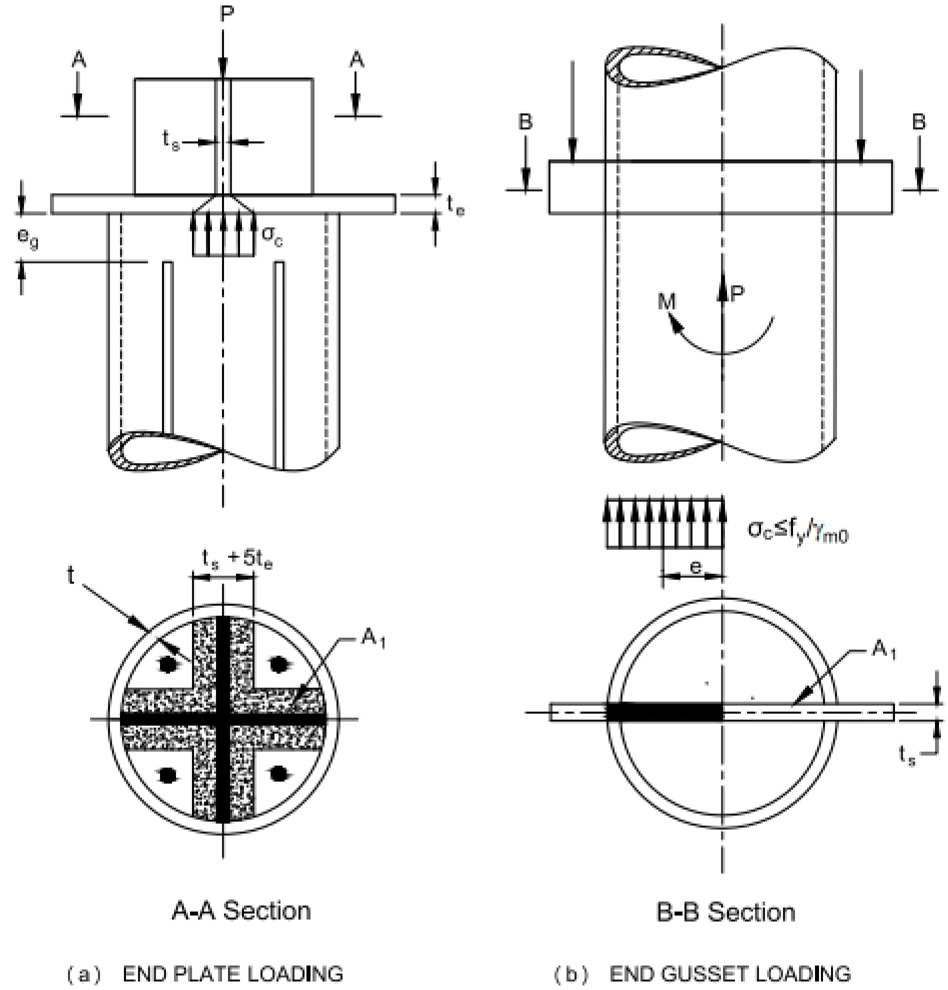


FIG. 20 PARTIALLY LOADED CIRCULAR CONCRETE FILLED HOLLOW SECTION

c) For concrete-filled circular hollow sections, longitudinal reinforcements may be taken into account while determining the resistance of the composite column, even if the reinforcement is not directly connected to the endplate, provided that the gap [Fig. 20(a)] between the end of reinforcement and the surface of the endplate does not exceed 30mm.

13.9.2*Shear connection*

When a mechanical connection is introduced in the form of stud connectors to the web of a fully or partially concrete-encased steel I-section, an account may be taken of the frictional forces that develop due to the restraint in lateral expansion of the concrete by the adjacent steel flanges. This resistance is assumed to be equal to $\mu.Q_u/2$ on each flange and each horizontal row of studs, as shown in Fig. 21 and may be added to the calculated resistance of the shear connectors. μ is the relevant coefficient of friction and be taken as 0.5. Q_u is the resistance of a single stud as per **11.2**.

The clear distance between the flanges should not exceed the values given in Fig. 21 to ensure the development of the frictional forces between concrete and steel flanges.

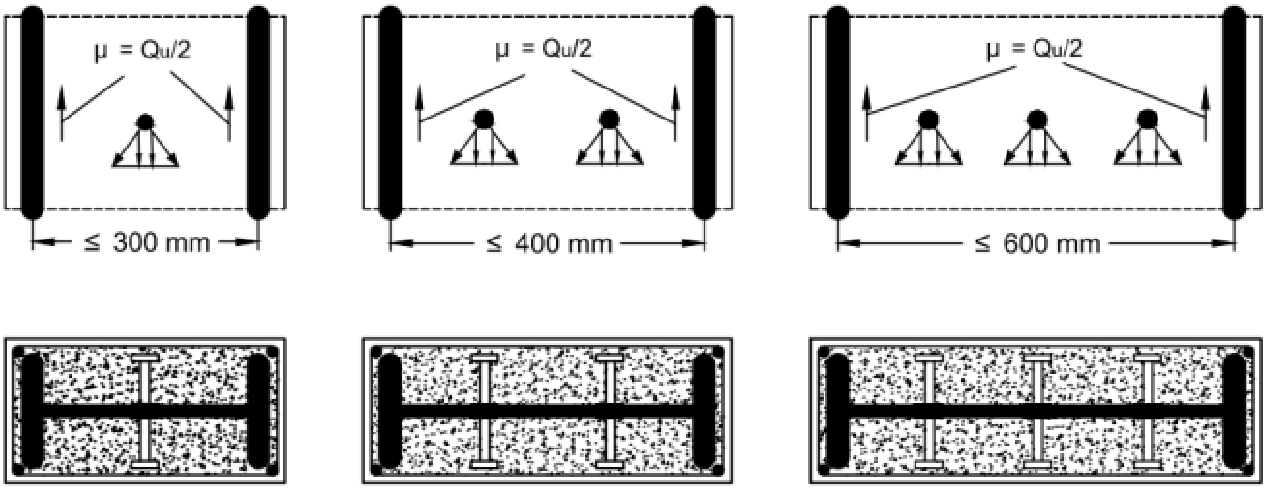


FIG. 21 ADDITIONAL FRICTIONAL FORCES IN COMPOSITE COLUMNS BY USE OF HEADED STUDS

13.9.3 *Longitudinal Shear Outside Area of Load Introduction*

a) Outside the area of load introduction, longitudinal shear at the interface between concrete and steel should be verified where it is caused by transverse loads and /or end moments. Shear connectors should be provided based on the distribution of the design value of longitudinal shear, where this exceeds the design shear resistance τ. In the absence of a more accurate method, elastic analysis, considering long-term effects and cracking of the concrete, may be used to determine the longitudinal shear at the interface. 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 15 may be assumed for τ. The value of τ given in Table 15 for fully concrete-encased steel sections applies to sections with a minimum concrete cover of 40 mm. For greater concrete cover and adequate reinforcement, higher values of τ may be used. Unless verified by tests, for completely encased sections, the increased value βc. τ may be used, with βc given by:

$$\beta_c = 1 + 0.02\, c_z \left[1 - \frac{c_{z,min}}{c_z}\right] \leq 2.5$$

Where,

c_z - is the wall thickness of the steel tube; and

$c_{z,min}$ - diameter of the tube or width of the square section.

b) Unless otherwise verified, for partially encased I-sections with transverse shear due to bending about the weak axis caused by lateral loading or end moments, shear connectors should always be provided. If the resistance of the structural steel section alone against transverse shear is not sufficient to take care of the total transverse shear on the composite section, then the required transverse reinforcement for the shear force Vc according to **12.5.2**, should be welded to the web of the steel section or should pass through the web of the steel section.

13.10 Shear Check

The factored shear force in the compression members should be less than the design shear strength of the member, which is the sum total of the shear resistance given by the concrete section along with steel reinforcements as per [6-6B(4)] and the shear resistance given by the steel section as per **8.3.6**. The shear force shall be distributed between the steel section and the concrete section in accordance with **13.9**.

14 CONSTRUCTION AND ERECTION

14.1 Fabrication and Inspection Procedure for Structural Steel

Fabrication and erection of steel sections and components, as and where situated in a composite structure would include fabrication procedures, both shop and site fabrications, along with fabrication tolerances, inspection, testing, handling, transportation, site storage, erection along with erection tolerances, etc.

Fabrication and erection specifications of all steel components of a composite structure shall refer to stipulations laid down in [6-6B(5)].

15 FIRE RESISTANCE AND FIRE DESIGN

15.1 In closed structures, an accidental fire may lead to a rise in temperature under which failure of the material may take place unless a proper design and construction against fire is carried out. Open structures such as bridges are not generally vulnerable to failure under fire since the temperature does not go up to the level which may cause material damage. Also, in an open structure, the fire can be extinguished easily and quickly. Fire tests on open structures such as elevated parking lots and bridges have shown that the structure does not undergo any material damage due to reasons indicated above. However, all structures, including both closed and open ones, shall be protected from all possible accidental fires caused by different kinds of hazards.

Fire-resistant designs for open and closed structures at specialized locations, such as those in proximity to oil installations or pipelines carrying inflammable materials, etc., as in the case of industrial buildings and structures, shall be done based on recommendations given in specialized literature. Also, adequate provisions may be made for fire fighting equipment to access all parts of the structure. After the occurrence of fire in a structure, it should be mandatory for the concerned authorities to have the structure inspected by competent experts in order to ascertain the condition of the structure before it can be declared safe for re-use.

In addition to the above, locations in any structural system that may be prone to accidental occurrence of fire, shall be adequately provided with basic fire protection methods as per specialist literature. These will include both active as well as passive fire protection.

15.2 Response to Fire

Steel is a good conductor of heat that experiences an almost uniform temperature increase over its entire volume. Concrete is a poor conductor of heat has a large temperature gradient within its volume. Due to this, the entire steel member gains the heat faster, whereas the surfaces of concrete exposed directly to heat experience high temperature, with the temperature dropping off drastically in the interior of the concrete mass.

Both steel and concrete exhibit a drastic reduction in strength with an increase in temperature above 400°C. Due to this, the resistance of steel members tends to decrease drastically above 400°C, whereas concrete at the surfaces exposed to high temperature tends to spall off due to high temperature gradient. The steel structure, if it does not fail under fire, regains its strength after the fire, the damage at the surface of the concrete is permanent due to chemical changes. The steel reinforcement in concrete is protected from loss of strength due to an increase in temperature by providing adequate concrete cover as per [6-6B(4)]. Composite floor system deck sheeting is to be provided with fire protection.

However, literature shows that the minimum necessary fire resistance can be achieved by using the composite floor systems and the composite columns, without applying fire protection. Interaction of the steel beam and the concrete slab in the composite flooring leads to a higher critical temperature and higher fire resistance.

The requirements for fire resistance shall apply to steel elements of steel-concrete composite structure designed to exhibit a required fire resistant level as per the relevant specifications.

15.2.1 *Design Fire*

The design of a structure against fire load is dependent on the required fire resistant level (FRL), which is dependent on the function of the structure itself and the period of structural adequacy (PSA). These shall be calculated based on the stipulation laid down in [6-6B(5)].

The response of steel elements of a composite structure against fire as laid down in [6-6B(5)] shall be binding on all steel elements of the composite structures which are susceptible to rise in temperature during its design life. For all general buildings, an adequate fire protection methodology shall be adopted.

15.2.2 *Fire Protection Methodology*
Apart from the direct design of steel components against fire as required in [6-6B(5)] (protected as well as unprotected section), other protective measures), both active and passive), also may be adopted as recommended in literature as fire-resistant procedures.

For active fire resistance, provisions of fire locating and fighting measures like the smoke detectors, fire extinguishers inside a building along with accessory fire water supply, sprinkler system, etc. shall be made available at vantage points. The planning of the structure shall be made in such a way that it is accessible from all sides to fire extinguishing vehicles.

For passive fire resistance, protective paints and materials like intumescent paints, vermiculite boards etc, may be used on exposed steel surfaces, and their provision shall be made as per the required fire-resistant level and as per their properties and specification provided by the manufacturers. The design of protected sections shall be done as per the stipulations laid down in the relevant section of [6-6B(5)]. Detailed design against the fire may be based on provisions of other composite codes and specialized literature.

15.2.3 *Fire Resistance of Composite Slabs*

The fire resistance is assumed based on the following two criteria:

a) Thermal insulation criterion, concerned with limiting the transmission of heat by conduction; and
b) Integrity criterion is concerned with preventing the flames and hot gases from nearby compartments. It is met by specifying an adequate thickness of insulation.

16 TESTING METHODS

For testing methods, Annex E may be referred.

ANNEX A

(*Clause* 2)

LIST OF STANDARDS

The following list records those standards which are acceptable as 'good practice' and 'accepted standards' in the fulfilment of the requirements of the Code. The latest version of a standard shall be adopted at the time of enforcement of the Code. The standards listed may be used by the Authority for conformance with the requirements of the referred clauses in the Code.

In the following list, the number appearing in the first column within parentheses indicates the number of the reference in this Part/Section.

(1)	456 : 2000	Plain and reinforced concrete — Code of practice (*fourth revision*)
	800 : 2007	General construction in steel — Code of practice (*third revision*)
(2)	875	Code of practice for design loads (other than earthquake) for buildings and structures
	Part 1 : 1987	Dead loads – Unit weights of building materials and stored materials (*second revision*)
	Part 2 : 1987	Imposed loads (*second revision*)
	Part 3: 2015	Wind loads (*third revision*)
	Part 4: 1987	Snow loads (*second revision*)
	Part 5: 1987	Special loads and combinations (*second revision*)
(3)	1893	Criteria for earthquake resistant design of structures,
	Part 1 : 2016	Part 1 General provisions and buildings (*fifth revision*)
	Part 2 : 2014	Part 2 Liquid retaining tanks (*fifth revision*)
	Part 3 : 2014	Part 3 Bridges and retaining walls
	Part 4 : 2005	Part 4 Industrial structures including stack like structures
(4)	456 : 2000	Plain and reinforced concrete — Code of practice (*fourth revision*)
(5)	800 : 2007	General construction in steel — Code of practice (*third revision*)
(6)	2062 : 2011	Hot rolled medium and high tensile structural steel — Specification (*seventh revision*)
(7)	1608 (Part 1) : 2018	Metallic materials tensile testing: Part 1 Method of test at room temperature (*fourth revision*)
(8)	808 : 2021	Dimensions for hot rolled steel beam, column, channel and angle sections (*fourth revision*)
	1161 : 2014	Steel tubes for structural purposes – Specification (*fourth revision*)
	1239 (Part 1) : 2004	Steel tubes, tubulars and other wrought steel fittings — Specification : Part 1 Steel tubes (*sixth revision*)
	1239 (Part 2) : 2011	Steel tubes, tubulars and other wrought steel fittings — Specification : Part 2 Steel pipe fittings (*fifth revision*)
	1730 : 1989	Dimensions for steel plates, sheets, strips and flats for general engineering purposes (*second revision*)

	1732 : 1989	Dimensions for steel bars, round and square for structural and general engineering purposes (*second revision*)
	1852 : 1985	Specification for rolling and cutting tolerances for hot rolled steel products (*fourth revision*)
	2062 : 2011	Hot rolled medium and high tensile structural steel — Specification (*seventh revision*)
	4923 : 2017	Hollow steel sections for structural use – Specification (*third revision*)
	11587 : 1986	Specification for structural weather resistant steels
	12779 : 1989	Rolling and cutting tolerances for hot rolled parallel flange beam and column sections
(9)	1875 : 1992	Carbon steel billets, blooms, slabs and bars for forgings (*fifth revision*)
	6911 : 2017	Stainless steel plate, sheet and strip – Specification (*first revision*)
(10)	1030 : 1998	Carbon steel castings for general engineering purposes – Specification (*fifth revision*)
	1875 : 1992	Carbon steel billets, blooms, slabs and bars for forgings (*fifth revision*)
	2004 : 1991	Carbon steel forgings for general engineering purposes – Specification (*third revision*)
	2644 : 1994	High strength steel castings for general engineering and structural purposes – Specification (*fourth revision*)
	4367 : 1991	Alloy steel forgings for general industrial use – Specification (*first revision*)
(11)	1148 : 2009	Steel rivet bars (medium and high tensile) for structural purposes (*fourth revision*)
	1363	Hexagon head bolts, screws and nuts of product grade C:
	Part 1 : 2019	Part 1 Hexagon head bolts (size range M 5 to M 64) (*fifth revision*)
	Part 2 : 2018	Hexagon Head Screws (Size Range M 5 to M 64) (*fifth r*evision)
	Part 3 : 2018	(Style 1) Hexagon nuts (size range M 5 to M 64) (*fifth revision*)
	1364 (Part 1) : 2018	Hexagon head bolts, screw and nuts products grade A and B: Part 1 Hexagon head bolts (size range M 1.6 to M 64) (*fifth revision*)
	1367	Technical supply conditions for threaded steel fasteners:
	Part 1 : 2014	Part 1 General requirements for bolts, screws, studs and nuts (fourth revision)
	Part 2 : 2002	Part 2 Tolerances for fasteners — Bolts, screws, studs and nuts — Product grades a, b and c (third revision)
	Part 3 : 2017	Part 3 Mechanical properties of fasteners made of carbon steel and bolts, screws and studs (fifth revision)
	Part 5 : 2018	Part 5 Mechanical properties of fasteners made of carbon steel and alloy steel — Set screws and similar threaded fasteners with specified hardness classes — Coarse thread and fine pitch thread (fourth revision)
	Part 6 : 2018	Part 6 Mechanical properties of fasteners made of carbon steel and alloy steel — Nuts with specified property

		classes — Coarse thread and fine pitch thread (fourth revision)
	Part 7 : 1980	Part 7 Mechanical properties and test methods for nuts without specified proof loads (second revision)
	Part 8 : 2020	Part 8 Prevailing torque type steel nuts — Functional properties (fifth revision)
	Part 9/Sec 1 : 1993	Part 9 Surface discontinuities, Section 1 bolts, screws and studs for general applications (third revision)
	Part 9/Sec 2 : 1993	Part 9 Surface discontinuities, Section 2 bolts, screws and studs for special applications (third revision)
	Part 10 : 2002	Part 10 Surface discontinuities — Nuts (third revision)
	Part 11 : 2020	Part 11 Electroplated coatings systems (fourth revision)
	Part 12 : 1981	Part 12 Phosphate coatings on threaded fasteners (second revision)
	Part 13 : 2020	Part 13 Hot — Dip galvanized coatings on threaded fasteners (third revision)
	Part 14/Sec 1: 2018	Part 14 Mechanical properties of corrosion — Resistant stainless — Steel fasteners, Section 1 Bolts, screws and studs (fourth revision)
	Part 14/Sec 2 : 2018	Part 14 Mechanical properties of corrosion-resistant stainless steel fasteners, Section 2 Nuts (fourth revision)
	Part 16 : 2002	Part 16 Designation system for fasteners (third revision)
	Part 17 : 2005	Part 17 Inspections, sampling and acceptance procedure (fourth revision)
	Part 18 : 1996	Packaging (third revision)
	1929 : 1982	Specification for hot forged steel rivets for hot closing (12 to 36 mm diameter) (*first revision*)
	2155 : 1982	Specification for cold forged solid steel rivets for hot closing (6 to 16 mm diameter) (*first revision*)
	3640 : 1982	Specification for hexagon fit bolts (*first revision*)
	3757 : 1985	Specification for high strength structural bolts (*second revision*)
	4000 : 1992	High strength bolts in steel structures — Code of practice (*first revision*)
	5369 : 1975	General requirements for plain washers and lock washers (*first revision*)
	5370 : 1969	Specification for plain washers with outside diameter 3 *x* inside diameter
	5372 : 1975	Specification for taper washers for channels (ISMC) (*first revision*)
	5374 : 1975	Specification for taper washer for I-beams (1SMB) (*first revision*)
	5624 : 1993	Foundation bolts – Specification (*first revision*)
	6610 : 1972	Specification for heavy washers for steel structures
	6623 : 2004	High strength structural nuts (*second revision*)
	6649 : 1985	Specification for hardened and tempered washers for high strength structural bolts and nuts (*first revision*)
	7002 : 2018	Prevailing torque type hexagon nuts (with non-metallic insert), style – Property classes 5, 8 and 10 (*third revision*)
(12)	813 (Part 1) : 2018 ISO 2553:2013	Welding and Allied Processes Symbolic Representation on Drawings-Welded Joints (Second Revision)

