STANDARD SPECIFICATIONS
AND
CODE OF PRACTICE FOR ROAD BRIDGES

SECTION V
STEEL ROAD BRIDGES
(LIMIT STATE METHOD)
(Third Revision)

Published by

INDIAN ROADS CONGRESS

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

MAY 2010

Price Rs. 800/-
(Packing and Postage charges extra)

First Published	:	May, 1967
Reprinted	:	August, 1972
Reprinted	:	July, 1976
First Revision	:	August, 1984 (Incorporates Amendment No.1 December, 1982)
Reprinted	:	October, 1994
Second Revision	:	April, 2001
Reprinted	:	October, 2003 (Incorporates Amendment No.1 to Second Revision)
Reprinted	:	April, 2007
Third Revision	:	May, 2010

Printed at India Offset Press, New Delhi-110 064
(1000 copies)

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STANDARD SPECIFICATIONS AND CODE OF PRACTICE FOR ROAD BRIDGES
Section V – STEEL ROAD BRIDGES (LIMIT STATE METHOD)

INTRODUCTION

The Standard Specifications and Code of Practice for Road Bridges, Section V : Steel Road Bridges (*Second Revision*), IRC : 24-2001 was published by the Indian Roads Congress in 2001. Since this Code was based on Working Stress Method (WSM) of design, it was felt necessary to bring out a revised version of the Code based on the modern concept of Limit State Method (LSM) of design in line with current International practice. LSM represents definite advancement over WSM. It represents realistic and quantitative safety being based on statistical and probability analysis. It uses a multiple safety factor format that intends to provide adequate safety at ultimate loads (which could be collapse or elastic buckling or fatigue fracture) as also adequate serviceability at service loads.

The work of revision of this Code was accordingly taken up by the Steel and Composite Structures Committee (B-5) during its tenure from 2006. The draft was discussed at length during various meetings and finalized in December 2008. The draft was discussed in the Bridges Specifications and Standards Committee meeting held on 18 May 2009 and some comments were made for consideration of the B-5 Committee. The Committee re-constituted in 2009 consisting of the following personnel in its meeting on 11 July 2009 appointed a Sub-Committee to finalise the document. The Sub-Committee considered the points raised by IRC as well as other subsequent comments and finalized the draft which was approved by B-5 Committee in its meeting held on 8 October 2009 for placing before the Bridges Specifications and Standards (BSS) Committee. The names of the Personnel of Steel and Composite Structures (B-5) Committee are given below:

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Secretary General, IRC	(Indoria, R.P.)

Sub-Committee Members

Ghoshal, A.	Ghosh, Prof. Achyut
Ghosh, U.K.	Guha, Arijit

Purakayastha, Debasish

The draft approved by Steel and Composite Structures Committee (B-5) was discussed by Bridges Specifications and Standards (BSS) Committee in the meeting on 26 October 2009 and approved the same, subject to certain modifications. Subsequently, the draft was approved by the Executive Committee on 31 October 2009. Finally the draft was approved in the 189th Council meeting held at Patna on 14 November 2009.

The object of issuing the document is to establish a common procedure for design and construction of road bridges in steel construction in India.

The revised publication is meant to serve as a guide to both the design and construction engineers, but compliance with the rules therein does not relieve them in anyway of their responsibility for stability and soundness of the structures designed and erected by them.

The design and construction of road bridges in steel construction require extensive and thorough knowledge of the science and technique involved and should be entrusted only to specifically qualified engineers and having adequate practical experience in bridge engineering and capable of ensuring careful execution of work.

501 GENERAL

501.1 Scope

501.1.1 This Code deals mainly with the design of the structural steelwork of normal road bridges (e.g. beams, plate girders, open web girders).

501.1.2 Provisions of this Code generally apply to riveted, bolted and welded constructions using hot rolled steel sections only. Cold formed sections are not covered in the Code.

501.1.3 IRC : 22 (Section VI) may be referred, wherever applicable in case of concrete work composite with steel.

501.1.4 For loads and load combinations reference shall be made to IRC:6.

501.1.5 The present version of the Code embodies application of limit state principles of design, which envisage that the structure will remain fit for use during its life with an acceptable level of reliability. The principles of limit state design have been discussed in greater details in Clause **503**. The provisions of Limit State Method (LSM) of design in the IS 800-2007 have been generally followed in this Code with appropriate changes, where necessary. Certain formulae and tables have been adapted from this document.

501.1.6 Generally steel bridge structures shall be designed by limit state method. Where limit state method cannot be conveniently adopted, working stress design method as per **Annex-G** may be used at the discretion of the concerned authority.

501.2 Limitations

This Code generally applies to normal steel bridges. For the following types of bridges for which there are special requirements for design, special literature may be referred to.

a) Curved bridges
b) Cable - stayed bridges
c) Suspension bridges
d) Temporary bridges
e) Pedestrian bridges
f) Swing bridges
g) Bascule bridges
h) Box girder bridges

i) Prestressed steel bridges

j) Arch bridges

This Code applies to such bridges to the extent where the special literature covering the above areas refers to the provisions of the present Code. However, the design of structural members and connections of all types of steel bridges may be done in accordance with the provisions of this Code. Limitation of this Code is listed in **Annex-A.**

501.3 References

While preparing this Code, practices prevailing in this country in the design and construction of steel bridges have been primarily kept in view. However, recommendations offered in the following publications have also been considered :

a) IS 800 - 2007 : General Construction in Steel - Code of Practice (*Third Revision*): Bureau of Indian Standards, New Delhi

b) BS 5400 - Part 3 - 2000 Code of Practice for Design of Steel Bridges: British Standards Institute, U.K.

c) Eurocode - 3 BS-EN 1993-2: 2006 Design of steel structures. Steel bridges

d) IRS Code of Practice for the design of steel or wrought iron bridges carrying rail, road or pedestrian traffic incorporating latest addendum/corrigendum - 2004.

501.4 Definitions

For the purpose of this Code, the following definitions shall apply :

Accidental Loads - Loads due to explosion, impact of vehicles, or other rare loads for which the structure is considered to be vulnerable as per the user.

Accompanying Load - Live (Imposed) load acting along with leading imposed load but causing lower effects and/or deflections.

Bearing Type Connection - A connection made using bolts in 'snug tight' condition, or rivets, where the load is transferred by bearing of bolts and rivets against plate inside the hole.

Braced Member - A member in which the relative transverse displacement is effectively prevented by bracing.

Buckling load - The load at which an element, a member or a structure as a whole, develops excessive lateral deformation or instability.

Buckling Strength or Resistance - Force or moment, which a member can withstand without buckling.

Camber - Intentionally introduced pre-curving (usually upwards) in a system, member or any portion of a member with respect to its geometry. Frequently, camber is introduced to compensate for deflections at a specific level of loads.

Characteristic Load - The value of specified load, above which not more than a specified percentage (usually 5 percent) of samples of corresponding load is expected to be encountered.

Characteristic Yield/Ultimate Stress - The minimum value of stress below which not more than a specified percentage (usually 5 percent) of corresponding stresses (yield/ultimate) of samples tested is expected to occur.

Compact Section - A cross-section, which can develop plastic moment, but has inadequate plastic rotation capacity needed for formation of a plastic collapse mechanism of the member or structure.

Constant Stress Range - The amplitude between which the stress ranges under cyclic loading is constant during the life of the structure or a structural element.

Cumulative Fatigue - Total damage due to fatigue loading of varying stress ranges.

Cut-off Limit - The stress range, corresponding to the particular detail below which cyclic loading need not be considered in cumulative fatigue damage evaluation (corresponds to 10^8 numbers of cycles in most cases).

Deflection - It is the deviation from the unloaded position of a member or structure caused by load or change in the material properties.

Design Life - Intended time period for which a structure or a structural element is required to perform its function, satisfying the criteria of performance as set out in this code.

Design Load/Factored Load - A load value obtained by multiplying the characteristic load with a load factor.

Design Spectrum - Frequency distribution of the stress ranges from all the nominal loading events during the design life, (stress spectrum).

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

Ductility - It is the property of the material or a structure indicating the extent to which it can deform beyond the limit of yield deformation before failure or fracture. The ratio of ultimate to yield deformation is usually termed 'ductility'.

Durability - It is the ability of a material to resist deterioration over long periods of time.

Edge Distance - Distance from the centre of a fastener hole to the nearest edge of an element measured perpendicular to the direction of load transfer.

