(The Official amendments to this document would be published by the IRC in its periodical, 'Indian Highways' which shall be considered as effective and as part of the code/guidelines/manual, etc. from the date specified therein)

IRC:22-2015

STANDARD SPECIFICATIONS AND CODE OF PRACTICE FOR ROAD BRIDGES

SECTION VI

COMPOSITE CONSTRUCTION (LIMIT STATES DESIGN)

(Third Revision)

INDIAN ROADS CONGRESS
2015

STANDARD SPECIFICATIONS AND CODE OF PRACTICE FOR ROAD BRIDGES

SECTION VI

COMPOSITE CONSTRUCTION (LIMIT STATES DESIGN)

(Third Revision)

Published by:

INDIAN ROADS CONGRESS

Kama Koti Marg,
Sector-6, R.K. Puram,
New Delhi-110 022

January, 2015

Price : ₹ 800/-

(Plus Packing & Postage)

First Published	:	October, 1996
Reprinted	:	March, 1971
Reprinted	:	February, 1974
Reprinted	:	November, 1977 (Incorporates Amendment No. 1, February, 1977)
First Revision	:	October, 1986
Reprinted	:	March, 1991 (Incorporates Amendment No. 1, March, 1991)
Reprinted	:	September, 1998
Reprinted	:	July, 2001
Reprinted	:	November, 2002 (Incorporates Amendment No. 2, November, 2002)
Reprinted	:	October, 2005 (Incorporates Amendment No. 3, January, 2003)
Second Revision	:	September, 2008
Reprinted	:	October, 2009
Third Revision	:	January, 2015

Printed by India Offset Press, Delhi

1000 Copies

CONTENTS

PERSONNEL OF THE BRIDGES SPECIFICATIONS AND STANDARDS COMMITTEE

(As on 8th August, 2014)

1.	Das, S.N. (Convenor)	Director General (Road Development), Ministry of Road Transport & Highways, New Delhi
2.	Tawade, D.O. (Co-Convenor)	Addl. Director General, Ministry of Road Transport & Highways, New Delhi
3.	Chief Engineer (B) S&R (Member-Secretary)	Ministry of Road Transport & Highways, New Delhi

Members

4.	Agrawal, K.N.	DG(W), CPWD (Retd.) Ghaziabad
5.	Alimchandani, C.R.	Chairman & Managing Director, STUP Consultants (P) Ltd., Mumbai
6.	Arora, H.C.	Chief Engineer (Retd.), MORT&H, New Delhi
7.	Bagish, Dr. B.P.	C-2/2013, Vasant Kunj, Opp. D.P.S., New Delhi
8.	Bandyopadhyay, Dr. N.	Director, Stup Consultants (P) Ltd., New Delhi
9.	Bandyopadhyay, Dr. T.K.	Joint Director General (Retd.) INSDAG, Kolkata
10.	Banerjee, A.K.	Chief Engineer (Retd.) MoRT&H, New Delhi
11.	Banerjee, T.B.	Chief Engineer (Retd.) MoRT&H, New Delhi
12.	Basa, Ashok	Director (Tech.) B. Engineers & Builders Ltd., Bhubaneswar
13.	Bhasin, P.C.	ADG (B), (Retd.), MoRT&H, New Delhi
14.	Bhowmick, Alok	Managing Director, Bridge & Structural Engg. Consultants (P) Ltd., Noida
15.	Bongirwar, P.L.	Advisor, L&T, Mumbai
16.	Dhodapkar, A.N.	Chief Engineer (Retd.) MoRT&H, New Delhi
17.	Ghoshal, A.	Director and Vice President, STUP Consultants (P) Ltd., Kolkata
18.	Joglekar, S.G.	Vice President, STUP Consultants (P) Ltd. , Mumbai
19.	Kand,, C.V.	Chief Engineer (Retd.), MP, PWD Bhopal
20.	Koshi, Ninan	DG(RD) & Addl. Secy., (Retd) MOST, New Delhi
21.	Kumar, Ashok	Chief Engineer (Retd.), MoRT&H, New Delhi
22.	Kumar, Prafulla	DG (RD) & AS, MoRT&H (Retd.), New Delhi
23.	Kumar, Vijay	E-in-Chief (Retd.) UP, PWD,
24.	Manjure, P.Y.	Director, Freyssinet Prestressed Concrete Co. Mumbai

25.	Mukherjee, M.K.	Chief Engineer (Retd.) MoRT&H, New Delhi
26.	Nagpal, A.K.	Prof. IIT, New Delhi
27.	Narain, A.D.	DG (RD) & AS, MoRT&H (Retd.), New Delhi
28.	Ninan, R.S.	Chief Engineer (Retd.) MoRT&H, New Delhi
29.	Pandey, R.K.	Chief Engineer (Planning), MoRT&H, New Delhi
30.	Parameswaran, Dr. (Mrs.) Lakshmy	Chief Scientist (BAS), CRRI, New Delhi
31.	Raizada, Pratap S.	Vice President (Corporate Affairs). Gammon India Ltd., Mumbai
32.	Rao, Dr. M.V.B.	A-181, Sarita Vihar, New Delh
33.	Roy, Dr. B.C.	Senior Executive Director, M/s. Consulting Engg. Services India (Pvt.) Ltd., Gurgaon
34.	Saha, Dr. G.P.	Executive Director Construma Consultancy (P) Ltd., Mumbai
35.	Sharan, G.	DG (RD) & Spl. Secy (Retd.) MoRT&H, New Dellhi
36.	Sharma, R.S.	Chief Engineer (Retd.) MoRT&H, New Delhi
37.	Sinha, N.K.	DG(RD) & SS, (Retd.) MoRT&H, New Delhi
38.	Subbarao, Dr. Harshavardhan	Chairman & Managing Director, Construma Consultancy (P) Ltd. Mumbai
39.	Tandon, Mahesh Prof.	Managing Director, Tandon Consultants (P) Ltd., New Delhi
40.	Thandavan, K.B.	Chief Engineer (Retd.) MoRT&H, New Delhi
41.	Velayutham, V.	DG (RD) & SS (Retd.) MoRT&H, New Delhi
42.	Viswanathan, T.	7046, Sector B, Pocket 10, Vasant Kunj, New Delhi
43.	The Executive Director (B&S)	RDSO, Lucknow
44.	The Director and Head,	(Civil Engg.), Bureau of Indian Standards, New Delhi

Corresponding Members

1.	Raina, Dr. V.K.	Consultant (World Bank)
2.	Singh, R.B.	Director, Projects Consulting India (P) Ltd. New Delhi

Ex-Officio Members

1.	President, Indian Roads Congress	(Sunil Bhowmik), Engineer-in-Chief, PWD (R&B), Govt. of Tripura
2.	Honorary Treasurer, Indian Roads Congress	(S.N. Das), Director General (Road Development), Ministry of Road Transport & Highways
3.	Secretary General, Indian Roads Congress	

STANDARD SPECIFICATION AND CODE OF PRACTICE FOR ROAD BRIDGES SECTION VI : COMPOSITE CONSTRUCTION (LIMIT STATES DESIGN)

INTRODUCTION

The first Bridge Code in Limit State Philosophy published by IRC in 2008 formatted as a "Stand alone" document with minimum reference to other bridge codes. IRC:24, the basic code for Steel Road Bridges in limit state philosophy was prepared in line with BIS publication IS:800-2007 (General Construction in Steel Limit State Method) was published in 2010. IRC:24 was extensively revised in January 2014. IRC:112, Code for Concrete Road Bridges, in Limit State Method was published in 2011. IRC Technical Committee for Steel and Composite Structures (B-5) felt the need for a through revision of IRC:22 to make it compatible with the revised version of IRC:24 for Steel Road Bridges and IRC:112 for Concrete Road Bridges. The present revised version is a result of such an endeavour.

The personnel of the Steel and Composite Structures Committee (B-5) is given below:

Ghoshal, A.	Convenor
Basa, Ashok	Co-Convenor
Ghosh, U.K.	Member Secretary

Members

Bagish, Dr. B.P.	Mohanty, G.R.
Banerjee, T.B.	Parameswaran Dr. (Mrs.) Lakshmy
Baul, Saibal	Purakayastha, D.
Bhattacharya, A.K.	Roy, Dr. B.C.
Bhattacharya, D	Singh, A.
Chopra, A.K.	Singhal, U.
Ghosh, Prof. Achyut	Sood, V.B.
Guha, Arijit	Subbarao, Dr. H.
Kalyanaraman, Dr. V.	Tandon, Prof. Mahesh
Kumar, Sudhanshu	Venkatram, P.G.
	Vijay, Late P.B.

Corresponding Members

Bhowmick, A.	Singh, R.B.

Ex-officio Members

President, Indian Roads Congress	(Bhowmik, Sunil), Engineer-in-Chief, PWD (R&B), Govt. of Tripura
Honorary Treasurer, Indian Roads Congress	(Das, S.N.), Director General (Road Development), Ministry of Road Transport and Highways
Secretary General, Indian Roads Congress	

The B-5 Committee finally approved the draft document in its meeting held on 12.4.2014 for placing before the BSS Committee. The Bridges Specifications and Standards Committee (BSS) approved the draft document in its meeting held on 8th August 2014. The Executive Committee in its meeting held on 18th August 2014, approved the same document for placing it before the Council. The IRC Council in its 203rd meeting held at New Delhi on 19th and 20th August 2014, approved the draft revision of IRC:22-2014 "Standard Specifications and Code of Practice for Road Bridges" Section VI Composite Construction (Limit States Design) for publishing.

600 GENERAL

600.1 Scope

This code is applicable to simply supported as well as continuous bridges and supporting column systems, with Steel-Concrete Composite Construction. The code is based on Limit States Method of Design.

600.2 Type

This code is restricted to steel-concrete composite construction where steel girders are used as primary members and cast-in-situ reinforced concrete and/or pre-cast concrete slab with necessary in-situ concrete as deck slab. Wherever appropriate, the provisions of this code may be applied to steel-concrete composite elements/components of other types of bridges.

600.3 Terminology

Accidental Load: The load not normally expected in design life but has major impact if it ever occurs, such as ramming of a ship or barge against piers or accidents caused by ramming of vehicles on Bridge Piers of Wall Type, columns or the frames built in the median or in the vicinity of the carriageway supporting the superstructure.

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 integrated action between steel and its encasement or in fill concrete. Shear transfer is to be ensured through use of mechanical devices known as shear connectors in composite beams.

Design Loads: The applied loads multiplied by the load factors.

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

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

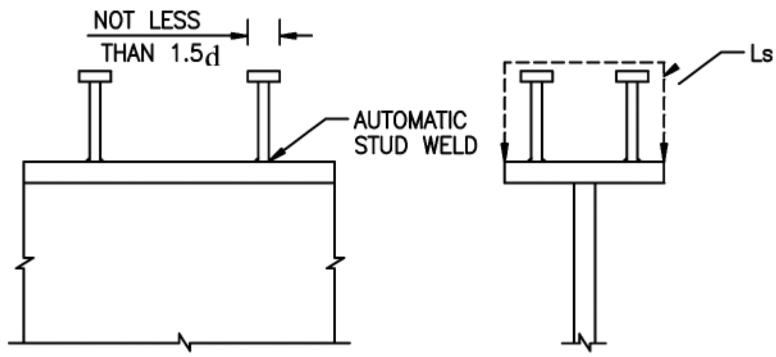
Differential Shrinkages: It is entirely due to shrinkage of concrete from the time composite action comes into effect. When the coefficient of thermal expansion varies significantly between steel and concrete (concrete with limestone or granite aggregate), it also includes the difference in thermal strain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

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

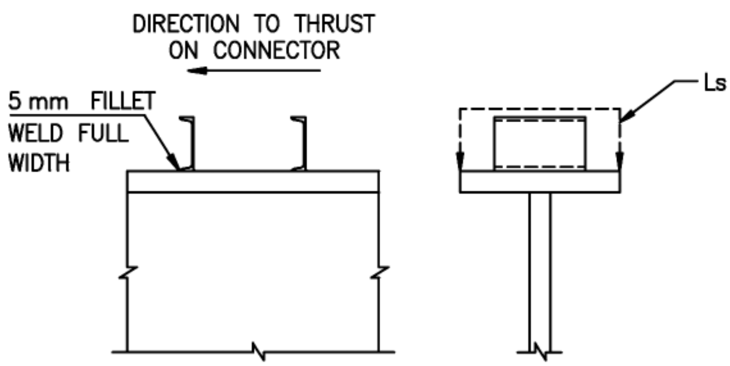
Fatigue Limit State: The state of failure through fatigue damage due to repeated application of loads.

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

Flexible Shear Connectors: Consists of studs, channels etc. welded as in **Fig. 1** to steel member to develop integral action and deriving resistance to shear through the bending of connectors, without permitting the slab to lift from girder flange through anchorage action.



(a) Stud Connector



(b) Channel

Fig. 1 Typical Flexible Shear Connectors

Hybrid Section: A fabricated steel section with a web that has a specified minimum yield strength lower than that of one or both flanges.

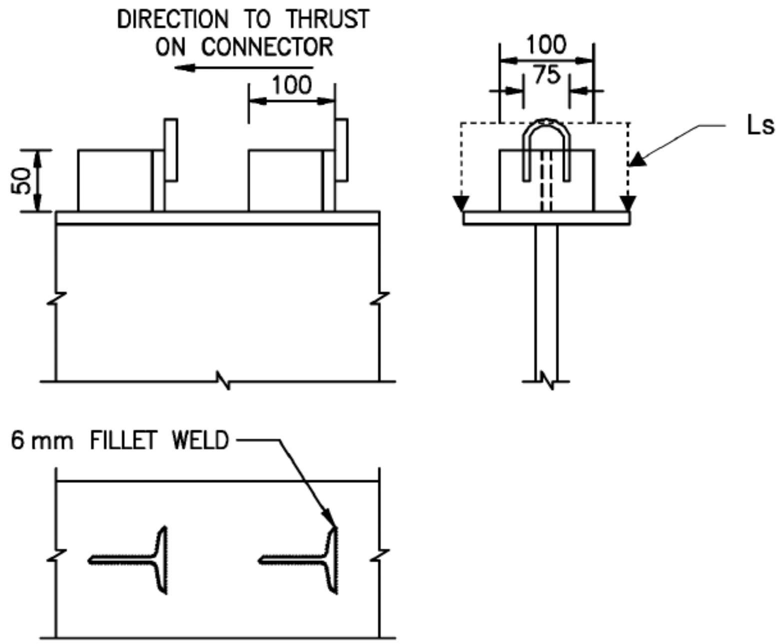
Initial Dead Load: The combination of weight of steel structure and the portion of concrete deck that are supported by the steel structure alone before the development of full composite action with concrete reaching 75 percent of its 28 days strength.

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

Loads: Applied forces as mentioned in IRC:6-2014 that the structure is subjected during its life time.

Load Factors: The factors multiplied with the loads or their combinations to obtain design loads, while checking performance under various limit states.

Rigid Shear Connectors: Consist of bars, angles, channels, tees welded to steel member to develop composite action, as in **Fig. 2**, deriving their resistance to shear from concrete bearing on the vertical face. They exhibit negligible deformation under shear transfer. These are not recommended for adoption.



Tee Connector with Hooped Bars for Anchorage

Fig. 2 Typical Rigid Shear Connectors
(L_s Indicates Lane of Shear (Type)

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.

Serviceability Loads: The actual loads on the structure against which the serviceability of the structure has to be checked.

Shear Connectors: These are the mechanical attachments to steel members to transfer interface shear to develop composite action and are composed of various types, viz. rigid shear connectors, flexible shear connectors etc.

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.

Strength Factors: The factors that divide the specified strength to obtain design strength; while assessing the safety under limit states of strength.

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

Superimposed Dead Load: The dead loads added subsequent to concrete hardening of deck slab that are resisted by composite action.

Transient Load: The loads that are assumed to be varying over a short time interval like vehicular live load with dynamic effect, pedestrian load, braking and tractive forces, temperature effects, wind loads on structure and vehicles, earthquake loads, vehicular centrifugal forces, accident loads such as vehicular collision loads, etc.

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

600.4 Symbols

The symbols, other than those used for load categorization as per **Clause 601**.3, are as follows:

A	Area
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_{ec}	Area of concrete effective in compression
A_{st}	Area of steel reinforcements
A_f	Area of each flange of steel section
A_e	Effective cross sectional area
A_v	Shear area
B	Centre-to-centre distance between girders and is equal to transverse span of inner girder
b	Outstand/width of the plate elements
b_e	Effective width of 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 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 centroid of concrete slab and centroid of steel beam
d_s	Overall depth of 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
E, E_s	Modulus of elasticity of structural steel
E_{cm}	Secant Modulus of elasticity of concrete
E_{st}	Modulus of elasticity of reinforcements

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_{ck}	Characteristic compressive cube strength of concrete at 28 days
f_{ctk}	Characteristic 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_t	Tensile strength of reinforcements
f_o	Proof stress
f_{yk}	Characteristic yield strength of reinforcement
f_u	Characteristic ultimate tensile stress
f_{up}	Characteristic ultimate tensile stress of the connected plate
f_y	Characteristic yield stress of steel
f_{yp}	Characteristic yield stress of connected plate
f_{yw}	Characteristic yield stress of the web material
h	Depth of the section
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 axis of bending for column
I_{co}	Moment of inertia of composite section
I_{fc}	Moment of Inertia of the compression flange about the minor axis of the steel beam
I_{ft}	Moment of Inertia of the tension flange about the minor axis of the steel beam
I_s	Moment of inertia of steel section about axis of bending for column
I_{st}	Moment of inertia of reinforcement about axis of bending for column
I_x	Moment of inertia about the major axis
K_L	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_x	Effective slenderness ratio of the section about the major axis
L	Actual span of girder
L_C	Effective span of cantilever for overhang
L_o	Length between points of zero moment (inflection) in the span
L_1	End span of continuous girder
L_2	Interior span of continuous girder
L_3	End span of continuous girder adjacent to cantilever span/overhang
L_4	Length of overhang or cantilever span
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_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_u	Design bending strength
M_y	Factored applied moments about the minor axis of the cross-section
M_x	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 buckling load
P_p	Plastic resistance of encased steel column section or concrete filled rectangular or square column section
q	Shear stress at service load
R_h	Flange stress reduction factor for hybrid section
r	Appropriate radius of gyration
r_y	Radius of gyration about the minor axis
r_x	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 plate
t_w	Thickness of web of steel section,
V, V_v, V_L	Factored applied shear force
$V_{d,}$	Design shear strength
V_p	Plastic shear resistance under pure shear
W	Total load
X	Distance from centre-line of edge girder to edge of slab
x_e	Depth of elastic neutral axis of composite section from centroid of steel section
x_u	Depth of neutral axis at limit state of flexure from top of 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 point of application of the load and shear centre of the cross-section
α	Imperfection factor
α_c	Strength coefficient of concrete
δ	Steel distribution ratio
χ	Stress reduction factor due to buckling under compression
χ_m	Stress reduction factor, χ, at f_{ym}
χ_{LT}	Stress reduction factor for lateral torsion 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_m	Partial safety factor for material (structural steel)
γ_{m0}	Partial safety factor against yield stress and buckling (structural steel)
γ_{m1}	Partial safety factor against ultimate stress (structural steel)
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 strength of weld
γ_s	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	Poisson's ratio
μ_c	Correction factor
η_1, η_2	Coefficients
η_{10}, η_{20}	Coefficients
τ	Actual shear stress range for the detail category
τ_f	Fatigue shear stress range
τ_{fd}	Design fatigue shear stress range
τ_{fmax}	Highest shear stress range
τ_{fn}	Fatigue shear stress range at N_{SC} cycle for the detail category

601 LIMIT STATE METHOD OF DESIGN

601.1 General

Normal elastic method is valid for analysis of the structure after considering load history, sequence of concrete casting and development of composite strength. In case of propped construction, most of the initial dead load is resisted through girder-prop system and the main girder remains basically unstressed at that stage. In case of un-propped construction the steel girders alone has to carry the initial dead load and is consequently stressed. The necessary distinction has to be made in the analysis. In ultimate limit state, however, this distinction is not necessary while checking for flexural strength. For design of steel components and concrete deck, stipulations of IRC:24-2010, IRC:112-2011 and this code may be applied.

601.2 Limit States

Structural safety has to be assessed for each of the limit states as mentioned below:

601.2.1 *Serviceability Limit State*

Is the state at which any of the following conditions occur

i) Stress in structural steel has reached the prescribed limit

ii) Deflection reaches the prescribed limit

iii) Concrete crack width reaches the prescribed limit

iv) Slip at the interface between steel and concrete exceeds permissible limits

v) Vibration becomes excessive specially at overhanging foot or cycle paths

601.2.2 *Fatigue Limit State*

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

601.2.3 *Ultimate Limit State*

Is the state when under the worst combination of factored loads the structure or its components either reach design strength and collapse or becomes unstable. Both stability and strength need to be checked under Ultimate Limit State.

601.3 Design Actions and their Combinations

For Load Combinations and Load factors IRC:6-2014 shall be referred to.

601.4 Material Strength & Partial Safety Factor for Material (γ_m)

Partial safety factors for materials as mentioned below in **Table 1** are to be used for assessment of strength:

Table 1 Material Safety Factors (γ_m,)

	Partial Safety Factor γ_m	
	Ultimate Limit	**Serviceability Limit**
Structural Steel against Yield Stress	1.10	1.00
Structural Steel against Ultimate stress	1.25	1.00
Steel Reinforcement (γ_s) against Yield Stress	1.15	1.00
Shear Connectors against Yield Stress	1.25	1.00
Bolts & Rivets for Shop & Site Fabrication against Yield Stress	1.25	1.00
Welds for Shop Fabrication	1.25	1.00
Welds for Site Fabrication	1.50	1.00
Concrete (γ_c) For Basic and Seismic Combinations	1.50	1.00
Concrete (γ_c) For Accidental Combinations	1.20	1.00

Note: Partial safety factors are not only given for f_y and f_{ck} but also for f_u

602 MATERIAL

The Materials and their properties are given in **Annexure-III**.

603 DESIGN FOR ULTIMATE LIMIT STATE

A typical composite girder system is as shown in **Fig.3** below. The neutral axis may be in the concrete slab, or in top flange of steel section or in the web of the steel sections. [Note: reference to **Annexure–I** is given in **Clause 603.3.1**, where it is more appropriate].

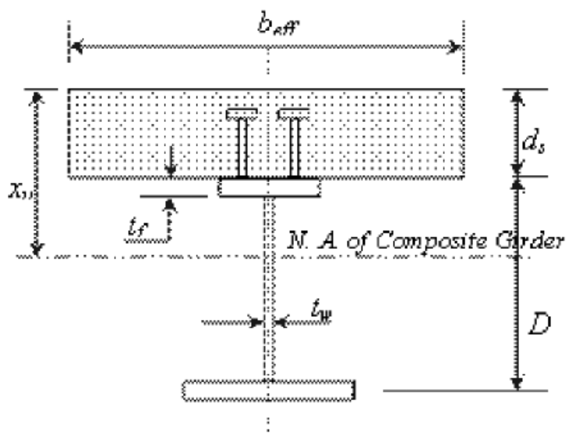


Fig.3 Typical Composite Girder

603.1 General

Distribution of moments and shears due to application of factored loads may be analyzed by elastic theory assuming concrete to be un-cracked and unreinforced.

Negative moments over internal supports as calculated above should be checked against section strength assuming steel girder acting integrally with concrete (considering un-cracked and un-reinforced). If the flexural tensile stress in concrete thus calculated exceeds the tensile strength of concrete, $f_{ctk,0.05}$ as given in **Table 6.5** of IRC:112-2011 then,

i) a new analysis neglecting concrete (but including reinforcements) over 15 percent of span on either side of supports should be done to check the strength,

ii) provided adjacent spans do not differ appreciably, positive maximum moments in the adjacent spans should be increased by 40 f_{ct}/f_{ck} percent for checking of strength without decreasing support moment (f_{ct} = tensile stress in uncracked concrete flange). This provision gives partial recognition to the philosophy of plastic design.

Bottom flange of girder in negative moment zone should be adequately braced against lateral buckling.

603.1.1 *Sectional Classification of Girder*

The section strength at ultimate limit state should be considered on their ability to resist local buckling before full plastic strength is developed. In this respect the sections may be classified as

Class – 1 or	**Plastic:**	Cross-sections which can develop plastic hinges and have the rotation capacity required for failure of the structure by formation of a plastic mechanism.
Class – 2 or	**Compact:**	Cross-sections which can develop plastic moment of resistance but have inadequate plastic hinge rotation capacity for formation of a plastic mechanism due to local buckling.

Class – 3 or Semi-compact: Cross-sections in which the extreme fibers in compression can reach yield stress, but cannot develop the plastic moment of resistance due to local buckling.

Class – 4 or Slender: Cross-sections in which the elements buckle locally, even before reaching yield stress. This code does not deal with these types of section.