	814 : 2004	Covered electrodes for manual metal arc welding of carbon and carbon manganese steel – Specification (*sixth revision*)
	3613 : 1974	Acceptance tests for wire-flux combinations for submerged-arc welding of structural steels
	1395 : 1982	Specification for low and medium alloy steel covered electrodes for manual metal arc welding (*third revision*)
	6419 : 1996	Welding rods and bare electrodes for gas shielded arc welding of structural steel – Specification (*first revision*)
	6560 : 2017	Welding consumables – wire electrodes, wires rods and deposits for gas shielded arc welding of creep resisting steel – Classification (*second revision*)
	7280 : 1974	Specifications for bare wire electrodes for submerged arc welding of structural steel
	812 : 1957	Glossary of terms relating to welding and cutting of metals
	816 : 1969	Code of practice for use of metal arc welding for general construction in mild steel (*first revision*)
	822 : 1970	Code of procedure for inspection of welds
	1024 : 1999	Use of welding in bridges and structures subject to dynamic loading – Code of practice (*second revision*)
	1182 : 1983	Recommended practice for radiographic examination of fusion welded butt joints in steel plates (*second revision*)
	4853 : 1982	Recommended practice for radiographic inspection of fusion welded butt joints in steel pipes (*first revision*)
	5334 : 2003	Code of practice for magnetic particle flaw detection of welds (*third revision*)
	7307 (Part 1) : 1974	Approval tests for welding procedures: Part 1 Fusion welding of steel
	7310 (Part 1) : 2019	Approval tests for welders procedures: Part 1 Fusion welding of steel (*first revision*)
	7318 (Part 1) : 1974	Approval tests for welders when welding procedure approval is not required: Part 1 Fusion welding of steel
	9595 : 1996	Metal – Arc welding of carbon and carbon manganese steels – Recommendations (*first revision*)
	15977 : 2013	Classification and acceptance tests for bare solid wire electrodes and wire flux combination for submerged arc welding of structural steel — Specification
(13)	1785 (Part 1)	Specification for plain hard-drawn steel wire for pre-stressed concrete: Part 1 Cold drawn stress relieved wire (*second revision*)
	1785 (Part 2)	Specification for plain hard-drawn steel wire for pre-stressed concrete: Part 2 As-drawn wire (*first revision*)
	2266 : 2019	Steel wire ropes for general engineering purposes – Specification (*fifth revision*)
	2315 : 1978	Specification for thimbles for wire ropes (*first revision*)
(14)	432 (Part 1) : 1982	Specification for mild steel and medium tensile steel bars and hard-drawn steel wire for concrete reinforcement: Part 1 Mild steel and medium tensile steel bars (*third revision*)

(15)	1786 : 2008	High strength deformed steel bars and wires for concrete reinforcement - Specification (f*ourth revision*)
(16)	13620 : 1993	Fusion bonded epoxy coated reinforcing bars — Specification
(17)	16651 : 2017	High Strength Deformed Stainless Steel Bars and Wires for Concrete Reinforcement - Specification
(18)	1343 : 2012	Prestressed Concrete - Code of Practice (*second revision*)
(19)	808 : 1989	Dimensions for hot rolled steel beam, column, channel and angle sections (*third revision*)
(20)	875 (Part 1) : 1987	Dead loads – Unit weights of building materials and stored materials (*second revision*)
(21)	1893 (Part 1)	Criteria for earthquake resistant design of structures: Part 1 General provisions and buildings (*fifth revision*)
(22)	875 (Part 5): 1987	Code of practice for design loads (other than earthquake) for buildings and structures : Special loads and combinations (*second revision*)
(23)	801 : 1975	Code of practice for use of Cold - Formed Light Gauge Steel Structural Members in General Building Construction (*first revision*)
(24)	11587 : 1986	Specification for structural weather resistant steels
(25)	4000 : 1992	High strength bolts in steel structures — Code of practice (*first revision*)

ANNEX B
BENDING RESISTANCE OF COMPOSITE BEAMS

(*Clauses* 8.1.1, 8.1.2, 8.3.5.2, 8.3.5.3 *and* 8.3.7)

The stress distribution and the methods of evaluation of the sagging and hogging bending resistances under full and partial shear interaction of composite beams are discussed in this Annex.

B-1 SAGGING MOMENT RESISTANCE OF COMPOSITE SECTION WITH PLASTIC OR COMPACT STRUCTURAL STEEL BEAMS

B-1.1 Sagging Bending Resistance with Full Shear Interaction

The equations for calculating the bending resistance of composite beams are presented in this section. It is assumed that sufficient shear connectors are provided between the structural steel and concrete to develop full shear with no-slip during the service load.

Initially, evaluating the resistance is based on parabolic stress block as per [6-6B(4)], over the entire depth of concrete in compression is presented. This is theoretically more accurate than the rectangular stress block (maximum of 2-3 percent difference), but computationally more tedious, particularly when it comes to sections subjected to combined bending and compression.

Subsequently, equations based on rectangular stress block for concrete in compression is presented. This is assumed so that the calculations are simplified. This method is easy to use, particularly when the neutral axis is in or closer to compression flange of the steel section.

In both the above methods, the stress in the reinforcing steel in the concrete slab under compression is disregarded.

B-1.1.1 Based on Parabolic Stress Block satisfying stress strain relationship

Calculation of bending resistance is based on parabolic stress block, which is compatible with strain diagram. The design bending resistance depends on the location of the plastic neutral axis. Table 16 gives the various cases of design bending moments of a composite section depending upon the location of the plastic neutral axis as shown in Fig. 22, 23 and 24. For hybrid sections, appropriate yield strength in flanges and web shall be considered for calculation of the plastic moments.

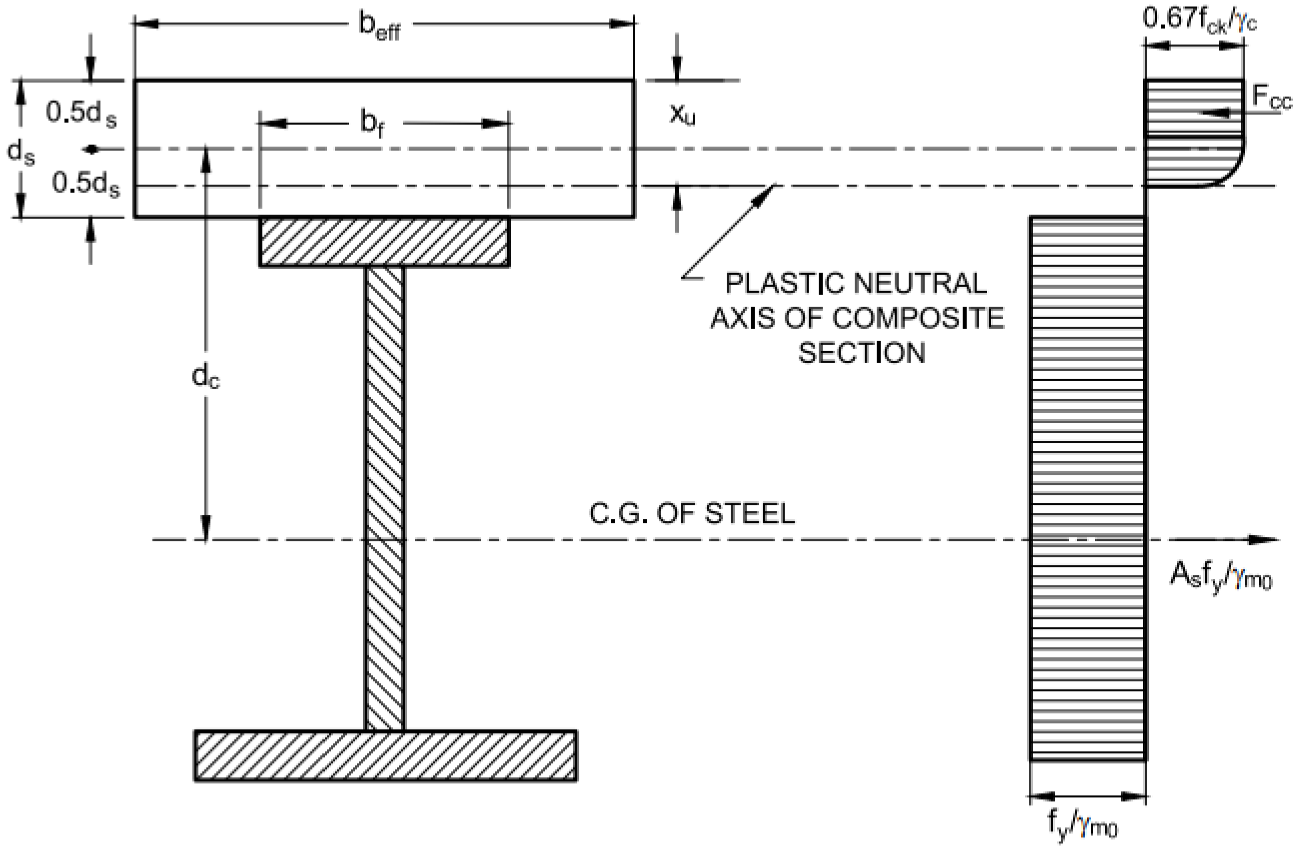


FIG.22 STRESS DISTRIBUTION IN A COMPOSITE BEAM WITH PLASTIC NEUTRAL AXIS WITHIN THE CONCRETE SLAB AT ULTIMATE MOMENT

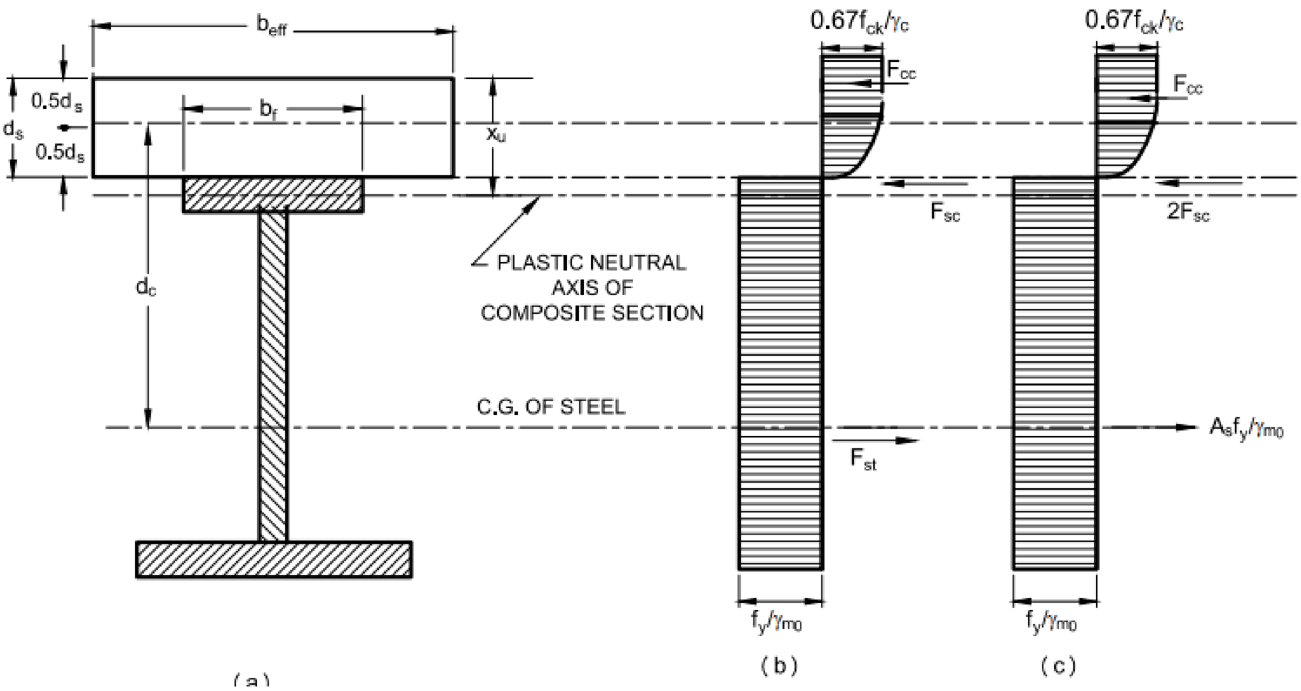


FIG.23 STRESS DISTRIBUTION IN A COMPOSITE BEAM WITH PLASTIC NEUTRAL AXIS WITHIN THE FLANGE OF STEEL BEAM AT ULTIMATE MOMENT

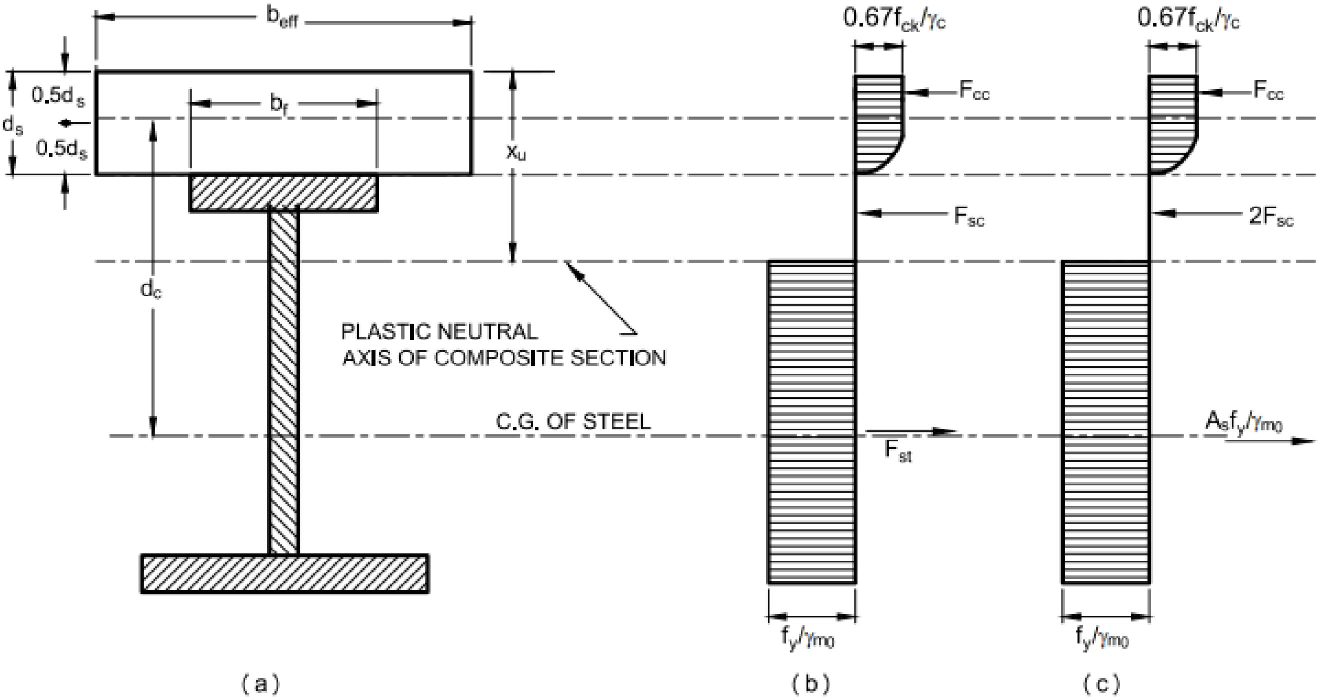


FIG. 24 STRESS DISTRIBUTION IN COMPOSITE BEAM WITH PLASTIC NEUTRAL AXIS WITHIN THE WEB OF STEEL BEAM AT ULTIMATE MOMENT

Table 16 Sagging Moment Resistance of Composite Section with full Shear Interaction (Parabolic Stress Block)

(*Clause* B-1.1.1)

Case	Plastic Neutral Axis in	Location of PNA x_u	Design Moment Capacity M_d
(1)	(2)	(3)	(4)
1	Within slab (Fig.22) $b_{eff}\, d_s > a.A_s$	$x_u = aA_s / b_{eff}$	$M_d = A_s\,(f_y/\gamma_{m0})\,(d_c + 0.5d_s - 0.42\,x_u)$
2	In steel flange (Fig. 23) $b_{eff}.d_s < a\,A_s < (b_{eff}\,d_s + 2a\,A_f)$	$x_u = d_s + (aA_s - b_{eff}\,d_s)/(2b_f\,a)$	$M_d = (f_y/\gamma_{m0})\,[\{A_s\,(d_c + 0.08d_s)\} - \{b_f\,(x_u - d_s)\,(x_u + 0.16\,d_s)\}]$
3	In web (Fig. 24) $a\,A_s > (b_{eff}\,d_s + 2a\,A_f)$	$x_u = d_s + t_f + \{a(A_s - 2A_f) - b_{eff}\,d_s\}/(2at_w)$	$M_d = (f_y/\gamma_{m0})\,[\{A_s\,(d_c + 0.08d_s)\} - \{2A_f\,(0.5t_f + 0.58d_s)\} - \{2t_w\,(x_u - d_s - t_f)\,(0.5x_u + 0.08d_s + 0.5t_f\}]$
Where $a = \dfrac{\frac{f_y}{\gamma_{m0}}}{0.36\,f_{ck}}$			

B-1.1.2 *Based on Rectangular Stress Block*

The design bending resistance depends on the location of the plastic neutral axis, as shown in

Figs. 25, 26 and 27. Table 17 gives the equations for evaluating the bending resistance of a composite section, depending upon the location of the plastic neutral axis. For hybrid sections appropriate yield strength in flanges and web shall be considered for calculation of the plastic moments.