Effective Lateral Restraint - Restrain which produces sufficient resistance to prevent deformation in the lateral direction.

Effective Length - Member length of a member between points of effective restraint or effective restraint and free end, multiplied by a factor to take account of the end conditions in buckling strength calculations.

Elastic Critical Moment - The elastic moment, which initiates lateral-torsional buckling of a laterally unsupported beam or girder.

Elastic Design - Design, which assumes elastic behaviour of materials throughout the service load range.

Elastic Limit - It is the stress below which the material regains its original size and shape when the load is removed. In steel design, it is taken as the yield stress/0.2 percent of proof stress.

End Distance - Distance from the centre of a fastener hole to the edge of an element measured parallel to the direction of load transfer.

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

Fatigue Loading - Set of nominal loading events, cyclic in nature, described by the distribution of the loads, their magnitudes and the number of applications in each nominal loading event.

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

Flexural Stiffness - Stiffness of a member against rotation as evaluated by the value of bending deformation moment required to cause a unit rotation while all other degrees of freedom of the joints of the member except the rotated one are assumed to be restrained.

Friction Type Connection - Connection effected by using pre-tensioned high strength bolts where shear force transfer is due to mobilization of friction between the connected plates due to clamping force developed at the interface of connected plates by the bolt pre-tension.

Gauge - The spacing between adjacent parallel lines of fasteners, transverse to the direction of load/stress.

High Shear - High shear condition is caused when the actual shear due to factored load is greater than a certain fraction of design shear resistance (Clause **510.2.2**).

Instability - The phenomenon which disables an element, member or a structure to carry further load due to excessive deflection lateral to the direction of loading and vanishing stiffness.

Lateral Restraint - See Effective lateral restraint.

Limit State - Any limiting condition beyond which the structure ceases to fulfill its intended function.

Load - An externally applied force causing stress or deformations in a structure such as dead, live, wind, seismic or temperature loads.

Load Effect - The internal force, axial shear, bending or twisting moment, due to external loads.

Main Member - A structural member, which is primarily responsible for carrying and distributing the applied load.

Member Length - The length between centre-to-centre of intersection points with connecting members or between the intersection point of the connecting member to the free end in case of a free standing member.

Mill Tolerance - Amount of variation allowed from the nominal dimensions and geometry, with respect to cross sectional area, non-parallelism of flanges, and out of straightness such as sweep or camber, in a product, as manufactured in a steel mill.

Normal Stress - Stress component acting normal to the face, plane or section.

Partial Safety Factor - The factor normally greater than unity by which either the loads are multiplied or the resistances are divided to obtain the design values.

Pitch - The centre-to-centre distance between individual fasteners in a line, in the direction of load/stress.

Plastic Collapse - The failure stage at which sufficient number of plastic hinges have formed due to the loads in a structure leading to a failure mechanism.

Plastic Design - Design against the limit state of plastic collapse.

Plastic Hinge - A yielding zone with significant inelastic rotation, which forms in a member, when the plastic moment is reached at a section.

Plastic Moment - Moment capacity of a cross-section when the entire cross-section has yielded due to bending moment.

Plastic Section - Cross-section, which can develop a plastic hinge and sustain plastic moment over sufficient plastic rotation required for formation of plastic failure mechanism of the member or structure.

Poisson's Ratio - It is absolute value of the ratio of lateral strain to longitudinal strain under uni-axial loading.

Prying Force - Additional tensile force developed in a bolt as a result of the flexing of a connection component such as a beam end plate or leg of an angle.

Rotation - The change in angle at a joint between the original orientation of two linear members and their final position under loading.

Secondary Member - Member which is provided for overall stability and/or for restraining the main members from buckling or similar modes of failure.

Semi-Compact Section - Cross-section, which can attain the yield moment, but not the plastic moment before failure by plate buckling.

Serviceability Limit State - A limit state of acceptable service condition exceedence of which causes serviceability failure.

Shear Force - The in-plane force at any transverse cross-section of a straight member.

Shear lag - The in-plane shear deformation effect by which concentrated forces tangential to the surface of a plate gets distributed over the entire section perpendicular to the load over a finite length of the plate along the direction of the load.

Shear Stress - The stress component acting parallel to a face, plane or cross-section.

Slender Section - Cross-section in which the elements buckle locally before reaching yield moment.

Slenderness Ratio - The ratio of the effective length of a member to the radius of gyration of the cross-section about the axis under consideration.

Slip Resistance - Limit shear that can be applied in a friction grip connection before slip occurs.

S-N curve - The curve defining the relationship between the number of stress cycles to failure (N_{sc}) at a constant stress range (S_c), during fatigue loading of a structure.

Snug Tight - The tightness of a bolt achieved by a few impacts of an impact wrench or by the full effort of a person using a standard spanner.

Stability Limit State - A limit state corresponding to the loss of static equilibrium of a structure by excessive deflection transverse to the direction of predominant loads.

Stiffener - An element used to retain or prevent the out-of-plane deformations of plates.

Strain - Deformation per unit length or unit angle.

Strain Hardening - The phenomenon of increase in stress with increase in strain beyond yielding.

Strength - Resistance to failure by yielding or buckling.

Strength Limit State - A limit state of collapse or loss of structural integrity.

Stress - The internal force per unit area of the original cross-section.

Stress Analysis - The analysis of the internal force and stress condition in an element member, or structure.

Stress Cycle Counting - Sum of individual stress cycles from stress history, arrived at using any rational method.

Stress Range - Algebraic difference between two extremes of stresses in a cycle of loading.

Stress Spectrum - Histogram of stress cycles produced by a nominal loading event design spectrum during design life.

Sway - The lateral deflection of a frame.

Sway Member - A member in which the transverse displacement of one end, relative to the other, is not effectively prevented.

Tensile Stress - The characteristic stress corresponding to rupture in tension, specified for the grade of steel in the appropriate Indian Standard.

Test Load - The factored load, equivalent to a specified load combination appropriate for the type of test being performed.

Transverse - Direction along the stronger axes of the cross section of the member.

Ultimate Limit State - The state which, if exceeded can cause collapse of a part or the whole of the structure.

Ultimate Stress - See Tensile Stress

Yield Stress - The characteristic stress of the material is tension before the elastic limit of the material is exceeded, as specified in the appropriate Indian Standard.

501.5 Symbols

Symbols used in this Code shall have the following meanings with respect to the structure or member or condition, unless otherwise defined elsewhere in this Code:

A	Area of cross-section
A_c	Area at root of threads
A_e	Effective cross-sectional area
A_f	Total flange area
A_g	Gross cross-sectional area
A_{gf}	Gross cross-sectional area of flange
A_{go}	Gross cross-sectional area of outstanding (not connected) leg of a member
A_n	Net area of the total cross-section
A_{nb}	Net tensile cross-sectional area of bolt
A_{nc}	Net cross-sectional area of the connected leg of a member
A_{nf}	Net cross-sectional area of each flange
A_{no}	Net cross-sectional area of outstanding (not connected) leg of a member
A_q	Cross-sectional area of a bearing (load carrying) stiffener in contact with the flange
A_s	Tensile stress area
A_{sb}	Gross cross-sectional area of a bolt at the shank
A_{tg}	Gross sectional area in tension from the centre of the hole to the toe of the angle section/channel section etc. (block shear failure, Clause **506.1.3**) perpendicular to the line of force.