603.1.2 *General Rules for Sectional Classification of Composite-Section*

1. A composite section should be classified according to the least favourable class of steel elements in compression. The class of a composite section normally depends on the direction of the bending moment at that section.
2. A steel compression element restrained by its connection to a reinforced concrete element may be placed in a more favourable class, after ensuring its improved performance due to the above connection.
3. Plastic stress distribution should be used for section classification except at the boundary between Class 2 and 3 where the elastic stress distribution should be used taking into account sequence of construction and the effects of creep and shrinkage.
4. For classification, design values of 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 flanges.
5. Welded mesh should not be included in the effective section unless it has sufficient ductility to withstand fracture when embedded in concrete,
6. In global analysis for stages of construction, account should be taken of the class of steel section at the particular stage considered.

603.1.3 *Classification of Composite Section without Concrete Encasement*

1. A steel compression flange which is restrained against buckling by effective attachment to a concrete flange by shear connectors may be assumed to be in class 1 if the spacing of the connector is in accordance with 606.9.
2. Other steel flanges and webs in compression in composite girders should be classified on the basis of width to thickness ratios (width and thickness of individual elements shown in **Fig.5**) and proneness to local buckling. Accordingly sections are categorized in three groups as indicated in **Table 2**.
3. 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. 4**. 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.

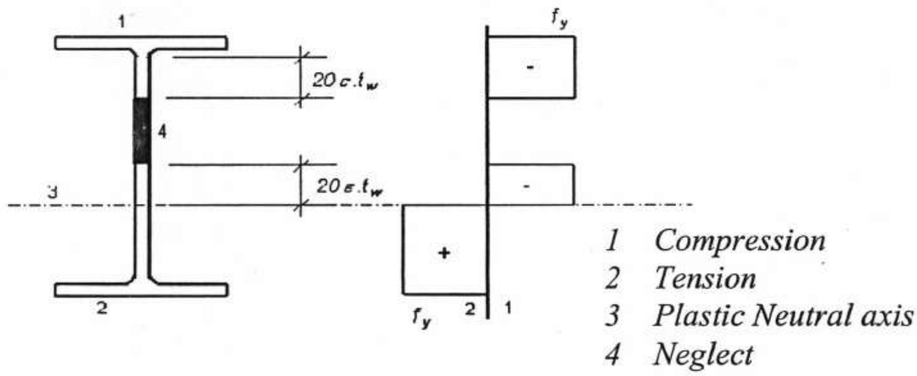


Fig. 4 Effective Class 2 Web

603.1.4 *Types of Elements*

1. Internal elements are elements attached along both longitudinal edges to other elements or to longitudinal stiffeners connected at suitable intervals to transverse stiffeners, e.g., web of I-section and flanges and web of box section.
2. 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 e.g., flange overhang of an I-section, stem of T-section and legs of an angle section.
3. Tapered elements may be treated as flat elements having average thickness as defined in BIS publication, IS:808-1989 (Reaffirmed 2004), titled, "Indian Standard Dimensions for Hot rolled Steel Beam, Column, Channel and Angle sections.

The limiting width to thickness ratios of elements for different classifications of sections are given in **Table 2** based on end conditions of elements as shown in **Fig. 5**.

Table 2 Limiting Width to Thickness Ratios

Compression Element		Ratio	Class of Section		
			Class 1 Plastic	**Class 2 Compact**	**Class 3 Semi-Compact**
Outstanding element of compression flange	Rolled section	b/t_f	9.4ε	10.5ε	15.7ε
	Welded section	b/t_f	8.4ε	9.4ε	13.6ε
Internal element of compression flange	Compression due to bending	b/t_f	29.3ε	33.5ε	42ε
	Axial compression	b/t_f	Not applicable		

Web of an I-H-or box section [c]	Neutral axis at mid-depth		d/t_w	84ε	105ε	126ε
	Generally	If r_1 is negative:	d/t_w	$\frac{84\varepsilon}{1+r_1}$ but $\geq 42\varepsilon$	$\frac{105.0\varepsilon}{1+r_1}$	$\frac{126.0\varepsilon}{1+2r_2}$ but $\geq 42\varepsilon$
		If r_1 is positive :	d/t_w		$\frac{105.0\varepsilon}{1+1.5r_1}$ but $\geq 42\varepsilon$	
	Axial compression		d/t_w	Not applicable		42ε
Web of a channel			d/t_w	42ε	42ε	42ε
Angle, compression due to bending (Both criteria should be satisfied)			b/t d/t	9.4ε 9.4ε	10.5ε 10.5ε	15.7ε 15.7ε
Single angle, or double angles with the components separated, axial compression (All three criteria should be satisfied)			b/t d/t $(b+d)/t$	Not applicable		15.7ε 15.7ε 25ε
Outstanding leg of an angle in contact back-to-back in a double angle member			d/t	9.4ε	10.5ε	15.7ε
Outstanding leg of an angle with its back in continuous contact with another component						
Stem of a T-section, rolled or cut from a rolled I-or H-section			D/t_f	8.4ε	9.4ε	18.9ε
Circular hollow tube, including welded tube subjected to moment			D/t	$42\varepsilon^2$	$52\varepsilon^2$	$146\varepsilon^2$
Circular hollow tube, including welded tube subjected to axial compression			D/t	Not applicable		$88\varepsilon^2$

Notes:

a elements which exceed semi-compact limits are to be taken as slender

b $\varepsilon = (250/f_y)^{1/2}$

c Check webs for shear buckling in accordance with 603.5.3.2 (2) when d/t > 67 ε. Where, b is the width of the element may be taken as clear distance between lateral supports or between lateral support and free edge, as appropriate, t is the thickness of element, d is the depth of the web, D outer diameter of the element, Refer **Fig. 3.1**,**Section 603.2.1.**

d Different elements of a cross-section can be in different classes. In such cases the section is classified based on the least favorable classification.

e The stress ratio r_1 and r_2 are defined as

$$r_1 = \frac{actual\ average\ axial\ stress\ (negative\ if\ tensile)}{design\ compressive\ stress\ of\ web\ alone},\quad r_2 = \frac{actual\ average\ axial\ stress\ (negative\ if\ tensile)}{design\ compressive\ stress\ of\ overall\ section}$$

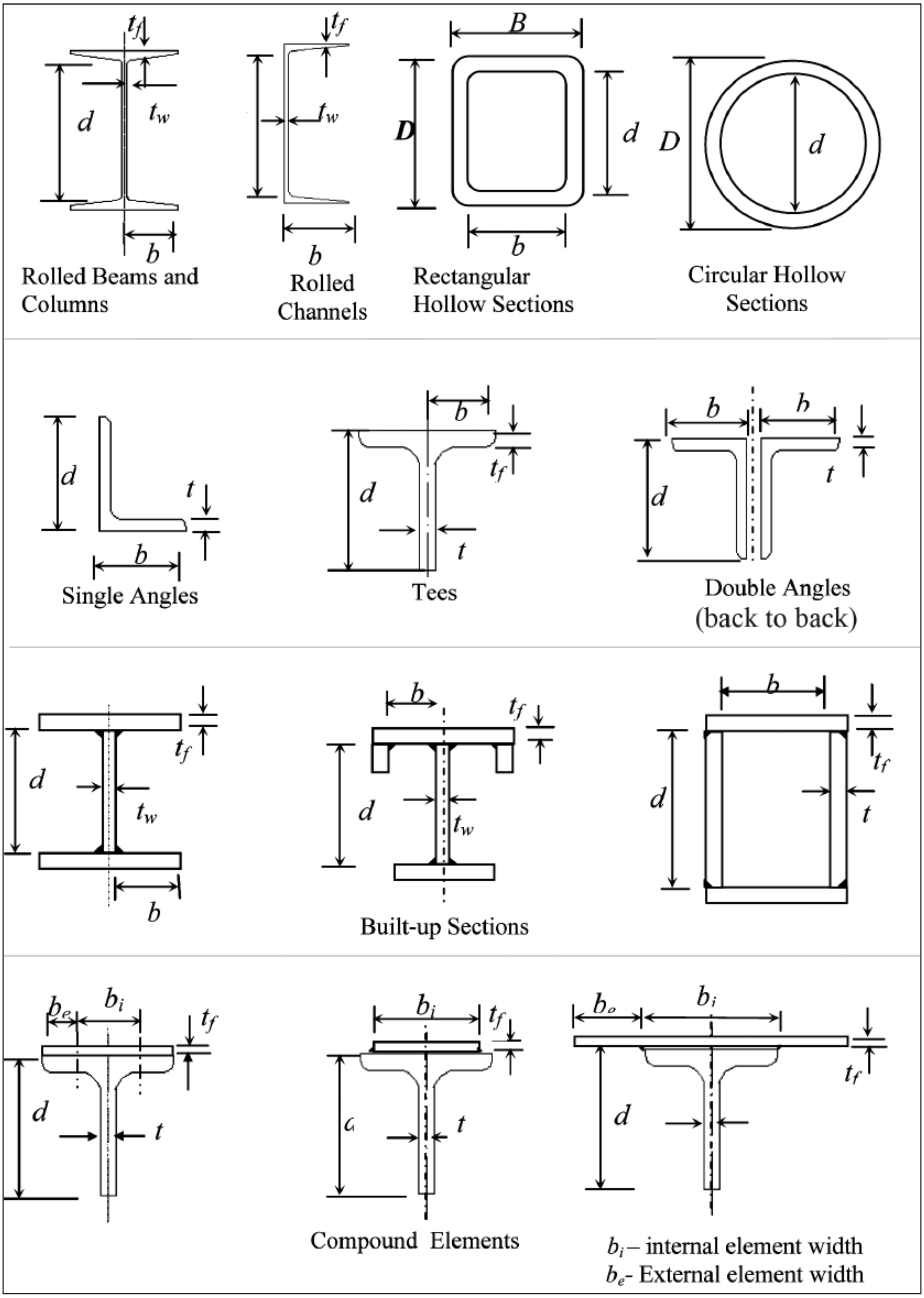


Fig. 5 Dimensions of Sections

603.2 Effective Width of Concrete Slab

For strength calculation of composite girders, effective width b_{eff1} of deck slab on either side of the girder should satisfy (Ref: **Fig. 6a**).

$$b_{eff1} = \frac{L_o}{8} \le \frac{B_1}{2} or \frac{B_2}{2} \quad ... 3.1$$

Therefore the total effective width beff (Ref: **Fig. 6a**) of deck slabs should be restricted to the limits as indicated below in Eq. 3.2 and Eq. 3.3:

603.2.1 *Effective Width of Simply Supported Girder*

1. For inner beams

$$b_{eff} = \frac{L_o}{4} \le \frac{B_1 + B_2}{2} \quad ... 3.2$$

For equal spacing of the girders i.e. $B_1 = B_2 = B$

$$b_{eff} = \frac{L_o}{4} \le B \quad ... 3.3$$

Where,

L = Actual span of the girder

L_o = The effective span taken as the distance between points of zero moments (L_o = L for simply supported girders)

B = Equal Centre to centre distance of transverse spans of inner slabs (See **Fig. 6a**).

2. For outer edge beams

$$b_{eff} = \frac{L_o}{8} + X \le \frac{B1}{2} + X \quad ... 3.4$$

Where, $\frac{L_o}{8} \le \frac{B_1}{2}$ and $X = B_0 \le \frac{L_0}{8}$

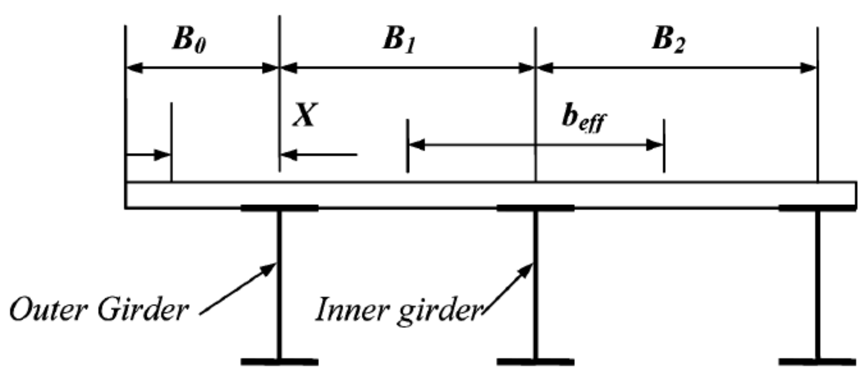


Fig. 6a Effective Widths for Composite Beams

603.2.2 *Effective Width for Continuous Girder*

The effective width for continuous girder shall be calculated on the same lines as discussed in **Clause 603.2.1** above. The effective span L_0 for span moment and support moment for continuous girders will be as given illustrated by **Fig. 6b** below.

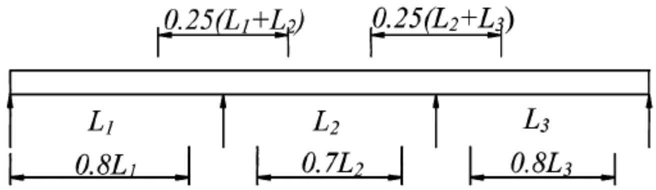


Fig. 6b Value of L_0 for Continuous Beam

The effective span for outer spans is 0.8 times the actual span. $L_0 = 0.8L_1$

The effective span for inner spans is 0.7 t imes the actual span. $L_0 = 0.7L_2$

The effective span for intermediate support is 0.25 times the sum of the width of the adjacent spans. $L_0 = 0.25(L_1 + L_2)$ or $0.25\ (L_2 + L_3)$

Reinforcements placed parallel to the span of the steel beam within the effective width of concrete slab at the continuous support will only be effective in analyzing the hogging moment capacity of the composite girder at the continuous support.

603.2.3 *Effective Cross-Section for Strength Calculation* – In calculating the strength of the cross section of the composite girders the following should be considered:

For Positive moment - Concrete in effective width to be included but not the reinforcements.

For Negative Moment - Concrete to be neglected but longitudinal reinforcement within effective width are to be included.

603.3 Analysis of Structure

603.3.1 *Elastic Analysis*

Design moments and shears may be calculated by normal elastic method. In case of continuous structures with negative moments over supports adjustments may be necessary as mentioned in **Clause 603.1.** Appropriate load combinations with corresponding load factors are to be used to find out the maximum values of moments and shears.

Design strength for different sections is to be worked out on the basis of their capacity to resist local buckling based on classification given in **Clause 603.1.** Design bending strength under various stress conditions have been given in **Annexure–I**. The bending moments, given in **Annexure–I**, are for full shear connection. Necessary correction due to partial shear connection shall be done as also indicated in **Annexure–I.**

The stability of the bridge as a whole against overturning shall be ensured under ultimate limit state as per provisions of IRC:6-2014. The bridge components shall also be safe against sliding under adverse condition of the applied characteristic loads. Following factor of safety shall be ensured:

- Factor of safety against Overturning - 1.30
- Factor of safety against Sliding - 1.25

603.3.2 *Plastic Analysis*

Plastic analysis is not permitted to members subjected to impact loading requiring fracture assessment or fluctuating loading requiring fatigue assessment. Thus, in bridges plastic analysis is not permitted.

603.3.3 *Design of structure (bending moment)*

Considering local buckling, sections are to be analyzed as plastic, compact or semi-compact as already mentioned with additional consideration.

1. Load history and development in composite action are to be taken into consideration with appropriate values of modular ratio '**m**' at each stage and stresses and deflections are to be the summation of values of successive stage.
2. Effective width of concrete may be as mentioned in **Clause 603.2.**

The bending moment and shear force distribution in a continuous bridge girder has to be determined by structural analysis.

603.3.3.1 Design of structure (effect of lateral buckling on moment):

Lateral Buckling may govern the design under the following conditions:

1. At construction stage, in the top flange closer to mid span in both simply supported and continuous girders.
2. At construction and service stage, in the bottom flange closer to support in continuous girders.

At the construction stage the effect of lateral torsional buckling on the bottom flange in a continuous girder shall be taken care of by considering the girder as a cantilever upto the point of inflection from the support. After hardening of concrete, the same lateral torsional buckling need to be checked with revised point of inflection and changed stiffness of section. The flexural strength corresponding to lateral buckling shall be determined as per **Clause I.5** of **Annexure–I**.

Suitable horizontal bracings or members may be provided at the bottom flange to reduce effective length of compression flange near support. For girders which 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.

603.3.3.2 Design against vertical shear and its effect on plastic moment capacity

The factored design shear force, V, in a beam due to external actions shall satisfy

$$V \leq V_d$$

where,

V_d = design shear strength calculated as given below:

V_d = V_n / γ_{m0}

γ_{m0} = partial safety factor against shear failure (601.4)

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

1. Plastic Shear Resistance

The nominal plastic shear resistance under pure shear is given by:

$$V_n = V_p$$

where,

$$V_p = \frac{A_v . f_{yw}}{\sqrt{3}} \quad \text{... 3.5}$$

where,

A_v is the shear area, f_{yw} is the yield strength of the web and partial safety factor γ_{m0} = 1.10.

Note: Shear area may be calculated as below: -

i) **For I and Channel Section:**

Major Axis Bending:

Hot Rolled = depth of section x web thickness = $h.t_w$

Welded = depth of web x web thickness = $d.t_w$

Minor Axis Bending:

Hot Rolled or Welded = 2 x flange width x flange thickness

ii) **Rectangular Hollow Sections of Uniform Thickness:**

Loaded parallel to depth (d), $A_v = Ad / (b + d)$

Loaded parallel to width (b), $A_v = Ab / (b + d)$

where,

A = Actual area of cross-section

b = Overall breadth of tubular section, breadth of I section flanges

d = Clear depth of the web between flanges

iii) **Circular Hollow Tubes of Uniform Thickness:**

$A_v = 2A / \pi$ (Where A = actual area of cross section)

2. Shear Buckling Resistance

The nominal shear strength, V_n, of webs with or without intermediate stiffeners as governed by buckling may be evaluated using one of the following methods:

a) **Simple Post-Critical Method** − The simple post critical method, based on the shear buckling strength can be used for webs of I-section girders, with or without intermediate transverse stiffener, provided that the web has transverse stiffeners at the supports. The nominal shear strength is given by:

$$V_n = V_{cr}$$

where,

V_{cr} = shear force corresponding to web buckling

V_{cr} = $A_v . \tau_b$

where,

τ_b = shear stress corresponding to web buckling, determined as follows:

When $\lambda_w \leq 0.8$ $\tau_b = f_{yw}/\sqrt{3}$ ... 3.6

When $0.8 < \lambda_w < 1.2$ $\tau_b = [1 - 0.8(\lambda_w - 0.8)]\left(f_{yw}/\sqrt{3}\right)$ ... 3.7

When $\lambda_w \geq 1.2$ $\tau_b = f_{yw}/\left(\sqrt{3}\lambda_w^{\ 2}\right)$ ... 3.8

where,

λ_w = non -dimensional web slenderness ratio for shear buckling stress, given by

$\lambda_w = \sqrt{f_{yw}/(\sqrt{3}\,\tau_{cr,e})}$ ... 3.9

The elastic critical shear stress of the web, $t_{cr,e}$ is given by:

$$\tau_{cr,e} = \frac{k_v \pi^2 E}{12\left(1-\mu^2\right)\left(d/t_w\right)^2} \quad \text{... 3.10}$$

where,

μ = Poisson's ratio

k_v = 5.35 when transverse stiffeners are provided only at supports

= $4.0 + 5.35 / (c/d)^2$ for $c/d < 1.0$

= $5.35 + 4.0 / (c/d)^2$ for $c/d \geq 1.0$

where,

c and *d* are the spacing of transverse stiffeners and depth of the web, respectively.

b) **Tension Field Method** − The tension field method, based on the post-shear buckling strength, may be used for webs with intermediate transverse stiffeners, in addition to the transverse stiffeners at supports, provided the panels adjacent to the panel under tension field action, or the end posts provide anchorage for the tension fields and further c/d ≥1.0, where c, d are the spacing of transverse stiffeners and depth of the web, respectively. In the tension field method, the nominal shear resistance, Vn, is given by

$$V_n = V_{tf}$$

where,

$$V_{tf} = \left[A_v\,\tau_b + 0.9\ w_{tf}\,t_w\,f_v \sin\phi\right] \leq V_p \quad \text{... 3.11}$$

τ_b = buckling strength, as obtained from **603.3.3.2 (2) (a)**

f_v = yield strength of the tension field obtained from

$$f_v = \left[f_{yw}^{\ 2} - 3\,\tau_b^{\ 2} + \psi^2 \right]^{0.5} - \psi$$

$\Psi = 1.5\ \tau_b \sin 2\phi$

ϕ = inclination of the tension field = $\tan^{-1}\left(\dfrac{d}{c}\right)$

The width of the tension field, w_{tf}, is given by:

$w_{tf} = d \cos\phi + (c - s_c - s_t) \sin\phi$

f_{yw} = yield stress of the web

d = depth of the web

c = spacing of stiffeners in the web

τ_b = shear stress corresponding to buckling of web **603.3.3.2 (2) (a)**

s_c, s_t = anchorage lengths of tension field along the compression and tension flange respectively, obtained from:

$$s = \frac{2}{\sin\phi}\left[\frac{M_{fr}}{f_{yw}\, t_w}\right]^{0.5} \le c$$

where,

M_{fr} = reduced plastic moment capacity of the respective flange plate (disregarding any edge stiffener) after accounting for the axial force, N_f in the flange, due to overall bending and any external axial force in the cross-section, and is calculated as given below:

$$M_{fr} = 0.25 b_f t_f^{\ 2} f_{yf} \left[1 - \left\{N_f / \left(b_f t_f f_{yf} / \gamma_{mo}\right)\right\}^2\right] \qquad \text{... 3.12}$$

where,

b_f, t_f = width and thickness of the relevant flange respectively

f_{yf} = Yield stress of the flange

603.3.3.3 Reduction in bending resistance under high shear force

If V is less than $0.6V_d$ there is no reduction in the plastic bending resistance of the section. When $V > 0.6V_d$, the bending resistance is reduced as the contribution of web to bending gets diminished. Therefore, reduced bending capacity is given as:

1. Plastic or Compact Section

$$M_{dv} = M_d - \beta\left(M_d - M_{fd}\right) \le 1.2\, z_e\, f_y / \gamma_{m0}, \qquad \text{... 3.13}$$

where,

β = $(2\ V/V_d - 1)^2$

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

V = factored applied shear force as governed by web yielding or web buckling.

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

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

2. **Semi-compact Section**

$$M_{dv} = Z_e f_y / \gamma_{m0} \quad \text{... 3.14}$$

where,

Z_e = elastic section modulus of the whole section

603.3.3.4 Design against longitudinal shear

Design against longitudinal shear and strength of shear connector has to be done as in **Section 606.0**.

603.4 Hybrid Sections

Use of hybrid sections are permitted, with necessary adjustment (reduction) in permissible stresses of the flange element in the cross section with higher yield stress by reduction factor R_h. R_h may be determined using the procedure as elaborated in **Annexure–I**.

604 DESIGN FOR SERVICEABILITY LIMIT

604.1 General

Normal elastic analysis is to be used for finding out design moments and stresses under various load combinations and load factors as mentioned in **Clause 601.3**. Concrete is to be assumed as unreinforced and uncracked.

604.1.1 *Method of construction*

The stress and strain at serviceability limit state depend on the method of construction, that is, whether the steel beam is propped or un-propped during construction.

1. **Un-propped construction :**

   In un-propped construction, the steel beam has to carry the construction load including shuttering, wet concrete and its own weight. However under limit state of collapse, the total load including transient loads shall be resisted by the composite section.

2. **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 characteristic strength equal to approximately twice the stress to which the concrete may be subjected shortly after removal of props.

This difference in the above two methods of construction does not, however, affect the ultimate limit load.

604.2 Negative Moments

Negative moments over intermediate supports are to be adjusted as mentioned in **Clause 603.1**.

604.3 Stresses and Deflection

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

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

$m = \dfrac{E_s}{K_c E_{cm}} \geq 15.0$ For permanent or long-term loads (K_c = *Creep factor = 0.5*)

where,

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 (Ref: **Table-III.1** of **Annexure-I**)

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

The equivalent area of concrete slab at any stage, however shall be determined by dividing the effective width of the concrete slab by the modular ratio,

$m = \dfrac{E_s}{E_{ci}}$,

where,

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

Final stresses and deflection is 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.

604.3.1 *Limiting Stresses for Serviceability*

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

1. **Concrete:**

   The allowable compressive stress in concrete shall be as per **Clause 12.2.1** of IRC:112-2011.

2. **Reinforcement Steel:**

   The allowable tensile stress in steel reinforcement shall be as per **Clause 12.2.2 of IRC:112-2011**.

3. **Structural Steel:**

   The concept of equivalent stress shall be adopted to determine the limiting stress permissible for steel beam or girder.