Figs. 26b and 27b, show the actual stress distribution and Figs.26c and 27c, show the equivalent stress distribution to simplify the equations.

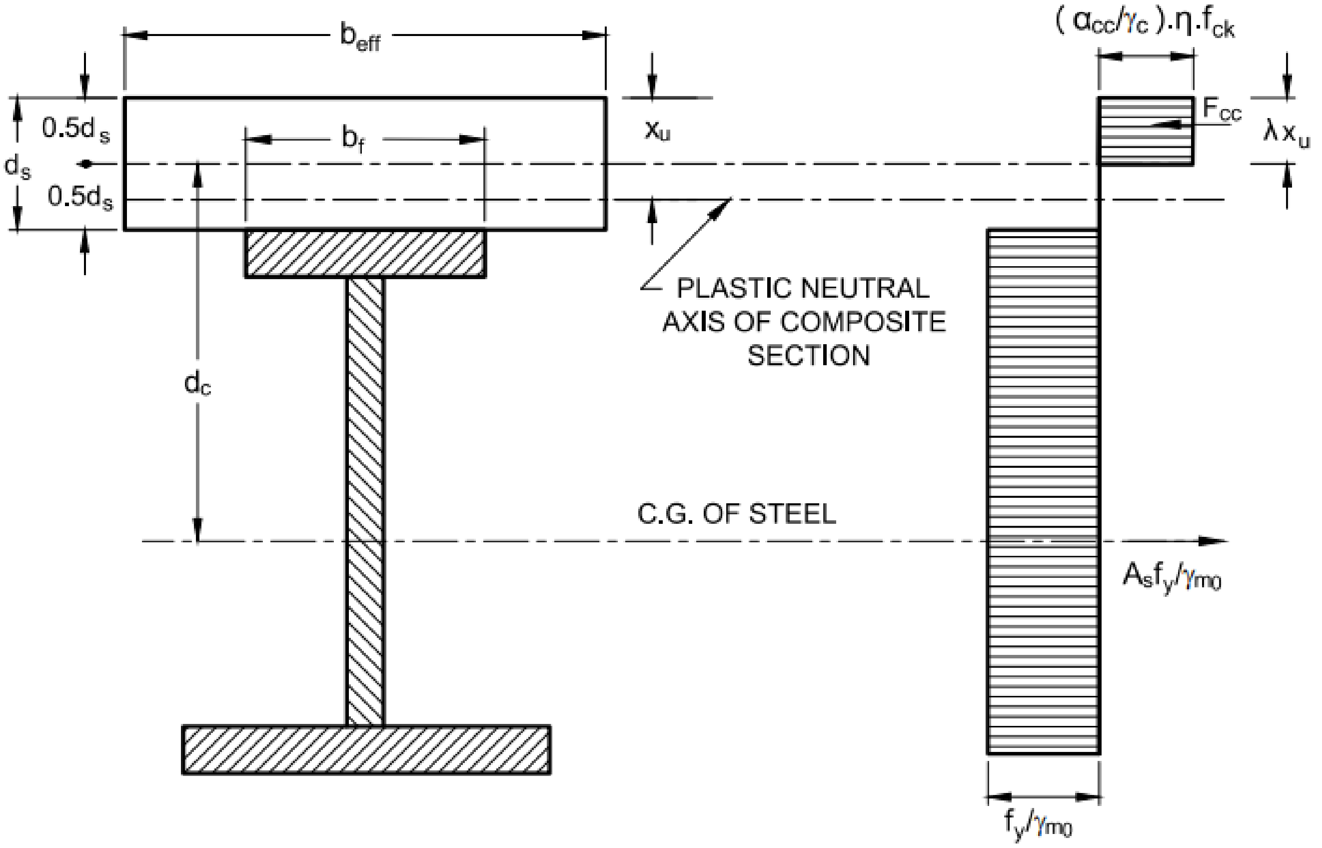


FIG. 25 STRESS DISTRIBUTION IN A COMPOSITE BEAM WITH PLASTIC NEUTRAL AXIS WITHIN CONCRETE SLAB AT ULTIMATE MOMENT

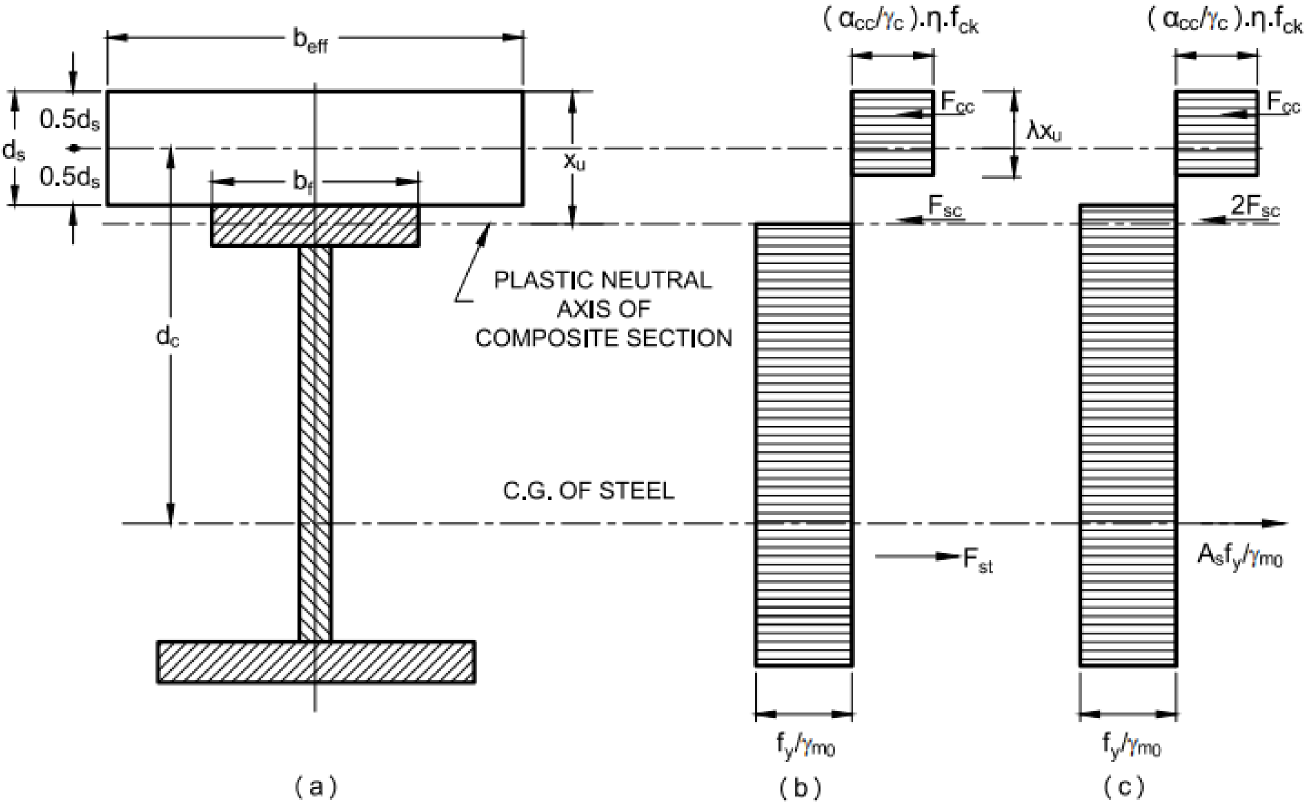


FIG. 26 STRESS DISTRIBUTION IN A COMPOSITE BEAM WITH NEUTRAL AXIS WITHIN THE FLANGE OF THE STEEL BEAM AT ULTIMATE MOMENT

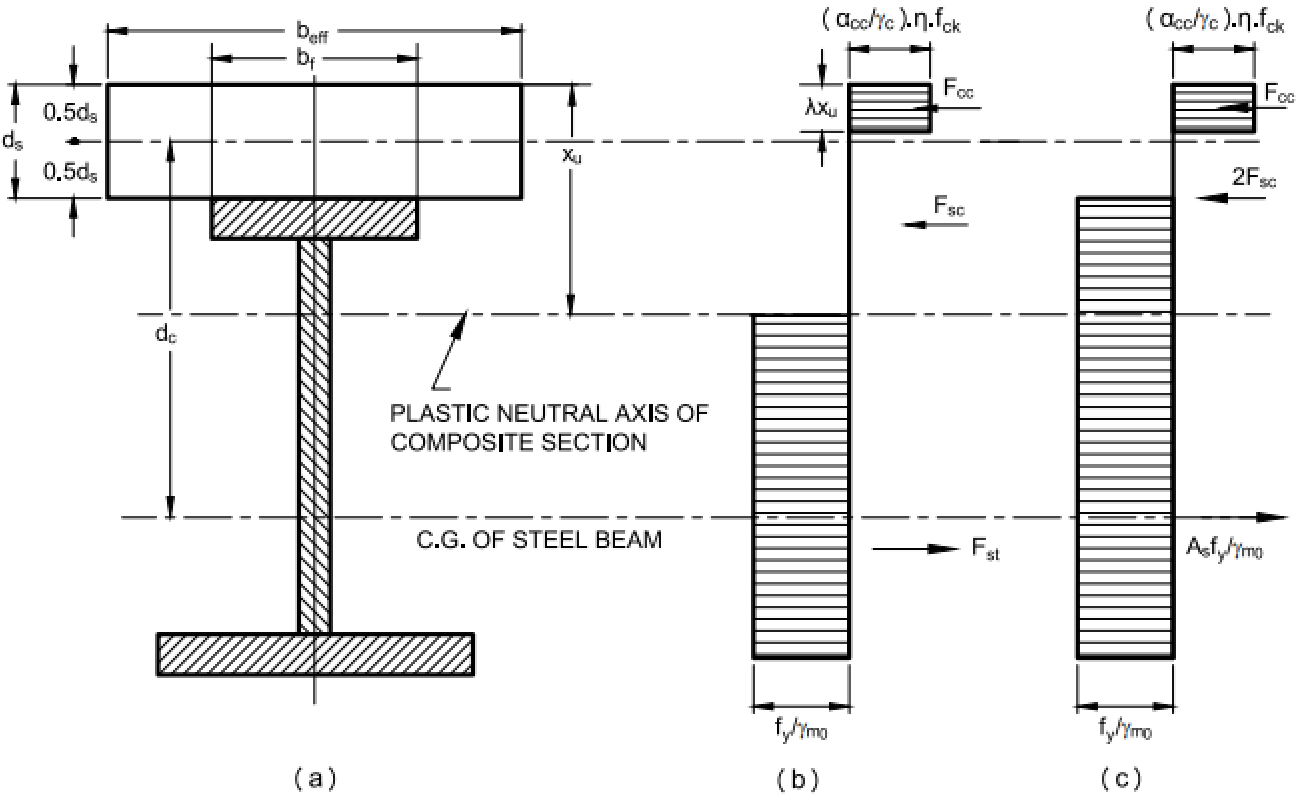


FIG. 27 STRESS DISTRIBUTION IN A COMPOSITE BEAM WITH NEUTRAL AXIS WITHIN THE WEB OF THE STEEL BEAM AT ULTIMATE MOMENT

Table 17 Sagging Moment Resistance of Composite Section with full Shear Interaction (Rectangular Stress Block)
(*Clause* B.1.1.2)

Case	Position of Plastic Neutral Axis	Location of PNA x_u	Design Moment Capacity, M_d
1	Within slab (Fig. 25) $b_{eff}\, d_s > a\, A_s$	$x_u = aA_s / b_{eff}$	$M_d = A_s f_y/\gamma_{m0}\,(d_c + 0.5 d_s - \lambda\, x_u/2)$
2	Plastic neutral axis in steel flange (Fig. 26) $b_{eff}\, d_s < a\, A_s < (b_{eff}\, d_s + 2a\, A_f)$	$x_u = d_s + \frac{(aA_s - b_{eff} d_s)}{2 b_f a}$	$M_d = f_y/\gamma_{m0}\,[A_s\,\{d_c + 0.5 d_S\,(1 - \lambda)\} - b_f\,(x_u - d_s)\,\{x_u + (1 - \lambda)\, d_s\}]$
3	Plastic Neutral axis in web (Fig. 27) $b_{eff}\, d_s + 2a\, A_f < a\, A_s$	$x_u = d_s + t_f + \frac{a(A_s - 2A_f) - b_{eff} d_s}{2 a t_w}$	$M_d = f_y/\gamma_{m0}\,[A_s\,\{d_c + 0.5\, d_s\,(1 - \lambda)\} - 2A_f\{0.5 t_f + (1 - \lambda/2)\, d_s\} - t_w\, x_u - d_s - t_f)\,\{x_u + (1 - \lambda)\, d_s + t_f\}]$

where $a = \dfrac{\frac{f_y}{\gamma_{m0}}}{\frac{\alpha_{cc}}{\gamma_c}.\eta.\lambda.(f_{ck})}$

NOTE - *Notations used in the determination of plastic bending resistance*

A_f = area of the top flange of steel beam of a composite section.
A_s = cross sectional area of structural steel beam of a composite section.
b_{eff} = effective width of concrete slab.
b_f = width of top flange of steel section.
d_s = overall depth of concrete slab.
d_c = vertical distance between centroids of concrete slabs and steel beam in a composite section.
t_f = average thickness of the top flange of the steel section.
t_w = thickness of the web of the steel section.
x_u = depth of neutral axis at ultimate limit state of flexure from top of concrete
M_d = design bending resistance.
α_{cc} = 0.67.
γ_c = material safety factor for concrete .
= 1.50 (for basic and seismic combinations).
= 1.20 (for accidental combinations).
γ_{m0} = material safety factor for structural steel = 1.10
γ_k = material safety factor for reinforcing steel = 1.15

η = 1.0 [for $f_{ck} \le 60$ MPa]
= 1.0 - (f_{ck} - 60) / 250 [for $60 < f_{ck} \le 110$ MPa]
λ = 0.8 [for $f_{ck} \le 60$ MPa]
= 0.8 - (f_{ck} - 60) / 500 [for $60 < f_{ck} \le 110$ MPa]

B-1.2 Bending Moment with Partial Shear Interaction

B-1.2.1 Partial shear connection is applicable only in the case of plastic resistance of sections with plastic and compact classifications. Provisions for partial shear connection is applicable either for attaining economy without losing much in moment capacity of the composite section or in conditions where the number of shear connectors required for full shear interactions cannot be provided due to lack of space or increased spacing of shear connectors due to provision of lesser numbers of shear connectors than that needed for full composite action.

Degree of shear connection S_c is given as

$$S_c = \frac{n_p}{n_f} = \frac{F_{cp}}{F_{cf}} = \frac{M - M_{ds}}{M_d - M_{ds}}$$

n_p = Number of shear connectors provided for partial shear connection,

n_f = Number of shear connectors required for full shear connection,

F_{cp} = Capacity of shear connectors in partial shear connection with n_p numbers of connectors,

F_{cf} = Capacity of shear connectors in full shear connection with n_f numbers of connectors,

M = Required bending resistance of the section,

M_d = Design bending resistance of the composite section

M_{ds} = Design bending resistance of steel section alone

To obtain required bending resistance M, the number of shear connectors required (assuming all connectors have equal capacity) is given as:

$$n_p = n_f \{(M - M_{ds})/(M_d - M_{ds})\}$$

B-1.3 SAGGING BENDING RESISTANCE OF COMPOSITE SECTION WITH NON-COMPACT STRUCTURAL STEEL BEAMS

Since the compression flange of semi-compact steel sections buckle locally under compression on reaching yield stress f_y, the design resistance of the composite section consisting of semi-compact sections is evaluated using equations of compact sections as per **B-1**, wherein the effective width of elements of section like the compression flange is restricted to limiting value of that of the compact section.

B-2 MOMENT OF RESISTANCE OF COMPOSITE SECTION (NEGATIVE MOMENT-CONTINUOUS BEAMS)

Figure 28 shows stress distribution across a composite beam section subjected to hogging bending moment. Since the steel bottom flange is in compression section classification shall be done as per [6-6B(5)]. For classification of the web, the distance, $\bar{y}$, of the plastic neutral axis above the centroid of the area of the steel section shall first be found.