A_{tn}	Net sectional area in tension from the centre of the hole to the toe of the angle perpendicular to the line of force (block shear failure, Clause **506.1.3**)
A_v	Shear area
A_{vg}	Gross cross-sectional area in shear along with line of transmitted force (block shear failure, Clause **506.1.3**)
A_{vn}	Net cross-sectional area in shear along the line of transmitted force (block shear failure Clause **506.1.3**)
a,b	Larger, and smaller projection of the slab base beyond the rectangle circumscribing the compression member respectively (Clause **507.4**)
a_o	Peak acceleration
a_l	Unsupported length of individual elements being laced between lacing points
B	Length of side of cap or base plate of a compression member
b	Outstand/width of the element
b_1	Stiff bearing length, Stiffener bearing length
b_e	Effective width of flange between pair of bolts
b_f	Width of the flange
b_p	Panel zone width between column flanges at beam-column junction
b_s	Shear lag distance
b_t	Width of tension field
b_w	Width of outstanding leg
C	Centre-to-centre longitudinal distance of battens
C_m	Coefficient of thermal expansion
C_{my}, C_{mz}	Moment amplification factor about respective axes
c	Spacing of transverse stiffener
c_b	Moment amplification factor for braced member
c_m	Moment reduction factor for lateral torsional buckling strength calculation
c_s	Moment amplification factor for sway frame

D	Overall depth/diameter of the cross-section
d	Depth of web, Nominal diameter
d_2	Twice the clear distance from the compression flange angles, plates or tongue plates to the neutral axis
d_h	Diameter of a bolt/rivet hole
d_0	Nominal diameter of the pipe compression member or the dimensions of the compression member in the depth direction of the base plate
d_p	Panel zone depth in the beam-column junction
E	Modulus of elasticity for steel
E_p	Modulus of elasticity of the panel material
F_d	Factored design load
F_n	Normal force
F_0	Minimum proof pretension in high strength friction grip bolts
F_q	Stiffener force
F_{qd}	Stiffener buckling resistance
F_{test}	Test load
$F_{test,a}$	Load for acceptance test
$F_{test.min}$	Minimum test load from the test to failure
$F_{test.R}$	Test load resistance
$F_{test.s}$	Strength test load
F_w	Design capacity of the web in bearing
F_x	External load, force or reaction
F_{xd}	Buckling resistance of load carrying web stiffener
f	Actual normal stress range for the detail category
f_a	Calculated stress due to axial force at service load
f_{abc}	Permissible bending stress in compression at service load
f_{ac}	Permissible compressive stress at service load
f_{abt}	Permissible bending stress in tension at service load
f_{apb}	Permissible bearing stress of the bolt at service load

f_{asb}	Permissible stress of the bolt in shear at service load
f_{at}	Permissible tensile stress at service load
f_{atb}	Permissible tensile stress of the bolt at service load
f_{aw}	Permissible stress of the weld at service load
f_b	Actual bending stress at service load
f_{bc}	Actual bending stress in compression at service load
f_{bd}	Design bending compressive stress corresponding to lateral buckling
f_{br}	Actual bearing stress due to bending at service load
f_{bt}	Actual bending stress in tension at service load
f_{bs}	Permissible bending stress in base of compression member at service load
f_c	Actual axial compressive stress at service load
f_{cc}	Elastic buckling stress of a compression member, Euler buckling stress
f_{cd}	Design compressive stress
$f_{cr,b}$	Extreme fibre compressive stress corresponding elastic lateral buckling moment
f_e	Equivalent stress at service load
f_f	Fatigue stress range corresponding to 5 x 10^6 cycles of loading.
f_{feq}	Equivalent constant amplitude stress
f_{fMax}	Highest normal stress range
f_{fn}	Normal fatigue stress range
f_{nw}	Normal stress in weld at service load
f_o	Proof stress
f_p	Actual bearing stress at service load
f_{pb}	Actual bearing stress in bending at service load
f_{psd}	Bearing strength of the stiffeners
f_r	Frequency
f_{sb}	Actual shear stress in bolt at service load

f_t	Actual tensile stress at service load
f_{tb}	Actual tensile stress of the bolt at service load
f_u	Characteristic ultimate tensile stress
f_{ub}	Characteristic ultimate tensile stress of the bolt
f_{um}	Average ultimate stress of the material as obtained from test
f_{up}	Characteristic ultimate tensile stress of the connected plate
f_v	Applied shear stress in the panel designed utilizing tension field action
f_w	Actual stress of weld at service load
f_{wd}	Design stress of weld at service load
f_{wn}	Nominal strength of fillet weld
f_x	Maximum longitudinal stress under combined axial force and bending
f_y	Characteristic yield stress
f_{yb}	Characteristic yield stress of bolt
f_{yf}	Characteristic yield stress of flange
f_{ym}	Average yield stress as obtained from test
f_{yp}	Characteristic yield stress of connected plate
f_{yq}	Characteristic yield stress of stiffener material
f_{yw}	Characteristic yield stress of the web material
G	Modulus of rigidity for steel
g	Gauge length between centre of the holes perpendicular to the load direction, acceleration due to gravity
h	Depth of the section
h_b	Total height from the base to the floor level concerned
h_c	Height of the column
h_e	Effective thickness
h_l	Height of the lip
h_y	Distance between shear centre of the two flanges of a cross section

I	Moment of inertial of the member about an axis perpendicular to the plane to the frame
I_{fc}	Moment of inertia of the compression flange of the beam about the axis parallel to the web.
I_{ft}	Moment of inertia of the tension flange of the beam about minor axis
I_q	Moment of inertia of a pair of stiffener about the centre of the web, or a single stiffener about the face of the web.
I_s	Second moment of inertia
I_{so}	Second moment of inertia of the stiffener about the face of the element perpendicular to the web
I_t	St. Venant's torsion constant
I_w	Warping constant
I_y	Moment of inertia about the minor axis of the cross-section
I_z	Moment of inertia about the major axis of the cross-section
K_b	Effective stiffness of the beam and column
KL	Effective length of the member
KL/r	Appropriate effective slenderness ratio of the section
KL/r_y	Effective slenderness ratio of the section about the minor axis of the section
KL/r_z	Effective slenderness ratio of the section about the major axis of the section
$\left(\frac{KL}{r}\right)_0$	Actual maximum effective slenderness ratio of the laced column
$\left(\frac{KL}{r}\right)_e$	Effective slenderness ratio of the laced column accounting for shear deformation
K_v	Shear buckling co-efficient
K_w	Warping restraint factor
L	Member length, Unsupported length, Length centre to centre distance of the intersecting members, Cantilever length
L_c	Length of end connection in bolted and welded members, taken as the distance between outermost fasteners in the end connection, or the length of the end weld, measured along the length of the member.

L_m	Maximum distance from the restraint to the compression flange at the plastic hinge to an adjacent restraint (limiting distance)
L_o	Length between points of zero moment (inflection) in the span
l	Centre to centre length of the supporting member
l_e	Distance between prying force and bolt centre line
l_g	Grip length of bolts in a connection
l_j	Length of the joint
l_s	Length between points of lateral support to the compression flange in a beam.
l_v	Distance from bolt centre line to the toe of fillet weld or to half the root radius for a rolled section
l_w	Length of weld
M	Bending moment
M_a	Applied bending moment
M_{cr}	Elastic critical moment corresponding to lateral torsional buckling of the beam.
M_d	Design flexural strength
M_{dv}	Moment capacity of the section under high shear
M_{dy}	Design bending strength about the minor axis of the cross-section
M_{dz}	Design bending strength about the major axis of the cross-section
M_{eff}	Reduced effective moment
M_{fr}	Reduced plastic moment capacity of the flange plate
M_{fd}	Design plastic resistance of the flange alone
M_{nd}	Design bending strength under combined axial force and uniaxial moment
M_{ndy}, M_{ndz}	Design bending strength under combined axial force and the respective uniaxial moment acting alone
M_p	Plastic moment capacity of the section
M_{pb}	Moment in the beam at the intersection of the beam and column centre lines

M_{pc}	Moments in the column above and below the beam surfaces
M_{pd}	Plastic design strength
M_{pdf}	Plastic design strength of flanges only
M_q	Applied moment on the stiffener
M_s	Moment at service (working) load
M_{tf}	Moment resistance of tension flange
M_y	Factored applied moment about the minor axis of the cross-section
M_{yq}	Moment capacity of the stiffener based on its elastic modulus
M_z	Factored applied moment about the major axis of the cross-section
N	Number of parallel planes of battens
N_d	Design strength in tension or in compression
N_f	Axial force in the flange
N_{SC}	Number of stress cycles
n	Number of bolts in the bolt group/critical section
n_e	Number of effective interfaces offering frictional resistance to slip
n_n	Number of shear planes with the threads intercepting the shear plane in the bolted connection
n_s	Number of shear planes without threads intercepting the shear plane in the bolted connection
P	Factored applied axial force
P_{cc}	Elastic buckling load
P_d	Design axial compressive strength
P_{dy}, P_{dz}	Design compressive strength as governed by flexural buckling about the respective axis
P_e	Elastic Euler buckling load
P_{Min}	Minimum required strength for each flange splice
P_r	Required compressive strength
P_s	Actual compression at service load