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

$$f_{ec} = \sqrt{f_{bc}^{\ 2} + f_p^{\ 2} + f_{bc}.f_p + 3\tau_b^{\ 2}} \qquad \text{En... 4.1}$$

and

$$f_{et} = \sqrt{f_{bt}^{\ 2} + f_p^{\ 2} + f_{bt}.f_p + 3\tau_b^{\ 2}} \qquad \text{En... 4.2}$$

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

τ_b = actual shear stress in steel section

The value of permissible 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.

604.3.2 *Limit of Deflection and Camber*

Calculated deflection of composite girder under live load and impact shall not exceed 1/800 of span of the girder.

In any case under the worst combination of dead load, super-imposed dead load, live load and impact effects, the total deflection of the girder shall not exceed 1/600 of span. Necessary camber may be adopted as per clause 504.6 of IRC:24-2010 to offset the effect of all permanent loads to comply with the above requirement.

The deflection of cantilever arms at the tip due to dead load, live load and impact shall not exceed 1/300 of the cantilever arm and the deflection due to live load and impact only, shall not exceed 1/400 of the cantilever arm. Sidewalk live load may be neglected in calculating deflection.

When cross bracings or diaphragms of sufficient stiffness and strength are provided between beams to ensure the lateral distribution of loads, the deflections may be calculated considering all beams acting together. In such cases the gross moment of inertia of equivalent section may be used for calculating the deflection of the composite bridge girder system.

604.4 Control of Cracking in Concrete

Minimum reinforcements in terms of diameter and spacing required for crack control at top of concrete as per **Clause 12.3.3** of IRC:112-2011 is to be provided in composite girders, at the zone of negative moment, to prevent cracking adversely affecting appearance and durability of structure. Crack width calculation as well as limiting crack width as given in **Clause 12.3.4** and **12.3.2** respectively of IRC:112-2011 may be followed subject to discretion of engineers.

For control of cracking without direct calculation, reference shall be made to the directions provided in **Clause 12.3.6** of IRC:112-2011.

604.5 Temperature Effect

For Temperature effect, **Section 215** of IRC:6-2014 may be referred to.

605 DESIGN FOR FATIGUE LIMIT

605.1 General

Fatigue is to be checked under fatigue vehicular live load with impact and with the application of appropriate load factor. Stresses are to be assessed by elastic theory and elastic properties of the section with no adjustment for support moments.

For the purpose of design against fatigue, different details (of members and connections) are classified under different fatigue class. The design stress range corresponding to various number of cycles, are given for each fatigue class. The requirements of this section shall be satisfied with, at each critical location subjected to cyclic loading, considering relevant number of cycles and magnitudes of stress range expected to be experienced during the life of the structure.

Definitions:

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

Detail Category − Designation given to a particular detail to indicate the S-N curves to be used in fatigue assessment.

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

Fatigue Strength − Stress range capacity for a category of detail, depending upon the number of cycles it is required to withstand during its design life.

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 a specific detail category of a structure.

605.2 Fatigue Design

The standard S-N Curves for each detail category are given for the following conditions:

a) The detail is located in a redundant load path, wherein local failure at that detail alone will not lead to overall collapse of the structure.

b) The nominal stress history at the local point in the detail is estimated/ evaluated by a conventional method without taking into account the local stress concentration effects due to the detail

c) The load cycles are not highly irregular

d) The details are accessible for and subject to regular inspection

e) The structure is exposed to only mildly corrosive environment as in normal atmospheric condition and suitably protected against corrosion (pit depth < 1 mm)

f) The structure is not subjected to temperature exceeding 150ºC

g) The transverse fillet or butt weld connects plates of thickness not greater than 25 mm

h) Holes in members and connections subjected to fatigue shall not be made

- Using punching in plates having thickness greater than 12 mm unless the holes are subsequently reamed to remove material strain hardened during punching
- Using gas cutting unless the holes are reamed to remove the material in the Heat Affected Zone (HAZ)

The values obtained from the standard S-N Curve shall be modified by a capacity reduction factor μ_r, when plates greater than 25 mm in thickness are joined together by transverse fillet or butt-welding, which is given by

$$\mu_r = (25/t_p)^{0.25} \leq 1.0$$

where,

t_p = actual thickness in mm of the thicker plate being joined

No thickness correction is necessary when full penetration butt weld reinforcements are machined flush and proved free of defect through nondestructive testing.

Determination of stress: Elastic analysis of structure shall be done to obtain stress resultants and any conventional stress analysis may be done to determine the stress at the location. The normal and shear stresses on a member shall be done considering the effect of all design actions on the members. Stress concentration due to geometry of detail may be excluded. The stress concentration, however, not characteristic of the detail shall be accounted for in the stress calculation.

Low Fatigue: Fatigue assessment is not required for a member, connection or detail, if normal and shears design stress ranges f, satisfy the following conditions:

$$f < 27/\gamma_{mft}$$

or if the actual number of stress cycles, N_{SC}, satisfies

$$N_{SC} < 5x10^6 \left(\frac{27/\gamma_{mft}}{\gamma_{fft}\, f}\right)^3$$

where,

γ_{mft}, γ_{fft} = partial safety factors for strength and load, respectively

f = actual fatigue stress range for the detail

Partial Safety Factor for Actions and their effects (γ_{fft}): The partial safety factor for loads in the evaluation of stress range in fatigue design shall be taken as 1.0.

Partial Safety Factor for Fatigue Strength (γ_{mft}): The partial safety factor for strength is influenced by consequences of fatigue damage and level of inspection capabilities.

Based on consequences of fatigue failure as defined below, the partial safety factor for fatigue strength shall be as given in the **Table 3**.

- Fail-safe structural component/detail is the one where local failure of one component due to fatigue crack does not result in the failure of the total structure due to availability of alternate load path (redundant system).
- Non-fail-safe structural component/detail is the one where local failure of one component leads rapidly to failure of the total structure due to its non-redundant nature.

Table 3 Partial Safety Factor for Fatigue Strength (γ_{mft})

Inspection and Access	Consequence of Failure	
	Fail-Safe	Non-Fail-Safe
Periodic inspection and maintenance, accessibility to detail is good	1.00	1.25
Periodic inspection and maintenance, poor accessibility for detail	1.15	1.35

605.3 Fatigue Strength

The fatigue strength for any standard details as described in **Table 8** for normal or shear fatigue stress range not corrected for the effects as described in **Clause 605.2** is given below:

For Normal Stress range

When $N_{SC} \leq 5 \times 10^6$ $\quad f_f = f_{fn}\sqrt[3]{5x10^6 / N_{SC}}$ ... 5.1

When $5 \times 10^6 \leq N_{SC} \leq 10^8$ $\quad f_f = f_{fn}\sqrt[5]{5x10^6 / N_{SC}}$ ... 5.2

For Shear Stress range,

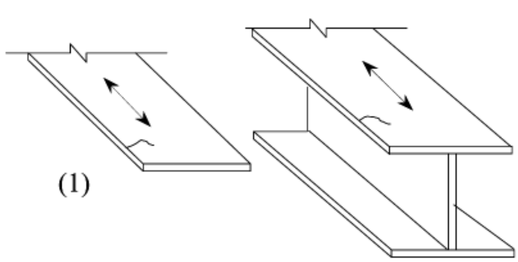
$\tau_f = \tau_{fn}\sqrt[5]{5x10^6 / N_{SC}}$ ... 5.3

where,

f_f, τ_f = design normal and shear fatigue stress range of the detail, respectively, for life cycle of N_{SC}

f_{fn},τ_{fn} = design normal and shear fatigue strength respectively of the detail for 5×10^6 cycles as given in **Table 4, 5 & 6**.

Table 4 Detail Category Classification Group -1; Non-Welded Details

Detail Category	Constructional Details	
	Illustration (See Note)	Description
118		**ROLLED AND EXTRUDED PRODUCTS** 1) Plates and flats 2) Rolled sections Sharp edges, surface and rolling flaws to be removed by grinding in the direction of applied stress.

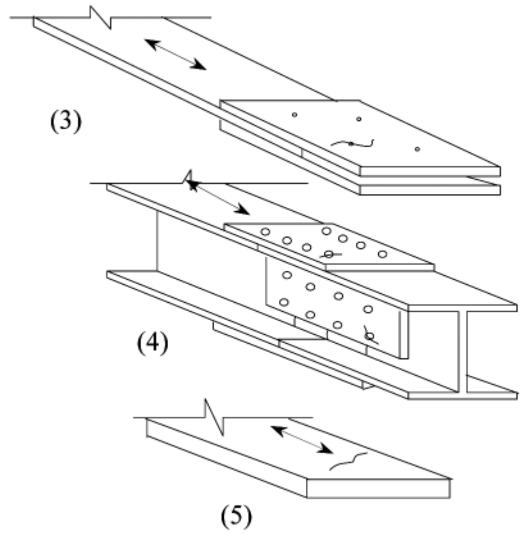
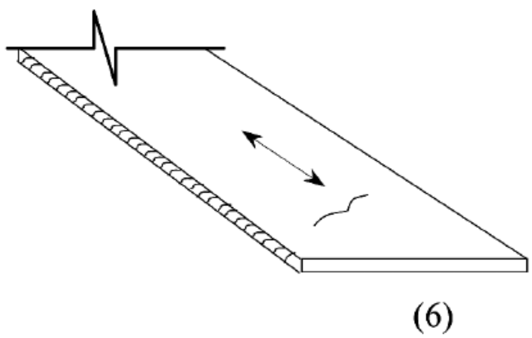
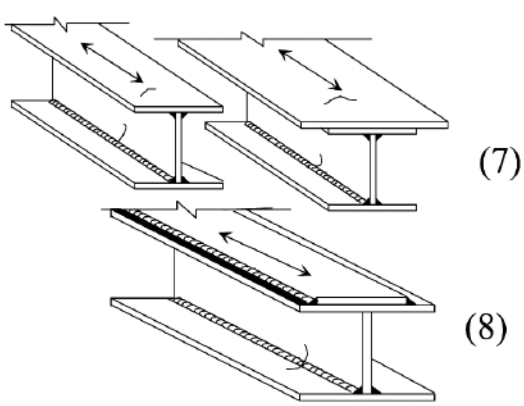
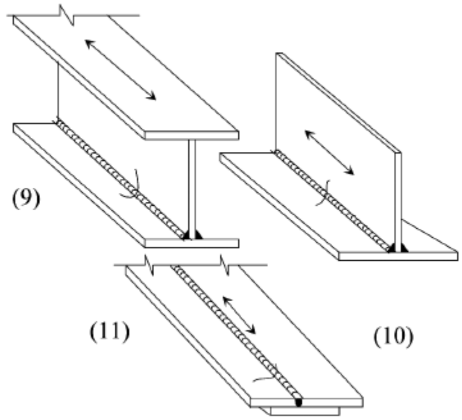
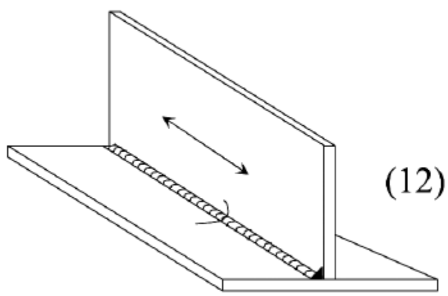
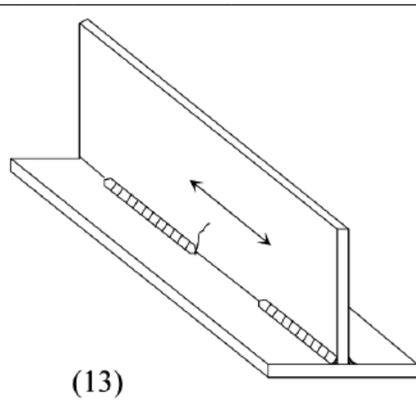
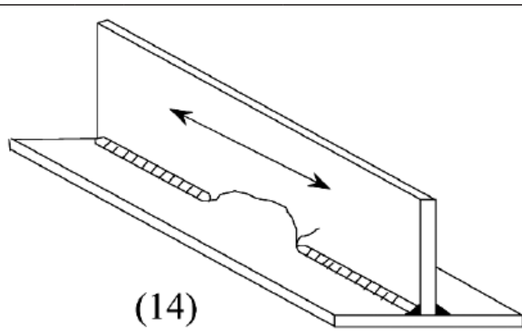
Detail Category	Constructional Details	
	Illustration (See Note)	Description
103		**BOLTED CONNECTIONS** (3) & (4): Stress range calculated on the gross section and on the net section. Unsupported one-sided cover plate connections shall be avoided or the effect of the eccentricity taken into account in calculating stresses. **MATERIAL WITH GAS-CUT OR SHEARED EDGES WITH NO DRAGLINES** (5): All hardened material and visible signs of edge discontinuities to be removed by machining or grinding in the direction of applied stress.
92		**MATERIAL WITH MACHINE GAS-CUT EDGES WITH DRAGLINES OR MANUAL GAS-CUT MATERIAL** (6): Corners and visible signs of edge discontinuities to be removed by grinding in the direction of the applied stress.

Table 5 Detail Category Classification Group - 2; Welded Details

Detail Category	Constructional Details	
	Illustration (See Note)	Description
92		**WELDED PLATE I-SECTION AND BOX GIRDERS WITH CONTINUOUS LONGITUDINAL WELDS** (7) & (8) Zones of continuous automatic longitudinal fillet or butt welds carried out from both sides and all welds not having un-repaired stop-start positions.

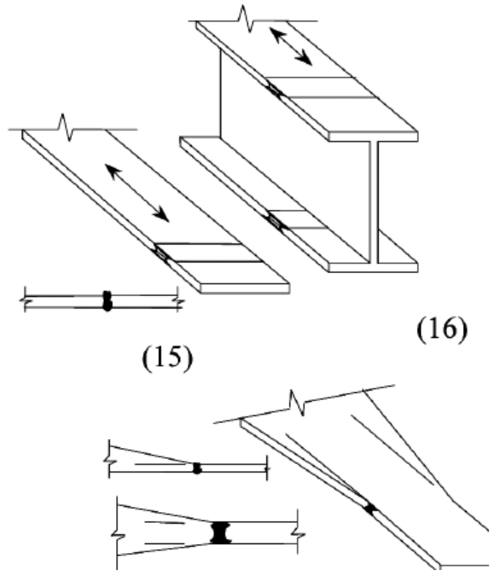
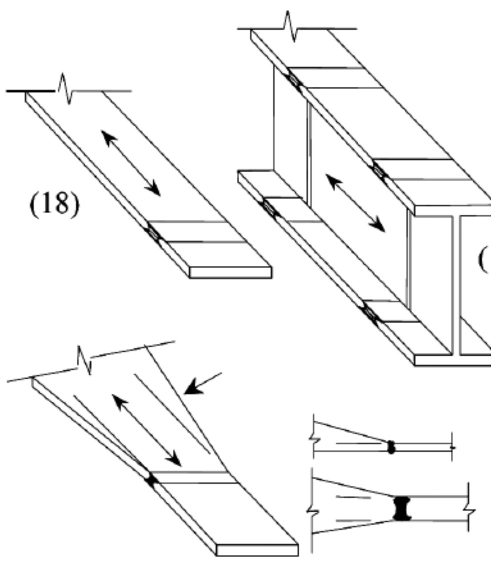
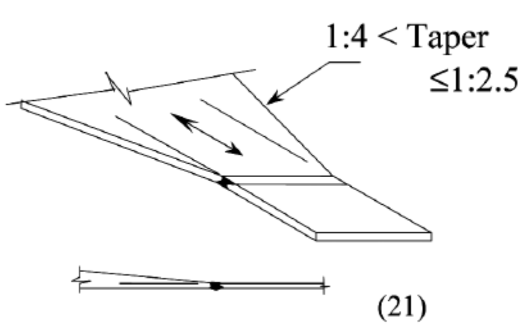
Note: The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

Table 5 (continued)

Detail Category	Constructional Details	
	Illustration (See Note)	**Description**
83		**WELDED PALTE I-SECTION AND BOX GIRDERS WITH CONTINUOUS LONGITUDINAL WELDS** (9) & (10) Zones of continuous automatic butt welds made from one side only with a continuous backing bar and all welds not having un-repaired stop-start positions. (11) Zones of continuous longitudinal fillet or butt welds carried, out from both sides but containing stop-start positions. For continuous manual longitudinal fillet or butt welds carried out from both sides, use Detail Category 92.
66		**WELDED PLATE I-SECTION AND BOX GIRDERS WITH CONTINUOUS LONGITUDINAL WELDS** (12) Zones of continuous longitudinal welds carried out from one side only, with or without stop-start positions.
59		**INTERMITTENT LONGITUDINAL WELDS** (13) Zones of intermittent longitudinal welds
52		**INTERMITTENT LONGITUDINAL WELDS** (14) Zones containing cope holes in longitudinally welded T joints. Cope hole not to be filled with weld.

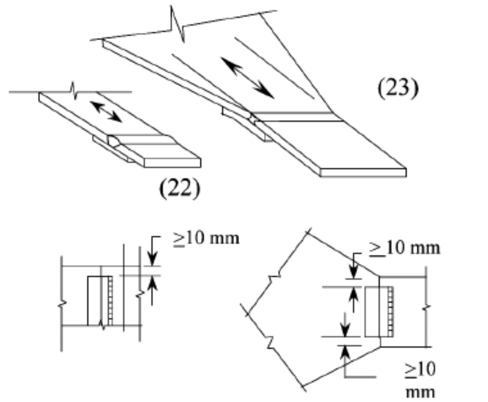
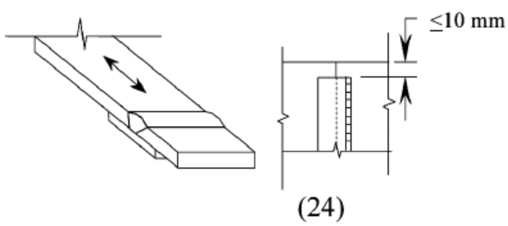
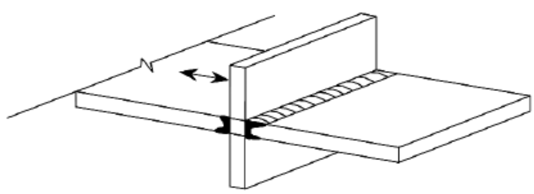
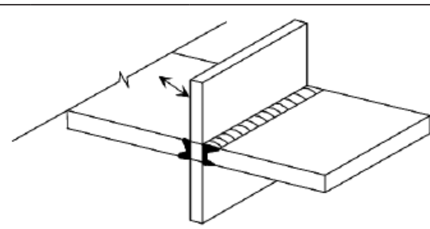
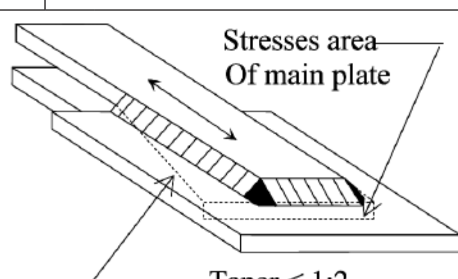
Note: The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

Table 5 (continued)

Detail Category	Constructional Details	
	Illustration (See Note)	Description
83		**TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)** Weld run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from two sides. (15) Transverse splices in plates, flats and rolled sections having the weld reinforcement ground flush to plate surface. 100 percent NDT inspection and weld surface to be free of exposed porosity in the weld metal. (16) Plate girders welded as (15) before assembly. (17) Transverse splices as (15) with reduced or tapered transition with taper ≤1:4.
66		**TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)** Welds run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from two sides. (18) Transverse splices of plates, rolled sections or plate girders. (19) Transverse splice of rolled sections or welded plate girders, without cope hole. With cope hole use Detail Category 52, as per (14). (20) Transverse splices in plates or flats being tapered in width or in thickness where the taper is ≤1:4 made from two sides
59		**TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)** Weld run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from two sides. (21) Transverse splices as per (20) With taper in width or >1:4.but ≤1:2.5.

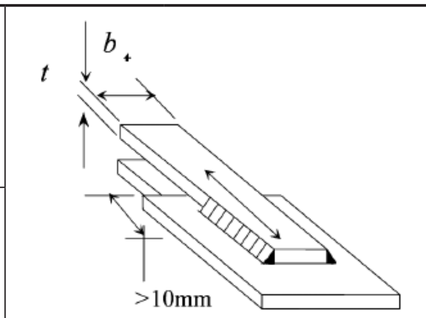
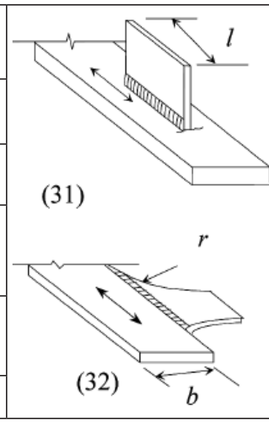
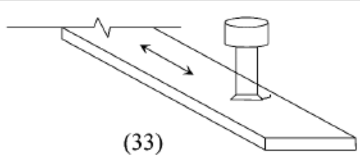
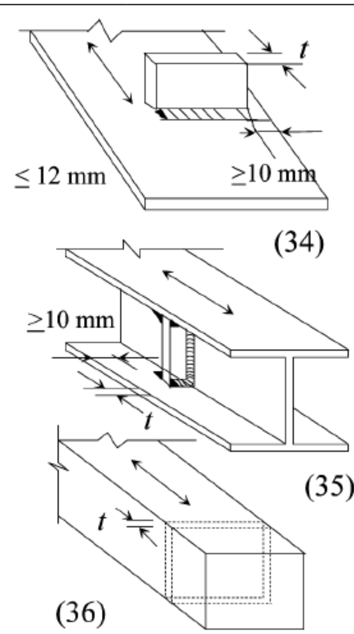
Note: The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

Table 5 (continued)

Detail Category	Constructional Details: Illustration (See Note)	Constructional Details: Description
52		**TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)** (22) Transverse butt-welded splices made on a backing bar. The end of the fillet weld of the backing strip shall stop short by greater than 10 mm from the edges of the stressed plate. (23) Transverse butt welds as for (22) With taper on width or thickness <1:2.5.
37		**TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)** (24) Transverse butt welds as (22) where fillet welds end closer than 10 mm to plate edge.
52	 (25)	**CRUCIFORM JOINTS WITH LOAD-CARRYING WELDS** (25) Full penetration welds with intermediate plate NDT inspected and free of defects. Maximum misalignment of plates either side of joint to be <0.15 times the thickness of intermediate plate.
41	(26) 	(26) Partial penetration or fillet welds with stress range calculated on plate area.
27	(27) (26) & (27)	(27) Partial penetration or fillet welds with stress range calculated on throat area of weld.
46		**OVERLAPPED WELDED JOINTS** (28) Fillet welded lap joint, with welds and overlapping elements having a design capacity greater than the main plate. Stress in the main plate to be calculated on the basis of area shown in the illustration.

Note: The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

Table 5 (continued)

Detail Category	Constructional Details			
	Illustration (See Note)			Description
41	(29)			(29) Fillet welded lap joint, with welds and main plate both having a design capacity greater than the overlapping elements.
33	(30)			(30) Fillet welded lap joint, with main plate and overlapping elements both having a design capacity greater than the weld.
66	(31)	(32)		**WELDED ATTACHEMENTS (NON-LOAD CARYING WELDS) LONGITUDINAL WELDS** (31) Longitudinal fillet welds. Class of detail varies according to the length of the attachment weld as noted. (32) Gusset welded to the edge of a plate or beam flange. Smooth transition radius (r), formed by machining or flame cutting plus grinding. Class of detail varies according to r/b ratio as noted.
	—	1/3≤*r*/*b*		
59	*l*≤50 mm	—		
52	50<*l* ≤100 mm	1/6 ≤ r/ b<1/3		
37	100mm < *l*	—		
33	—	r/b<1/6		
59				**WELDED ATTACHMENTS** (33) Shear connectors on base material (failure in base material).
59	t ≤ 12 mm			**TRANSVERSE WELDS** (34) Transverse fillet welds with the end of the weld ≥10 mm from the edge of the plate. (35) Vertical stiffeners welded to a beam or plate girder flange or web by continuous or intermittent welds. In the case of webs carrying combined bending and shear design actions, the fatigue strength shall be determined using the stress range of the principal stresses. (36) Diaphragms of box girders welded to the flange or web by continuous or intermittent welds.
52	t < 12 `mm			

Note: The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

Table 5 (continued)

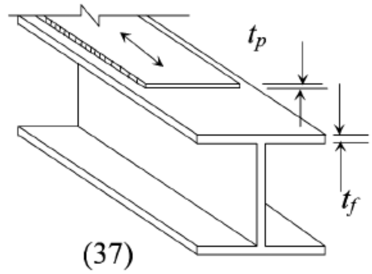
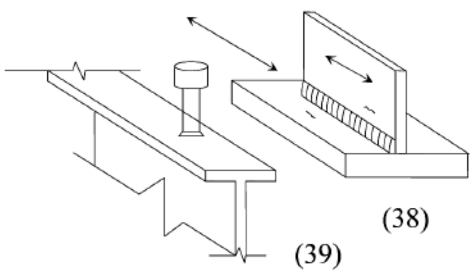
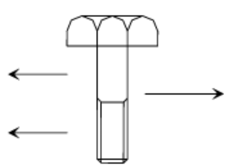
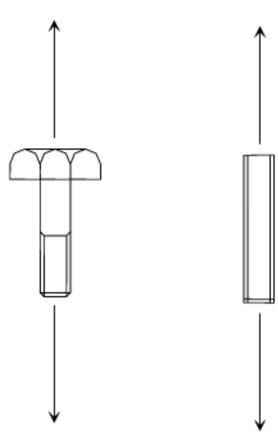
Detail Category	Constructional Details		
	Illustration (See Note)		Description
37	t_f or tp ≤ 25 mm	 (37)	**COVER PLATES IN BEAMS AND PLATE GIRDERS** (37) End zones of single or multiple welded cover plates, with or without a weld across the end. For a reinforcing plate wider than the flange, an end weld is essential.
27	t_f or tp > 25 mm		
67	 (38) (39)		**WELDS LOADED IN SHEAR** (38) Fillet welds transmitting shear. Stress range to be calculated on weld throat area. (39) Stud welded shear connectors (failure in the weld) loaded in shear (the shear stress range to be calculated on the nominal section of the stud).