B-2.1 Hogging Bending Resistance for Plastic and Compact Structural Steel Sections

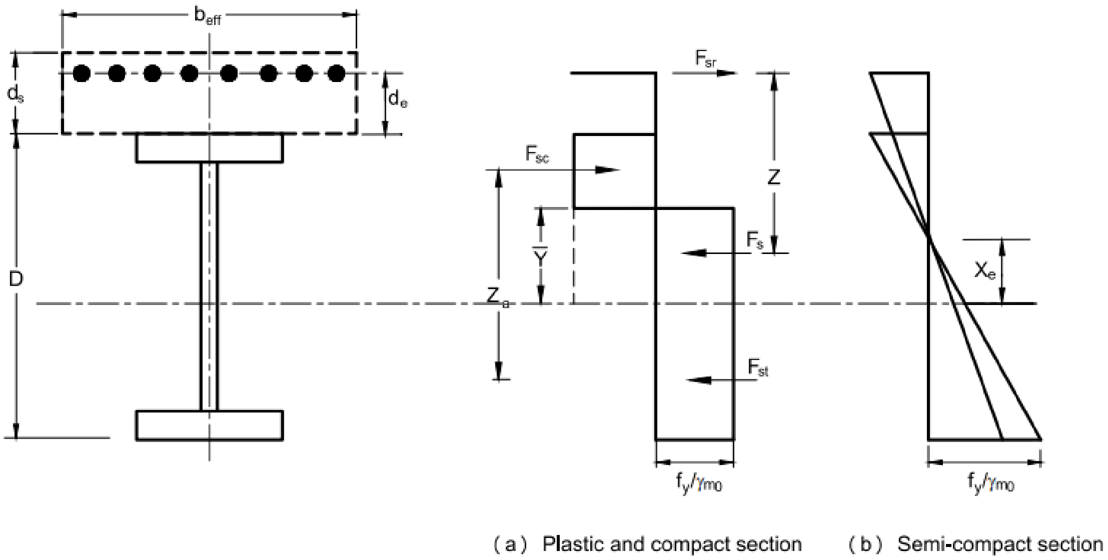


FIG. 28 STRESS DIAGRAM FOR HOGGING MOMENT REGION AT ULTIMATE MOMENT

The design tensile force in reinforcement is given as:

$$F_{sr} = f_{yk}.A_{st}/\gamma_k,$$

Where,

γ_k = partial safety factor for reinforcing steel = 1.15

f_{yk} = Characteristic yield strength of the reinforcing steel

A_{st} = the effective area of longitudinal reinforcement within the effective width b_{eff} of the beam

The plastic moment of resistance for plastic and compact structural steel section is given as:

$$M_{ds} = \frac{Z_p.f_y}{\gamma_{m0}}$$

Where,

Z_p is the plastic section modulus of the structural steel section and γ_{m0} is the material safety factor to be taken as 1.10. In the absence of any tensile reinforcements, the bending resistance of the section would be that of the structural steel section as given by M_{ds} above. To allow for reinforcements, it is assumed that the stress in a depth $\overline{y}$ changes from tension to compression for Plastic and compact section. The corresponding depth for a non-compact section is x_e. For plastic and compact sections, stress distribution $\overline{y}$may be determined from

$$\overline{y}\, t_w \frac{2f_y}{\gamma_{m0}} = F_s$$

The locations of the neutral axis and the bending resistance for the plastic and compact section are given in Table 18.

Table 18 Hogging Bending Resistance of plastic and compact Composite Section

(*Clause* B-2.1)

Case	Position of Plastic Neutral Axis	Value of $\overline{y}$	Moment Capacity M_{dh}
(1)	(2)	(3)	(4)
1	In Web	$\overline{y} \leq \frac{D}{2} - t_f$	$M_{dh} = M_{ds} + F_{sr} Z$
2	In Flange	$\frac{D}{2} \geq \overline{y} > \frac{D}{2} - t_f$	$M_{dh} = F_b \frac{D}{2} + F_{sr}\, d_s - \frac{(F_b - F_{sr})^2}{F_f} \frac{t_f}{4}$

Where,

$Z = \frac{D}{2} + d_s - \frac{\overline{y}}{2}$

F_f = Axial capacity of a single flange $= \frac{f_y.A_f}{\gamma_{m0}}$

F_b = Axial capacity of steel section $= \frac{f_y.A_s}{\gamma_{m0}}$

F_{sr} = Axial capacity of reinforcing steel $= \frac{f_{yk}.A_{st}}{\gamma_k}$

A_s = area of structural steel cross section

A_f = area of top flange of structural steel cross section

A_{st} = area of reinforcing steel within the effective width of concrete flange

d_s – overall depth of the concrete slab

d_e = Effective depth of the slab

t_f = Average thickness of the top flange of the steel section

t_w = Thickness of the web of the steel section

NOTE – The web shall be classified as being in compression throughout.

B-2.2 Hogging Bending Resistance for Non-Compact Section

Where elastic analysis is used, creep is considered in the choice of modular ratio m ($=E_s / E_{cm}$). Here, at the section considered, the loading causes hogging bending moment $M_{e(s)}$ in the steel member alone and ($M_{e(c)}$) in the composite member.

The location of the elastic neutral axis of the composite semi-compact section (*see* Fig. 28(b)) is given as:

$$x_e(A_s + A_{st}) = A_{st}\left(\frac{D}{2} + d_s\right)$$

and the second moment of area of the composite section is:

$$I_{co} = I_s + A_s.x_e{}^2 + A_{st}\left(\frac{D}{2} + d_s - x_e\right)^2$$

Where, I_s is the second moment of area of the steel section alone.

The yield moment is mostly governed by the total stress in the steel bottom flange. The locations of the neutral axis and the moment of resistance for non-compact section is given in Table 19.

The bending moment ($M_{e(s)}$) causes no stress in the slab reinforcements. In propped construction, the tensile stress in the reinforcement may govern the design. It is given in the following equation:

$$\sigma_{sr} = \frac{\left(\frac{f_y}{\gamma_{m0}} - f_s\right)\left(\frac{D}{2} + d_s - x_e\right)}{\left(\frac{D}{2} + x_e\right)} \leq \frac{f_{yk}}{\gamma_k}$$

Table 19 Hogging Bending Resistance of Non-compact Composite Section

(*Clause* B-2.2)

Location of Neutral Axis	**Moment capacity of Steel section alone [f_s] $\bar{y}$**	**Moment Capacity $M_{d(c)}$**
$x_e(A_s + A_{st}) = A_{st}\left(\frac{D}{2} + d_s\right)$	$f_s = M_{e(s)} \cdot \frac{D/2}{I_s}$	$M_{d(c)} = M_{e(s)} + \frac{\left(\frac{f_y}{\gamma_{m0}} - f_s\right) I_{co}}{\left(\frac{D}{2} + x_e\right)}$
NOTE - The web shall be classified as being in compression throughout.		

Where,

A_{st}= Cross sectional area of reinforcements within the effective width of the concrete flange
A_s= Cross sectional area of steel beam of a composite section
d_s= Overall depth of the slab
d_e= effective depth of the slab
$M_{e(s)}$ = Hogging moment in the steel section alone
f_s= Compressive stress in steel flange due to moment $M_{e(s)}$
I_s= moment of inertia of the steel beam alone
I_{co}= moment of inertia of the composite section
t_f= average thickness of the top flange of the steel section
t_w=Thickness of the web of steel section

B-3 FLANGE STRESS REDUCTION FACTOR (R_h) FOR HYBRID SECTIONS

Flange stress reduction factor is applicable for hybrid sections using higher-grade steel flanges where the section is non-compact, or in other words, where the section reaches the plastic moment capacity. In such cases, design limiting stress for both compression and tension shall be modified by the reduction factor R_h and shall be taken as:

$$f_n = R_h \,.\, f_{yf} / \gamma_{m0}$$

where,

$R_h = (D.f_{yw}) / (d.f_{yf}) \leq 1.0$
f_{yf}, f_{yw} = the yield strength of the flange and web respectively

D, d = the distance of the extreme compression/tension flange and the corresponding extreme fiber of the web from the neutral axis of the composite section respectively.

For homogeneous grade steel sections, R_h shall be 1.0. The reduction factor shall not apply to compact or plastic sections because the effect of lower strength material in the web is accounted for in calculating the plastic moment as specified in **B-1.1**.

B-4 MOMENT OF RESISTANCE AGAINST LATERAL-TORSIONAL BUCKLING (CONSTRUCTION STAGE)

At the construction stage, before the concrete hardens, the structural steel beam strength may be dictated by the lateral buckling strength. This may be evaluated as per [6-6B(5)].

Effect of lateral-torsional buckling on flexural strength need not be considered if $\lambda_{LT} \leq 0.4$.

B-4.1 Sagging Lateral Buckling Resistance of Structural Steel Beams

The design buckling resistance moment of a laterally unrestrained beam under un-propped condition during the construction stage shall be taken as:

$$M_d = \beta_b Zp \left(\frac{\chi_{LT} f_y}{\gamma_{m0}}\right)$$

where,

$$\chi_{LT} = \frac{1}{[\varphi_{LT}+(\varphi_{LT}{}^2-\lambda_{LT}{}^2)^{\frac{1}{2}}]} \leq 1.0$$

and

and $\varphi_{LT} = 0.5\left[1 + \alpha_{LT}(\lambda - 0.2) + \lambda_{LT}{}^2\right]$

$\alpha_{LT} =$ As per Table 13

The non-dimensional slenderness ratio, λ_{Lt}, is given by:

$$\lambda_{LT} = \sqrt{\frac{\beta_b Z_p f_y}{M_{cr}}} \leq \sqrt{\frac{1.2\, Z_e f_y}{M_{cr}}}$$

$$\lambda_{LT} = \sqrt{\frac{f_y}{f_{crb}}}$$

Where

β_b = 1.0 for Plastic and Compact sections.
= Z_e / Z_p for semi-compact sections.

M_{cr} = the elastic critical moment corresponding to lateral torsional buckling.

In the case of simply supported prismatic members with symmetric cross-section, the elastic critical moment, M_{cr}, can be determined as:

$$M_{cr} = \sqrt{\left\{\left(\frac{\pi^2 EI_y}{(L_{LT})^2}\right)\left[GI_t + \frac{\pi^2 EI_w}{(L_{LT})^2}\right]\right\}} = \beta_b Z_p f_{crb}$$

The maximum bending compressive stress corresponding to lateral buckling, $f_{cr.b}$ of non-slender rolled I-sections may be approximately calculated using the following equation:

$$f_{crb} = \frac{1.1\pi^2 E}{(L_{LT}/r_y)^2}\left[1 + \frac{1}{20}\left(\frac{L_{LT}/r_y}{h_f/t_f}\right)^2\right]^{0.5}$$

The following simplified conservative equation may be used in the case of prismatic members made of standard rolled I-sections and welded doubly symmetric I-sections for calculating the elastic critical lateral buckling moment.

$$M_{cr} = \frac{\pi^2 EI_y h_f}{2L_{LT}^2}\left[1 + \frac{1}{20}\left(\frac{L_{LT}/r_y}{h_f/t_f}\right)^2\right]^{0.5} = \beta_b Z_p f_{cr,b}$$

where

$f_{cr,b}$ = lateral torsional buckling stress,
I_w = warping constant,
I_t = torsional constant = $\sum b_i t_i^3 / 3$ for open sections,
I_y = moment of inertia about the weak axis,
r_y = radius of gyration of the section about the weak axis],
L_{LT} = effective length for lateral torsional buckling,
h_f = Center to center distance between flanges, and
t_f = thickness of the flange.

M_{cr} for different beam sections, considering loading, support condition, and non-symmetric cross-section, shall be more accurately calculated using the method given in [6-6B(5)].

B-4.2 *Hogging Lateral Buckling Resistance of Composite Beams*

The effective unsupported length, L_{LT}, in case of bare steel section may be taken as recommended in [6-6B(5)]. When a slab connects in the hogging moment region the top flange of two or more beams running parallel, then the effect of U-Frame action in restraining the lateral buckling of bottom flange under compression (Fig. 29) may be considered as given below:

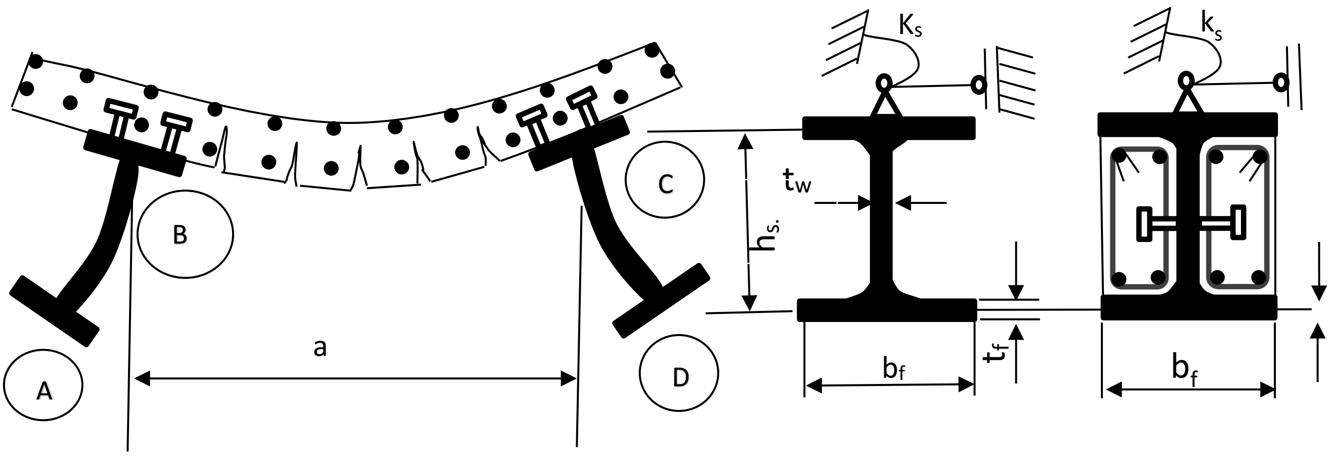


FIG.29 INVERTED U-FRAME ACTION TO RESTRAIN LATERAL BUCKLING OF BOTTOM FLANGE

B-4.2.1 The rotational spring restraint stiffness of the concrete slab, k_s, per unit length of steel beam is given by

$$k_s = k_1 k_2 / (k_1 + k_2)$$

where
k_1 = Flexural stiffness of the cracked concrete or composite slab transverse to the spanning direction of steel beam = α $(EI)_s/a$
α = 2 for an edge beam,
= 3 for inner beam and
= 4 for inner beam with four or more such beams running parallel and supporting the continuous slab;
a = span length of the slab transverse to the steel beams
$(EI)_s$ = cracked flexural stiffness of the concrete or composite slab per unit width parallel to the steel beam, taken as the lower of the sagging bending slab stiffness of the slab in between the steel beams and hogging bending stiffness of the slab at the supporting steel section;

k_2 = flexural stiffness of the steel web per unit length along the beam,
= $E_s t_w^3/[4(1-\upsilon_s^2)h_s)]$ in un-encased steel section
= $E_s t_w b_c^2/[16 h_s (1+ 4 m t_w/b_c)]$ in encased steel section as per **8.3.8**
E_s = modulus of elasticity of structural steel;
υ_s = Poisson's ratio of structural steel;
t_w = thickness of the steel section web;
b_c = width of concrete encasement (Fig. 27);
h_s = distance between centroids of top and bottom flanges
m =long term modular ratio of encasing concrete as per [6-6B(4)]

B-4.2.2 The torsional rigidity of the steel section may be taken as $G_s I_{st}$, where Gs is the shear modulus of steel, I_{st}, is the St. Venant's torsion constant of the steel section. The torsional rigidity stiffness contribution of encasing concrete may be taken as being equal to $G_c I_{ct}/10$, where G_c, may be taken as equal to 0.3 Es/m and I_{ct} is the St. Venant's torsion constant of encasing concrete of rectangular shape and m is the modular ratio for long term effects.

B-4.2.3 The elastic hogging critical buckling moment of the composite beam at an internal support of a continuous beam is given by

$$M_{cr} = (k_c C_4 / L) [(G_s I_{st} + k_s L^2/\pi^2) E_s I_{ybf}]^2$$

where
I_{ybf} = moment of inertia of the bottom flange about the minor axis of the steel section
I_{zco} = second moment of the equivalent area, A_{co}, of the cracked composite beam section for major axis bending
K_c = composite beam section property = $(h_s I_{zc}/I_{ys})/[(h_s^2/4+(I_{ys}+I_{zs})/A_s)/e+ h_s]$
e = $A_{co} I_{zs} /[A_s Z_c (A_{co}-A_s)]$
h_s = Distance between the centers of the flanges of the steel section
I_{ys}, I_{zs} = Second moment of area of steel section about the minor and major axis, respectively
Z_c = distance between the centroid of steel section and the mid depth of concrete slab.
C_4 = constant depending upon the bending stress distribution over the span length as given in Tables 20 and 21.

Table 20 Linear BMD over the span
(*Clause* B-4.2.3)

BMD	C_4

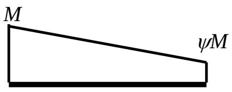
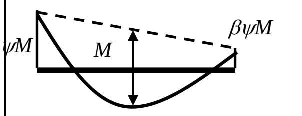
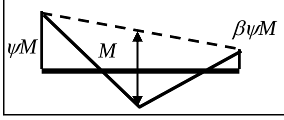
	ψ =-1	-0.75	-0.5	-0.25	0.0	0.25	0.5	0.75	1.0
	18.1	16.3	14.6	12.8	11.1	9.5	8.2	7.1	6.2

Table 21 Non-Linear BMD over the span
(*Clause* B-4.2.3)

BMD	β	C_4								
		ψ =0.5	0.75	1.0	1.25	1.5	1.75	2.0	2.25	2.5
	0.00	41.5	30.2	24.5	21.1	19.0	17.5	16.5	15.7	15.2
	0.50	33.9	22.7	17.3	14.1	13.0	12.0	11.4	10.9	10.6
	0.75	28.2	18.0	13.7	11.7	10.6	10.0	9.5	9.1	8.9
	1.00	21.9	13.9	11.0	9.6	8.8	8.3	8.0	7.8	7.6
	0.00	28.4	21.8	18.6	16.7	15.6	14.8	14.2	13.8	135
	1.00	12.7	9.8	8.6	8.0	7.7	7.4	7.2	7.1	7.0

B-5 MOMENT OF RESISTANCE FOR FILLER BEAM DECKS

Filler Beam decks most likely to be designed for structures with wide column-free zones having higher live loads. The stress distribution diagram for a standard filler beam decks is as shown in Fig. 30.