P_y	Yield strength of the cross section under axial compression
p	Pitch length between centres of holes parallel to the direction of the load
p_s	Staggered pitch length along the direction of the load between lines of the bolt holes (**Fig. 2**)
Q	Prying force
Q_a	Accidental load
Q_c	Characteristics load
Q_d	Design load
Q_P	Permanent loads
Q_v	Variable loads
q	Shear stress at service load
R	Ratio of the mean compressive stress in the web (equal to stress at mid depth) to yield stress of the web reaction of the beam at support
R_i	Net shear in bolt group at bolt "i"
R_r	Response reduction factor
R_{tf}	Flange shear resistance
r	Appropriate radius of gyration
r_1	Minimum radius of gyration of the individual element being laced together
r_{vv}	Radius of gyration about the minor axis(v-v) of angle section
r_y	Radius of gyration about the minor axis
r_z	Radius of gyration about the major axis
S	Minimum transverse distance between the centroid of the rivet or bolt or weld group
S_c	Constant stress range
S_d	Design strength
S_o	Original cross-sectional area of the test specimen
S_p	Spring stiffeness

S_u	Ultimate strength
s_c	Anchorage length of tension field along the compression flange
s_t	Anchorage length of tension field along the tension flange
s_a	Actual stiffener spacing
T	Factored tension
T_b	Applied tension in bolt
T_{cf}	Thickness of compression flange
T_d	Design strength under axial tension
T_{dg}	Yielding strength of gross section under axial tension
T_{dn}	Rupture strength of net section under axial tension
T_{db}	Design strength of bolt under axial tension, Block shear strength at end connection
T_e	Externally applied tension
T_f	Factored tension force of friction type bolt
T_{nb}	Nominal strength of bolt under axial tension
T_{nd}	Design tension capacity
T_{ndf}	Design tension capacity of friction type bolt
T_{nf}	Nominal tensile strength of friction type bolt
T_s	Actual tension under service load
t	Thickness of element/angle, time in minutes
t_f	Thickness of flange
t_p	Thickness of plate
t_{pk}	Thickness of packing
t_q	Thickness of stiffener
t_s	Thickness of base slab
t_t	Effective throat thickness of welds
t_w	Thickness of web
V	Factored applied shear force
V_b	Shear in batten plate

V_{bf}	Factored frictional shear force in friction type connection
V_{cr}	Critical shear strength corresponding to web buckling
V_d	Design shear strength
V_{db}	Block shear strength
V_{nb}	Nominal shear strength of bolt
V_{nbf}	Bearing capacity of bolt for friction type connection
V_p	Plastic shear resistance under pure shear
V_n	Nominal shear strength
V_{npb}	Nominal bearing strength of bolt
V_{nsb}	Nominal shear capacity of a bolt
V_{nsf}	Nominal shear capacity of bolt as governed by slip in friction type connection
V_s	Transverse shear at service load
V_{sb}	Factored shear force in the bolt
V_{sd}	Design shear capacity
V_{sdf}	Design shear strength in friction type bolt
V_{sf}	Factored design shear force of friction bolts
V_t	Applied transverse shear
V_{tf}	Shear resistance in tension field
W	Total load
w	Uniform pressure from below on the slab base due to axial compression under the factored load
w_{tf}	Width of tension field
x_t	Torsional index
Z_e	Elastic section modulus
Z_{ec}	Elastic section modulus of the member with respect to extreme compression fibre
Z_{et}	Elastic section modulus of the member with respect to extreme tension fibre

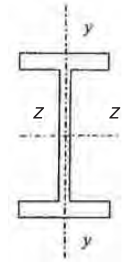
Z_p	Plastic section modulus
Z_v	Contribution to the plastic section modulus of the total shear area of the cross section
y_g	Distance between point of application of the load and shear centre of the cross section
y_s	Co-ordinate of the shear centre in respect to centroid
α	Imperfection factor for buckling strength in compression members and beams
α_t	Coefficient of thermal expansion
β_M	Ratio of smaller to the larger bending moment at the ends of a beam column.
β_{My}, β_{Mz}	Equivalent uniform moment factor for flexural buckling for y-y and z-z axes respectively.
β_{MLT}	Equivalent uniform moment factor for lateral torsional buckling
χ	Strength reduction factor to account for buckling under compression
χ_m	Strength reduction factor, χ, at f_{ym}
χ_{LT}	Strength reduction factor to account for lateral torsional buckling of beams
δ	Deflection
δ_p	Load amplification factor
ϕ	Inclination of the tension field stress in web
γ	Unit weight of steel
γ_f	Partial safety factor for load
γ_m	Partial safety factor for material
γ_{mo}	Partial safety factor against yield stress and buckling
γ_{ml}	Partial safety factor against ultimate stress
γ_{mb}	Partial safety factor for bolted connection with bearing type bolts
γ_{mf}	Partial safety factor for bolted connection with high strength friction grip bolts

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
ε	Yield stress ratio, $(250/f_y)^{½}$
λ	Non dimensional slenderness ratio = $\sqrt{f_y (KL/r)^2 / \pi^2 E} = \sqrt{f_y / f_{cc}} = \sqrt{P_y / P_{cc}}$
λ_{cr}	Elastic buckling load factor
λ_e	Equivalent slenderness ratio
μ	Poisson's ratio
μ_c	Correction factor
μ_f	Coefficient of friction (slip factor)
μ_r	Capacity reduction factor
θ	Ratio of the rotation at the hinge point to the relative elastic rotation of the far end of the beam segment containing plastic hinge
ρ	Unit mass of steel
τ	Actual shear stress range for the detail category
τ_b	Buckling shear stress
τ_{ab}	Permissible shear stress at the service load
$\tau_{cr,e}$	Elastic critical shear stress
τ_f	Fatigue shear stress range
τ_{fMax}	Highest shear stress range
τ_{fn}	Fatigue shear stress range at N_{SC} cycle for the detail category
τ_v	Actual shear stress at service load
Ψ	Ratio of the moments at the ends of the laterally unsupported length of a beam

NOTE: The subscripts *y, z* denote the *y-y* and *z-z* axes of the section, respectively. For symmetrical section, *y-y* denotes the minor principal axis whilst *z-z* denotes the major principal axis (Clause **501.6**).

501.6 Convention for Member Axes

Unless otherwise specified convention used for member axes is as follows :



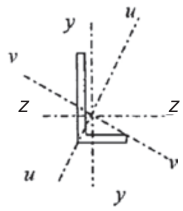
x-x along the member

y-y an axis of the cross-section

- perpendicular to the flanges
- perpendicular to the smaller leg in an angle section

z-z an axis of the cross-section

- axis parallel to flanges
- axis parallel to smaller leg in angle section



u-u major axis (when it does not coincide with *z-z* axis)

v-v minor axis (when it does not coincide with *y-y* axis)

501.7 Units

For the purpose of design calculations the following units are recommended :

a) Forces and loads : kN, kN/m, kN/m^2

b) Unit mass : kg/m^3

c) Unit weight : kN/m^3

d) Stresses and strengths : N/mm^2 (= MN/m^2 or MPa)

e) Moments (bending, etc.) kNm

For conversion of system of units to another system, IS 786 (Supplement) may be referred.

502 MATERIALS AND PROPERTIES

502.1 General

The material properties given in this clause are nominal values, as given by various IS Codes defining the material properties to be accepted as characteristic values in design calculations.

502.2 Structural Steel

502.2.1 Provisions in this clause are applicable to the structural steels commonly used in steel bridge construction namely:

a) Mild Steel

b) Medium and High Strength Steel

502.2.2 *Properties of steel*

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

a) Unit mass of steel, ρ = 7850 kg/m^3

b) Modulus of elasticity, E = 2.0 x 10^5 N/mm^2 (MPa)

c) Poisson's Ratio, μ= 0.3

d) Modulus of rigidity, G = 0.77 x 10^5 N/mm^2 (MPa)

e) Coefficient of thermal expansion α_t= 12 x 10^{-6}/^{o}C

502.2.2.2 The principal mechanical properties of the structural steel important in design, are the yield stress, f_y, the tensile or ultimate stress, f_u, the maximum percent elongation on a standard gauge length and notch toughness. Except for notch toughness, other properties are determined by conducting tensile tests on samples cut from the plates, sections etc., according to IS 1608. For notch toughness test IS 1499 may be referred.