Table 6 Detail Category Classification Group - 3 Bolts

Detail Category	Constructional Details	
	Illustration (See Note)	Description
83	 (40)	**BOLTS IN SHEAR (8.8/TB BOLTING CATEGORY ONLY)** (40):Shear stress range calculated on the minor diameter area of the bolt (A_c). **Note:** If the shear on the joint is insufficient to cause slip of the joint the shear in the bolt need not be considered in fatigue.
27	 (41)	**BOLTS AND THREADED RODS IN TENSION (tensile stress to be calculated on the tensile stress area A_t)** (41): Additional forces due to prying effects shall be taken into account. For tensional bolts, the stress range depends on the connection geometry. **Note:** In connections with tensioned bolts, the change in the force in the bolts is often less than the applied force, but this effect is dependent on the geometry of the connection. It is not normally required that any allowance for fatigue be made in calculating the required number of bolts in such connections.

Note: The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

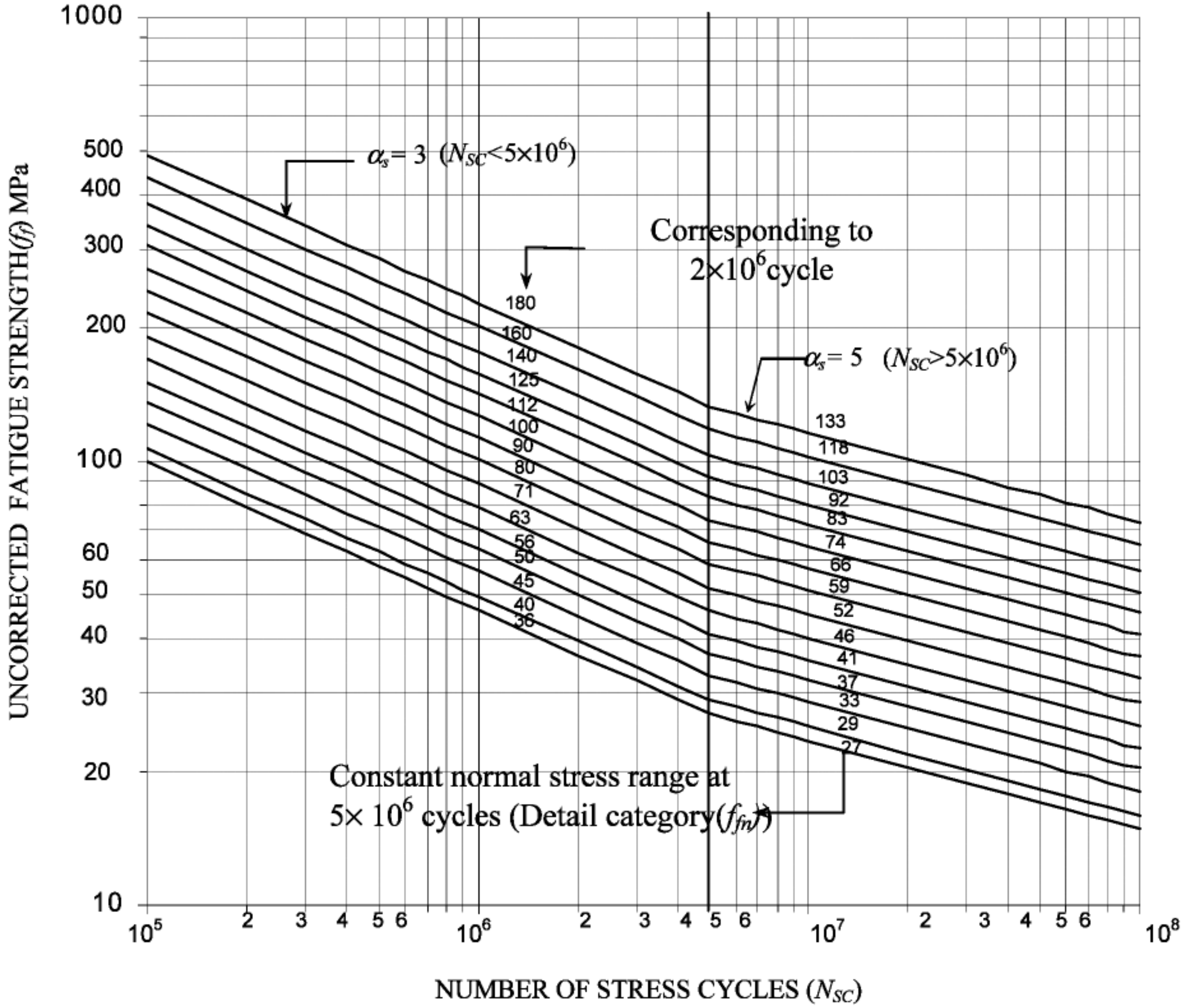


Fig. 7a S-N Curve for Normal Stress

605.4 Fatigue Assessment

The design fatigue strength for N_{SC} life cycles (f_{fd}, τ_{fd}) is given as

$$f_{fd} = \mu_r f_f / \gamma_{mft} \qquad \text{... 5.4}$$

$$\tau_{fd} = \mu_r \tau_f / \gamma_{mft} \qquad \text{... 5.5}$$

where,

μ_r = correction factor (discussed in 605.2)

γ_{mft} = partial safety factor against fatigue failure, given in **Table 3**.

f_f, τ_f = normal and shear fatigue strength ranges for the actual life cycle, N_{SC}, obtained from **Section 605.3** for the detail.

Stress limitations:

The (absolute) maximum value of the normal and shear stresses shall never exceed the elastic limit (f_y, τ_y) for the material under cyclic loading.

The maximum stress range shall not exceed 1.5 f_y for normal stresses and 1.5 $f_y/\sqrt{3}$ for the shear stresses under any circumstance.

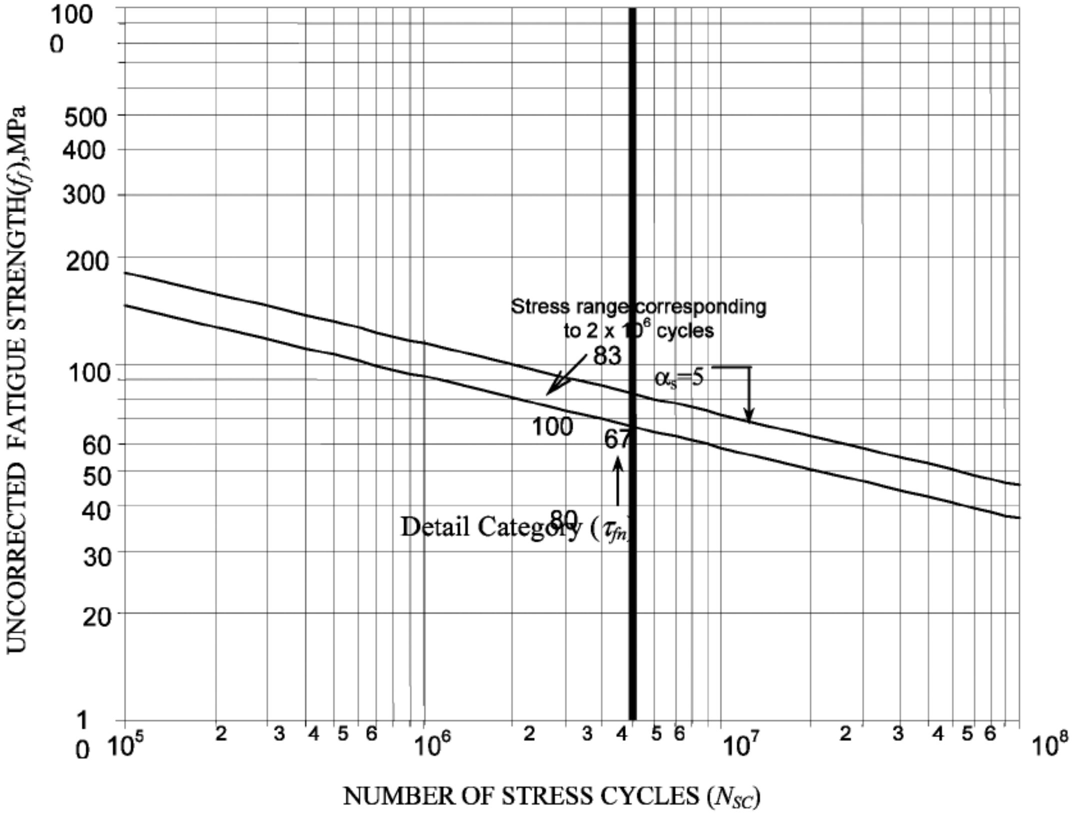


Fig. 7b S-N Curve for Shear Stress

Constant Stress range:

The actual normal and shear stress range f and τ at a point of the structure subjected to N_{SC} cycles in life shall satisfy.

$$f \leq f_{fd}$$

$$p \leq \tau_{fd}$$

Variable Stress range:

Fatigue assessment at any point in a structure, wherein variable stress ranges f_{fi} or τ_{fi} for n_i number of cycles (i =1 to r) are encountered, shall satisfy the following;

1. For Normal Stress (f)

$$\frac{\sum_{i=1}^{\gamma_5} n_i f_i^{\,3}}{5\times10^6\left(\mu_r f_{fn}/\gamma_{mft}\right)^3} + \frac{\sum_{j=\gamma_5}^{r} n_j f_j^{\,5}}{5\times10^6\left(\mu_r f_{fn}/\gamma_{mft}\right)^5} \leq 1.0 \qquad \text{... 5.6}$$

2. For Shear Stresses (τ)

$$\sum_{i=1}^{r} n_i \tau_{fi}^{\,5} \leq 5\times10^6\left(\mu_r \tau_{fn}/\gamma_{mft}\right)^5 \qquad \text{... 5.7}$$

Where lower case γ_s is the summation upper limit of all the normal stress ranges (f_i) having magnitude lesser than ($\mu_r f_{fn} / \gamma_{mft}$) for that detail and the lower limit of all the normal stress ranges (f_j) having magnitude greater than ($\mu_r f_{fn} / \gamma_{mft}$) for the detail. In the above summation all normal stress ranges, f_i, and t_i having magnitude less than 0.55 $\mu_r . f_{fn}$, and 0.55 $\mu_r . \tau_{fn}$ may be disregarded.

605.5 Exemption from Fatigue Assessments

No fatigue assessment is required if the following conditions are satisfied:

1. The highest normal stress range $f_{f\,max}$ satisfies

   $$f_{f\,max} \leq 27\mu_r/\gamma_{mft} \qquad \text{... 5.8}$$

2. The highest shear stress range τ_{fmax} satisfies

   $$\tau_{fmax} \leq 67\mu_r/\gamma_{mft} \qquad \text{... 5.9}$$

3. The total number of actual stress cycles N_{SC}, satisfies

   $$N_{SC} \leq 5 \times 10^6 \left(\frac{27\,\mu_r}{\gamma_{mft}\, f_{feq}} \right)^3 \qquad \text{... 5.10}$$

where,

$f_{f\,eq}$ = equivalent constant amplitude stress range in MPa given by

$$f_{feq} = \left[\frac{\sum_{i=1}^{\gamma_s} n_i f_{fi}^{\,3} + \sum_{j=\gamma_s}^{r} n_j f_{fj}^{\,5}}{n} \right]^{1/3} ; \qquad n = \sum_{i=1}^{\gamma} n_i$$

f_{fi}, f_{fj} = stress ranges falling above and below the f_{fn}, the stress range corresponding to the detail at 5 x 10^6 number of life cycles.

606 SHEAR CONNECTOR

606.1 Design of Shear Connectors

1. Shear connection and transverse reinforcement shall be provided to transmit the longitudinal shear force between the concrete and the structural steel element, ignoring the effect of natural bond between the two.
2. Shear connectors may be either of mild steel or high tensile steel irrespective of the grade of steel used in the parent girder. Flexible shear connectors are preferred because of their better performance. Channel, Angle and Tee shear connectors may be of mild steel, whereas, the shear studs may be mad of high tensile steel.
3. Shear connector shall be generally designed for full transfer of longitudinal shear. Shear connector strength and spacing are to be checked separately for all the limit states using appropriate factored load combinations and factored

strength. All shear connectors should be capable of resisting uplift of slab from steel section. Channel and stud shear connectors provide adequate safety against uplift. Headed stud shear connectors may be assumed to provide sufficient resistance to uplift, unless the shear connection is subjected to direct tension, in which case they should be supplemented by anchoring devices.

4. For verification for ultimate limit states, the shear connectors provided in terms of size and shape may be kept constant over any length where the design longitudinal shear per unit length does not exceed the design shear resistance by more than 10 percent. Over every such length, the total design longitudinal shear force should not exceed the total design shear resistance.

606.2 Longitudinal Shear

Longitudinal Shear load on connectors of composite section, whether simply supported or continuous is to be calculated for service and fatigue limit states on the basis of elastic theory using appropriate sectional properties based on effective widths and modular ratios as per the load history and development of composite action.

606.3 Design Strength of Shear Connectors

Shear connectors are to be checked for ultimate limit and fatigue limit states.

606.3.1 *Ultimate strength of shear connectors*

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

1. Stud Connectors

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

$$Q_u = \frac{0.8 f_u .\pi .d2/4}{\gamma_v} \leq \frac{0.29\alpha .d^2 \sqrt{f_{ck(cy)} .E_{cm}}}{\gamma_v} \qquad \text{... 6.1}$$

where,

$$\alpha = 0.2\left\{\frac{h_s}{d}+1\right\} \text{ for } 3<\frac{h_s}{d}<4 \quad \text{and} \quad \alpha = 1.0 \text{ for } \frac{h_s}{d} \geq 4$$

Q_u = design strength of stud in newton (N)

γ_v = partial safety factor for stud connector = 1.25

d = diameter of the shank of the stud in millimeters (mm)

f_u = ultimate tensile strength of the stud material $\leq$ 500 N/mm^2

$f_{ck(cy)}$ = Characteristic cylindrical compressive strength of concrete = 0.8 fck

h_s = nominal height of stud in millimeters (mm)

E_{cm} = Secant modulus of elasticity of concrete (Refer: **Table–III.1** of **Annexure-III)**

2. Channel Connectors

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

$$Q_n = \left[20b.(h)^{3/4}.(f_{ck(cy)})^{1/3}\right] / \gamma_v \qquad \text{... 6.2}$$

where,

Q_u = design strength of channel in newton (N)
b = length of the channel in millimeters (mm)
h = height of the channel in millimeters (mm)

While using channel shear connectors the following recommendation need to be followed.

1. The height h of the channel should not exceed 20 times 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 leg length of the weld connecting the channel to the plate should not exceed half the plate thickness.

The design strengths of some standard shear connectors have been given in **Table 7** for easy reference.

Table 7 Ultimate Static Strengths of Shear Connectors (Q_u for Different Concrete Strengths)

Type of Connector		Connector Material	Ultimate Static Strength in kN Per Connector for Concrete Strengths f_{ck} (MPa)			
			25	**30**	**40**	**50**
Stud connectors		Material with a characteristic yield strength of 385 MPa, minimum elongation of 18% and a characteristic tensile strength of 495 MPa				
Nominal Diameter (mm)	Overall height (mm)					
25	100		112	125	149	156
22	100		87	97	115	120
20	100		72	80	95	100
20	75		68	76	91	100
16	75		46	51	61	64
12	65		26	29	34	36
Channels: 150 mm long (min)		As per IS 2062				
ISMC 125			244	259	285	307
ISMC 100			206	219	241	260
ISMC 75			166	176	194	209

Notes: 1. f_{ck} is the specified characteristic cube strength at 28 days.
2. Strengths for concrete of intermediate grade may be obtained by linear interpolation.

3. For channels of lengths different from those quoted above, the capacities are proportional to the lengths for lengths greater than 150 mm.
4. 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.
5. For stud connectors of overall height greater than 100 mm the design static strength should be taken as the values given in the table for 100 mm high connectors.
6. The above provisions of Stud Connectors are not applicable to composite slab using profiled deck. Static strength of shear connector in such cases can be established by experimental push-out tests.
7. The number of shear connectors given by the above table shall be distributed in the zone between the maximum and the zero moment sections. The number of connectors required from fatigue consideration will usually exceed the requirement from flexural strength. However, if the flexural requirement exceeds the number required from fatigue point of view as mentioned in **Clause 606.4.2** additional connectors should be provided to ensure that the ultimate strength of the composite section is achieved.
8. 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 be as specified in **Clause 606.4.1**.

606.3.2 *Fatigue Strength of Shear Connectors*

The fatigue shear stress range (fatigue Strength) of shear connector shall be obtained from **Fig. 7b** corresponding to the design load life cycle, N_{SC}.

The strength shall be determined as given below:

$$\tau_f = \tau_{fn} \sqrt[5]{5x10^6 / N_{SC}} \qquad \text{... 6.3}$$

τ_{fn} = design normal and shear fatigue stress range respectively of the detail for 5 x10^6 cycles as given in **Table 5**.

τ_{fn} = 67 N/mm^2 for stud connector (refer:- **Table 5**, Detail Category–67)

τ_{fn} = 59 N/mm^2 for channel connector (Ref. Table 5, Detail Category–59) [provided that the thickness of the top flange of steel girder 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 have been indicated in **Table 8**.

Table 8 Nominal Fatigue Strengths Q_r (in *kN*)

Type of Connectors	Connector Material	N = Nos. of Cycles				
		1 x 10^5	5 x 10^5	2 x 10^6	1 x 10^7	1 x 10^8
Headed Studs ϕ 25	f_y = 385	71	52	39	28	18
Headed Studs ϕ 22	f_u = 495	55	40	30	22	14
Headed Studs ϕ 20	Elongation = 18%	46	33	25	18	11
Headed Studs ϕ 16		29	21	16	11	7
Channel 150 long for a nominal weld of 8 mm	IS:2062	109	79	60	43	27

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

606.4 Spacing and Design of Shear Connectors

606.4.1 *Ultimate Limit State (Strength Criteria)*

Calculated shear V_L at interface corresponding to vertical shear is as given below:

$$V_L = \sum \left[\frac{V.A_{ec}.Y}{I} \right]_{dl,ll} \qquad \text{... 6.4}$$

where,

V_L Longitudinal shear per unit length

V The vertical shear forces due to dead load and live load (including impact) separately at each state of load history.

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

Y C.G. distance of transformed concrete area from neutral axis.

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

dl, ll Different load history condition, i.e. sustained load or composite action dead load, transient load or composite action live load. These loads are to be considered with appropriate load factor at this stage.

Spacing of Shear connectors is given as $S_{L1} = \dfrac{\sum Q_u}{V_L}$

Q_u is the Ultimate static strength of one shear connector (Clause 606.3.1 and **Table 7**) and the summation is over the number of shear studs at one section.

606.4.1.1 For full shear connection

The maximum longitudinal force due to bending moment is to be calculated over the shear span from zero moment to maximum moment section and is given by

$$H_1 = A_{sl} \cdot f_y \cdot 10^{-3}/\gamma_m \qquad \text{... 6.5}$$

$$H_2 = 0.36 \cdot f_{ck} \cdot A_{ec} \cdot 10^{-3} \qquad \text{... 6.6}$$

where,

H_1, H_2 = Longitudinal Force due to bending (kN)

A_{sl} = Area of Tensile Steel (mm^2) in longitudinal direction

A_{ec} = Effective area of concrete

= $b_{eff} \cdot x_u$ (for neutral axis within the slab)

= $b_{eff} \cdot d_s$ (for neutral axis in steel section)

The ultimate flexural strength of any composite construction is governed by either of the aforesaid equation. Therefore, the maximum possible compressive force in the composite beam will be governed by H, which is the smaller of H_1 and H_2 and sufficient connectors should be provided to resist the longitudinal force H.

Spacing of Shear Connectors is given as $S_{L2} = \frac{\sum Q_u}{H}.L$

606.4.2 *Serviceability Limit State (Limit State of Fatigue)*

Calculated longitudinal shear per unit length, V_r at interface due to live load and impact is as given below:

$V_r = \sum\left[\frac{V_R.A_{ec}.Y}{I}\right]_{ll}$ {V_r, A_{ec}, Y and I, are as explained above}

V_R Vertical Shear difference due to maximum and minimum shear envelop due to live load and impact

ll is live load with impact

Spacing of Shear connectors from fatigue consideration is given as $S_R = \frac{\sum Q_r}{V_r}$

Q_r is the Nominal fatigue Strength one shear connector which is to be taken from **Table 8**.

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

606.5 Partial shear Connection

When all cross-sections are in Class 1 or Class 2, partial shear connection may be used for girders. The number of connectors shall then be determined by a partial connection theory taking into account the deformation capacity of shear connector. The bending resistance of the section under partial shear connection shall be determined as per **Clause I.1.2** of **Annexure–I**.

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 following limits for the degree of shear connection, which may be generally defined as, $S_c = n_p/n_f$

[Where, S_c is the Degree of shear connection n_f is the number of shear connectors required for full shear connection determined for the length of a beam under consideration and n_p is the number of shear connectors provided within that same length]

1. For steel sections with equal flanges:

$L_e \leq 25 \quad S_c \geq 1 - \left[\frac{355}{f_y}\right].(0.75 - 0.03L_e) \qquad S_c \geq 0.4$ ... 6.7

$L_e > 25 \quad S_c \geq 1.0$ ... 6.8

2. For steel sections having a bottom flange with an area of three times the area of top flange with equal flanges:

$$L_e \leq 20 \quad S_c \geq 1 - \left[\frac{355}{f_y}\right].(0.30 - 0.015L_e) \quad S_c \geq 0.4 \qquad \text{... 6.9}$$

$$L_e > 20 \quad S_c \geq 1.0 \qquad \text{... 6.10}$$

Where L_e overall span for a simply supported girder and is the distance between the point of zero bending moment within one span of a continuous girder.

3. 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 (1) and (2) above by linear interpolation.

606.6 Detailing of Shear Connector

Details as shown in following sketch (**Fig. 8**) are to be followed.

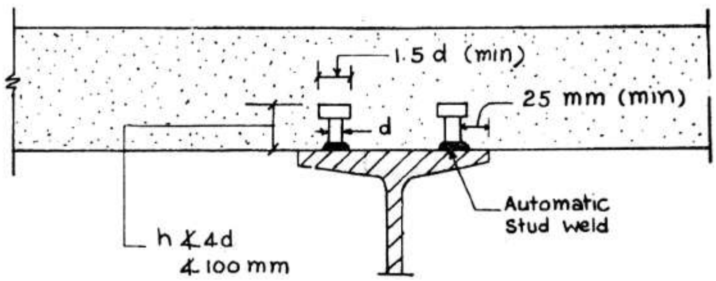


Fig. 8a Details of Stud Connector

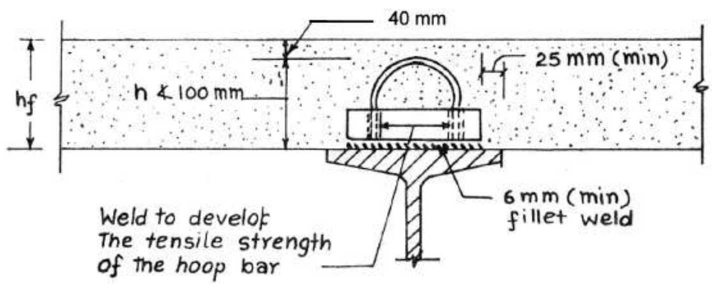


Fig. 8b Details of Angle/Channel Connector

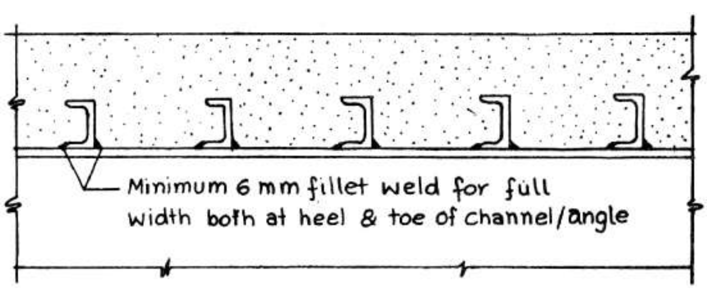


Fig. 8c Longitudinal Section

Fig. 8 Details of Connector on Steel Girders

The diameter of the stud connector welded to the flange plate shall not exceed twice the plate thickness. The height of the stud connectors shall not be less than four times their diameter or 100 mm. The diameter of the head of the stud shall not be less than one and a half times the diameter of the stud. The leg length of the weld joining other types of connectors to the flange plate shall not exceed half the thickness of the flange plate. 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.