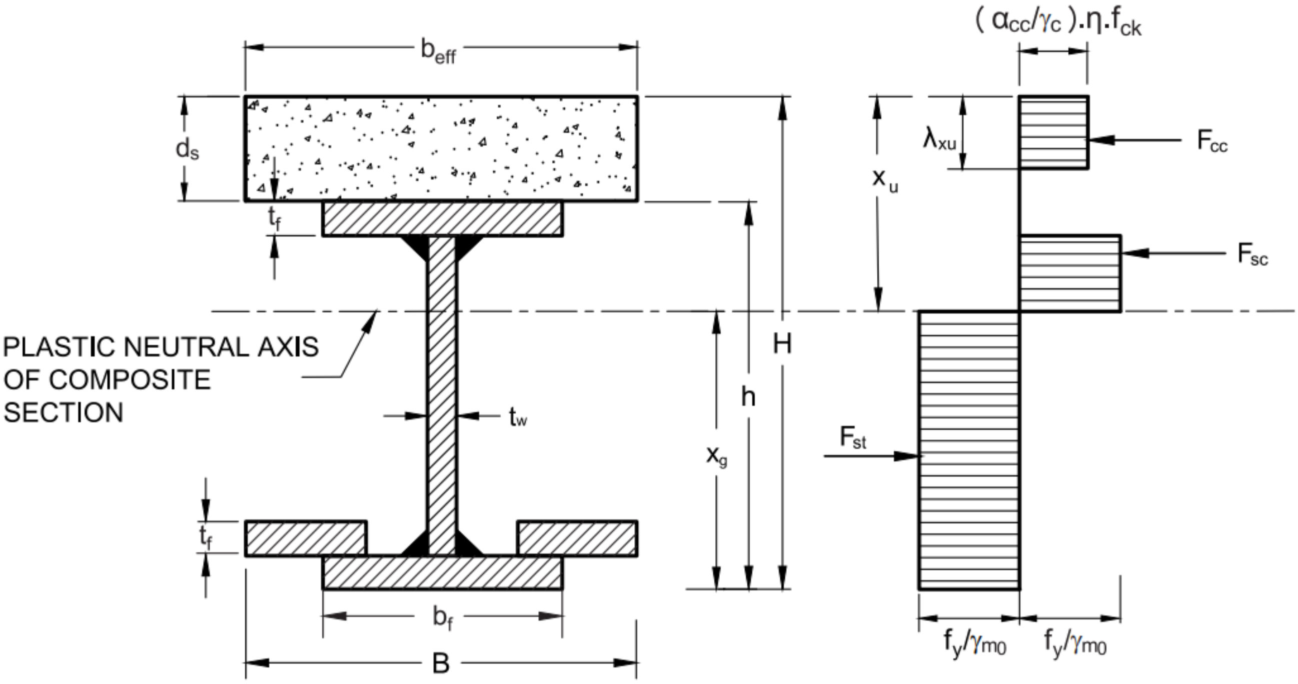


FIG 30 STRESS DIAGRAM FOR FILLER BEAM

For equilibrium, $F_{st} = F_{sc} + F_{cc}$

where,

F_{st} = Tensile force in the steel section below the neutral axis
F_{sc} = Compressive force the steel section above the neutral axis
F_{cc} = Compressive force in the concrete above the neutral axis

The depth of the neutral axis is given as,

$$x_u = H - x_g$$

where,

$$x_g = \frac{\frac{\alpha_{cc}}{\gamma_c}\,\eta\, f_{ck}\,\left[BH - b_f t_f - t_w\left(h - t_f\right)\right] + t_w h \frac{f_y}{\gamma_{m0}}}{\frac{\alpha_{cc}}{\gamma_c}\eta f_{ck}(B - t_w) + 2t_w \frac{f_y}{\gamma_{m0}}}$$

Bending resistance,

$$M_p = F_{sc} \,.\, X_{sc} + F_{cc} \,.\, X_{cc} + F_{st} \,.\, X_{st}$$

where

X_{sc}, X_{cc} *and* X_{st} are respectively the distance between the neutral axis of the composite beam and the individual centre of gravities of the corresponding forces].
B = center-to-center distance between two filler beams
b_{eff} = Effective width of concrete for one filler beam
b_f = width of the top flange of the steel section
t_w =Thickness of the web of steel section
H = Distance between top of concrete and bottom of bottom flange of steel beam
h = Total depth of steel girder
x_g = Distance of neutral axis from bottom of bottom flange of steel beam
A_{st} = Cross sectional area of reinforcements within the effective width of the concrete flange
A_s = Cross sectional area of steel beam of a composite section
d_s = Overall depth of the slab
d_e = effective depth of the slab
M_p = Ultimate bending moment

ANNEX C

COMPOSITE SLABS

(*Clauses* 12.4.3.1 *and* 12.5.2)

C-1 GENERAL

Composite slabs consist of metal decking spanning between the supports and acting integrally with the concrete after the concrete hardens (Fig 31). The steel decking performs several roles as given below:

a) It supports loads during construction and acts as a working platform

b) It develops adequate composite action with concrete to resist the imposed loading

c) It transfers in-plane loading by diaphragm action to vertical bracing or shear walls

d) It may stabilize the compression flanges of the beams against lateral buckling until the concrete hardens.

e) It reduces the volume of concrete in the tension zone of the slab

f) It distributes shrinkage strains, thus preventing serious cracking of concrete.

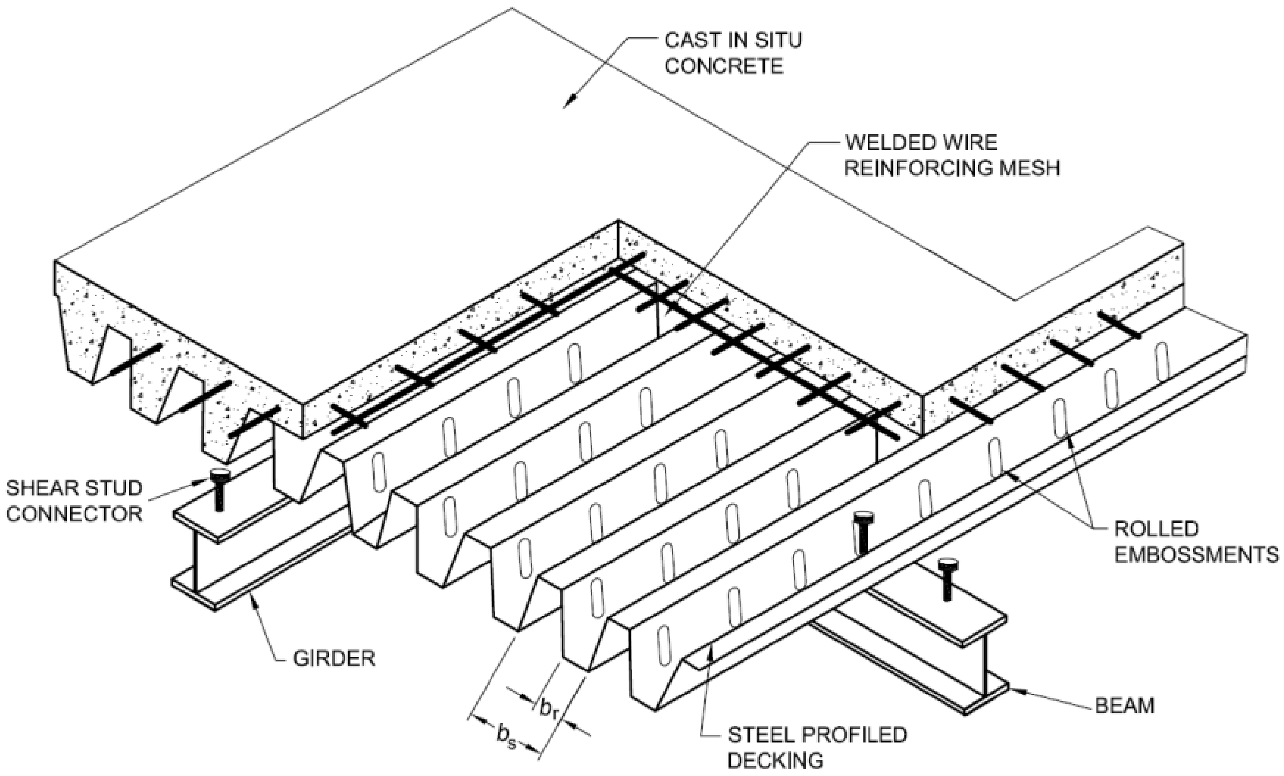


FIG. 31 COMPOSITE SLAB

C-2 STRUCTURAL ELEMENTS

C-2.1 Composite floors with profiled decking consist of the following structural elements along with in-situ concrete and steel beams:

a) Profiled Decking

b) Shear Connectors

c) Reinforcement for shrinkage and temperature stresses

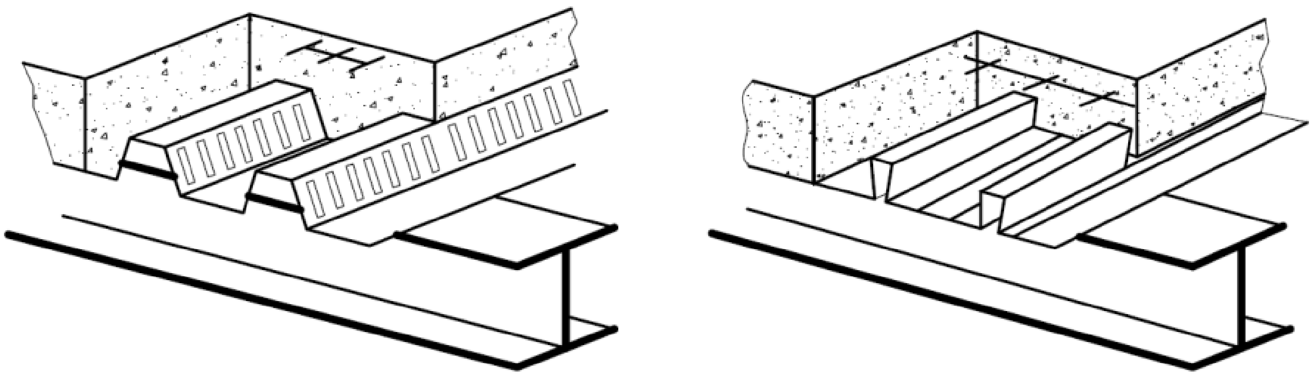
Connections between the structural steel beam and decking elements are generally designed as ‘simple’, that is, not moment-resisting. Stud shear connectors are welded through the sheeting onto the top flange of the beam to facilitate the shear transfer between concrete and steel decking, serving as end anchorage, as well as the shear transfer between the slab and steel beam for mobilising composite action along the beam span.

C-2.2 Profile Decking

The steel deck is normally rolled into the desired profile from 0.9 mm to 1.5 mm galvanized coil sheets. It is profiled such that the profile heights are usually in the range of 38-75 mm, and the pitch of corrugations is between 150 mm and 350 mm. Generally, composite slab spans of the order of *2.5 m* to 3.5 m between the beams are chosen, and the beams are designed to span between 6 m to 12 m. There are two well-known generic types of profiles as given below (Fig.32).

a) Dovetail profile with flange indentations

b) Trapezoidal profile with web indentations



(a) (b)

FIG. 32 COMPOSITE SLAB DECKING PROFILE

C-2.3 Shear Connectors

To enable composite action to be assumed between the profiled steel sheet and the concrete, the longitudinal shear force be transferred by the sheet by the above following form of connection (Fig.33):

a) Mechanical interlock through the provision of indentations or embossments rolled into the profile.
b) Frictional interlock of re-entrant profile.
c) Through-deck welded stud connectors or any other local connection between the steel and the concrete.
d) Deformation of the ends of the ribs at the ends of the sheeting.

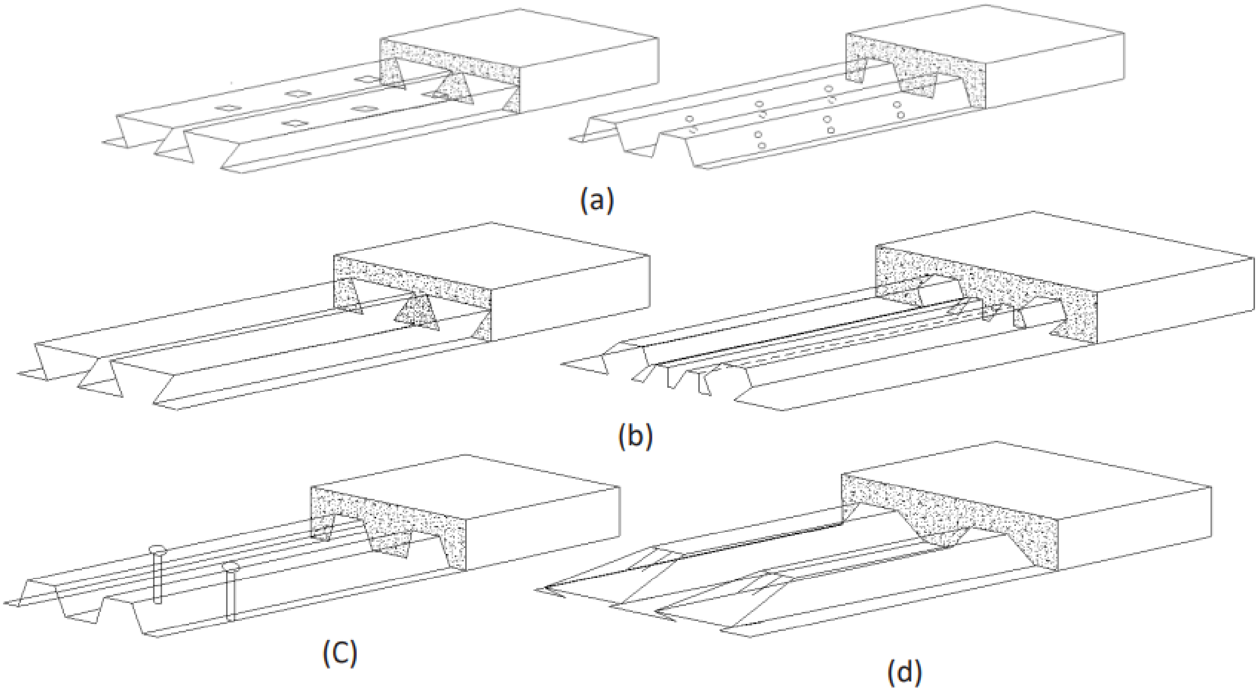


FIG. 33 TYPICAL FORMS OF SHEAR CONNECTORS IN COMPOSITE SLAB

C-2.4 Reinforcement for shrinkage and temperature stresses

In buildings, the temperature difference in the slabs is negligible; thus, there is no need to provide reinforcement to account for temperature stresses. The effect of shrinkage is to be considered, and the total shrinkage strain for design may be taken as 0.003 in the absence of test data.

C-3 PONDING OF PROFILED DECKING

Fig. 34 shows ponding of the profiled deck. In unpropped construction, the profiled decking acting as a formwork to support wet concrete and construction live load. Due to this, the deck sheeting may deflect and cause an increase in the thickness of concrete at or closer to mid-span. This is referred to as 'Ponding.' Ponding increases the volume of concrete consumed as well as dead load. These effects should be accounted for, particularly in long spans.

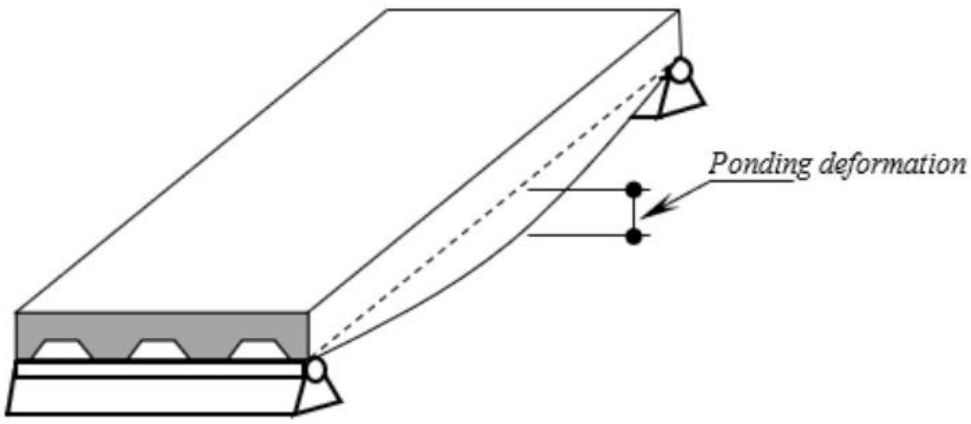


FIG. 34 PONDING IN PROFILE DECKING DUE TO WEIGHT OF CONCRETE SLAB

C-4 BENDING RESISTANCE OF COMPOSITE SLAB WITH FULL SHEAR CONNECTION

NOTE – In **C-4**, instead of the usual F_{cc}, the concrete compressive force is denoted as F_{ccf} to represent the concrete compressive force with a full shear connection.

C-4.1 Sagging Bending Resistance [Neutral axis is above the steel decking (Fig. 35)]

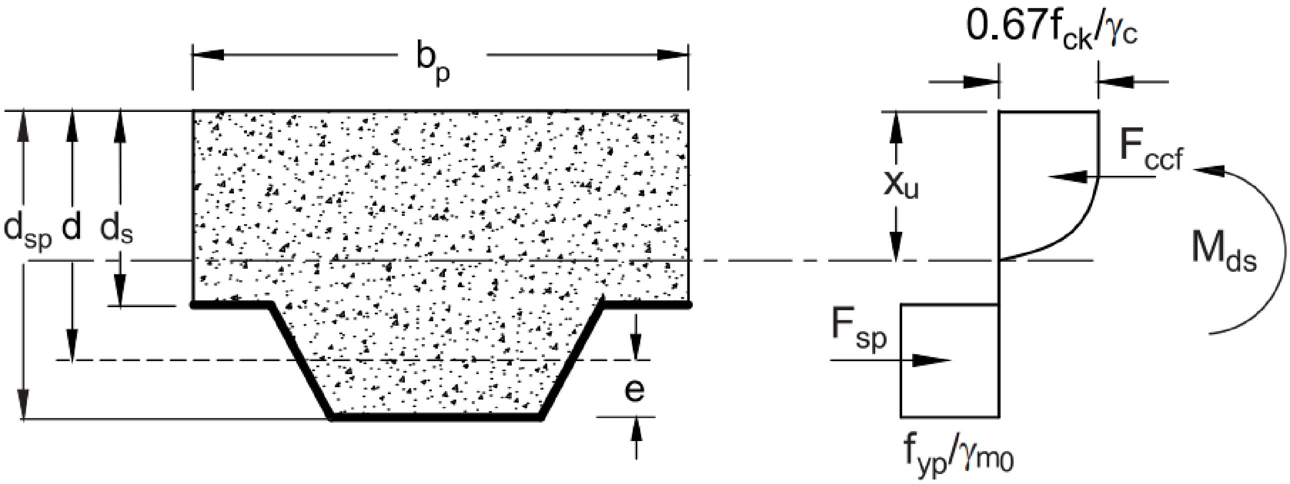


FIG. 35 STRESS DISTRIBUTION UNDER SAGGING BENDING MOMENT ($X_u < D_s$)

The design stress distribution is shown in Fig. 35. This is valid when $x_u \leq d_s$, i.e. when the neutral axis lies above steel decking. Centroid of concrete force lies at $0.42x_u$ from top concrete surface. The equations for the design resistance are given in Table 22.