502.2.3 *Structural steels*

All structural steel shall, before fabrication comply with the requirements of the latest revisions of the following Indian Standards :

IS 808	Dimensions for hot rolled steel beam, column, channel and angle sections
IS 1161	Steel tubes for structural purposes
IS 1239 (Pt 1)	Steel tubes, tubulars and other wrought steel fittings: Part 1 Steel tubes
IS 1239 (Pt 2)	Mild steel tubes, tubulars and other wrought steel fittings : Part 2 Mild steel tubulars and other wrought steel pipe 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 strength structural steel
IS 4923	Hollow Steel sections for structural use
IS 11587	Structural weather resistant steels

IS 12778	Hot-rolled parallel flange steel sections for beams, columns and bearing piles.

502.2.4 *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

502.3 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 2708	1.5 percent manganese steel castings
IS 4367	Alloy steel forgings for general industrial use

502.4 Fasteners

Bolts, nuts, washers and rivets shall comply with the following or relevant IS 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 M 5 to M 64)
IS 1364 (Pt 1 to Pt 3)	Hexagon head bolts, screws & nuts products grade A & B (size range M 1.6 to M 64)
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 & 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 (ISMB)
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

502.5 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 and 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 steels

502.6 Welding

IS 812	Glossary of terms relating to welding and cutting of metals
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 1 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 test 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

502.7 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)	Plain hard-drawn steel wire for prestressed concrete : Part 1 Cold drawn stress relieved wire
IS 1785 (Pt 2)	Plain hard-drawn steel wire for prestressed 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

503 LIMIT STATE DESIGN

503.1 Basis of Design

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

503.1.2 Steel bridge structures are to be designed and constructed to satisfy the design requirements with regard to stability, strength, brittle fracture, serviceability, fatigue and durability such that they

a) shall remain fit with adequate reliability and be able to sustain all loads and other influences experienced during construction and use,

b) have adequate durability under normal maintenance,

c) shall not suffer overall damage or collapse under accidental events like fire hazards, explosions, vehicle impact, or due to consequences of human error to an extent beyond local damage. The potential for catastrophic damage shall be limited or avoided by appropriate choice of one or more the following:

   i) Avoiding, eliminating or reducing exposure to hazards, which the structure is likely to suffer.

   ii) Choosing structural forms, layouts and details, and designing such that the structure has low sensitivity to hazardous conditions.

   iii) Introducing redundancy in the structural system, so that in the event of failure of a member, the structure does not collapse, and suffers only local damage.

   iv) Choosing suitable material, design and detailing procedure, construction specifications, and control procedures for shop fabrication and field construction.

   v) Providing adequate bracing system.

503.2 Limit State Design

503.2.1 Design shall be based on the characteristic values of material strengths and applied loads, which take into account the probability of variations in the material strengths and the applied loads. The characteristic values shall be based on statistical data, if available. Where such data are not available, these shall be based on experience. The design values, are derived from the characteristic values through the use of partial safety factors both for material strengths and for loads. These factors are dependent on the type of the material, the type of load and the limit state being considered. The reliability of the design is ensured when :

Design load $\leq$ Design strength

503.2.2 Limit states are the states beyond which the structure no longer satisfies the specified performance requirements. The limits states are classified as :

a) Limit State of Strength
b) Limit State of Serviceability
c) Limit State of Fatigue

503.2.2.1 Limit state of strength

Limit state of strength is associated with the failure (or imminent failure), under the action of probable and most unfavourable combination of loads on the structure using the appropriate partial safety factors, which may endanger the safety of life and/or property. The limit state of strength includes:

a) Loss of equilibrium of the structure as a whole or any of its parts or components
b) Loss of stability of the structure (including the effect of overturning)
c) Failure by excessive deformation (including buckling induced deformation), rupture of the structure or any of its parts or components.
d) Brittle fracture

503.2.2.2 Limit state of serviceability

Limit state of serviceability includes:

a) Deformation or deflection, which may adversely affect the appearance or effective use of the bridge structure.
b) Vibration in the structure or any of its components causing discomfort to the user or damages to the structure or which may limit its functional effectiveness
c) Corrosion and durability

503.2.2.3 Limit state of fatigue

Limit state of fatigue is the state at which stress range due to application of live loads reaches the limiting values as per Clause **511**, corresponding to the number of load cycles and detail configuration.

503.3 Design Loads

The loads specified in IRC:6 shall be considered along with the load factors.

503.4 Design Strength

The design strength, S_d is obtained as given below from ultimate strength, S_u and partial safety factors for materials, γ_m (**Table 1**):

$$S_d = S_u/\gamma_m$$

NOTE*:* Partial safety factor for materials (γ_m) account for the possibilities of :

a) unfavourable deviation of material strength from the characteristic value

b) unfavourable variation of member sizes,

c) unfavourable reduction in member strength due to fabrication and tolerances,

d) uncertain calculation of strength of the members.

Table 1 Partial Safety Factor for Materials, γ_m
(Clause **503.4**)

Sl.No.	Definition	Partial Safety Factor	
1)	Resistance, governed by yielding γ_{m0}	1.10	
2)	Resistance of member governed by buckling γ_{m0}	1.10	
3)	Resistance, governed by ultimate stress γ_{ml}	1.25	
4)	Resistance of connection	Shop fabrications	Field fabrications
	a) Bolts-friction type γ_{mf}	1.25	1.25
	b) Bolts-bearing type γ_{mb}	1.25	1.25
	c) Rivets γ_{mr}	1.25	1.25
	d) Welds γ_{mw}	1.25	1.50

503.5 Factors Governing Ultimate Strength

503.5.1 *Stability* - Stability shall be ensured for the structure as a whole and for each of its elements. This should include, overall frame stability against overturning given below:

Stability Against Oveturning - The structure as a whole or any part of it shall be designed to prevent instability due to overturning, uplift or sliding under factored load as given below:

a) The loads shall be divided into components aiding instability and components resisting instability.

b) The permanent and variable loads and their effects causing instability shall be combined using appropriate load factors as per the Limit States requirements to obtain maximum destabilizing effect.

c) The permanent loads and effects contributing to resistance shall be multiplied by a partial safety factor 0.9 and added together with design resistance (after multiplying by appropriate partial safety factor). Variable loads and their effects contributing to resistance shall be disregarded.

d) The resistance effect shall be greater than or equal to the destabilizing effect. Combination of imposed and dead loads should be such as to cause most severe effect on overall stability.

503.5.2 *Fatigue* - Fatigue design shall be as per Clause **511** of this Code. When designing for fatigue the partial safety factor for loads (f) shall be considered as 1.00 for loads causing stress fluctuation and stress range.

503.6 Geometrical Properties

The geometrical properties of the gross and the effective cross-sections of a member or part thereof, shall be calculated on the following basis:

a) The properties of the gross cross-section shall be calculated from the specified size of the member or part thereof or read from appropriate table.

b) The properties of the effective cross-section shall be calculated by deducting from the area of the gross cross-section the following:

   i) The sectional area in excess of effective plate width, in case of slender sections (Clause **503.7.2**).

   ii) The sectional areas of all holes in the section except for parts in compression. In case of punched holes, hole size 2 mm in excess of the actual diameter may be deducted.

503.7 Classification of Cross-Sections

503.7.1 The local buckling of plate elements of a cross-section can be avoided before the limit state is achieved by limiting the width to thickness ratio of each element of a cross-section, subjected to compression due to axial force, moment or shear.

503.7.1.1 When plastic analysis is used, the members shall be capable of forming plastic hinges with sufficient rotation capacity (ductility) without local buckling to enable the redistribution of bending moment required before formation of the failure mechanism.

503.7.1.2 When elastic analysis is used, the member shall be capable of developing the yield stress under compression without local buckling.

503.7.2 On the basis of the above, four classes of sections are defined as follows:

Class 1: ***Plastic*** - Cross-sections, which can develop plastic hinges and have the rotation capacity required for failure of the structure by formation of plastic mechanism. The width to thickness ratio of plate elements shall be less than that specified under Class 1 (Plastic) in **Table 2**.

Class 2: ***Compact*** - Cross-sections, which can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of plastic mechanism, due to local buckling. The width to thickness ratio of plate elements shall be less than that specified under Class 2 (compact), but greater than that specified under Class 1 (Plastic) in **Table 2**.

Class 3 : ***Semi-compact*** - Cross-sections, in which the extreme fibre in compression can reach yield stress, but cannot develop the plastic moment of resistance, due to local buckling. The width to thickness ratio of plate elements shall be less than that specified under Class 3 (Semi-compact), but greater than that specified under Class 2 (Compact) in **Table 2**.