606.6.1 *Precautions for separation of steel girder from concrete*

To resist separation with the steel girder, top flange of stud and channel shear connectors shall extend into the deck slab at least 40 mm above bottom transverse reinforcements and also a minimum of 40 mm into the compression zone of concrete flange. Where a concrete haunch is used, between the steel flange and the soffit of the slab, top flange of the stud or channel shear connectors shall extend upto at least 40 mm above the transverse reinforcements in the haunches, provided the reinforcements are sufficient to transfer longitudinal shear. Where shear connectors are placed adjacent to the longitudinal edge of the slab, transverse reinforcement provided in accordance with **Clause 606.11** shall be fully anchored in the concrete between the edge of the slab and the adjacent row of connectors.

606.6.2 *Overall height of connector*

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.

606.7 Details of Haunches

The dimensions for haunches if provided between top of steel girder and soffit of slab shall be as indicated in **Fig. 9**, the sides of haunches being located outside a line drawn at 45 degrees from the outside edge of the base of the connector.

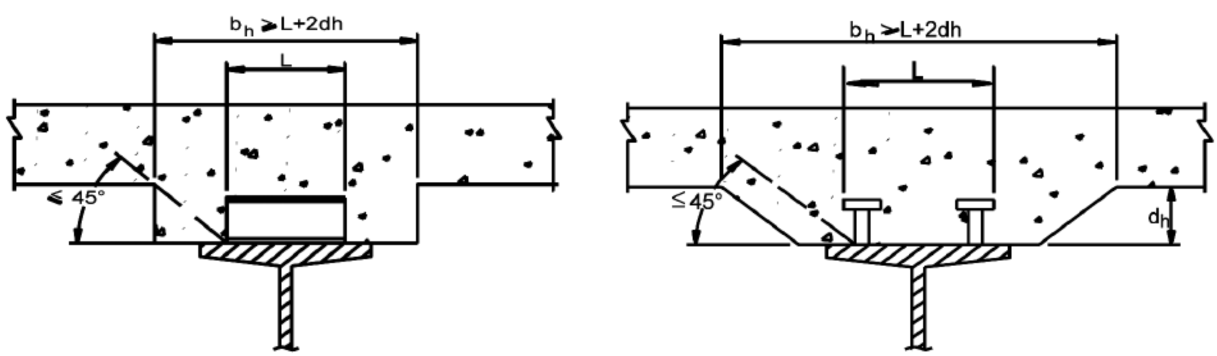


Fig. 9 Dimension of Haunches

606.8 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.10**.

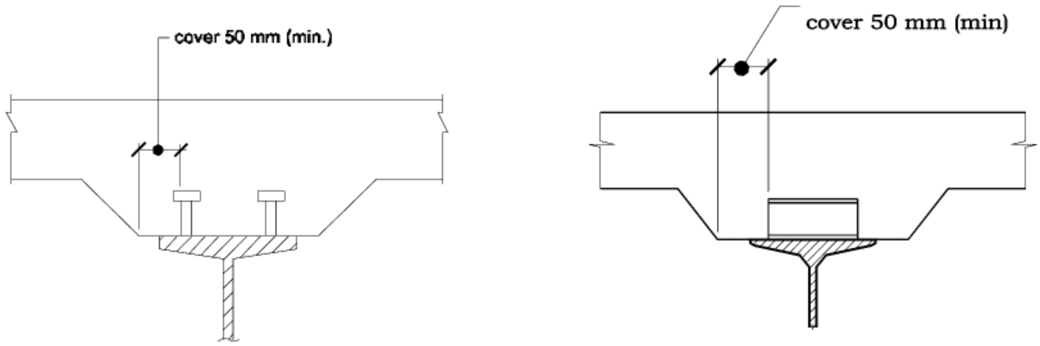


Fig. 10 Cover to Connectors

606.9 Limiting Criteria for Spacing of Shear Connectors

1. Where a steel compression flange that would otherwise be in a lower class is assumed to be in Class 1 or Class 2 because of 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 (e.g. solid slab)

$$S_L \leq 21+_f\sqrt{250/f_y} \qquad \text{... 6.11}$$

b) Where the slab is not in contact over the full length (e.g. slab with ribs transverse to the beam)

$$S_L \leq 14+_f\sqrt{250/f_y} \qquad \text{... 6.12}$$

where,

t_f is the thickness of the flange

f_y is the nominal yield strength of the flange N/mm^2.

S_L is the 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 $9.t_f\sqrt{250/f_y}$ or 50 mm which whichever is less.

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

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

606.10 Transverse Shear Check

Shear connectors transfer longitudinal shear from steel girder to slab concrete abutting them, where from the same is transferred to the rest of the slab through transverse shear strength of slab as well as transverse reinforcements provided. The strength and amount of reinforcement is to be checked by the following relations.

The shear force transferred per meter length V_L shall satisfy both the following conditions:

1. $V_L \le 0.632 L \sqrt{f_{ck}}$ ... 6.13

or

$V_L \le 0.232 L \sqrt{f_{ck}} + 0.1 . A_{st} . f_{yk} . n$ ... 6.14

where,

V_L = Longitudinal shear force per unit length calculated for 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 plans envelop as indicated in **Fig. 11**

n = Number of times each lower transverse reinforcing bar is intersected by a shear surface (i.e. the number of rows of shear connector at one section of the beam). Generally for T-beam n = 2 and for L-beam n = 1

A_{st} = Sectional areas (in cm^2) of transverse reinforcements per metre run of beam

The amount of transverse steel in the bottom of the slab shall not be less than $\frac{2.5 . V_L}{f_{yk}}$ cm^2/m, where V_L is in KN/m.

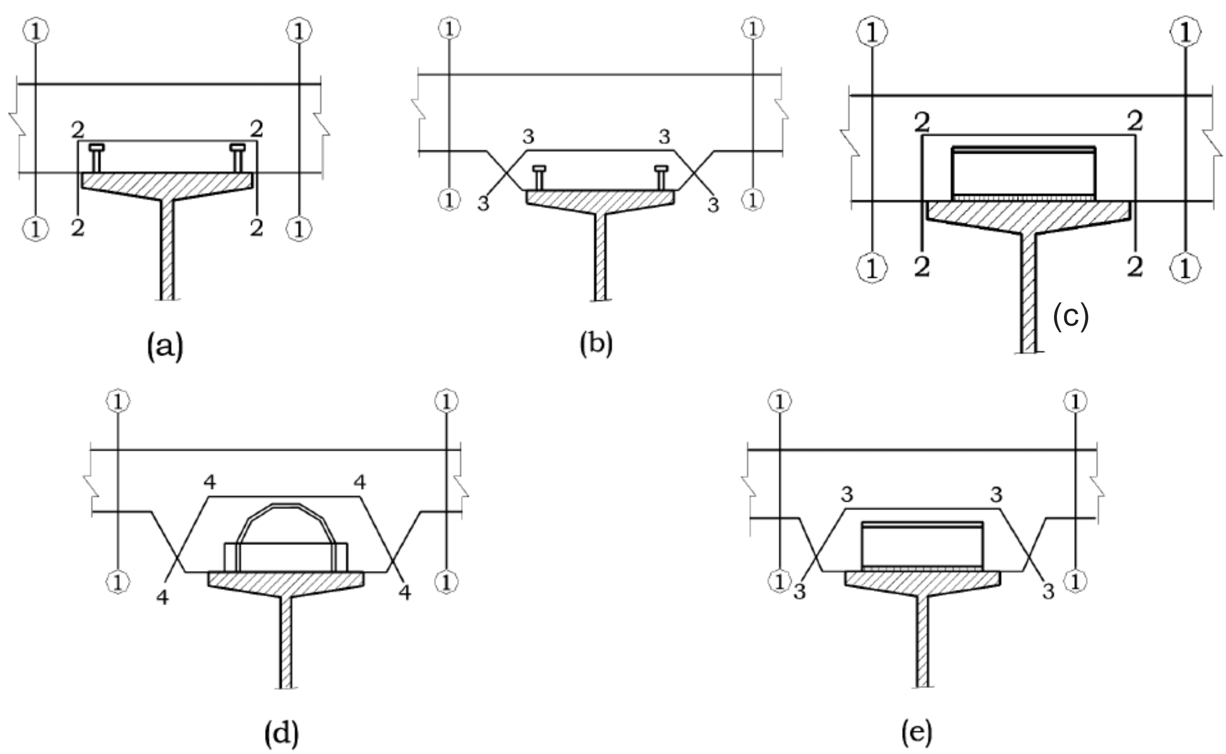


Fig. 11 Typical Shear Planes

606.11 Transverse Reinforcements

Planes, which are critical for longitudinal shear failure, in the process of transfer of longitudinal shear from the girder to the slab, are of four main types, as shown in **Fig.11**. 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 girder to the effective width of the slab. The area of transverse reinforcement per unit length of beam will be the sum total of all the reinforcement (A_t, A_h or A_b as shown in **Fig. 12 a, b and c**), which are intersected by the shear plane and are fully anchored on both the sides of the shear plane considered.

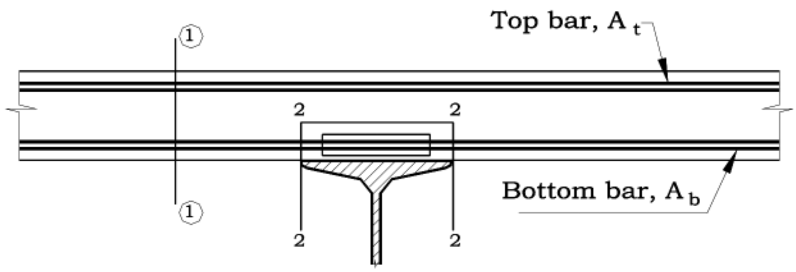


Fig. 12a Non-Haunched Beam

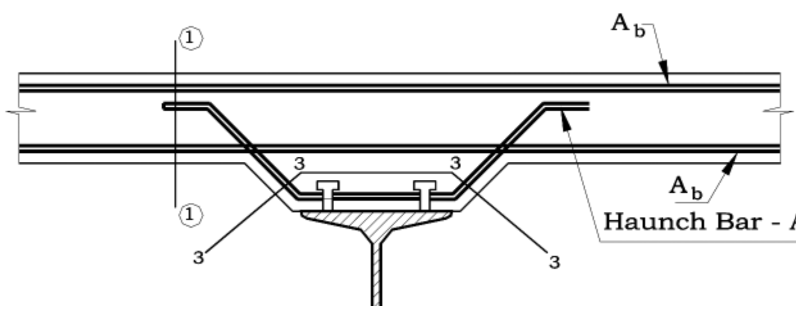


Fig. 12b Large Haunched Beam

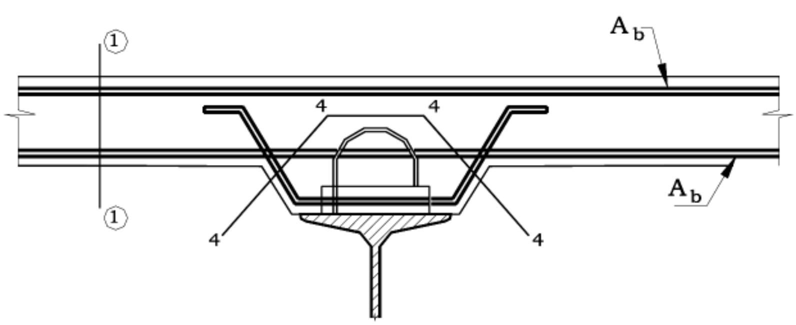


Fig. 12c Small Haunched Bean

Fig. 12 Transverse Reinforcement Across Shear Planes

606.12 Total Transverse Reinforcements

The total transverse reinforcements A_{st}, per unit length of 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) (see **Fig. 12a**). Area of reinforcements A_t and A_b include those provided for flexure. The total transverse reinforcements across plane 2–2 (**Fig. 12a**) is $A_{st} = 2A_b$ and that across plane 3–3 (**Fig. 12b**) is $A_{st} = 2A_h$ as these planes do not cross the full thickness of the slab. In case of shear plane 4–4 (**Fig. 12c**), the total transverse reinforcement is $A_{st} = 2 (A_b + A_h)$.

The transverse reinforcements shall be placed at locations as shown in **Fig. 13**. The haunch bars shall be extended beyond the junction of bottom bars by a length equal to the anchorage length.

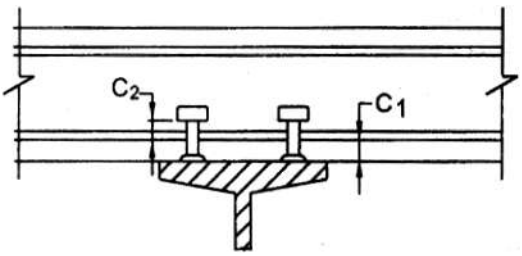


Fig. 13a Stud Connector in Un-Haunched Beam

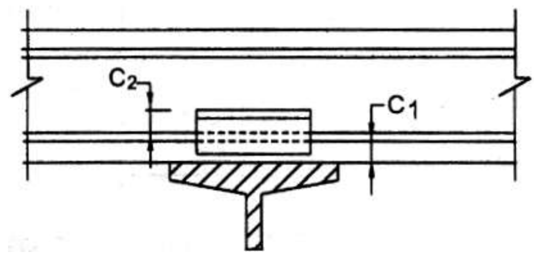


Fig. 13b Channel Connector in Un-Haunched Beam

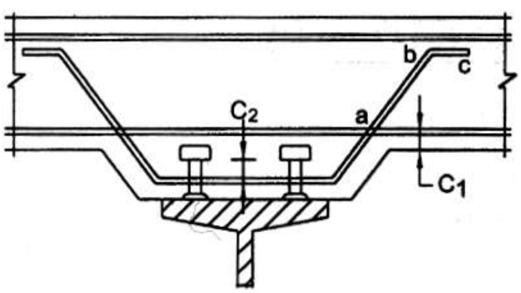


Fig. 13c Stud Connector in Haunched Beam

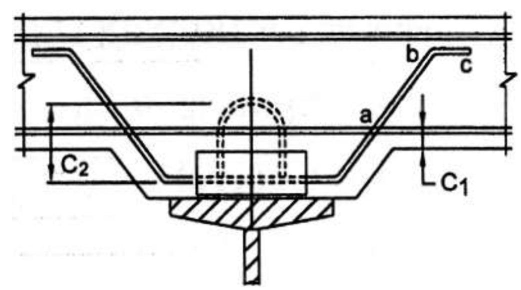


Fig. 13d Channel Connector in Haunched Beam

Fig. 13 Arrangement of Transverse reinforcements

607 COMPOSITE COLUMNS

607.1 General

1. This Clause applies to columns and compression members with steel grade conforming to IS 2062 and normal weight concrete of strength M25 to M90.
2. This Clause applies to isolated columns and compression members in framed structures where the other structural members are either composite or steel members.
3. The steel contribution ratio δ should fulfill the criteria:

   $0.2 \leq \delta \leq 0.9$ where δ is defined in section 607.6.
4. The influence of local buckling of the steel section on the resistance of the composite section as a whole shall be considered for design.
5. The effects of local buckling may be neglected for a steel section fully encased in accordance with **Section 607.2**, and for other types of cross-section provided the maximum width to thickness ratio given in **Section 607.4** are not exceeded.
6. Composite Columns can be of two types:
   a) Encased where concrete encases the steel section [**Fig. 14a**]

b) In-filled where concrete fills the hollow tubular or hollow box Section [**Fig. 14b**].

607.2 Construction Particular

1. In composite columns consisting of fully encased steel sections, concrete shall be adequately held by steel wires and stirrups and with all round cover of at least 40 mm or one-sixth of the breadth b of the flange, over steel section that should be unpainted but cleaned at abutting surface to ensure protection against corrosion and spalling of concrete. Shear transfer between steel concrete interfaces is ensured basically through bond for which calculated shear stress at interface shall be kept limited in accordance with **Table 13**, beyond which mechanical shear connectors are to be provided.
2. The cover to reinforcement should be in accordance with IRC:112-2011.

607.3 Members under Axial Compression

Standard composite sections used as columns are as shown below:

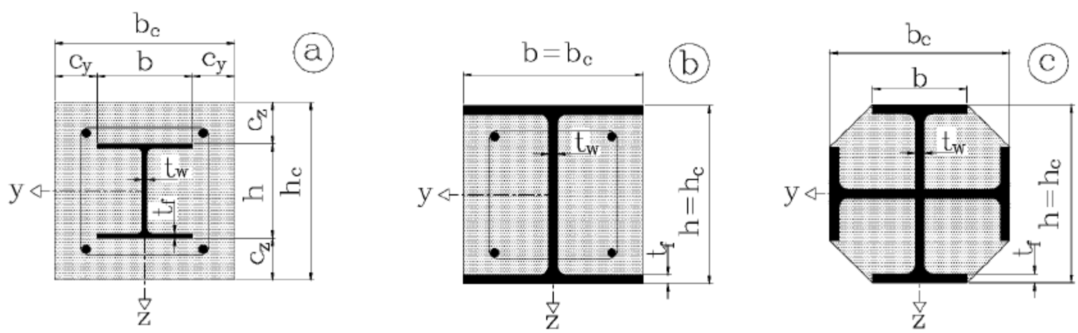


Fig. 14a Fully and Partially Concrete Encased Columns

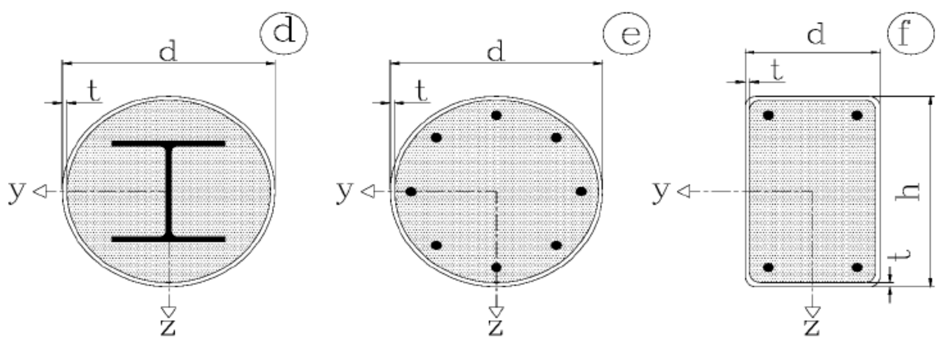


Fig. 14b Concrete Filled Hollow Sections

607.3.1 *General Design Philosophy*

1. Design for structural stability shall consider second-order effects including residual stresses, geometrical imperfections, local instability, cracking of concrete, creep and shrinkage of concrete and yielding of structural steel and of reinforcement.
2. Second-order effects shall be considered in any direction in which failure might occur, if they affect the structural stability significantly.

3. Internal forces should be determined by elasto-plastic analysis.
4. Full composite action between steel and concrete shall be considered up to failure.
5. 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 and longitudinal force resulting from permanent loads is not greater than 10 percent.

607.3.2 *Design Guidelines*

1. For a fully encased steel section, see **Fig. 14a**, limits to the maximum thickness of concrete cover that may be used in calculation are:

   $C_{z\,(max)} = 0.3\,h$

   $C_{y\,(max)} = 0.4\,b$

2. The longitudinal reinforcement that may be used in calculation should not exceed 6 percent of the concrete area.
3. The ratio of the cross-section's depth h_c to width b_c, see **Fig. 14a**, should be within the limits.

607.4 Local Buckling of Steel Section

The equations for determining the plastic resistance of an encased steel section or concrete filled rectangular/square or circular tubular section P_p as discussed subsequently in the next clauses is valid provided that local buckling in the steel sections does not occur. To prevent premature local buckling, the width to thickness ratio of the steel sections in compression must satisfy the following limits:

- $\frac{d}{t} \leq 85\varepsilon^2$ for concrete filled circular tubular sections
- $\frac{h}{t} \leq 50\varepsilon$ for concrete filled rectangular tubular sections
- $\frac{b}{t_f} \leq 43\varepsilon$ 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

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

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.

607.5 Short Compression Members

A compression members is termed as a 'short compression member' when its non-dimensional slenderness ratio $\bar{\lambda} \leq 0.2$.

Non-dimensional slenderness ratio $\bar{\lambda}$ is given as

$$\bar{\lambda} = \sqrt{\frac{P_{pu}}{P_{cr}}},$$

where,

P_{pu} is the plastic resistance of the cross-section to compression and is given

$$P_{pu} = A_s.f_y + \alpha_c.A_c.[0.80 f_{ck}] + A_{st}.f_{yk} \qquad \text{... 7.1}$$

where,

f_y, yield strength of steel section.

f_{ck} characteristic compressive strength (cube strength) of the concrete

f_{yk} yield strength of the reinforcing steel

α_c strength coefficient for concrete

= 1.0 for confined concrete in tubular sections

= 0.85 for fully or partially concrete encased steel sections with lateral ties

= 0.89 for fully or partially encased concrete columns with spiral ties.

` [**Note:** While providing spiral ties it must be ensured that the ratio of the volume of helical reinforcement to the volume of the core is not less than $0.36(A_g/A_{co}-1)f_{ck}/f_{yk}$

(A_g = Gross area of section; A_{co} = Area of core of the helically reinforced column measured to the outer diameter of the helix)]

P_{cr} is the elastic buckling load of the column

$$P_{cr} = \frac{\pi^2 (EI)_e}{l^2}$$

Where $(EI)_e$ is the effective elastic flexural stiffness of the composite column. l is the effective length of the column, which may be conservatively taken as system length L for an isolated non-sway composite column.

607.5.1 *Encased Steel Sections and Concrete filled Rectangular/Square Tubular Sections*

The plastic resistance of an encased steel section or concrete filled rectangular or square section (i.e. α. the so-called "squash load") is given by

$$P_p = A_s.f_y / \gamma_m + \alpha_c.A_c.[0.80 f_{ck}] / \gamma_c + A_{st}.f_{yk} / \gamma_s \qquad \text{... 7.2}$$

where, f_y, (f_{ck}), f_{yk} and αc are as discussed earlier.

607.5.2 *Concrete Filled Circular Tubular Sections*

There is an increased resistance and ductility of concrete due to the confining effect of the circular tubular section. However, this effect is significant only in stocky columns. In composite columns with a non-dimensional slenderness of $\bar{\lambda} \leq 0.5$ (see **Section 607.6 & 607.7**) or where the eccentricity of the applied load does not exceed the value $d/10$, (where d is the outer dimension of the circular tubular section) this effect has to be considered.

The plastic compression resistance of concrete filled circular tubular sections is calculated by using two coefficients η_1 and η_2 as given below.

$$P_p = A_s \eta_2 f_y / \gamma_m + 0.8 A_c \alpha_c f_{ck} \left[1 + \eta_1 \frac{t}{d} \frac{f_y}{f_{ck}} \right] / \gamma_c + A_{st} . f_{yk} / \gamma_s \qquad \text{... 7.3}$$

where,

t is the thickness of the circular tubular section, and η_1 and η_2 two coefficients which account for confinement effect and are given by

$\eta_1 = \eta_{10} \left[1 - \frac{10e}{d} \right]$ and

$\eta_2 = \eta_{20} + (1 - \eta_{20}) \frac{10\,e}{d}$

The resistance of a concrete filled circular tubular section to compression may increase by 15 percent under axial load only when the effect of tri-axial confinement is considered. Linear interpolation is permitted for various load eccentricities of $e \leq d/10$. The basic values η_{10} and η_{20} depend on the non-dimensional slenderness $\bar{\lambda}$, which can be read off from **Table 9**.

If the eccentricity '*e*' exceeds the value $d/10$, or if the non-dimensional slenderness exceeds the value 0.5 then $\eta_1 = 0$ and $\eta_2 = 1.0$.