C-4.2 Sagging Bending Resistance [Neutral axis is within the steel decking (Fig. 36)]

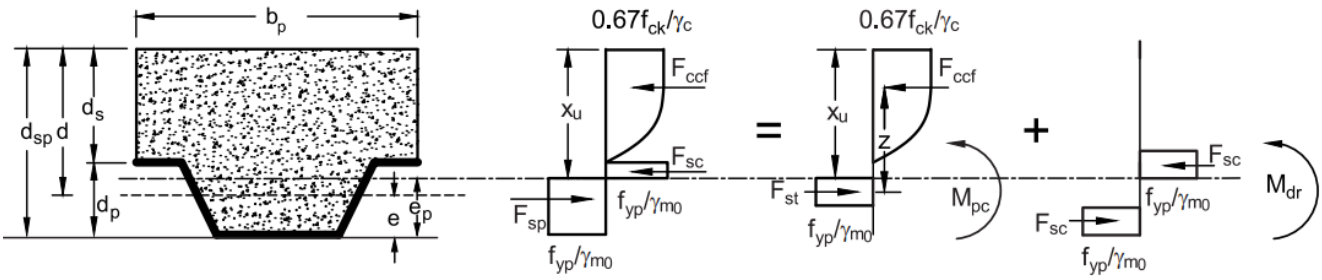


FIG. 36 STRESS DISTRIBUTION UNDER SAGGING BENDING MOMENT ($x_u > d_s$)

The design stress distribution is shown in Fig. 36. This is valid when $x_u > d_s$, that is. when the plastic neutral axis lies in the steel decking. The depth of the parabolic compressive stress block in concrete is between the top of the concrete slab to the top fibre of the deck sheet as shown in Fig.36. Also, e_p is the distance between the plastic neutral axis of the composite slab to the extreme fibre of the deck sheet in tension and e is the distance between the centroidal axis of the profiled steel sheeting to the extreme fibre of the deck sheet in tension. The centroid of concrete force F_{ccf} lies at $0.42d_s$ from top of the concrete surface. The equations for the design resistance are given in Table 22. The value M_{dp} used in Table 22 is plastic moment capacity of the deck sheet which may be provided by the manufacturer or calculated independently.

C-4.3 Hogging bending resistance of composite Deck

The contribution of steel decking is neglected. The design stress distribution is shown in Fig. 37. This is valid when x_u is in the range of $(d_s < x_u < d_{cp})$, that is., when the neutral axis lies in the steel decking. The centroid of concrete force lies at $0.42x_u$ from the bottom concrete surface b_0 the concrete width at the neutral axis is to be evaluated by trial and error. The equations for the design resistance are given in Table 22.

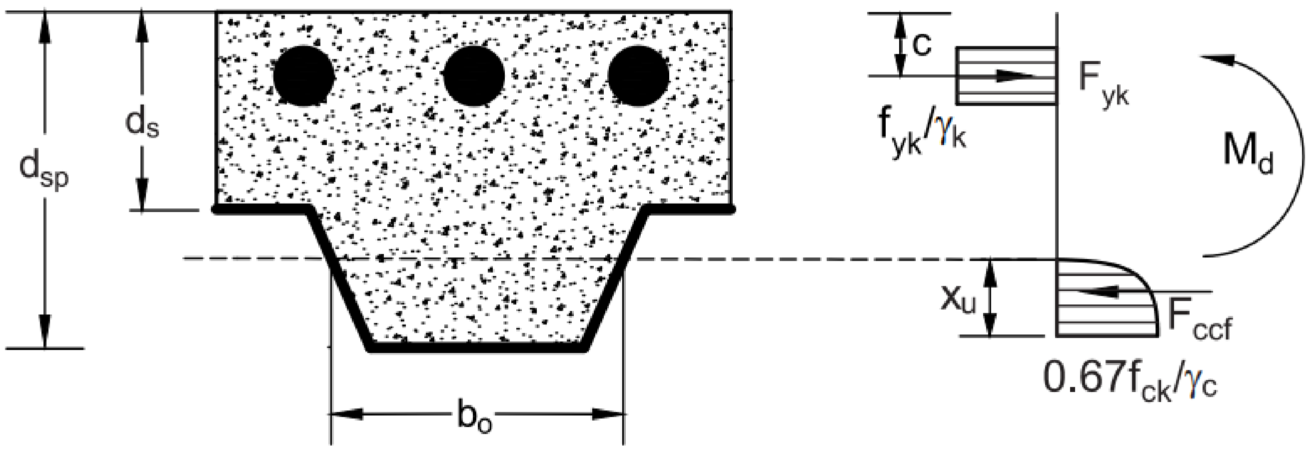


FIG. 37 STRESS DISTRIBUTION UNDER HOGGING BENDING MOMENT (d_s<x_u< d_{sp})

Table 22 Design Moment Capacity, M_{ds}, of Composite Slabs with Full Shear Connection
(*Clauses* C-4.1, C-4.2 *and* C-4.3)

Case	Position of Plastic Neutral Axis	Value of x_u	Design Moment Capacity, M_{ds}
(1)	(2)	(3)	(4)
1	Sagging moment, NA above sheeting material (Fig. 35)	$x_u = \frac{A_p f_{yp}/\gamma_{mo}}{0.36 f_{ck} b_p} \leq d_s$	$M_{ds} = \frac{A_p f_{yp}}{\gamma_{mo}}(d - 0.42x_u)$
2	Sagging moment NA within the steel decking (Fig. 36)	x_u>d_s *A simplified stress block is shown in Fig. 36. Using which M_{ds} is calculated without the need for* x_u	$M_{ds} = F_{ccf} z + M_{dr}$ $M_{dr} = 1.25\, M_{dp}\left[1 - \left(\frac{F_{ccf}}{A_p f_{yp}/\gamma_{mo}}\right)\right] < M_{dp}$ $z = d_{cp} - 0.42 d_S - e_p + \frac{F_{ccf}}{F_{sp}}(e_p - e)$ $F_{ccf} = 0.36 f_{ck} b_p d_s$ $F_{sp} = \frac{A_p f_{yp}}{\gamma_{mo}}$
3	Hogging moment (Fig. 37)	$x_u = \frac{\frac{A_{st} f_{yk}}{\gamma_k}}{0.36 f_{ck} b_o}$	$M_d = \frac{A_{st} f_{yk}}{\gamma_k} \times (d_{sp} - c - 0.42x_u)$

Notations:

A_{st}–area of the reinforcing bar within one wave length b_s of decking sheet
d – distance between the C.G axis of the profiled steel sheeting and the extreme fibre of the composite slab in compression $\left(d_{sp} - e \right)$
e – distance between the C.G axis of the profiled steel sheeting and the extreme fibre of the composite slab in tension
e_p – distance between the plastic neutral axis and the extreme tension fibre of the composite slab.
b_p – distance between the centres of adjacent ribs or one wave length of a profile deck
A_p – effective cross-sectional area of profile decking sheeting
M_{dr} – design bending resistance of profile decking sheeting
M_{ps} _ Plastic moment of resistance of the profile decking sheeting
f_{ck} – characteristic compressive strength of concrete
f_{yp} – yield strength of profile decking sheeting
d_{sp} – total depth of the composite slab $\left(d_{sp} = d_s + d_p \right)$

C.5 BENDING RESISTANCE OF COMPOSITE SLAB WITH PARTIAL SHEAR CONNECTION

NOTE – In **C-5**, instead of the usual F_{cc}, the concrete compressive force is denoted as F_{ccp} to represent the concrete compressive force with a partial shear connection.

C-5.1 The stress distribution in slabs with partial shear connection is similar to Fig. 37. However, the compressive force in the concrete above the neutral axis ($F_{ccp} < F_{ccf}$) is controlled by the interface shear resistance due to the partial shear connection. The interface shear resistance is obtained from tests. The details of tests to obtain the shear strength between the interface of decking and concrete is given in **C-5.2**.

C-5.2 Shear Resistance of Composite Slab

The longitudinal shear behaviour may be considered as ductile if the failure load exceeds the load causing a recorded end slip of 0.1 mm by more than 10%. If the maximum load is reached at a mid-span deflection exceeding *L*/50, the failure load shall be taken as the load at the mid-span deflection of *L*/50. Otherwise, the behaviour is classified as brittle.

The shear resistance due to the chemical bond shall not be considered. This clause is applicable to the composite slabs with the mechanical or frictional interlock. Literature gives two main methods for determining the shear transfer for partial shear connection from test results. They are (i) the m-k method and (ii) the partial shear connection method. The m-k method is valid for both ductile and brittle failure cases but disregards the effect of overhang beyond the support, whereas the partial shear connection method is valid only for ductile behaviour, but accounts for the effect of the support reaction and the overhang beyond the support.

C-5.2.1 *m-k Method for Composite Slabs without End Anchorage*

The m-k method is based on establishing the gradient and intercept of an assumed linear relationship between two parameters of the slab, obtained from two groups of composite slab tests. The evaluation of m-k values is illustrated in Fig. 38.

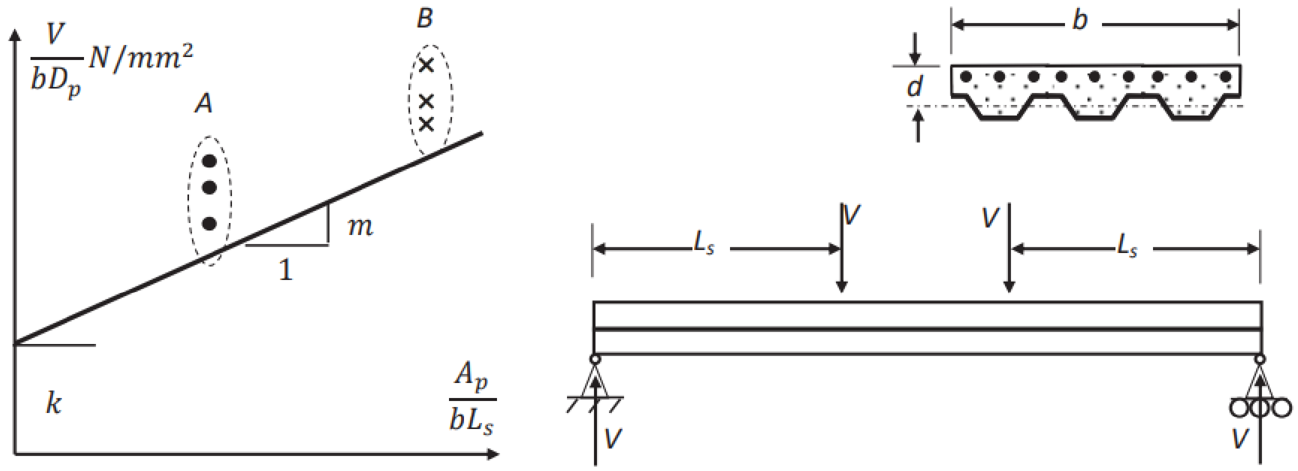


FIG. 38 TEST SETUP TO EVALUATE *M-K* VALUES

For cases when the longitudinal shear behaviour may be considered ductile, V is taken as the value of the support reaction at the failure load. However, if the behavior is brittle, the value should be reduced using a factor 0.8. By plotting the results from the composite slab tests in terms of the vertical shear parameter [$V/(b\ D_p)$] against the shear bond parameter [$A_p/(b\ L_s)$] for two groups of data corresponding to the long specimens (Group-A) and the short specimens (Group-B). The relationship between vertical shear and shear bond capacity is approximated by constructing a straight line through the two groups of data (Fig. 38). The effect of any overhang in the test specimen is neglected in the m-k method, unlike in the partial shear connection approach.

From all the values of V, 5% fractile of the characteristic shear strength linear regression line should be calculated to define the characteristic m and k values (Fig. 38). The minimum value of each group is further reduced by 10% for design consideration if two groups of three tests are used and the deviation of any individual test result in a group does not differ by more than 10% from the mean.

If the m-k method is used, it should be shown that the maximum design vertical shear V for a width of slab 'b' does not exceed the design shear resistance V_{ld} specified in Table 23.

C-5.2.2 *Partial Interaction Method*

The degree of shear connection η is defined as,

$$\eta = \frac{F_{ccp}}{F_{ccf}}$$

where

F_{ccp} = Compressive force in the concrete as governed by partial shear transfer, as obtained from a standard tests
F_{ccf}= Compressive force in the concrete for full shear connection as given in **C-4**.

The variation in bending resistance with the degree of shear connection is shown in Fig. 39. In case $\eta = 0$ composite action between the steel sheet and the concrete does not exist, and it is assumed that the bending resistance is provided by the bending resistance, M_{dp}, of profiled steel sheet alone. For the case $\eta = 1$, in the full shear connection, the full tensile resistance of the sheet is developed, or the full compressive resistance of concrete above the ribs of the sheet is attained. For intermediate cases such that $0 < \eta < 1$, the partial shear connection exists. Typical behaviour of open trough profile steel sheeting with partial shear connection is illustrated in Fig. 39.

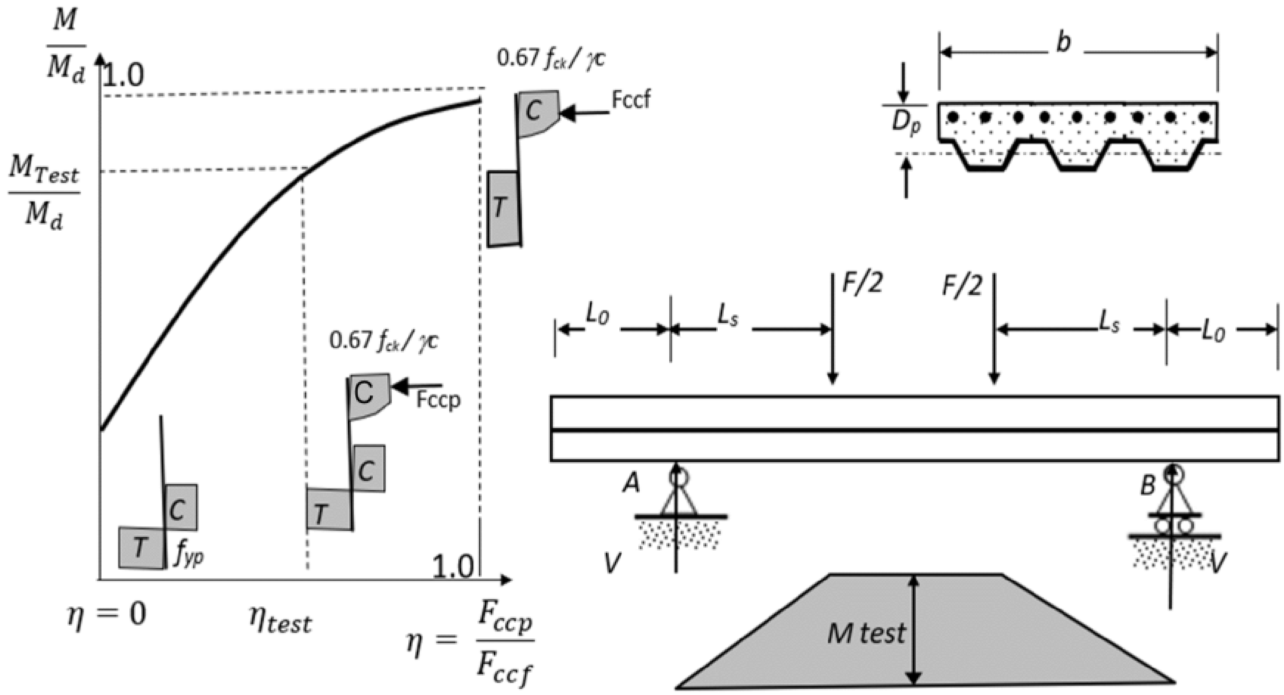


FIG. 39 TEST TO DETERMINATION OF THE DEGREE OF THE PARTIAL SHEAR CONNECTION

For a given bending resistance, the degree of shear connection provided in the test η_{test} can be assessed from the points on the partial shear interaction curve in Fig.39. By neglecting the effect of the support reaction, the longitudinal shear strength τ_u can be obtained from

$$\tau_u = \frac{\eta_{test}\, F_{ccf}}{b\,(L_s + L_o)}$$

where,

F_{ccf} – Compressive normal force in the concrete flange with full shear connection
b - width of the slab
L_s – Shear span
L_o –Length of the overhang (Fig. 39)
If the additional longitudinal shear, the additional resistance caused by the friction due to the support reaction is taken into account, the equation becomes

$$\tau_u = \frac{\eta_{test}\, F_{ccf} - \mu V}{b\,(L_s + L_o)}$$

where,

μ – frictional coefficient taken as 0.5
V – support reaction under the test load

The characteristic shear strength $\tau_{u.fk}$ should be calculated from the test values as the minimum value of stress below which not more than a specified percentage (usually 5 percent) of the corresponding stresses of samples tested are expected to occur. This is divided by the partial safety factor governed by the ultimate state to obtain the design value, $\tau_{u.d}$.

The determination of the design moment resistance of the composite slab with the partial shear connection is similar to the case (Fig. 39). The design bending resistance of the partial shear connection slab is evaluated using equations given in Table 23.