Class 4: ***Slender*** - Cross-sections in which the elements buckle locally even before reaching yield stress. The width to thickness ratio of plate element shall be greater than that specified under Class 3 (Semi-compact) in **Table 2**. In such cases the effective sections for design shall be calculated by deducting width of compression plate element in excess of the Semi-compact section limit. The design of slender compression element is outside the scope of this code.

When different elements of a cross-section fall under different classes, the section shall be classified as governed by the most critical element.

The maximum value of limiting width to thickness ratios of elements for different classifications of sections are given in **Table 2.**

Table 2 Limiting Width to Thickness Ratio
(Clauses **503.7.2** and **503.7.4**)

<table>
<tr><th colspan="3" rowspan="2">Compression Element
(1)</th><th rowspan="2">Ratio
(2)</th><th colspan="3">Class of Section</th></tr>
<tr><th>Class 1
Plastic
(3)</th><th>Class 2
Compact
(4)</th><th>Class 3
Semi-Compact
(5)</th></tr>
<tr><td colspan="2" rowspan="2">Outstanding element of compression flange</td><td>Rolled Section</td><td>b/t_f</td><td>9.4ε</td><td>10.5ε</td><td>15.7ε</td></tr>
<tr><td>Welded Section</td><td>b/t_f</td><td>8.4ε</td><td>9.4ε</td><td>13.6ε</td></tr>
<tr><td colspan="2" rowspan="2">Internal element of Compression Flange</td><td>Compression due to Bending</td><td>b/t_f</td><td>29.3ε</td><td>33.5ε</td><td rowspan="2">42ε</td></tr>
<tr><td>Axial Compression</td><td>b/t_f</td><td colspan="2">Not applicable</td></tr>
<tr><td rowspan="3">Web of an I-H- or Box Section</td><td colspan="2">Neutral axis at mid-depth</td><td>d/t_w</td><td>84ε</td><td>105ε</td><td>126ε</td></tr>
<tr><td rowspan="2">Generally</td><td>If r_1 is negative:</td><td>d/t_w</td><td rowspan="2">$84\varepsilon/(1+r_1)$ but $\geq 42\varepsilon$</td><td>$105.0\varepsilon/(1+r_1)$</td><td rowspan="2">$126.0\varepsilon/(1+2r_2)$ but $\geq 42\varepsilon$</td></tr>
<tr><td>If r_1 is positive:</td><td>d/t_w</td><td>$105.0\varepsilon/(1+1.5r_1)$ but $\geq 42\varepsilon$</td></tr>
<tr><td></td><td colspan="2">Axial compression</td><td>d/t_w</td><td colspan="2">Not applicable</td><td>42ε</td></tr>
<tr><td colspan="3">Web of a channel</td><td>d/t_w</td><td>42ε</td><td>42ε</td><td>42ε</td></tr>
<tr><td colspan="3">Angle, compression due to bending (Both criteria should be satisfied)</td><td>b/t
d/t</td><td>9.4ε
9.4ε</td><td>10.5ε
10.5ε</td><td>15.7ε
15.7ε</td></tr>
<tr><td colspan="3">Single angle, or double angles with the components separated, axial compression (All three criteria should be satisfied)</td><td>b/t
d/t
$(b+d)/t$</td><td colspan="2">Not Applicable</td><td>15.7ε
15.7ε
25ε</td></tr>
<tr><td colspan="3">Outstanding leg of an angle in contact back-to-back in a double angle member</td><td>d/t</td><td>9.4ε</td><td>10.5ε</td><td>15.7ε</td></tr>
<tr><td colspan="3">Outstanding leg of an angle with its back in continuous contact with another component</td><td>d/t</td><td>9.4ε</td><td>10.5ε</td><td>15.7ε</td></tr>
<tr><td colspan="3">Stem of a T-section, rolled or cut from a rolled I-or-H-section</td><td>D/t_f</td><td>8.4ε</td><td>9.4ε</td><td>18.9ε</td></tr>
<tr><td colspan="3" rowspan="2">Circular hollow tube, including welded tube subjected to (a) moment (b) axial compression</td><td>D/t_f</td><td>$42\varepsilon^2$</td><td>$52\varepsilon^2$</td><td>$146\varepsilon^2$</td></tr>
<tr><td>D/t</td><td colspan="2">Not applicable</td><td>$88\varepsilon^2$</td></tr>
</table>

(Contd.)

(Table 2 Contd.)

NOTE 1	:	Elements which exceed semi-compact limits are to be taken as of slender cross-section
NOTE 2	:	$\varepsilon = (250/f_y)^{1/2}$
NOTE 3	:	*Webs shall be checked for shear buckling in accordance with Clause* ***509.4.2*** *when d/t > 67ε. 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 is outer diameter of the element, Refer* **Fig 1** *Clauses* ***503.7.3*** *and* ***503.7.4.***
NOTE 4	:	*Different elements of a cross-section can be different classes. In such cases the section is classified based on the least favourable classification.*
NOTE 5	:	*The stress ratio r_1 and r_2 are defined as*
r_1	=	*(actual average axial stress(negative, if tensile))/(design compressive stress of web alone)*
r_2	=	*(actual average axial stress(negative, if tensile))/(design compressive stress of overall Section)*

503.7.3 *Types of Elements*

a) 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.

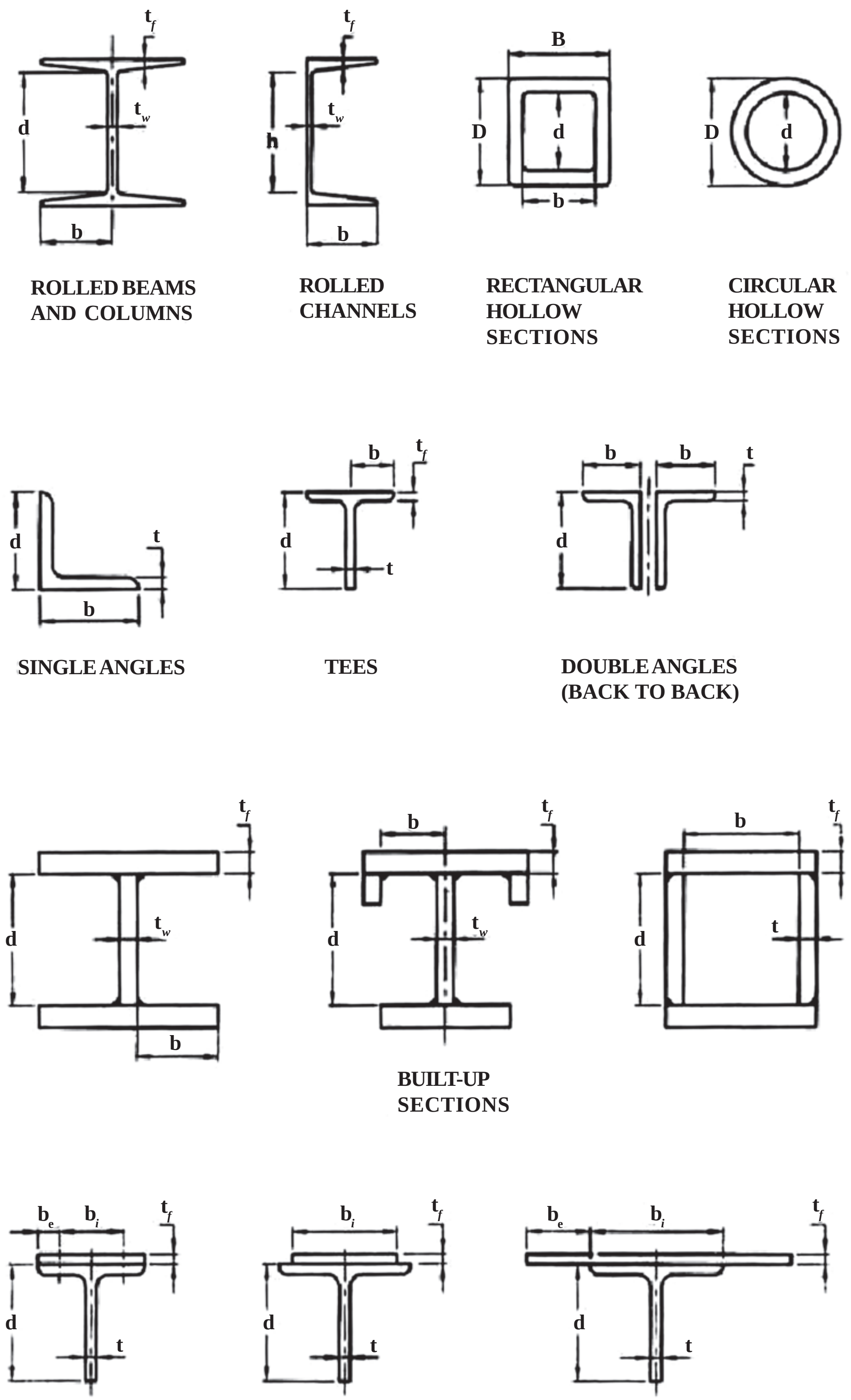
b) Outside elements or Outstands are elements attached along only one of the longitudinal edges to an adjacent element, the other edge being free to displace out of plane e.g. flange overhang of an I-section, stem of T-section and legs of an angle section.

c) Tapered elements may be treated as flat elements having average thickness defined in SP:6 Part 1 of BIS.