Table 9 Basic Value η_{10} and η_{20} to Allow for the Effect of Tri-Axial Confinement

	$\bar{\lambda} = 0.0$	$\bar{\lambda} = 0.1$	$\bar{\lambda} = 0.2$	$\bar{\lambda} = 0.3$	$\bar{\lambda} = 0.4$	$\bar{\lambda} \geq 0.5$
η_{10}	4.90	3.22	1.88	0.88	0.22	0.00
η_{20}	0.75	0.80	0.85	0.90	0.95	1.00

607.6 Effective Elastic Flexural Stiffness

1. **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:

$$(EI)_e = E_s I_s + 0.6\, E_{cm} I_c + E_{st} . I_{st} \qquad \text{... 7.4}$$

where,

E_s and E_{st} are the modulus of elasticity of the steel section and the reinforcement respectively

E_{cm} is the secant modulus of the concrete

2. **Long Term Loading**: For slender columns the effect of long-term loading should be considered $\bar{\lambda} > 0.2$.

If the eccentricity '*e*' of loading is more than twice the cross-section dimension '*D*' or $e > 2D$, 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. Moreover, effect of long-term loading need not be considered if the non-dimensional slenderness, $\bar{\lambda}$ (Ref: **Section 607.6**) of the composite column is less than the limiting values given in **Table 10**.

Table 10 Limiting Values of $\bar{\lambda}$ for Long Term Loading

	Braced Non-Sway Systems	Un-Braced and/or Sway System
Concrete encased cross-section	0.8	0.5
Concrete filled cross-section	$\frac{0.8}{1-\delta}$	$\frac{0.5}{1-\delta}$
Note: δ is the steel contribution ratio defined as		

$$\delta = \frac{A_s . f_y}{P_p \gamma_m} \qquad \text{... 7.5}$$

When $\bar{\lambda}$ exceeds the limits prescribed above and $e/D < 2$, the effect of creep and shrinkage of concrete should be allowed for by adopting modulus of elasticity of concrete E_{cs} instead of E_{cd} where E_{cs} is defined as follows:

$$E_{cs} = 0.75\, E_{cm} \left[1 - \frac{0.5 P_{dd}}{P}\right] \qquad \text{... 7.6}$$

where,

P the applied factored load

P_{dd} the part of the applied factored load permanently acting on the column.

The effect of long-term loading may be ignored for concrete filled tubular sections with $\bar{\lambda} \leq 2.0$ provided that δ is greater than 0.6 for braced (non-sway) columns, and 0.75 for Unbraced (sway) columns.

607.7 Resistance of Member Subjected to Axial Compression

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

a) The axial force in the column is less than 0.1 P_{cr} where P_{cr} is the elastic buckling load of the column

b) The non-dimensional slenderness $\bar{\lambda}$ is less than 0.2.

To check the safety of a compression member, check for buckling about each principal axis of the composite section need to be done as below,

$$P \leq \chi P_p$$

Now,

P_p is the plastic resistance to compression as discussed in **Clause 607.3**.

$$\chi = \frac{1}{\phi + \sqrt{\phi^2 - \bar{\lambda}^2}} \qquad \text{... 7.7}$$

Where,

$$\phi = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right] \qquad \text{... 7.8}$$

$\bar{\lambda}$ = Non-dimensional slenderness ratio as discussed in **Clause 607.6**

α = Imperfection factor as given in **Table 11** which allows for different levels of imperfections and residual stresses in columns corresponding to curves *a*, *b* and *c*

The buckling curve to be adopted for design shall be selected according to type of section and the axis of bending as given blow:

Curve *a* for concrete filled tubular sections with reinforcement percentage less than 3 percent of gross cross section area

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

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

Table 11 Imperfection Factor α for the Buckling Curves

Buckling Curve	**a**	**b**	**c**
Imperfection Factor	0.21	0.34	0.49

607.8 Combined Compression and Bending

1. When the bending moment in the section is zero (i.e. M = 0), the permissible compression is as given in **Section 607.5** and **Section 607.7**. (Ref: Point A in interaction curve shown in **Fig. 15**.).

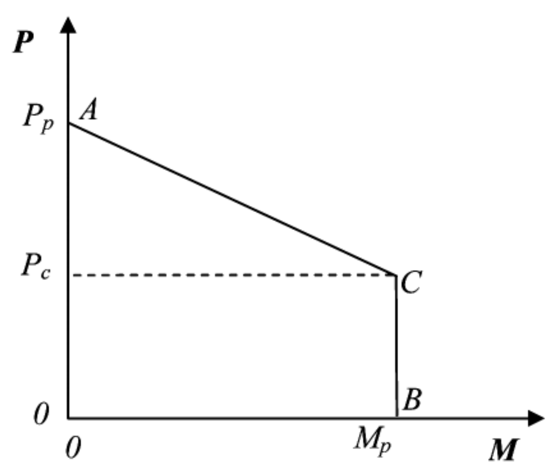


Fig. 15 Interaction Curve for Compression and Uni-Axial Bending Using the Simplified Method

2. For zero compressive force in the section the plastic moment of resistance of the cross section is given as (Ref: Point B in curve shown in **Fig. 15**)

$$M_p = (Z_{ps} - Z_{psn})\, f_y / \gamma_m + (Z_{pr} - Z_{prn})\, f_{yk} / \gamma_s + \alpha_c . 0.8 . (Z_{pc} - Z_{pcn})\, f_{ck} / \gamma_c \qquad \text{... 7.9}$$

where,

Z_{ps}, Z_{pr}, and Z_{pc} plastic section modulii of the steel section, reinforcement and concrete about their own centroids respectively

Z_{psn}, Z_{prn} and Z_{pcn} plastic section modulii of the steel section, reinforcement and concrete about neutral axis of gross cross section respectively

While determining the plastic resistance of a section the following criteria shall be considered:

1. The influence of transverse shear forces on the resistance to bending and axial force should be considered when determining the interaction curve. If the shear force V on the steel section exceeds 50 percent of the design shear resistance V_p of the steel the influence of the transverse shear on the resistance in combined bending and compression should be taken into account by a reduced design steel strength (1 - β) f_y/γ_m in the shear area A_v (β is determined as per **Section 603.3.3.3**.
2. Unless a more accurate analysis is used, V may be distributed into V_s acting on the structural steel and V_c, acting on the reinforced concrete section by :

$$V_s = V . \frac{M_{ps}}{M_p}$$

$$V_c = V - V_s$$

where,

M_{ps} is the plastic moment of resistance of steel section alone

M_p is the plastic moment of resistance of the entire composite section

607.8.1 *Second Order Effects on Bending Moment*

The second order effects on bending moments for isolated non-sway columns shall be considered if both of the following conditions are satisfied:

i) $\frac{P}{P_{cr}} > 0.1$

Where P is the design applied load, and P_{cr} is the elastic critical load of the composite column.

ii) Elastic slenderness conforms to:

$\bar{\lambda} > 0.2$

Where $\bar{\lambda}$ is the non-dimensional slenderness of the composite column.

In case the above two conditions are met, the second order effects may be allowed for by modifying the maximum first order bending moment (moment obtained initially), M_{max}, with a correction factor k, which is defined as follows:

$$k = \frac{C_m}{1 - \frac{P}{P_{cr}}} \geq 1.0$$

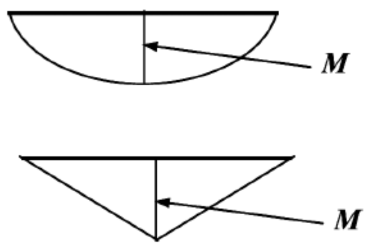
where,

P applied design load and

P_{cr} is the elastic critical load for the relevant axis and corresponding to a modified effective flexural stiffness given by $(EI)_{em}$ with the effective length taken as the composite column length

C_m equivalent moment factor given in **Table 12**.

Table 12 Imperfection Factor α for the Buckling Curves

Moment Distribution	Moment Factor (C_m)	Comment
	First-order bending moments from member imperfection or lateral load:: $C_m = 1.0$	M is the maximum bending moment within the column length ignoring second-order effects
	End Moments: $C_m = 0.66 + 0.44\ r \geq 0.44$	M or $r.M$ are the end moments from first-order or second-order global analysis.

A simplified approach in considering the value of C_m is as indicated below:

C_m = 0.85 for members whose ends are restrained against rotation

= 1.0 for members whose ends are un-restrained against rotation

$(EI)_e$ = $0.9\ (E_s I_s + 0.5\ E_{cm} I_c + E_{st}.I_{st})$

607.8.2 *Resistance of Members Subjected to Combined Compression and Uniaxial Bending Moment*

1. The section shall be checked for resistance under axial compression for both x and y-axes.
2. 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 is adequate when the following condition is satisfied:

$$M \leq 0.9\ \mu.M_p$$

Where M is the design bending moment, which may be factored to allow for second order effects, if necessary, as described in **Section 607.8.1(2)**.

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

$$\mu = \frac{(\chi - \chi_d)}{(1-\chi_c)\chi} \qquad \text{when } \chi_d \geq \chi_c \qquad \text{... 7.10}$$

and

$$\mu = 1 - \frac{(1-\chi)\chi_d}{(1-\chi_c)\chi} \qquad \text{when } \chi_d \geq \chi_c \qquad \text{... 7.11}$$

Where,

χ_c = Axial resistance ratio due to concrete = $\frac{P_c}{P_p}$

Where, P_c = $0.8\alpha_c.f_{ck}.A_c/\gamma_c$

χ_d = Design axial resistance ratio = $\frac{P}{P_p}$

χ = reduction factor due to column buckling

607.8.3 *Resistance of Members Subjected to Combined Compression and bi-axial Bending Moment*

1. The section shall be checked for resistance under axial compression for both x and y axes.
2. The resistance of the section shall then be checked for combined axial compression and bi-axial bending moment as described below.

The three necessary conditions to be satisfied are:

$$\frac{M_x}{\mu_x M_{px}} \leq 0.9 \quad \text{and} \quad \frac{M_y}{\mu_y M_{py}} \leq 0.9$$

The interaction of the moments must also be checked using the following interaction formula:

$$\frac{M_x}{\mu_x M_{px}} + \frac{M_y}{\mu_y M_{py}} \leq 1.0 \qquad \text{... 7.12}$$

The moment resistance ratios μ_x and μ_y for both the axes shall be evaluated as given below:

$$\mu_x = \frac{(\chi_x - \chi_d)}{(1-\chi_c)\chi_x} \qquad \text{when } \chi_d \geq \chi_c \qquad \text{... 7.13}$$

$$\mu_x = 1 - \frac{(1-\chi_x)\chi_d}{(1-\chi_c)\chi_x} \qquad \text{when } \chi_d < \chi_c \qquad \text{... 7.14}$$

and

$$\mu_y = \frac{(\chi_y - \chi_d)}{(1-\chi_c)\chi_y} \qquad \text{when } \chi_d \geq \chi_c \qquad \text{... 7.15}$$

$$\mu_x = 1 - \frac{(1-\chi_y)\chi_d}{(1-\chi_c)\chi_y} \qquad \text{when } \chi_d < \chi_c \qquad \text{... 7.16}$$

where,

χ_x and χ_y are the reduction factors for buckling in the x and y directions respectively.

When the effect of geometric imperfections is not considered the moment resistance ratio is evaluated as given below:

$$\mu = \frac{(1-\chi_d)}{(1-\chi_c)} \qquad \text{when } \chi_d > \chi_c$$

$$= 1.0 \qquad \text{when } \chi_d \leq \chi_c$$

607.9 Mechanical Shear Connection and Load Introduction

Proper sharing of loads between steel section and concrete 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 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, provision shall be made for the transfer of the corresponding longitudinal shear stress at the interface between steel and concrete.

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

607.9.1 *Load Introduction*

Shear connectors should be provided at areas of load introduction and at areas with change in cross-section if the design shear strength τ (**Table 13**) is exceeded at the interface between steel and concrete. The shear forces should be determined from the change of sectional forces of the steel or reinforced concrete section within the introduction length. If the loads are introduced into the concrete cross-section only, the values 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 absence of a more accurate method, the introduction length should not exceed $2d$ or $L/3$, where d is the minimum transverse dimension of the column and L is the column length.

Table 13 Design Shear Strength (τ)

Type of Cross-Section	τ (N/mm²)
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 endplates where the full interface between steel and concrete is permanently under compression. Otherwise the load application/introduction should be verified as elaborated below:

1. If the cross section is partially loaded as shown in **Fig. 16A**, the loads may be distributed with a ratio of 1:2.5 over the thickness t_e of the end plate. The concrete stresses should then be limited in the area of effective load introduction.
2. For concrete filled circular hollow section or square hollow section, under partial loading as shown in **Fig. 16B**, 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

$$\sigma_c = \frac{0.8 f_{ck}}{\gamma_c}\left[1+\eta_0 \frac{t}{d}\frac{f_y}{0.8 f_{ck}}\right]\sqrt{\frac{A_c}{A_1}} \leq \frac{0.8.A_c.f_{ck}.}{A_1.\gamma_c} \qquad \text{... 7.17}$$

where,

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. 16**);

η_0 = 4.9 for circular steel tubes and 3.5 for square sections

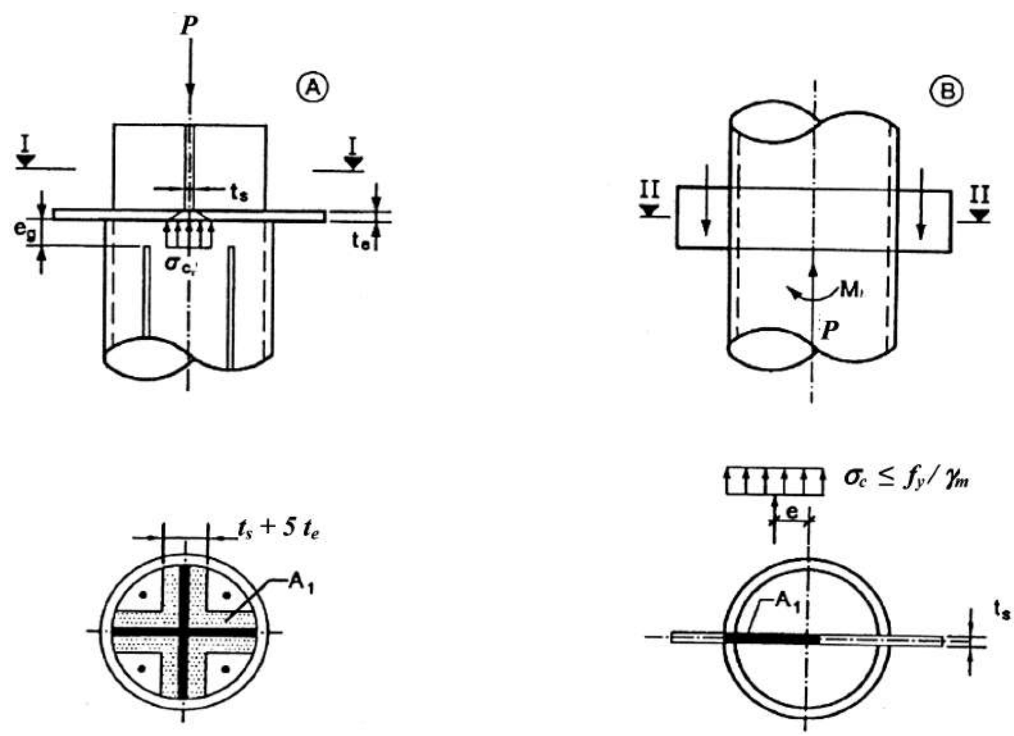


Fig. 16 Partially Loaded Circular Concrete filled Hollow Section

3. For concrete filled circular hollow section longitudinal reinforcements may be taken in to account while determining the resistance of the composite column, even if the reinforcement is not directly connected to the end plate, provided that the gap e.g. (**Fig 16A**) between the end of reinforcement and the surface of the end plate does not exceed 30 mm.

607.9.1.1 Shear connection

When mechanical connection is introduced in the form of stud connectors to the web of a fully or partially concrete encased steel I-section, account may be taken of the frictional forces that develop from the prevention of 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. 16** 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 **Section 606.3**.

The clear distance between the flanges should not exceed the values given in **Fig. 17** to ensure the development of the frictional forces between concrete and steel flanges.

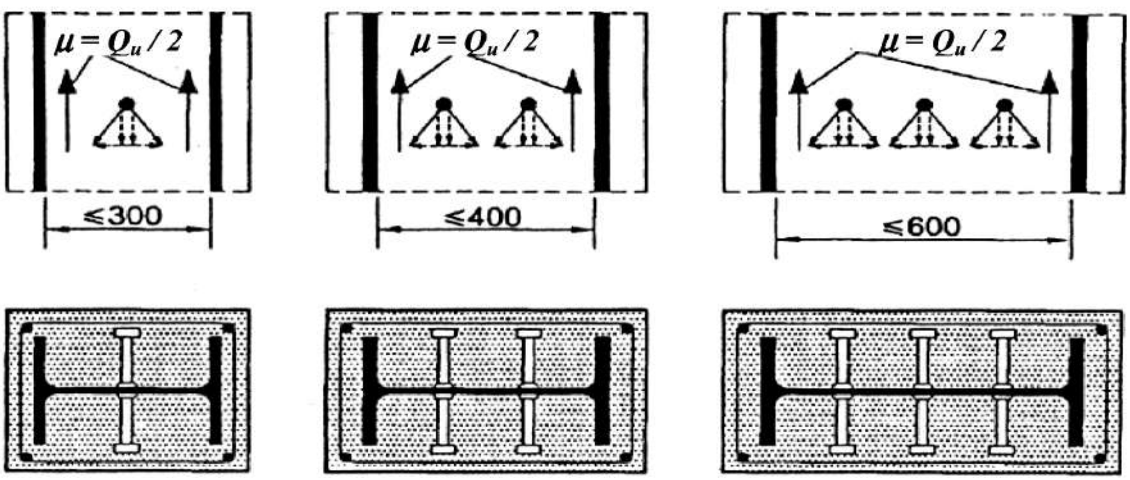


Fig. 17 Additional Frictional Forces in Composite Columns by use of Headed Studs

607.9.2 *Longitudinal Shear Outside the Area of Load Introduction*

1. 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 strength τ. In absence of a more accurate method, elastic analysis, considering long-term effects and cracking of 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 13** may be assumed for τ. The value of τ given in **Table 13** 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 $\beta_c . \tau$ may be used, with β_c given by:

$$\beta_c = 1 + 0.02.C_z.\left[1 - \frac{C_{z,\min}}{C_z}\right] \leq 2.5 \qquad \text{... 7.18}$$

where,

C_z is the wall thickness of the steel tube;

$C_{z,min}$ 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. 16**);

η_0 = 4.9 for circular steel tubes and 3.5 for square sections

2. 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 V_c, according to 607.8 should be welded to the web of the steel section or should pass through the web of the steel section.

607.10 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 alongwith steel reinforcements as per IRC:112-2011 and the shear resistance given by the steel section as per **Clause 603.3.3.2**. The shear force shall be distributed between the steel section and the concrete section in accordance with **Section 607.9**.

608 FILLER BEAM DECKS FOR BRIDGES

Filler beam decks shall be used with its own restrictions as far as geometrical dimensions, spacing of steel girder, section classifications and detailing.

608.1 Specific Requirements

The basic requirements for satisfactory application and adoption of filler beam decks are as indicated below:

1. Spans may be both simply supported and continuous and shear connectors are not required.
2. The structural steel sections may be both rolled and welded section and should be constant throughout the entire length of the section.
3. Decks with longitudinal filler beams should comply the following conditions:
   a) The steel beams shall be straight in horizontal projection.
   b) The skew angle of the supports shall not be greater than 300.
   c) The nominal depth *h* (Ref: **Fig. I.5** of **Annexure-I**) of the steel beams shall be minimum 250 mm and maximum 1100 mm.

d) The spacing of webs of the steel beam shall not exceed the lesser of [h/3 + 600] mm or 750 mm, h is the nominal depth of the steel beam.

e) The concrete cover c, above the steel beam shall satisfy the following three conditions,

$c \geq 70$ mm

$c \leq 150$ mm

$c \leq h/3$

f) The soffit of the lower flange of the steel beam shall not be encased.

g) A bottom layer of transverse reinforcements shall be provided which will pass though the webs of the steel and shall be anchored beyond the end steel beam upto the edge of concrete and should be high bond bars. The minimum diameter of these reinforcement bars shall be 16 mm with a maximum spacing of 300 mm.

h) The concrete in the filler beam decks shall be normal-weight concrete.

4) The clear distance between the upper flanges of the steel beams should not be less than 150 mm to allow proper pouring of concrete.

5) The minimum concrete cover for the flanges of the steel beam at the side of the deck shall be 80 mm.

6) The surface of the steel beam shall be de-scaled.

7) The exposed surfaces of the lower flange of the steel beams shall be protected against corrosion.

608.2 Global Analysis

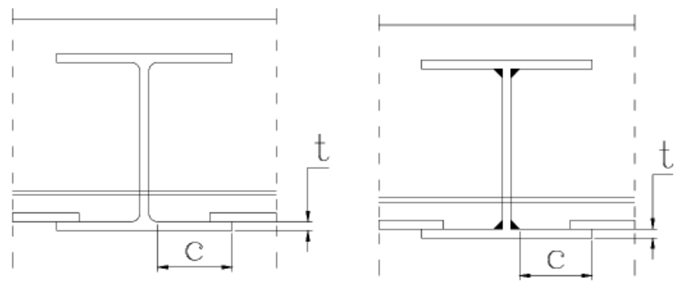
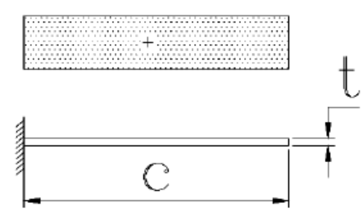
1. **Clause 603.1** shall be applicable for analysis of girders, both simply supported and continuous.
2. Under serviceability limit states, deflection shall be calculated as per **Clause 604.3** using equivalent section under cracked condition.
3. In case of non-uniform application of load in the transverse direction of the filler beam deck, after concrete hardening; the analysis shall take into account the difference between the deflections between adjacent filler beams. However, if it is verified that sufficient accuracy is obtained by a simplified analysis considering one rigid cross section, the same can be done based on the designer’s discretion.
4. The influence of shrinkage in concrete may be ignored.
5. In longitudinal bending, the effect of slip between steel beams and concrete may be ignored.
6. In transverse bending, the presence of steel beams shall be ignored.
7. The influence of shear lag may be ignored.

8. The contribution of formwork supported from the bottom flange of steel beams, which becomes a part of the permanent construction as shown in **Fig. I.5** of **Annexure-I** shall be ignored.

608.3 Section Classifications

The steel outstand flange of a composite section should be classified as shown in **Table 3** below:

Table 14 Classification of Steel Flange of Filler Beams

		
$\varepsilon = \sqrt{\frac{250}{f_y}}$ with f_y in N/mm^2		Stress Distribution (Compression Positive)
Class	**Type**	**Limit**
1	Rolled or Welded	$c/t \le 9\,\varepsilon$
2		$c/t \le 13\,\varepsilon$
3		$c/t \le 19\,\varepsilon$

A web in Class 3 which is encased in concrete may be represented by an effective web of the same cross-section in Class 2.

608.4 Bending Moments

The permissible longitudinal bending moment or the bending strength of a filler beam has been given in **Clause I.6** of **Annexure-I**. Check for construction stage bending moment must be verified. The calculations shall be done and checked as per IRC:24-2010.

608.5 Vertical Shear

The resistance to vertical shear of a filler beam girder shall conservatively be taken as the resistance of the structural steel section alone and shall be as per procedure laid down in **Clause 603.3.3.2**. However, the contribution from the reinforced concrete part between adjacent filler beams may be taken into account if it is determined and verified as per **Section 10 of IRC:112-2011**.

608.6 Minimum Reinforcement

The minimum reinforcements in the concrete section shall be as per guideline given in IRC:112-2011.

609 PRECAST SLAB ON STEEL BEAMS

The use of precast slab, both full depth and partial depth is allowed for composite construction as one of the components of Composite girders.

609.1 Full Depth Pre-cast Slab

Full-depth precast concrete deck panels may be used for new bridge construction as well as for replacement of deteriorated, concrete decks on existing bridges. This system typically consists of precast concrete panels, placed adjacent to one another on bridge girders. The typical requirements for these types of girders are as given below:

1. Panels shall either span the full width of the bridge deck or shall be in panels that span between the girders. Minimum thickness of slab shall be 200 mm.
2. The panels shall be connected to the girders using shear pocket connectors, which consist of mechanical connectors such as shear studs encapsulated in grouted pockets. These connections cause the panels to develop composite action with the girders.

609.2 Partial Depth Pre-cast Slab

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

1. Minimum thickness of the precast panels shall be 75 mm.
2. Dimensions of the pre-cast 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 concrete.
3. Partial-depth panels must be capable of developing sufficient composite action with the cast-in-situ concrete to be an effective bridge deck system.
4. To ensure full bond between the cast-in-situ concrete and precast panels, it is recommended that the top surface of the panel is intentionally roughened before the placement of the cast-in-situ concrete. In addition, the surface must also be cleaned by removing the laitance or other contaminates on the surface.
5. As a composite deck system, the cast-in-situ concrete and the partial-depth panels together create the total thickness of the slab, with the panel's reinforcing steel serving as the positive moment reinforcement in the transverse direction of the bridge.
6. After the pre-cast panels are in place, the top layers of reinforcing steel shall be placed, and the cast-in-situ concrete shall be placed on top of the panels.