Table 23 Design Moment Capacity, M_{dp}, of Composite Slabs with Partial Shear Connection
(*Clauses* C-5.2.1 *and* C-5.2.2)

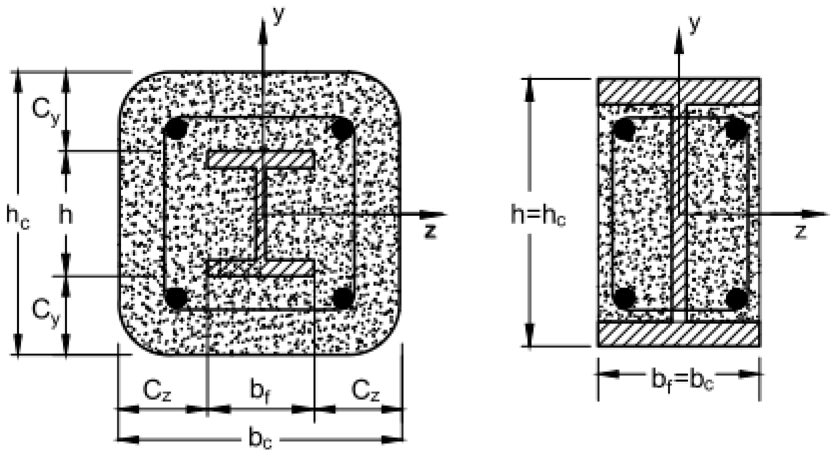
Case	**Position of Plastic Neutral Axis**	**Methods/ Value of x_u**	**Design Moment Capacity, M_{dp}**
(1)	(2)	(3)	(4)
1	sagging moment (Fig. 38)	m-k method	$M_{dp} = V_{lp}\, L_s$ $V_{lp} = \frac{bd_p}{\gamma_{m1}}\left(\frac{mA_p}{bL_s} + k\right)$
2	Sagging moment, NA above profile decking sheet (Fig. 39)	Partial shear connection method $x_u = \frac{F_{ccp}}{0.36 f_{ck} b_p} \leq d_s$	$M_{dp} = \frac{A_p f_{yp}}{\gamma_{mo}}(d - 0.42x_u)$ $F_{ccp} = \tau_{ud} b_p L_x \leq F_{ccf}$
3	Sagging moment, NA within the profile decking sheet (Fig. 39)	Partial shear connection method A simplified stress block is shown in Fig.39. Using which M_{dp} is calculated with $x_u = \frac{F_{ccp}}{0.36 f_{ck} b_p}$	$M_{dp} = F_{ccp} z + M_{pr}$ $F_{ccp} = \tau_{ud} b_p L_x \leq F_{ccf}$ $z = d_{cp} - 0.42x_u - e_p + \frac{F_{ccp}}{A_p f_{yp}/\gamma_{mo}}(e_p - e)$ $M_{dr} = 1.25\, M_{dp}\left[1 - \left(\frac{F_{ccp}}{A_p f_{yp}/\gamma_{mo}}\right)\right] < M_{dp}$
Where L_x is the distance of the cross section being considered to the nearest support.			

ANNEX D
PLASTIC DESIGN RESISTANCE OF SYMMETRIC COMPOSITE COLUMNS

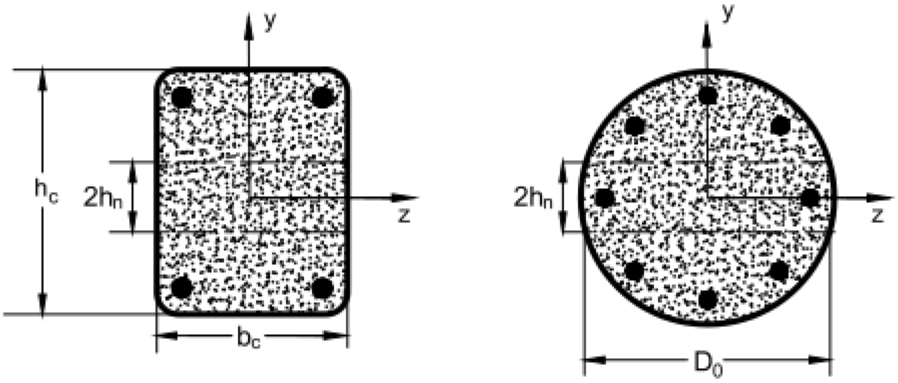
(*Clause* 13.8.1)

D-1 GENERAL

This section deals with the calculation of the design resistance of the doubly symmetric composite columns. The resistance equations are derived based on the rectangular stress block in concrete. The general dimensional view of the fully or partially concrete-encased single I-section is as given in Fig. 40a. The general dimensional view of the concrete infilled tubular composite column section is given in Fig. 40b.



a) Fully and partially concrete encased



b) Concrete infilled tubular columns

FIG. 40 TYPICAL COMPOSITE COLUMN DETAILS

The following are the notations used in this section, in addition to the dimensions shown in Fig. 40.

A_c = cross sectional area of concrete
A_s = cross sectional area of structural steel
A_{st} = cross sectional area of total steel reinforcement
A_{stc} = cross sectional area of steel reinforcement in the compression zone
f_{ck} = characteristic compressive cube strength of concrete

f_y = Minimum Guaranteed yield strength of structural steel section
f_{yk} = Minimum Guaranteed yield strength of steel reinforcement
M_d = Design bending resistance under only bending moment
M_n = Ultimate bending resistance under only bending moment
P_d = Design compressive resistance under compression only
P_n = Ultimate compressive resistance under compression only
M'_d = Design bending resistance under combined bending and compression
M'_n = Ultimate bending resistance under combined bending and compression
P'_d = Design compressive resistance under combined bending and compression
P'_n =Ultimate compressive resistance under combined bending and compression
r =Inner radius of the rounded corners in RHS
γ_c = partial safety factor for the compressive strength of concrete = 1.5
γ_{m0} = partial safety factor for the yield strength of structural steel = 1.1
γ_k = partial safety factor for the yield strength of steel reinforcement = 1.15

D-2 DESIGN COMPRESSIVE RESISTANCE OF SHORT COMPOSITE COLUMN

The stresses on different components of the composite columns at the ultimate stage under only axial force are as shown in Fig. 41. The ultimate design compressive resistance, P_n, of the plastic, compact and semi-compact sections are given by:

$$P_n = A_s f_y + A_{st} f_{yk} + 0.8 A_c \alpha_c f_{ck}$$

where,

α_c = 0.85, for partially or fully encased column
α_c = 1.0, for infilled columns

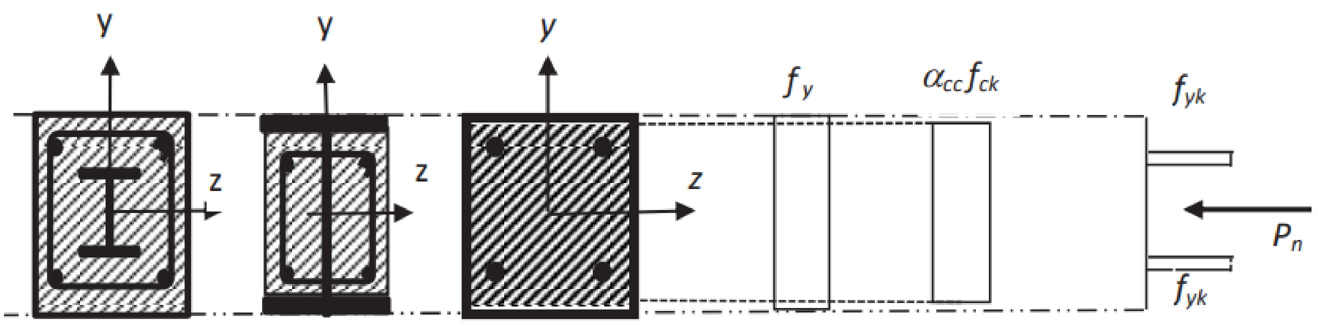


FIG. 41 STRESS DISTRIBUTION AT ULTIMATE STAGE UNDER ONLY AXIAL FORCE

The axial design resistances of different types of the composite column are given in Table 24.

To account for minimum eccentricity and grade of reinforcing steel, the design axial capacity can be estimated using the following expression,

$$P_d = A_s \frac{f_y}{\gamma_{m0}} + \propto_k A_{st} \frac{f_{yk}}{\gamma_k} + 0.48 \propto_c A_c \frac{f_{ck}}{\gamma_c}$$

Where α_k values can be taken as

Grade of Reinforcing Steel	Value of α_k
Fe250	0.90
Fe415	0.82
Fe500	0.77

Table 24 Design Compressive Resistance of Short Composite Compression Members
(*Clause* D-2)

Type of section	Design compressive resistance
(1)	(2)
Partially or fully encased composite section	$P_d = A_s \frac{f_y}{\gamma_{m0}} + A_{st} \frac{f_{yk}}{\gamma_k} + 0.68\, A_c \frac{f_{ck}}{\gamma_c}$
Concrete filled rectangular tubular section	$P_d = A_s \frac{f_y}{\gamma_{m0}} + A_{st} \frac{f_{yk}}{\gamma_k} + 0.8\, A_c \frac{f_{ck}}{\gamma_c}$
Concrete filled circular tubular section	$P_d = A_s \eta_2 \frac{f_y}{\gamma_{m0}} + A_{st} \frac{f_{yk}}{\gamma_k} + 0.8\, A_c \frac{f_{ck}}{\gamma_c} \left[1 + \eta_1 \frac{t}{d} \frac{f_y}{0.8 f_{ck}}\right]$ $\eta_1 = \eta_{10}\left[1 - \frac{10e}{d}\right]$ $\eta_2 = \eta_{20} + (1 - \eta_{20}) \frac{10e}{d}$ $\eta_{10} = 4.9 - 18.5\,\lambda + 17\,\lambda^2 \geq 0.0$ $\eta_{20} = 0.25(3 + 2\,\lambda) \leq 1.0$

D-3 DESIGN BENDING RESISTANCE OF COMPOSITE COLUMN

The stresses on the different components of the composite columns at the ultimate stage under only the bending moment about the z-z axis are as shown in Fig. 42.

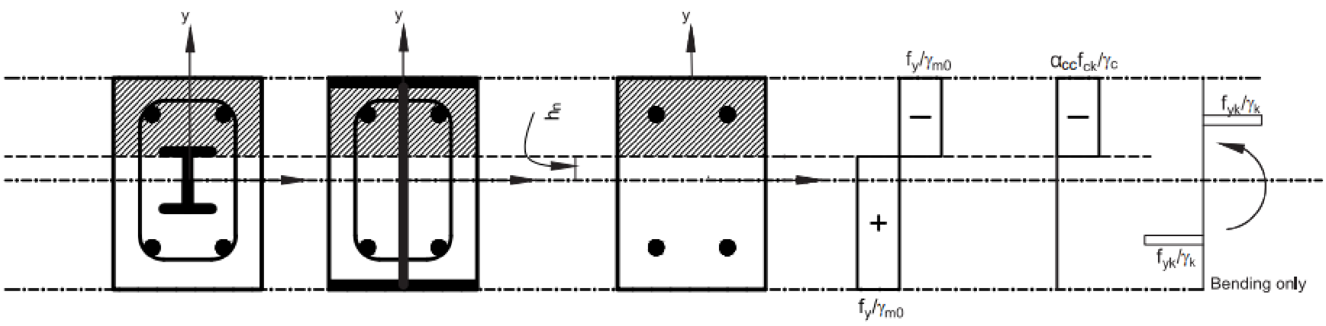


FIG. 42 STRESS DISTRIBUTION AT ULTIMATE STAGE UNDER ONLY BENDING MOMENT

The design Bending resistance is given by:

$$M_d = (Z_{ps} - Z_{psn})\, f_y / \gamma_{m0} + (Z_{pr} - Z_{prn})\, f_{yk} / \gamma_k + 0.4\,(Z_{pc} - Z_{pcn})\, \alpha_c\, f_{ck} / \gamma_c$$

where,

Z_{ps}, Z_{pr}, and Z_{pc} = Plastic section moduli of the steel section, reinforcement and concrete about their own centroidal axis, respectively (the concrete in tension region is assumed to be uncracked for this calculation).

Z_{psn},, Z_{prn} and Z_{pcn} = Plastic section moduli of the steel section, reinforcement and concrete, all within the depth h_n on both sides of the neutral axis of the gross section, about the neutral axis of gross section, respectively. (Table 25 gives equations for evaluating the value of h_n)

α_c = the strength coefficient of concrete,
= 0.85 for fully and partially encased composite sections,
= 1.0 for concrete filled hollow sections and
= 0.89 for fully or partially encased concrete columns with spiral ties

Table 25 Location of Neutral Axis in Composite Column at Ultimate Moment
(*Clause* D-3)

Section	***Case***	***Depth, h_n***
(1)	(2)	(3)
a)	*Major axis Bending*	
Concrete encased sections	*$h_n \leq$ [$h/2 - t_f$]*	$\dfrac{A_c\,0.8\,a_c\,\dfrac{f_{ck}}{\gamma_c} - A_{src}\left(\dfrac{2f_{yk}}{\gamma_k} - 0.8\,a_c\,\dfrac{f_{ck}}{\gamma_c}\right)}{2b_c\,0.8\,a_c\,\dfrac{f_{ck}}{\gamma_c} + 2\,t_w\left(2\dfrac{f_y}{\gamma_{m0}} - 0.8\,a_c\dfrac{f_{ck}}{\gamma_c}\right)}$
	[$h/2 - t_f$] $\leq h_n \leq$ $h/2$	$\dfrac{0.8\,a_cA_c\,\dfrac{f_{ck}}{\gamma_c} - A_{src}\left(2\,\dfrac{f_{yk}}{\gamma_k} - 0.8\,a_c\,\dfrac{f_{ck}}{\gamma_c}\right) - (b-t_w)\left(h-2\,t_f\right)\left(2\,\dfrac{f_y}{\gamma_{m0}} - 0.8\,a_c\,\dfrac{f_{ck}}{\gamma_c}\right)}{1.6\,b_c\,\dfrac{f_{ck}}{\gamma_c} + 2\,b\left(2\dfrac{f_y}{\gamma_{m0}} - 0.8\,\dfrac{f_{ck}}{\gamma_c}\right)}$
	$h/2 \leq h_n \leq h_c/2$	$\dfrac{0.8\,A_c\,\dfrac{f_{ck}}{\gamma_c} - A_{src}\left(2\dfrac{f_{yk}}{\gamma_k} - 0.8\,\dfrac{f_{ck}}{\gamma_c}\right) - A_s\left(2\dfrac{f_y}{\gamma_{m0}} - 0.8\,\dfrac{f_{ck}}{\gamma_c}\right)}{1.6\,b_c\dfrac{f_{ck}}{\gamma_c}}$
Concrete filled sections	*RHS*	$\dfrac{0.8\,A_c\dfrac{f_{ck}}{\gamma_c} - A_{st}\left(2\dfrac{f_{yk}}{\gamma_k} - 0.8\,\dfrac{f_{ck}}{\gamma_c}\right)}{1.6\,b_c\,\dfrac{f_{ck}}{\gamma_c} + 4\,t_w\left(2\dfrac{f_y}{\gamma_{m0}} - 0.8\,\dfrac{f_{ck}}{\gamma_c}\right)}$
	CHS	substitute D and t for b_c and t_w, respectively as an approximation
b)	*Major axis Bending*	
Concrete encased sections	*$h_n \leq t_w/2$*	$\dfrac{0.8\,A_c\dfrac{f_{ck}}{\gamma_c} - A_{src}\left(2\,\dfrac{f_{yk}}{\gamma_k} - 0.8\,\dfrac{f_{ck}}{\gamma_c}\right)}{1.6\,h_c\dfrac{f_{ck}}{\gamma_c} + 2\,h\left(2\dfrac{f_y}{\gamma_{m0}} - 0.8\,\dfrac{f_{ck}}{\gamma_c}\right)}$
	$t_w/2 < h_n < b/2$	$\dfrac{0.8\,A_c\dfrac{f_{ck}}{\gamma_c} - A_{src}\left(2\,\dfrac{f_{yk}}{\gamma_k}s - 0.8\,\dfrac{f_{ck}}{\gamma_c}\right) + t_w\left(2t_f - h\right)\left(2\dfrac{f_y}{\gamma_{m0}} - 0.8\,\dfrac{f_{ck}}{\gamma_c}\right)}{1.6\,h_c\,\dfrac{f_{ck}}{\gamma_c} + 4t_f\left(2\dfrac{f_y}{\gamma_{m0}} - 0.8\,\dfrac{f_{ck}}{\gamma_c}\right)}$

	$b/2 \leq h_n \leq b_c/2$	$$\frac{0.8\,A_c \frac{f_{ck}}{\gamma_C} - A_{st}\left(2\frac{f_{yk}}{\gamma_k} - 0.8\,\frac{f_{ck}}{\gamma_c}\right) - A_s\left(2\frac{f_y}{\gamma_{m0}} - 0.8\,\frac{f_{ck}}{\gamma_c}\right)}{1.6\,h_c\,\frac{f_{ck}}{\gamma_c}}$$
Concrete filled sections	RHS	$$\frac{0.8\,A_c \frac{f_{ck}}{\gamma_C} - A_{st}\left(2\frac{f_{yk}}{\gamma_k} - 0.8\,\frac{f_{ck}}{\gamma_c}\right)}{1.6\,b_c\frac{f_{ck}}{\gamma_c} + 4t_w\left(2\frac{f_y}{\gamma_{m0}} - 0.8\,\frac{f_{ck}}{\gamma_c}\right)}$$
	CHS	Substitute D and t for h_c and t_w, respectively as an approximation

D-4 DESIGN RESISTANCE OF COMPOSITE SHORT COLUMN UNDER COMBINED BENDING AND COMPRESSION

The stresses on the different components of the composite columns at the ultimate stage under only the bending moment about the z-z axis are as shown in Fig. 43.