503.7.4 Compound elements in built-up section (**Fig. 1**) - In case of compound elements consisting of two or more elements bolted or welded together, the limiting width to thickness ratios as given in **Table 2** should be considered as follows :

a) Outstanding width of compound elements (b_e) to its own thickness.

b) The internal width of each added plate between the lines of welds or fasteners connecting it to the original section to its own thickness.



b_i - Internal Element Width

b_e - External Element Width

Fig. 1 Dimensions of Sections

c) Any outstand of the added plates beyond the line of welds or fasteners connecting it to original section to its own thickness.

504 GENERAL DESIGN CONSIDERATIONS

504.1 Effective Span

The effective span shall be as given below :

a) ***For main girders*** - the distance between the centres of bearings

b) ***For cross girders*** - the distance between the centres of main girders or trusses

c) ***For stringers*** - the distance between the centres of cross girders

NOTE:- Where a cross girder or stringer terminates on an abutment or pier, the centre of bearing thereon shall be taken as one end of the effective span.

d) ***For pins in bending*** - the distance between the centres of bearings; but where pins pass through bearing plates having thickness greater than half the diameter of the pins, consideration may be given to the effect of the distribution of bearing pressures on the effective span.

504.2 Effective Depth

The effective depth of plate or truss girder should be taken as the distance between the centres of gravity of the upper and lower flanges or chords.

504.3 Spacing of Girders

The distance between centres of the main girders shall be sufficient to resist overturning or over stressing due to lateral forces and loading conditions. Otherwise special provisions must be made to prevent this. This distance shall not be less than 1/20 of the span.

504.4 Depth of Girders

Minimum depth preferably shall not be less than the following :

a) ***For trusses*** : 1/10 of the effective span

b) ***For rolled steel joists and plate girders*** : 1/25 of the effective span

The effective depth of open web girders shall not be greater than three times the distance between the centres of these girders.

504.5 Deflection of Girders

Deflection is to be checked by elastic analysis, using a partial safety factor for loads as 1.0.

504.5.1 Rolled steel beams, plate girders or lattice girders, either simple or continuous spans, shall be designed so that the total deflection due to dead load, live load and impact shall not exceed 1/600 of the span. However, this restriction shall not apply if minimum in-place precamber is provided to compensate for all dead and superimposed dead load deflections.

Additionally, the deflection due to live load and impact shall not exceed of 1/800 of the span.

504.5.2 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 deflection due to live load and impact shall not exceed 1/400 of the cantilever arm.

504.5.3 Sidewalk live load may be neglected in calculating deflection.

504.5.4 When cross bracings or diaphragms of sufficient depth and strength are provided between beams to ensure the lateral distribution of loads the deflection may be calculated considering all beams acting together.

504.5.5 The gross moment of inertia shall be used for calculating the deflection of beams or plate girders. In calculating the deflection of trusses the gross area of each truss member should be used.

504.6 Camber

504.6.1 Camber, if any, shall be provided as specified by the engineer. Camber may be required to maintain clearance under all conditions of loading or it may be required for the sake of appearance.

504.6.2 In the absence of specific guidance, the following principles may be observed.

a) Beams and plate girders up to and including 35 m span need not be cambered.

b) In open web spans the camber of the main girders and the corresponding variations in length of members shall be such that when the girders are loaded with full dead load plus 75 percent of the live load without impact producing maximum bending moment, they shall take up the true geometrical shape assumed in their design. The camber diagram shall be prepared as indicated in **Annex-B**.

504.7 Minimum Sections

504.7.1 For all members of the structure, except parapets and packing plates, the following minimum thicknesses of plates and rolled sections shall apply :

a) 8 mm when both sides are accessible for painting or are in close contact with other plates or rolled sections, or are otherwise adequately (refer **Annex-D**) protected against corrosion.

b) When one side is not readily accessible for painting or is not in close contact with another member, or is not otherwise adequately (refer **Annex-D**) protected and where the thickness required by calculation is less than 12.5 mm, 1.5 mm shall be added to the calculated thickness subject to the total thickness being not less than 10 mm.

c) 6 mm for box members when the inside of the member is effectively sealed.

d) For rolled steel beams and channels the controlling thickness shall be taken as the mean thickness of the flange, regardless of the web thickness

504.7.2 In floor plates and parapets not designed to carry stresses a minimum thickness of 6 mm shall be used if both sides are accessible or 8 mm if only one side is accessible. For packing plates the thickness shall not be less than 1.5 mm.

504.7.3 In riveted construction no angle less than 75 mm x 50 mm shall be used for the main members of the girders.

504.7.4 No angle less than 65 mm x 45 mm and no flat less than 50 mm wide shall be used in any part of a bridge structure, except for hand railings and shear connectors.

504.7.5 Thickness of end angles connecting stringers to cross girders or cross girders to main girders shall be not less in thickness than three quarters of the thickness of the web plates of the stringers and cross girders respectively.

504.8 Skew Bridges

For skew bridges, detailed analysis of forces shall be required. However, if the angle of skew is within 15^0, such detailed analysis may not be necessary.

504.9 Bearings

504.9.1 Provision for jacking of the steel girder for inspection and maintenance of the bearings shall be in-built in the bridge structure and the jacking positions shall be identified and clearly marked.

504.9.2 It shall be ensured that, while selecting the bearing type and designing it, the adequacy of the load transfer mechanisms from superstructure to bearing and bearing to sub-structure have been examined and provided for.

504.10 Fire Hazards

504.10.1 Adequate provision may be made as far as possible for fire fighting equipment to access all parts of the bridge.

504.10.2 In case of accidental occurrence of fire in a bridge 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 use.

505 ANALYSIS OF STRUCTURES

505.1 General

Effects of design loads on a bridge structure and its members and connections shall be determined by structural analysis using Elastic analysis

505.2 Elastic Analysis

505.2.1 *Assumption*- Individual members shall be assumed to remain elastic under the effects of factored design loads for all limit states.

505.2.2 The effect of haunching or any variation of the cross-section along the axis of a member shall be considered, and where significant shall be taken into account in the determination of the member stiffness.

505.2.3 Appropriate load combinations with corresponding load factors are to be used to find out the maximum values of load effects on members.

505.2.4 In a first-order elastic analysis, the equilibrium of the frame in the undeformed geometry is considered, the changes in the geometry of the frame due to the loading are not accounted for, and changes in the effective stiffness of the members due to axial force are neglected. The effect of these on the first order bending moments may be accounted for by carrying out second order elastic analysis.

506 DESIGN OF TENSION MEMBERS

506.1 Design

Tension members are linear members in which axial forces act causing elongation (stretch). Such members can sustain loads upto ultimate load, at which stage they may fail by rupture at a critical section. However, if the gross area of the member yields over a major portion of its length before the rupture load is reached, the member may become non-functional due to excessive elongation. Plates and other rolled sections in tension may also fail by block shear of end bolted riveted regions (See Clause **506.1.3**).