609.3 General Design Principle

1. The pre-cast slab together with any in-situ concrete (for partial depth slab) should be designed as continuous in both the longitudinal and the transverse direction. The joints between slabs should be designed to transmit membrane forces as well as bending moment and shear forces.
2. Effective width of pre-cast slab shall be calculated as per clause 603.2.
3. The design principles of composite girders involving either full depth or partial depth precast slabs are similar to standard composite decks using cast-in-situ reinforced concrete. The stress diagram as shown in **Figs. I.1, I.2** and **I.3** is applicable for positive bending moment and **Fig. I.4** is applicable for hogging moment at support under ultimate limit states as indicated in **Annexure-I**.
4. Vertical shear check shall be done as per **Clause 603.3.3.2**.
5. For serviceability limit states, guidelines given in **Section 604 and 605** shall be followed.

609.4 Joints Between Steel Beam and Pre-cast Concrete Slab

1. Where pre-cast slabs are supported on steel beams without bedding the influence of the vertical tolerances of the bearing surfaces shall be considered.
2. The shear transfer between steel flange and precast concrete though mechanical shear connector shall be designed as per **Section 606** with the following precautions:
   a) If shear connectors welded to the steel beam project 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 shall be such that it can be cast properly.
   b) The minimum infill around the shear connectors should be at least 25 mm.
   c) If shear connectors are arranged in groups, sufficient reinforcement should be provided near each group to prevent premature local failure in either the pre-cast or the in-situ concrete.
   d) Special provision for protection against corrosion shall be adopted, wherein, the steel flange under pre-cast slabs without bedding should have the same corrosion protection as the rest of the steel work but for a top coating provided after erection. Bedding with the purpose of protecting against corrosion may be designed to be non-load bearing.

609.5 Joints Between pre-cast Members

The critical sections of members close to joints should be designed to resist the worst 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 **Clause 609.4**), suitable precautions should be taken to ensure that if any crack develops, it will not excessively reduce the member's resistance to shear or axial force and will not lose any aesthetic value.

Where a space is left between two or more pre-cast units, to be filled later with in-situ concrete or sometimes mortar, the space should be large enough for easy placement and adequate compaction of the filling material, which should fill the gap completely.

609.6 Structural Connection at Joints

When designing and detailing the connections across joints between pre-cast members the overall stability of the structure, including its stability during construction, shall be considered. A typical Joint connection is as shown in **Fig. 18** below:

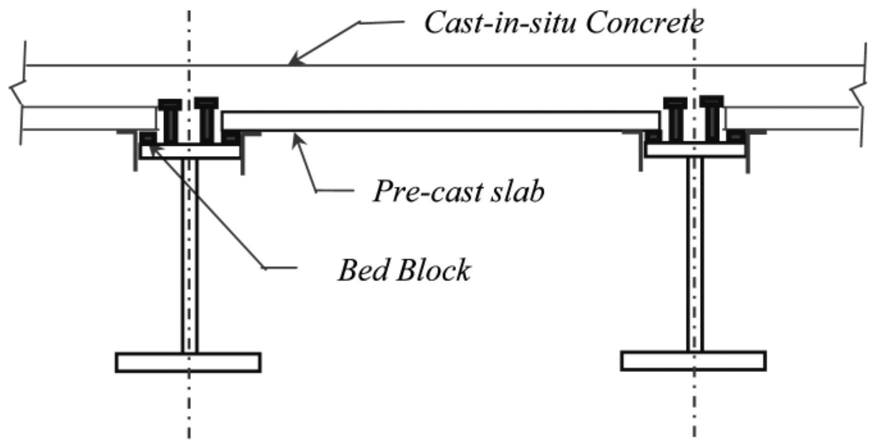


Fig. 18 Typical Joint Connection for Partial Depth Precast Slab

609.6.1 *Design Method*

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

609.6.2 *General Considerations in Design Details*

In additional to ultimate strength requirements the following should be considered:

1. **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.
2. **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.
3. **Manufacture Assembly and Erection:**- Methods of manufacture and erection should be considered during design, and the following points should be given particular attention.
   a) 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.

b) Fixing devices should be located in concrete sections of adequate strength.

c) The practicability of both casting and assembly should be considered.

d) 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 applicable.

609.6.3 *Factors Affecting Design Details*

The strength and stiffness of any connection can be significantly affected by workmanship on site. The following points should be considered where appropriate:

1. Sequence of forming the joint.
2. Critical dimensions allowing for tolerances, e.g. minimum permissible bearing.
3. Critical details, e.g. accurate location required for a particular reinforcing bar.
4. Method of correcting possible lack of fit in the joint.
5. Details of temporary propping and time when it may be removed.
6. Description of general stability of the structure with details of any necessary temporary bracing.
7. How far the uncompleted structure may proceed in relation to the completed and matured section.

609.6.4 *Reinforcement Continuity at Joint*

Where continuity of reinforcement is required through the connection, the jointing 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.

610 CONSTRUCTION & ERECTION

610.1 Fabrication and Inspection Procedure for Steel Section

Fabrication and inspection provisions of **Section 513** of IRC:24-2010 should apply for steel section.

For propped construction care is to be taken to ensure that props do not yield otherwise adequate provisions has to be kept in design analysis. In the analysis prop stability has to be assured through adequate bracings and proper arrangements.

610.2 Quality Control for Reinforced Concrete Construction

Section 18 of IR:112-2011 should apply to material, quality control and workmanship for reinforced concrete portion of composite girders. **Clause 18.2** or IRC:112-2011 shall be referred to for specifications of reinforcements and **Clause 18.4** shall be referred for material ingredients of concrete which include, cement, aggregates and water. Mixed proportions of concrete shall be done as per **Clause 18.5** of IRC:112-2011.

610.3 Transportation, Handling and Erection of Steel Section

For transportation, handling and erection of steel structure **Section 514**, of IRC:24-2010 shall be referred to.

611 TESTING METHOD

Testing of materials shall be done as per standard laid down norms. For testing of concrete reference shall be made to **Clause 18.5.4 & 18.6** of IRC:112-2011. For testing of steel sections and accessories like bolts, nuts, washers, welding consumables etc. **Clause 513.6** of IRC:24-2010 shall be referred. For testing of strength, flexibility and other relevant properties of shear connectors proper test procedures as indicated in **Clause 611.1** shall be adopted.

611.1 Testing of Stud Shear Connectors

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:

1. The dimensions of the standard test-piece are as shown in **Fig. 19**.
2. While preparing the test-piece, the bond at the steel-concrete interface should be prevented by greasing the flanges or by any other suitable method.
3. The thickness of slab and detailing of reinforcement should be either as given in **Fig. 18** or as in the actual beams for which the test is designed.
4. The rate of application of load should be uniform and such that the failure load is reached in not less than 10 minutes.
5. 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 ± 20 percent.
6. 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.

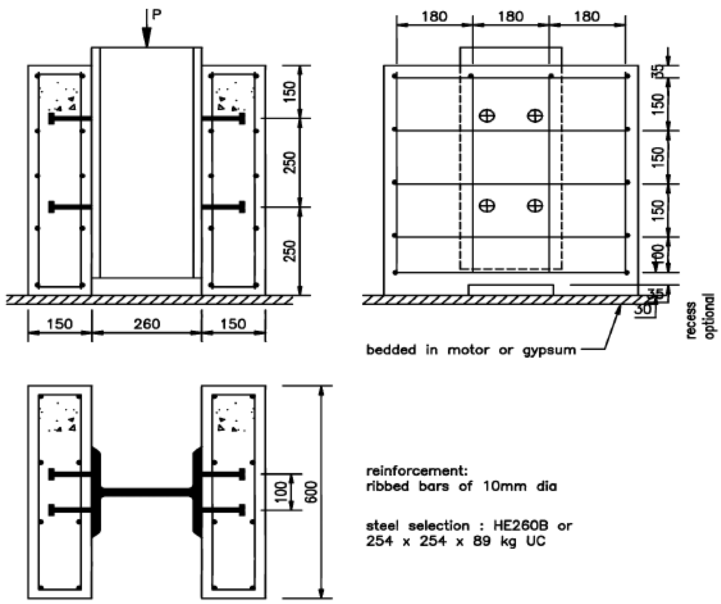


Fig. 19 Standard Push Test

612 FIRE RESISTANCE

Bridges being open structures are not generally vulnerable to failure under fire, since the temperature does not go upto the level which may cause damage to the materials. Also being an open structure, the fire can be extinguished easily and quickly. Fire tests on similar open structures like parking lots where composite construction has been adopted has shown that the structure does not undergo any material damage due to reasons indicated above. However, bridges shall be protected from possible accidental fire caused by hazards like lighting of stoves, lamps beneath a superstructure due to possible encroachments by road-side food stalls or families living under the bridges. These can be achieved by cordoning the entire area underneath the bridges either by fencing or with proper landscaping and gardening which will further improve the aesthetics of the area.

Fire designs at specialized locations, such as proximity to oil installations or pipelines carrying inflammable materials etc. shall be done based on recommendations given in specialized literatures. Also adequate provisions may be made as far as possible for fire fighting equipment to access all parts of the bridge. In case of accidental occurrence of fire in bridges it should be mandatory for the authorities to have the bridge inspected by competent experts in order to ascertain the health of the structure before it can be declared safe for re-use.

In addition to the above, locations in a bridge system which may be prone to accidental occurrence of fire as discussed earlier shall be adequately provided with basic fire protection methods as per specialist literature.

613 MAINTENANCE

For periodic inspection and maintenance of a bridge, procedures and guidelines primarily laid down in IRC:35 may be followed along with various other stipulations mentioned in IRC:24-2010 and IRC:112-2011.

ANNEXURE–I

(Clause 603.3.1; Clause 608; Clause 609)

MOMENT OF RESISTANCES

I.1 Moment of Resistance of Composite Section with Plastic or Compact Structural Steel Section (Positive Moments)

I.1.1 Bending Moment with Full Shear Interaction

The design plastic moment of resistance of a section will depend on the position of the neutral axis. **Table I.1** gives the various positive and negative bending moments of a composite section depending upon the location of the neutral axis as shown in **Figs. I.1, I.2 and I.3**. For hybrid sections appropriate yield strength shall be considered for calculation of the plastic moments.

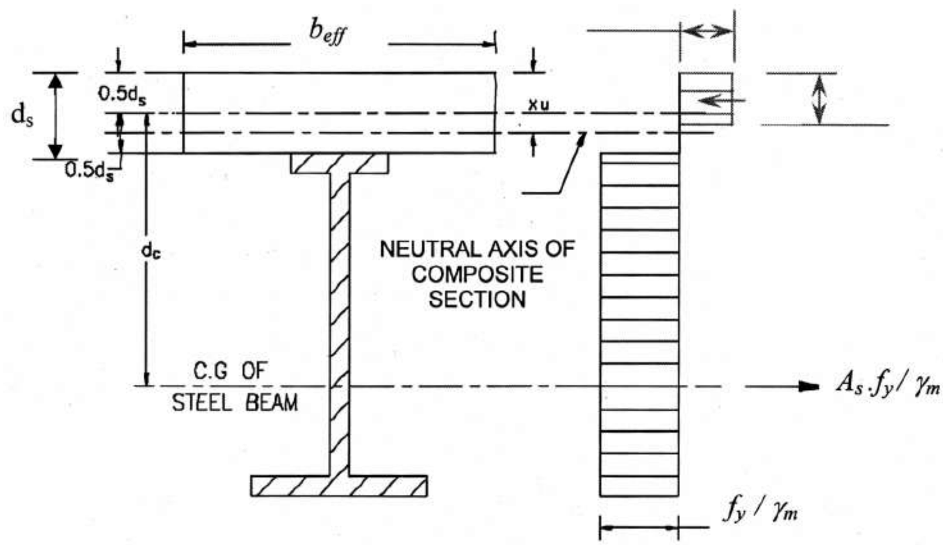


Fig. I.1 Stress Distribution in a Composite Beam with Neutral Axis within Concrete Slab

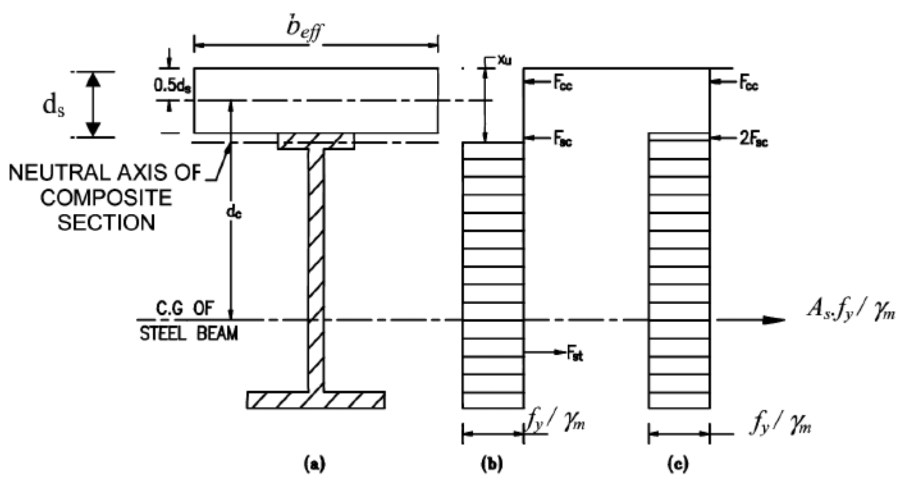


Fig. I.2 Stress Distribution in a Composite Beam with Neutral Axis within Flange of Steel Beam

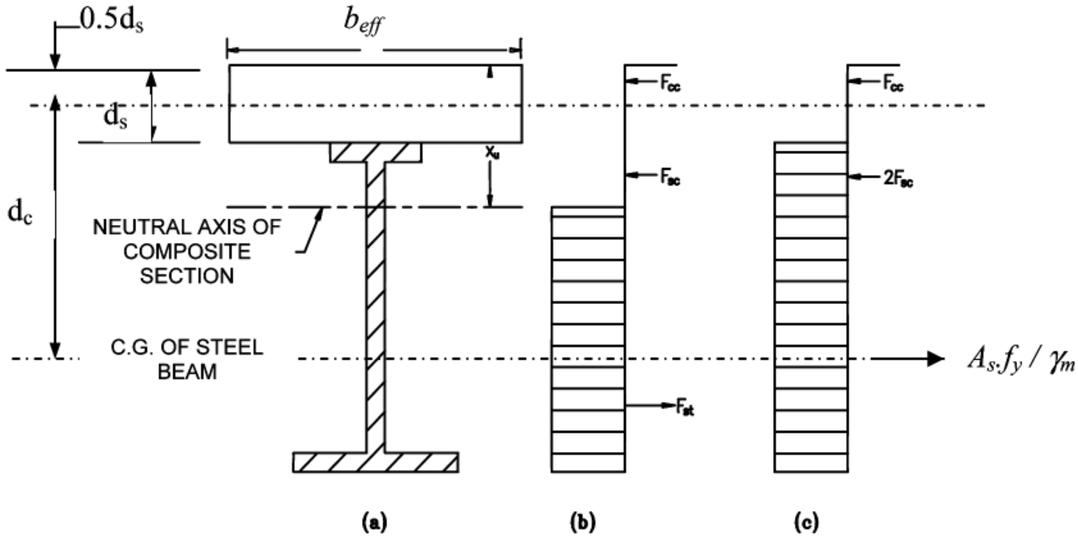


Fig. I.3 Stress Distribution in a Composite Beam with Neutral Axis within the Web of the Steel Beam

Table I.1 Positive Moment Capacity of Composite Section with full Shear Interaction

Case	Position of Plastic Neutral Axis	Value of x_u	Moment Capacity M_p
1	Within slab (**Fig. I.1**) $b_{eff} d_s > a.A_s$	$x_u = a\,A_s / b_{eff}$	$M_p = A_s f_y (d_c + 0.5 d_s - \lambda .\, x_u/2)/\gamma_m$
2	Plastic neutral axis in steel flange (**Fig. I.2**) $b_{eff} d_s < a.A_s < (b_{eff} d_s + 2a.A_f)$	$x_u = d_s + \frac{(aA_s - b_{eff} d_s)}{2 b_f a}$	$M_p = f_y [A_s \{d_c + 0.5 d_s .(1 - \lambda)\} - b_f (x_u - d_s).\{x_u + (1 - \lambda).d_s\}]/\gamma_m$
3	Plastic Neutral axis in web (**Fig. I.3**) $b_{eff} d_s + 2a.A_f < a.A_s$	$x_u = d_s + t_f + \frac{a(A_s - 2A_f) - b_{eff} d_s}{2 a t_w}$	$M_p = f_y .[A_s .\{d_c + 0.5\, d_s .(1 - \lambda)\} - 2A_f \{0.5 t_f + (1 - \lambda/2)\, d_s\} - t_w .(x_u - d_s - t_f)\,\{x_u + (1 - \lambda)\, d_s + t_f\}]/\gamma_m$

$$a = \frac{f_y / \gamma_m}{\frac{\alpha_{cc.}}{\gamma_c} .\eta ..\lambda .(f_{ck})}, \qquad \text{... I.1}$$

A_f = area of top flange of steel beam of a composite section.

A_s = cross sectional area of 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_p = ultimate bending moment.

α_{cc} = 0.67

γ_c = material safety factor for concrete

= 1.50 (for basic and seismic combinations)

= 1.20 (for accidental combinations)

γ_m = material safety factor for structural steel = 1.10

η = 1.0 [for $f_{ck} \leq 60$ MPa]

= $1.0 - (f_{ck} - 60) / 250$ [for $60 < f_{ck} \leq 110$ MPa]

λ = 0.8 [for $f_{ck} \leq 60$ MPa]

= $0.8 - (f_{ck} - 60) / 500$ [for $60 < f_{ck} \leq 110$ MPa]

I.1.2 *Bending Moment with Partial Shear Interaction*

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 (i.e. required minimum spacing of shear connectors cannot be provided).

Degree of shear connection S_c is given as $S_c = \dfrac{n_p}{n_f} = \dfrac{F_{cp}}{F_{cf}} = \dfrac{M - M_{ps}}{M_p - M_{ps}}$

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 no. of connectors

F_{cf} = Capacity of shear connectors in full shear connection with n_f no. of connectors

M = Required bending resistance of the section

M_p = Plastic moment of resistance of the composite section

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

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

$$n_p = \frac{M - M_{ps}}{M_p - M_{ps}} n_f$$

I.2 Moment of Resistance of Composite Section with Non-Compact Structural Steel Section (Positive moments)

Since the compression flange of non-compact steel sections buckle locally under compression before reaching yield stress f_y, the resistance of the composite section consisting of non-compact sections is guided by that of compact sections as above, wherein the effective width of the compression flange is restricted to that of the compact section limiting value.

I.3 Moment of Resistance of Composite Section (Negative Moment) (Continuous Girder)

Fig. I.4 shows stress distribution across a composite beam section subjected to hogging bending moment. The steel bottom flange is in compression. Section classification shall be done as per **Table 2** as discussed earlier. For classification of the web, the distance $\bar{\lambda}$ of the plastic neutral axis above the center of area of the steel section, must first be found.

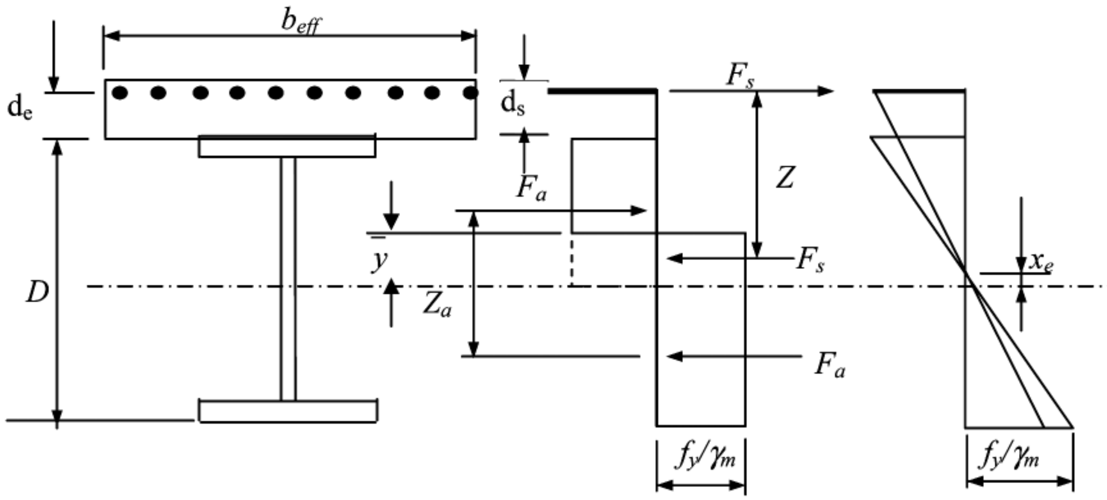


Fig. I.4 Stress Diagram for Hogging Moment

I.3.1 *Moment of Resistance for Plastic and Compact Section*

The design tensile force in reinforcement is given as,

$F_s = f_{yk}.\ A_{st}/\gamma_s$, Where, γ_s = partial safety factor for reinforcement = 1.15

f_{yk} = Characteristic yield strength of the reinforcements

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 section is given as,

$$M_p = \frac{Z_p . f_y}{\gamma_m} \qquad \text{... I.2}$$

Where, Z_p is the plastic section modulus and γ_m 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 steel section as given by M_p above. To allow for reinforcements it is assumed that the stress in a depth $\bar{\lambda}$ changes from tension to compression for Plastic and compact section. The corresponding depth for non-compact section is x_e. For plastic and compact stress distribution $\bar{\lambda}$ may be determined from

$$\bar{\lambda} ..t_w \frac{2f_y}{\gamma_m} = F_s \qquad \text{... I.3}$$

The locations of the neutral axis and the moment of resistance for plastic and compact section are given in **Table I.2** below:

Table I.2 Limiting Negative Moment Capacity of Composite Section with Steel Component having Plastic and Compact Steel

Case	Position of Plastic Neutral Axis	Value of $\bar{\lambda}$	Moment Capacity M_p
1	In Web	$\bar{y} \le \frac{D}{2} - t_f$	$M_{ph} = Mp + F_s.Z$
2	In Flange	$\frac{D}{2} \ge \bar{y} > \frac{D}{2} - t_f$	$M_{ph} = F_b.\frac{D}{2} + F_s.d_s - \frac{(F_b - F_s)^2}{F_f}.\frac{t_f}{4}$

$$Z = \frac{D}{2} + d_e - \frac{\bar{y}}{2}$$

$F_b = \dfrac{f_y.A_s}{\gamma_m}$ = axial capacity of steel section ((A_s = area of steel section)

F_f = axial capacity of a single flange = $\dfrac{f_y.A_f}{y_m}$

Note: The web shall classified as being in compression throughout

A_s = Cross sectional area of steel beam of a composite section.

d_s = Overall depth of concrete slab

d_e = Effective depth of slab (**Fig. I.4**).

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

t_w = Thickness of the web of the steel section

I.3.2 *Moment of Resistance for Non-Compact Section*

Where elastic analysis is used, creep is allowed for 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 height x_e of the elastic neutral axis of the composite section (**Fig I.4**) above that of the steel section is given as

$$x_e\,(A_s + A_{st}) = A_{st}\left(\frac{D}{2} + d_s\right) \qquad \text{... I.4}$$

and the second moment of area of the composite section is

$$I_{co} = I_s + A_s.x_e^{\,2} + A_{st}\left(\frac{D}{2} + d_s - x_e\right)^2 \qquad \text{... I.5}$$

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 I.3** below:

Table I.3 Negative Moment Capacity of Composite Section with Steel Component Having Plastic and Compact Steel

Location of Neutral Axis	Moment Capacity of Steel Section Alone $[f_s]\ \bar{\lambda}$	Moment Capacity $M_{e(c)}$
$x_e(A_s + A_{st}) = A_{st}\left(\frac{D}{2} + d_s\right)$	$f_s = M_{e(s)}.\frac{D/2}{I_s}$	$M_{e(c)} = M_{e(s)} + \frac{(f_y/\gamma_m - f_s)I_{co}}{(D/2 + x_e)}$

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

$$f_s = M_{e(s)}.\frac{D/2}{I_s}$$

Note: The web shall classified as being in compression throughout

A_s = Cross sectional area of steel beam of a composite section.

A_{st} = Cross sectional area reinforcements within the effective width of the concrete flange.

d_s = Overall depth of concrete slab

d_e = Effective depth of slab (**Fig. I.4**).