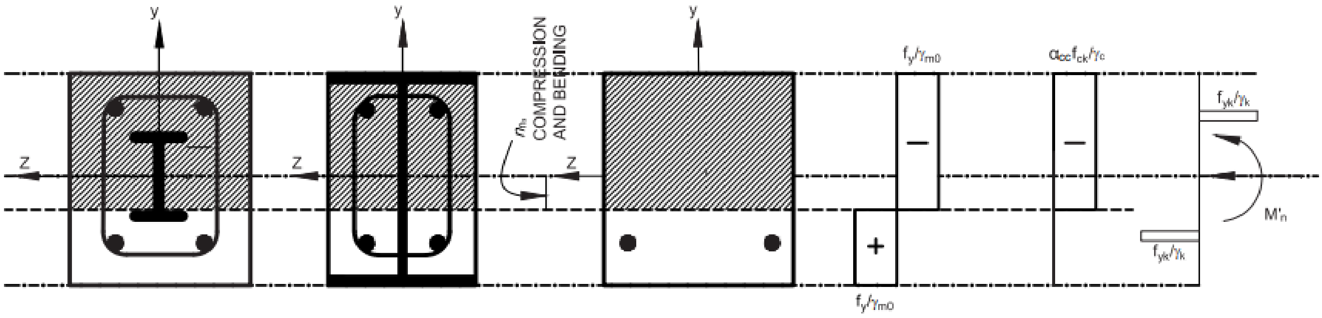


FIG. 43 SHORT COLUMN COMPRESSION AND BENDING INTERACTION

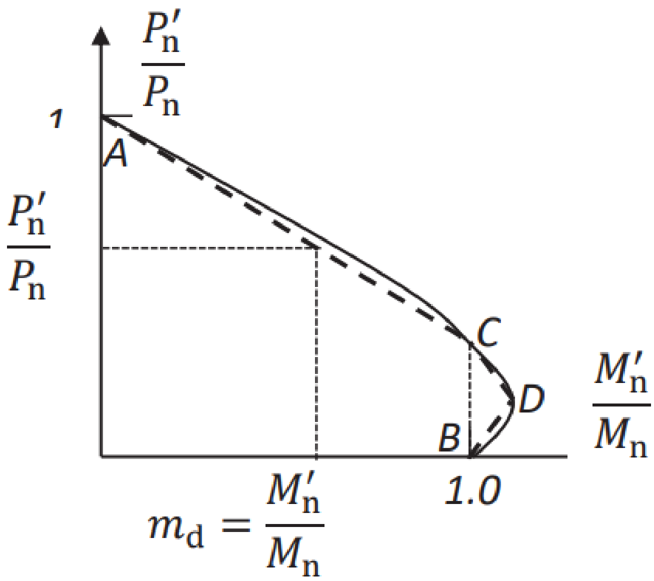


FIG. 44 INTERACTION CURVE OF A COMPOSITE COLUMN SHOWING THE POINTS A, B, C AND D

If the concrete section in compression is below the neutral axis of the gross section, h_n, is assumed. The integration of the stresses over the cross section gives the ultimate nominal compressive resistance, P_n', for the cross section. The integration of the moment of the stresses over the cross section about the neutral axis of the gross section gives the corresponding ultimate bending resistance, M'_n. For various values of h_n, the corresponding simultaneous compressive and bending resistance of the cross section can be calculated to obtain an interaction curve as shown in Fig. 44.

It is seen that the resistance under only compression (Table 23) is a point, (A), on the curve. The resistance under only bending is point, (B), on the curve. Further, from the case of pure bending, as the axial load is increased from zero value, the bending resistance also increases until point D. Similarly, the point C corresponds to the case, where in spite of the axial compression, the ultimate plastic bending resistance, M_n, is reached.

It is tedious and time-consuming to develop the full interaction curve for every section designed. In order to simplify, the values corresponding to the points A, B, C, and D are computed and in between a straight-line variation is assumed in the design. The simplified procedure and the equations are discussed in the following sections.

The values of P_d and M_d corresponding to plastic section under pure compression (point A, Fig. 41) and pure bending (Point B, Fig. 42) are obtained as in **D-2** and **D-3**.

D-4.1 The Design Resistance at Point C (Fig. 45)

The design bending resistance corresponding to point C is the same as pure bending resistance.

$$M'_{d,C} = M_d$$

The design compressive resistance corresponding to point C on the interaction diagram is obtained as given below. The typical position of the neutral axis for points *B* is shown in Fig. 43 and at the point C in the beam-column interaction curve in Fig. 44. The location of the neutral axis from the CG of section, h_n, can be determined from Table 21. The integration of the stresses shown in Fig 46 is equal to $P'_{pd,C}$ at point C. The axial force corresponding to point C in the interaction curve is given by the equation in Table 26.

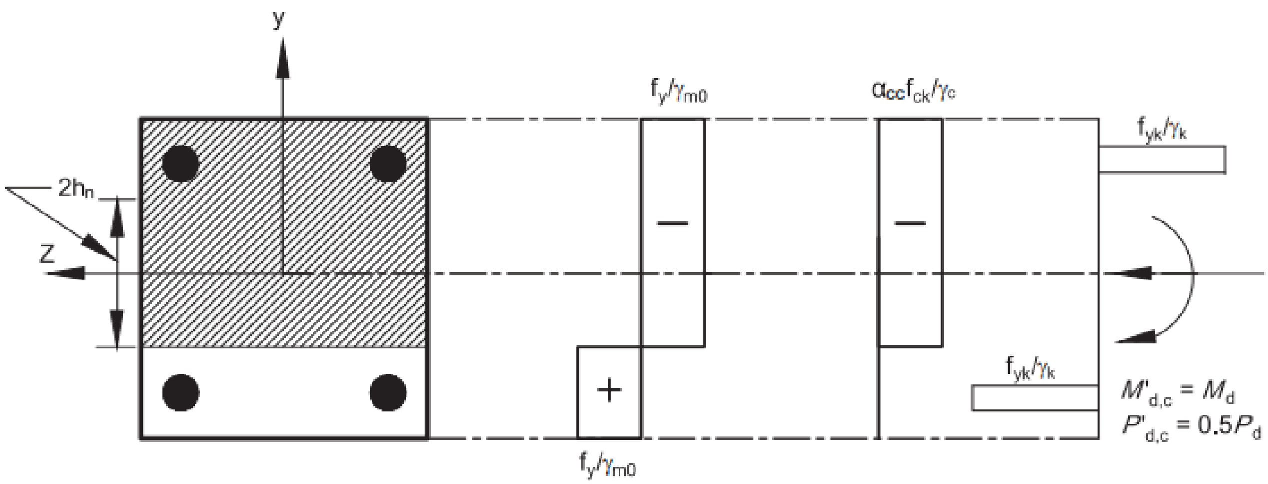


FIG. 45 STRESS DISTRIBUTIONS FOR THE POINT C OF THE INTERACTION CURVE FOR CONCRETE FILLED RECTANGULAR TUBULAR SECTIONS

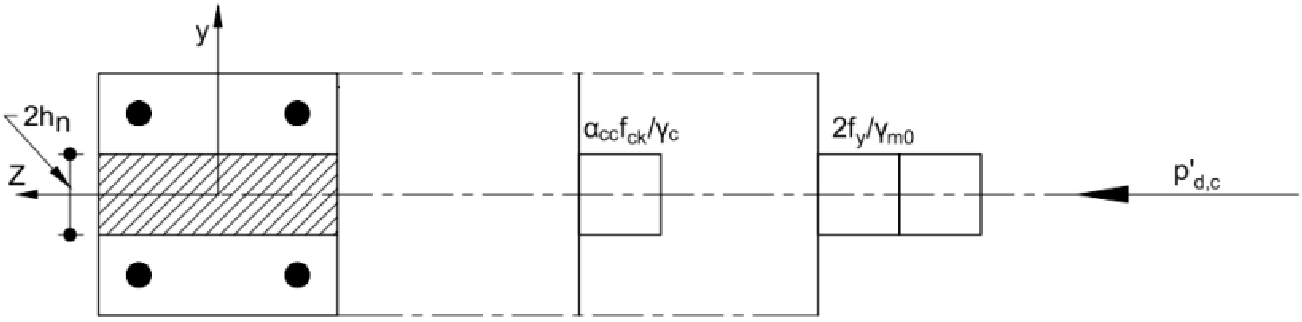


FIG. 46 STRESS CAUSING RESISTING COMPRESSION AT POINT C

Table 26 Design Compressive Resistance at Point C
(*Clause* D-4.1)

Section	**Case**	$P'_{d,C}$
(1)	(2)	(3)
a)	*Major axis Bending*	
Concrete encased column	***$h_n \leq [h/2 - t_f]$***	$P'_{d,C} = 1.6\, h_n b_c a_c \dfrac{f_{ck}}{\gamma_c} + 4 h_n t_w \left(\dfrac{f_y}{\gamma_{m0}} - 0.8\, \dfrac{f_{ck}}{\gamma_c}\right)$
	$[h/2 - t_f] \leq h_n \leq h/2$	$P'_{d,C} = 1.6\, h_n b_c a_c \dfrac{f_{ck}}{\gamma_c} + 4\left(0.5\, A_s (h - h_n)\, b_f\right)\left(\dfrac{f_y}{\gamma_{m0}} - 0.8\, a_c \dfrac{f_{ck}}{\gamma_c}\right)$
	$h/2 \leq h_n \leq h_c/2$	$P'_{d,C} = 1.6\, h_n b_c a_c \dfrac{f_{ck}}{\gamma_c} + (2\, A_s)\left(\dfrac{f_y}{\gamma_{m0}} - 0.8\, a_c \dfrac{f_{ck}}{\gamma_c}\right)$
Concrete filled sections	**RHS**	$P'_{d,C} = 1.6\, h_n b_c \dfrac{f_{ck}}{\gamma_c} + 8 h_n t_w \left(\dfrac{f_y}{\gamma_m} - 0.8 \dfrac{f_{ck}}{\gamma_c}\right)$
	CHS	substitute *D* for b_c and *t* for t_w
b)	*Minor axis Bending*	
Concrete encased column	***$h_n \leq t_w/2$***	$P'_{d,C} = 1.6\, h_n h_c \dfrac{f_{ck}}{\gamma_c} + 4 h_n \mathrm{h} \left(\dfrac{f_y}{\gamma_{m0}} - 0.8\, \dfrac{f_{ck}}{\gamma_c}\right)$
	$t_w/2 < h_n < b/2$	$P'_{d,C} = 1.6\, h_n h_c \dfrac{f_{ck}}{\gamma_c} 2 + \left(4 h_n t_f + t_W \left(h - 2\, t_f\right)\right)\left(\dfrac{f_y}{\gamma_{m0}} - 0.8 \dfrac{f_{ck}}{\gamma_c}\right)$
	$b/2 \leq h_n \leq b_c/2$	$P'_{d,C} = 1.6\, h_n h_c \dfrac{f_{ck}}{\gamma_c} + 2\, A_s \left(\dfrac{f_y}{\gamma_{m0}} - 0.8 \dfrac{f_{ck}}{\gamma_c}\right)$
Concrete filled	**RHS**	$P'_{d,C} = 1.6\, h_n h_c \dfrac{f_{ck}}{\gamma_c} + 8 h_n t_f \left(\dfrac{f_y}{\gamma_{m0}} - 0.8 \dfrac{f_{ck}}{\gamma_c}\right)$

D-4.2 The Design Resistance at Point D (Fig. 44)

The stress diagram on the section corresponding to point D on the interaction curve is shown in Fig. 47.

$$P'_{d,D} = 0.5\, P'_{d,C}$$

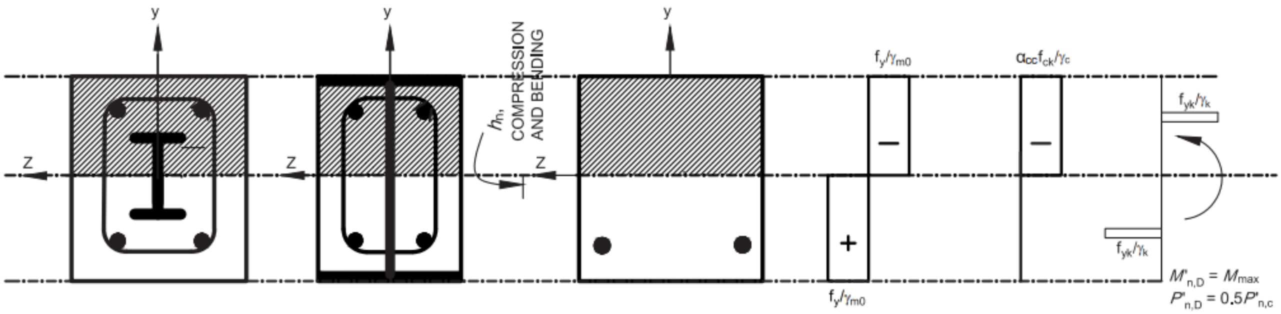


FIG. 47 STRESS DISTRIBUTION AT POINT D AT ULTIMATE STAGE UNDER COMBINED COMPRESSION AND BENDING

$$M'_{d,D} = M_{max} = Z_{ps}\frac{f_y}{\gamma_{m0}} + Z_{pr}\frac{f_{yk}}{\gamma_k} + 0.4a_c Z_{pc}\frac{f_{ck}}{\gamma_C}$$

Knowing the values of the compressive and bending design resistances at the points A, B, C and D the values of the resistance at any other point may be obtained approximately by linear interpolation, as shown by dashed lines in Fig. 44, assuming a linear variation.

ANNEX E
(*Clause* 16)

TESTING

E-1 TESTING OF MATERIALS

Testing of materials shall be done as per standard laid down norms given in **E-1.1** to **E-1.3**.

E-1.1 Concrete

For testing of concrete, reference shall be made to [6-6B(4)].

E-1.2 Steel Sections

All the structural steel and accessories like rivets, bolts, nuts, washers, welding consumables, steel forging, casting, etc., shall be tested for mechanical and chemical properties as applicable and shall conform to requirements of appropriate Indian standards referred to in [6-6B(5)]. Steel shall conform to the requirements of [6-6B(6)] and [6-6B(24)]. Bolts and bolted connection joints with high strength bolts shall be inspected according to [6-6B(25)].

For testing of the strength, flexibility, and other relevant properties of the shear connectors, the test procedures as indicated in **E-1.3** shall be adopted.

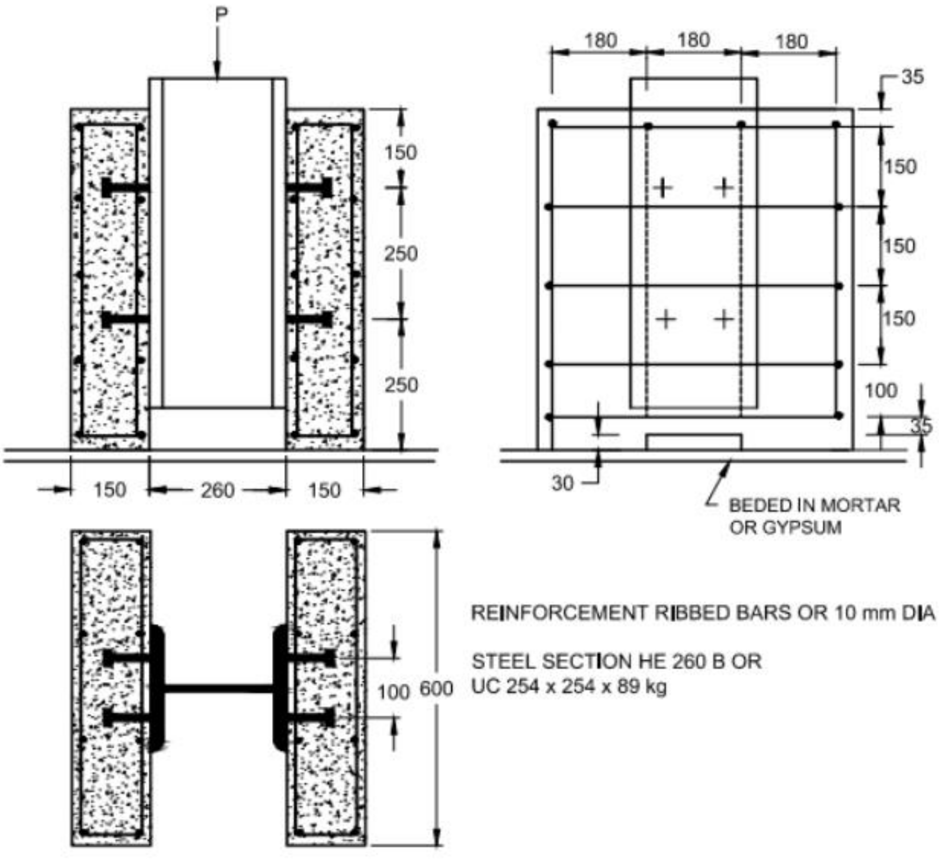


FIG. 48 STANDARD PUSH OUT TEST FOR SHEAR STUDS

E-1.3 Testing of Shear Studs for steel-concrete interface shear.

The nominal static strength of a shear connector may be determined by push-out tests. The conditions and procedures which shall be followed while performing the tests are as indicated below:

a) The dimensions of the standard test are as shown in Fig. 48.
b) The bond at the steel-concrete interface shall be prevented by greasing the flanges or by any other suitable method.
c) The rate of application of load should be uniform and such that the failure load is reached in not less than 10 minutes.
d) The strength of the concrete f_c, at the time of testing, should not differ from the specified cube strength f_{ck} of the concrete by more than $\pm$20 percent.
e) Not less than three tests shall be done, and the nominal static strength P_u shall be taken as the lowest value of $f_{ck}.P / f_c$ for any of the tests, where P is the failure loads of the connectors at concrete strength f_c and f_{ck} is the characteristic cube strength of the concrete.
f) The load-deformation curve, the ultimate resistance of the shear per shear connector, relative slip at ultimate load shall be recorded.

ANNEX F

(*Clauses* 5.3.2, 6.1.2, 8.1.2.1 *and* 11.2.1)

STRENGTH DETAILS AND ELASTIC MODULUS OF NORMAL CONCRETE

Sl No.	**Symbol**	**Grade of Concrete**	**M20**	**M25**	**M30**	**M35**	**M40**	**M45**	**M50**	**M55**	**M60**	**M65**	**M70**	**M75**
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
i)	f_{ck},*(MPa)*	Characteristic cube compressive strength	20	25	30	35	40	45	50	55	60	65	70	75
ii)	f_{cm} *(Mpa)*	Mean concrete compressive strength	30	35	40	45	50	55	60	65	70	75	80	85
iii)	f_{ctm}*(MPa)*	Mean tensile strength of concrete	1.91	2.21	2.50	2.77	3.03	3.28	3.52	3.75	3.97	4.42	4.54	4.66
iv)	f_{ctk} *(0.05)(MPa)*	Characteristic tensile strength at 5% fractile of concrete	1.34	1.55	1.75	1.94	2.12	2.29	2.46	2.62	2.78	3.09	3.18	3.26
v)	E_{cm}*(MPa)*	Elastic modulus	28608	29962	31187	32308	33346	34313	35220	36076	36887	37659	38395	39100

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