$M_{e(s)}$ = Hogging moment in the Steel section alone.

f_s = Compressive stress in steel flange due to moment $M_{e(s)}$.

I_s = Second moment of inertia of steel section alone

I_{co} = Second 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 the steel section

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 as

$$\sigma_{sr} = \frac{(f_y/\gamma_m - f_s)(D/2 + d_s - x_e)}{(D/2 + x_e)} \leq f_{yk}/\gamma_s \qquad \text{... I.6}$$

I.4 Flange Stress Reduction Factor R_h

Flange stress reduction factor is applicable for hybrid sections using higher grade flanges where the section is non-compact, or in other words, where plastic moment capacity cannot be generated. In such cases permissible limiting stress for both compression and tension shall be modified by the reduction factor R_h and shall be given as,

$$f_n = R_h . f_y/\gamma_m$$

For homogeneous sections, R_h shall be 1.0. The reduction factor should not be applied 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 **Clause I.1.1** The value of R_h may be ignored for calculating bending resistance of composite section considering the fact that, for non-homogeneous section, i.e. for fabricated girder it is always advisable to consider sections which are fully compact or plastic.

I.5 Buckling Resistance Moment (Construction Stage)

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

$$M_{pl\,(buck)} = \chi_{LT}.M_{pl} \qquad \text{or} \qquad M_{el\,(buck)} = \chi_{LT}.M_{el} \qquad \text{... I.7}$$

where,

$$\chi_{LT} = \frac{1}{[\phi_{LT} + (\phi_{LT}^{\ 2} - \bar{\lambda}_{LT}^{\ 2})^{\frac{1}{2}}]} \quad \text{but } \chi_{LT} \le 1.0 \qquad \text{... I.8}$$

Now

$$\phi_{LT} = 0.5[1 + \alpha_{LT}(\bar{\lambda}_{LT} - 0.2) + \bar{\lambda}_{LT}^{\ 2}] \qquad \text{... I.9}$$

α_{Lt} = 0.21 for rolled sections

α_{Lt} = 0.49 for welded sections

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

$$\lambda_{Lt} = \sqrt{\beta_b Z_p f_y / M_{cr}} \qquad \text{... I.10}$$

where,

β_b = 1.0 for Plastic and Compact sections

= Z_e/Z_p for semi-compact sections

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

The elastic lateral buckling moment is given by

$$M_{cr} = \sqrt{\left\{\left(\frac{\pi^2 EI_y}{(L_{LT})^2}\right)\left[GI_t + \frac{\pi^2 EI_w}{(L_{LT})^2}\right]\right\}} \qquad \text{... I.11}$$

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.

$$M_{cr} = \frac{\pi^2 EI_y h_f}{2L_{LT}^2}\left[1 + \frac{1}{20}\left(\frac{L_{LT}/r_y}{h_f/t_f}\right)^2\right]^{0.5} = \beta_b Z_p f_{cr,b} \qquad \text{... I.12}$$

where,

I_w = warping constant

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

$t_{f,}$ = thickness of the flange

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

I.6 Moment of Resistance for Filler Beam Decks

The stress distribution diagram for a standard filler beam decks is as shown in **Fig. I.6**.

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

where,

F_{st} = Tensile force in the steel section below neutral axis

F_{sc} = Compressive force the steel section above neutral axis

F_{cc} = Compressive force in the concrete above neutral axis

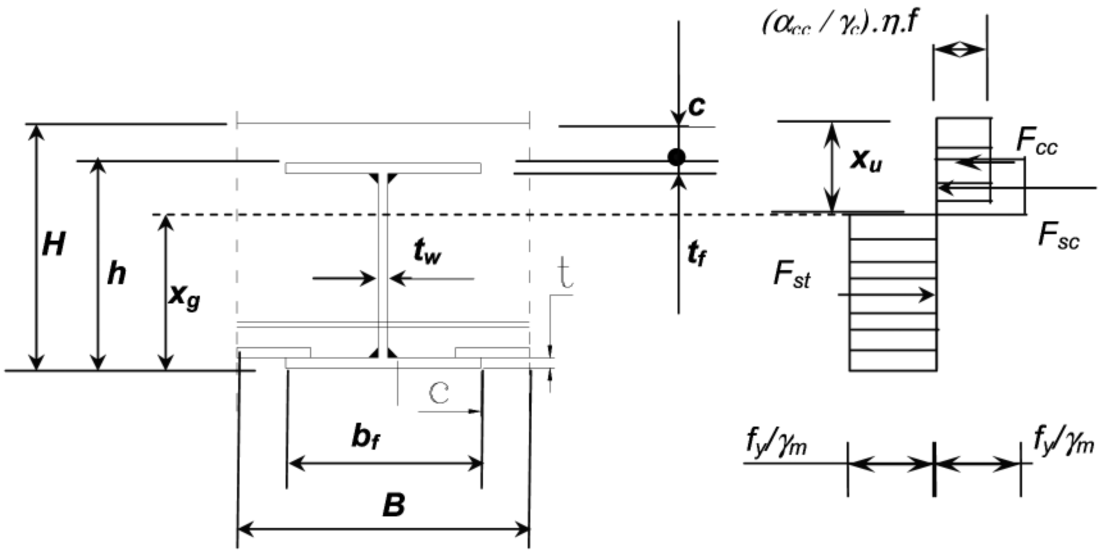


Fig. I.5 Stress Diagrain for Filer Beam

The depth of the neutral axis is given as,

$x_u = H - x_g$

where,

$$x_g = \frac{\frac{\alpha_{cc}}{\gamma_c}.\eta.f_{ck}.[B.H - b_f.t_f - t_w.(h - t_f)] + t_w.h.f_y / \gamma_m}{\frac{\alpha_{cc}}{\gamma_c}.\eta.f_{ck}.(B - t_w) + 2t_w.f_y / \gamma_m} \quad \text{... I.13}$$

Moment of resistance,

$$M_p = F_{sc}.\,X_{sc} + F_{cc}.\,X_{cc} + F_{st}.\,X_{st} \quad \text{... I.14}$$

[X_{sc}, X_{cc} and X_{st} are respectively the distance between the neutral axis of the composite girder and the individual centre of gravities of the corresponding forces]

Note:

A_f = area of top flange of steel beam of a composite section.

A_s = cross sectional area of steel beam of a composite section.

b_{eff} = effective width of concrete slab.

b_f	=	width of top flange of steel 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
B	=	Centre-to-Centre distance between two filler beams
	=	Effective width of concrete for one filler beam
H	=	Distance between top of concrete and bottom of bottom flange of steel girder
h	=	Total depth of steel girder
x_g	=	Distance of neutral axis from bottom of bottom flange of steel beam
M_p	=	Ultimate bending moment.
α_{cc}	=	0.67
γ_c	=	Material safety factor for concrete
	=	1.50 (for basic and seismic combinations)
	=	1.20 (for accidental combinations)
γ_m	=	Material safety factor for structural steel = 1.10

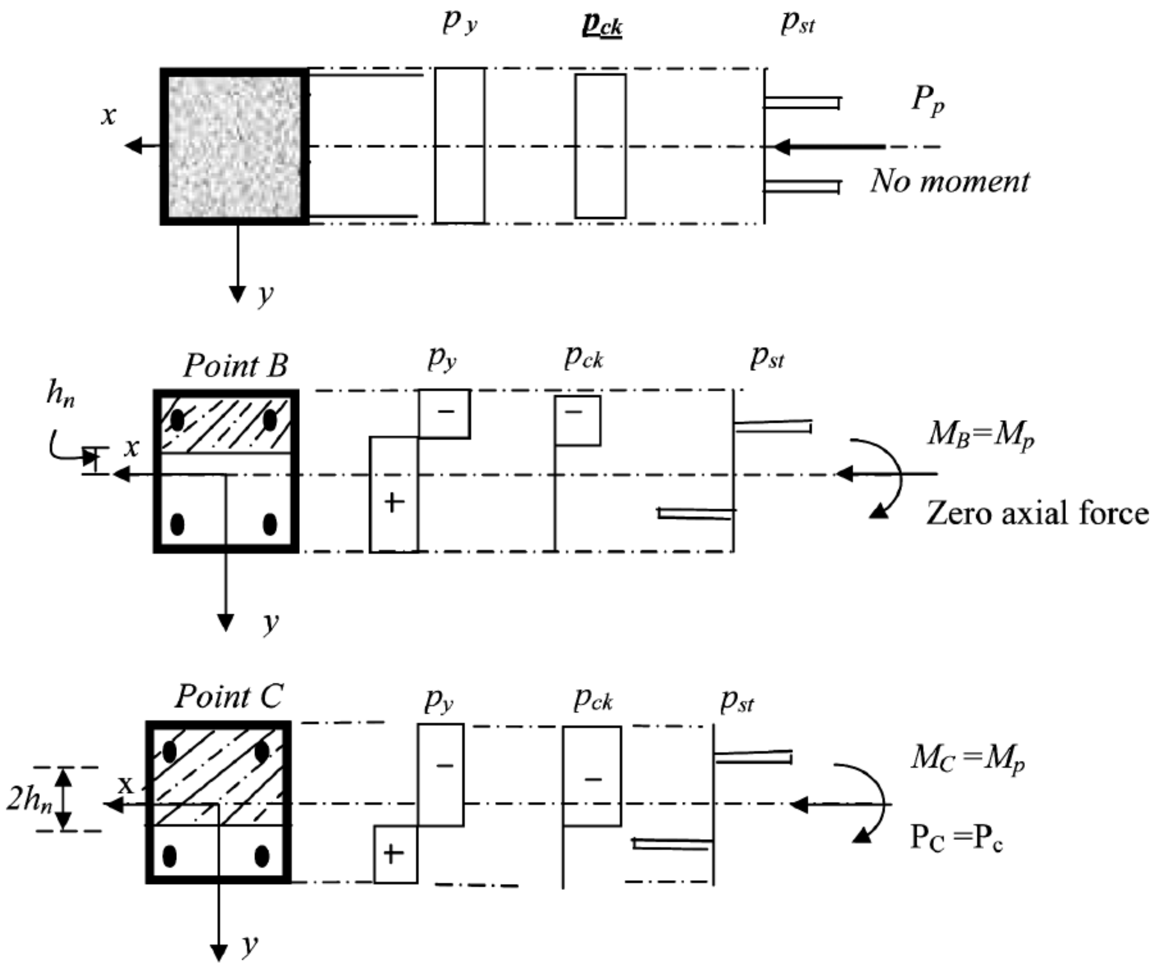
ANNEXURE–II

(Clause 607.8)

LOCATION OF PLASTIC NEUTRAL AXIS IN COMPOSITE COLUMN

II.1 General

Referring to **Fig. II.1**, it is important to note that the positions of the neutral axis for points B and C in the interaction curve is as shown in **Fig. II.1**, h_n, can be determined from the difference in stresses at points B and C.



$$p_y = \frac{f_y}{\gamma_m} \qquad p_{ck} = \frac{0.8.\alpha_c.f_{ck}}{\gamma_c} \qquad p_{st} = \frac{f_{yk}}{\gamma_s}$$

Fig. II.1 Stress Distributions for the Points of the Interaction Curve for Concrete Filled Rectangular Tubular Sections

The resulting axial forces, which are dependent on the position of the neutral axis of the cross-section, h_n, can easily be determined as shown in **Fig. II.2**. The sum of these forces is equal to P_c. This calculation enables the equation defining h_n to be determined, which is different for various types of sections.

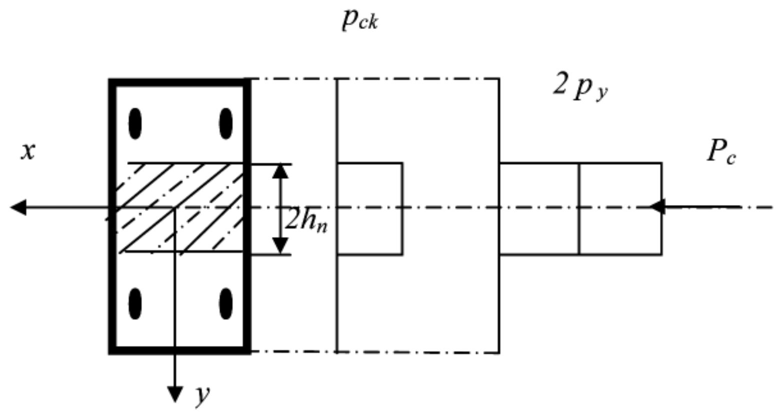


Fig. II.2 Variation in the Neutral Axis Positions

II.1.1 *For Concrete Encased Steel Sections:*

Major axis bending

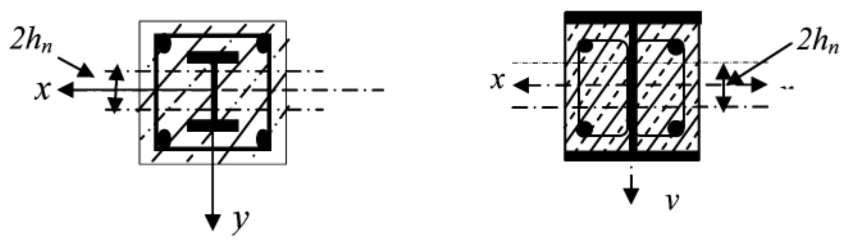


Fig. II.3

1. Neutral axis in the web: $h_n \leq [\, h/2 - t_f \,]$

$$h_n = \frac{A_c p_{ck} - A'_{st}(2p_{st} - p_{ck})}{2b_c p_{ck} + 2t_w(2p_y - p_{ck})} \qquad \text{... II.1}$$

2. Neutral axis in the flange: $[h/2 - t_f] \leq h_n \leq h/2$

$$h_n = \frac{A_c p_{ck} - A'_{st}(2p_{st} - p_{ck}) + (b - t_w)(h - 2t_f)(2p_y - p_{ck})}{2b_c p_{ck} + 2b(2p_y - p_{ck})} \qquad \text{... II.2}$$

3. Neutral axis outside the steel section: $h/2 \leq h_n \leq h_c/2$

$$h_n = \frac{A_c p_{ck} - A'_{st}(2p_{st} - p_{ck}) - A_s(2p_y - p_{ck})}{2b_c p_{ck}} \qquad \text{... II.3}$$

Minor axis bending

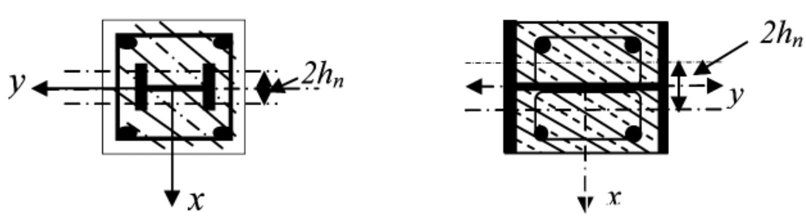


Fig. II.4

1. Neutral axis in the web: $h_n \leq t_w/2$

$$h_n = \frac{A_c p_{ck} - A'_{st}(2p_{st} - p_{ck})}{2h_c p_{ck} + 2h(2p_y - p_{ck})} \quad \text{... II.4}$$

2. Neutral axis in the flange: $t_w/2 < h_n < b/2$

$$h_n = \frac{A_c p_{ck} - A'_{st}(2p_{st} - p_{ck}) + t_w(2t_f - h)(2p_y - p_{ck})}{2h_c p_{ck} + 4t_f(2p_y - p_{ck})} \quad \text{... II.5}$$

3. Neutral axis outside the steel section: $b/2 \leq h_n \leq b_c/2$

$$h_n = \frac{A_c p_{ck} - A'_{st}(2p_{st} - p_{ck}) - A_s(2p_y - p_{ck})}{2h_c p_{ck}} \quad \text{... II.6}$$

Note: A'_{st} is the sum of the reinforcement area within the region of $2h_n$

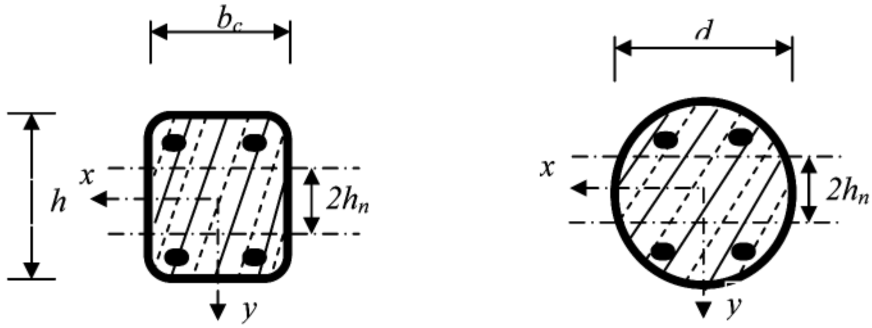


Fig. II.5

II.1.2 *For Concrete Filled Tubular Sections*

Major axis bending

$$h_n = \frac{A_c p_{ck} - A'_{st}(2p_{st} - p_{ck})}{2b_c p_{ck} + 4t(2p_y - p_{ck})} \quad \text{... II.7}$$

where,

$$p_y = \frac{f_y}{\gamma_m},\ p_{ck} = \frac{\alpha_c.0.8.f_{ck}}{\gamma_c} \text{ and } p_{st} = \frac{f_{yk}}{\gamma_s}$$

Note:

- For circular tubular section substitute $b_c = d$
- For minor axis bending the same equations can be used by interchanging h and b as well as the subscripts x and y.

ANNEXURE-III

MATERIALS AND PROPERTIES

III.1 Structural Steel

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

Young's Modulus (Modulus of Elasticity)	=	2.0×10^5 Mpa
Shear Modulus	=	0.77×10^5 Mpa
Poisson's Ratio	=	0.30
Coefficient of Thermal Evxpansion	=	0.0000 117 / ºc / unit length

Requirements of Structural Steel

Unless otherwise permitted herein all structural steel shall, before fabrication comply with the requirements of the following Indian Standards, or their latest revisions as appropriate:

IS:808	Dimensions for hot rolled steel beam, column, channel and angle sections
IS:1161	Steel tubes for structural purposes
IS:1239 (Pt. 1)	Mild steel tubes, tubular and other wrought steel fittings: Part-I Mild steel tubes
IS:1239 (Pt. 2)	Mild steel tubes, tubular and other wrought steel fittings: Part-2 Mild steel tubular and other wrought steel fittings
IS:1730	Dimensions for steel plates, sheets, strips and flats for general engineering purposes
IS:1732	Dimension for round and square steel bars for structural and general engineering purposes
IS:1852	Rolling and cutting tolerances for hot rolled steel products
IS:2062	Hot rolled low, medium and high tensile structural steel
IS:4923	Hollow steel sections for structural use
IS:11587	Structural weather resistant steels

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

Other Steels

Except where permitted with the specific approval of the authority, steels for machined parts and for uses in other than structural members or elements, shall comply with the following or relevant Indian Standards.

IS:1875	Carbon steel billets, blooms, slabs and bars for forgings
IS:6911	Stainless steel plate, sheet and strip

Castings and Forgings:

Steel casting and forgings shall comply with the requirements of the following Indian Standards as appropriate:

IS:1030	Carbon steel castings for general engineering purposes
IS:1875	Carbon steel billets, blooms, slabs & bars for forgings
IS:2004	Carbon steel forgings for general engineering purposes
IS:2644	High tensile steel castings
IS:4367	Alloy steel forgings for general industrial use

Fasteners:

Bolts, nuts, washers and rivets shall comply with the following or relevant Indian Standards, as appropriate:

IS:1148	Hot rolled rivet bars (upto 40 mm dia) for structural purposes
IS:1149	High tensile steel rivet bars for structural purposes
IS:1363 (Pt. 1 to Pt. 3)	Hexagon head bolts, screws and nuts of product grade C (size range M5 to M64)
IS:1364 (Pt. 1 to Pt. 3)	Hexagon head bolts, screw and nuts products grade A & B (size range M1.6 to M64).
IS:1367 (Pt. 1 to Pt. 18)	Technical supply conditions for threaded steel fasteners
IS:1929	Hot forged steel rivets for hot closing (12 to 36 mm diameter)
IS:2155	Cold forged solid steel rivets for hot closing (6 to 16 mm diameter)
IS:3640	Hexagon fit bolts
IS:3757	High strength structural bolts
IS:4000	High strength bolts in steel structures-code of practice
IS:5369	General requirements for plain washers and lock washers
IS:5370	Plain washers with outside dia = 3 x inside dia.
IS:5372	Taper washers for channels (ISMC)
IS:5374	Taper washer for I-beams (1SMB)
IS:5624	Foundation bolts
IS:6610	Heavy washers for steel structures
IS:6623	High strength structural nuts
IS:6649	Hardened and tempered washers for high strength structural bolts and nuts
IS:7002	Prevailing torque type steel hexagon nuts

Welding Consumables:

Welding consumables shall comply with the following Indian standards, as appropriate:

IS:814	Covered electrodes for manual metal arc welding of carbon and carbon manganese steel

IS:1395	Low and medium alloy, steel covered electrodes for manual metal arc welding
IS:3613	Acceptance tests for wire flux combination for submerged arc welding
IS:6419	Welding rods und bare electrodes for gas shielded arc welding of structural steel
IS:6560	Molybdenum and chromium-molybdenum low alloy steel welding rods and bare electrodes for gas shielded arc welding
IS:7280	Bare wire electrodes for submerged arc welding of structural steel

Welding:

IS:812	Glossary of terms relating to welding and cutting of metal
IS:816	Code of practice for use of metal arc welding for general construction in mild steel
IS:822	Code of procedure for inspection of welds
IS:1024	Code of practice for use of welding in bridges and structures subject to dynamic loading
IS:1182	Recommended practice for radiographic examination of fusion welded butt joints in steel plates
IS:4853	Recommended practice for radiographic inspection of fusion welded butt joints in steel pipes
IS:5334	Code of practice for magnetic particle flaw detection of welds
IS:7307 (Pt.1)	Approval tests for welding procedures: Part-I fusion welding of steel
IS:7310 (Pt.1)	Approval tests for welders working to approved welding procedures: Part-1 fusion welding of steel
IS:7318 (Pt.1)	Approval tests for welders when welding procedure is not required: Part- 1 fusion welding of steel
IS:9595	Recommendations for metal arc welding of carbon and carbon manganese steels

Wire Ropes and Cables:

These shall conform to the following or relevant Indian Standards except where use of other types is specifically permitted by the authority.

IS:1785 (Pt.1)	Specification for plain hard-drawn steel wire for pre-stressed concrete: Part-1; Cold drawn stress relieved wire
IS:1785 (Pt.2)	Specification for plain hard-drawn steel wire for pre-stressed concrete: Part-2; As- drawn wire
IS:2266	Steel wire ropes for general engineering purposes
IS:2315	Thimbles for wire ropes
IS:9282	Wire ropes and strands for suspension bridges

III.2 Concrete

All structural concrete shall be of minimum grade M25 and shall be cast in accordance with stipulations mentioned in IRC:112-2011. The strengths shall be specified in terms of the characteristic cube strengths, f_{ck}, measured at 28 days. Stress and deformation characteristics of various grades of concrete shall also be in accordance with IRC:112-2011. **Table III.1** gives the properties of different grades of concrete.

Table - III - 1: Properties of Concrete

	Strength Class of Concrete															
Strength Class	**M 15**	**M 20**	**M 25**	**M 30**	**M 35**	**M 40**	**M 45**	**M 50**	**M 55**	**M 60**	**M 65**	**M 70**	**M 75**	**M 80**	**M 85**	**M 90**
$(f_{ck})_{cu}$ (MPa)	15	20	25	30	35	40	45	50	55	60	65	70	75	80	85	90
$(f_{ck})_{cy}$ (MPa)	12	16	20	24	28	32	36	40	44	48	52	56	60	64	68	72
f_{ctm} (MPa)	1.6	1.9	2.2	2.5	2.8	3.0	3.3	3.5	3.7	4.0	4.4	4.5	4.7	4.8	4.9	5.0
E_{cm} (GPa)	27	29	30	31	32	33	34	35	36	37	38	38	39	40	40	41

Note:-

$(f_{ck})_{cu}$ ---- characteristic compressive (cube) strength of concrete

$(f_{ck})_{cy}$ ---- characteristic compressive (cylinder) strength of concrete, given by 0.8 times 28 days cube crushing strength of concrete

f_{ctm} ---- mean tensile strength of concrete

E_{cm} ---- Secant Modulus of elasticity of concrete.

The values of E_{cm} given above are for quartzite/granite aggregates. They should be multiplied by the following factors as given below:
Limestone = 0.9; Sandstone = 0.7; Basalt = 1.2

The values have been taken from Table - 6.5 of IRC:112-2011

III.3 Reinforcement Steel for Concrete

Reinforcing Steel grades used in the concrete construction should conform to IS:1786-2008. It should be noted that the ductility of reinforcing bars will have a significant effect on the behavior of continuous composite beam where the reinforcements provide sufficient tensile resistance at top of the section.