The factored design tension T, in the members shall satisfy the following requirement :

$$T < T_d$$

where

T_d = design strength of the member

The design strength of a member under axial tension, T_d is the lowest of the design strength due to yielding of gross section, T_{dg}, rupture of critical section, T_{dn} and block shear T_{db} given in Clauses **506.1.1, 506.1.2** and **506.1.3** respectively

506.1.1 *Design strength governed by yielding of gross section*

The design strength of members under axial tension, T_{dg} as governed by yielding of gross section, is given by

$$T_{dg} = A_g f_y / \gamma_{m0}$$

where

f_y = yield stress of the material

A_g = gross area of cross-section

γ_{m0} = partial safety factor for failure in tension by yielding (Table 1)

506.1.2 Design strength governed by rupture of critical section

506.1.2.1 *Plates* - The design strength in tension of a plate, T_{dn} as governed by rupture of net cross sectional area, A_n at the holes is given by

$$T_{dn} = 0.9 A_n f_u / \gamma_{m1}$$

where

γ_{m1} = partial safety factor for failure at ultimate stress (**Table 1**)

f_u = ultimate stress of the material

A_n = net effective area of the member given by,

$$A_n = \left[b - nd_h + \sum_i \frac{P_{si}^{\ 2}}{4g_i} \right] t$$

where

b, t	=	width and thickness of the plate, respectively
d_h	=	diameter of the hole
g	=	gauge length between the holes, as shown in **Fig. 2**
p_s	=	staggered pitch length between line of holes as shown in **Fig. 2**
n	=	number of holes in the critical section
i	=	subscript for summation of all the inclined legs

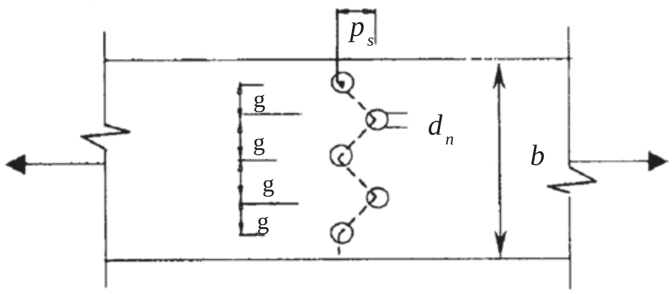


Fig. 2 Plates with Holes in Tension

506.1.2.2 *Threaded rods* - The design strength of threaded rods in tension, T_{dn}, as governed by rupture is given by

$$T_{dn} = 0.9\ A_n\ f_u\ /\gamma_{m1}$$

where

A_n	=	net root area at the threaded section,

506.1.2.3 *Single angles* - The rupture strength of an angle connected through one leg is affected by shear lag. The design strength T_{dn} , as governed by rupture at net section is given by

$$T_{dn} = 0.9\ A_{nc}\ f_u\ /\ \gamma_{m1} + \beta A_{go}\ f_y\ /\gamma_{m0}$$

where

$$\beta \ = \ 1.4 - 0.076\ (w/t)\ (f_y/\ f_u)\ (b_s/L_c) \leq (f_u\,\gamma_{m0}/f_y\ \gamma_{m1}) \geq\ 0.7$$

where

w = outstand leg width

b_s = shear lag width as shown in **Fig. 3**

L_c = length of the end connection, i.e., distance between the outermost bolts/ rivets in the end joint measured along the load direction or length of the weld along the load direction

For preliminary sizing, the rupture strength of net section may be approximately taken as

$$T_{dn} = \alpha A_n f_u / \gamma_{m1}$$

where

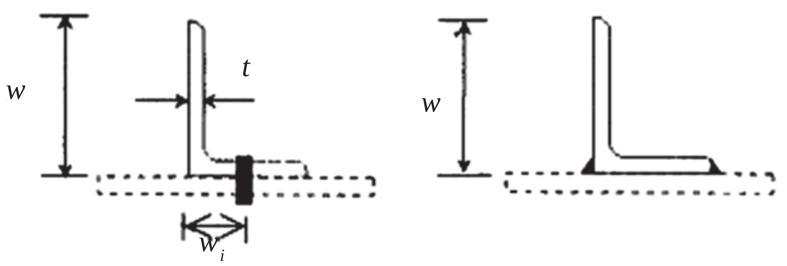
α = 0.6 for one or two bolts/rivets, 0.7 for three bolts/rivets and 0.8 for four or more bolts/rivets along the length in the end connection or equivalent weld length

A_n = net area of the total cross section

A_{nc} = net area of the connected leg

A_{go} = gross area of the outstanding leg

t = thickness of the leg



$b_s = w + w_i - t$ $b_s = w$

Fig. 3 Angles with Single Leg Connections

506.1.2.4 *Other sections* - The rupture strength, T_{dn} of the double angles, channels, I sections and other rolled steel sections, connected by one or more elements to an end gusset is also governed by shear lag effects. The design tensile strength of such sections as governed by tearing of net section may also be calculated using equation in Clause **506.1.2.3** where β is calculated based on the shear lag distance, b_s taken from the farthest edge of the outstanding leg to the nearest bolt/rivet/weld line in the connected leg of the cross section.

506.1.3 *Design strength governed by block shear* - The strength as governed by block shear at an end connection of plates and angles is calculated as given in Clause **506.1.3.1**.

506.1.3.1 *Bolted/riveted connections* - The block shear strength, T_{db} of connection shall be taken as the smaller of

$$T_{db} = (A_{vg} f_y / (\sqrt{3}\,\gamma_{m0}) + 0.9\, A_{tn} f_u / \gamma_{m1})$$

or

$$T_{db} = (0.9 A_{vn} f_u / (\sqrt{3}\,\gamma_{m1}) + A_{tg} f_y / \gamma_{m0})$$

where

A_{vg}, A_{vn} = minimum gross and net area in shear along bolt/rivet line parallel to external force, respectively [1-2 and 3-4 as shown in **Fig. 4 (a**) and 1-2 as shown in **Fig. 4 (b)**]

A_{tg}, A_{tn} = minimum gross and net area in tension from the bolt hole to the toe of the angle, end bolt line, perpendicular to the line of force, respectively [2-3 as shown in **Fig. 4 (b)**]

f_u, f_y = ultimate and yield stress of the material, respectively

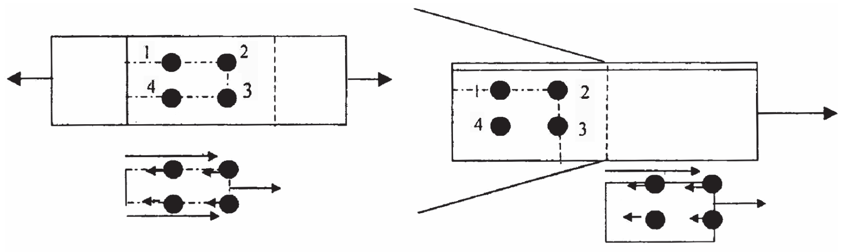


Fig. 4 Block Shear Failure

506.1.3.2 *Welded connection* - The block shear strength, T_{db}, shall be checked for welded end connections by taking an appropriate section in the member around the end weld, which can shear off as a block.

506.2 Design Details

506.2.1 *Slenderness ratio*

For main members the ratio of unsupported length to the least radius of gyration shall not exceed 300.

506.2.2 *Configuration*

Tension members should preferably be of solid cross section. However, when composed of two or more components these shall be connected as described in Clause **506.2.6, 506.2.7** and **506.2.8**.

506.2.3 *Effective sectional area*

When plates are provided solely for the purposes of lacing or battening, they shall be ignored in computing the radius of gyration of the section.

506.2.4 *Lacing and battening*

The open sides of built-up tension members of channel or beam sections shall be connected by lacing or battening where the length of the outstand towards the open side exceeds 16 times the mean thickness of the outstand for steel conforming to IS 2062 upto Grade E 250 (F_e 410 W) and 14 times the mean thickness of the outstand for steel conforming to IS 2062 Grade E 300 (F_e 440 W) and above.

506.2.5 Lacing and battening shall be designed in accordance with Clause **506.2.7** and **506.2.8** and shall be proportioned to resist all shear forces due to external forces, if any, in the plane of lacing. The shear shall be considered as divided equally among all transverse systems and plating in parallel planes.

506.2.6 *Tension members composed of two components back-to-back*

506.2.6.1 Tension members formed by sections placed back-to-back, either in contact or separated by a distance not exceeding 50 mm shall be connected together in their length at regular intervals by riveting, bolting or welding so spaced that the maximum ratio of slenderness of each element is not greater than that specified for main members in Clause **506.2.1**.

506.2.6.2 Where the components are in contact back-to-back riveting, bolting or welding shall be in accordance with clauses applicable.

506.2.6.3 When the components are separated they shall be connected through solid washers or packings, riveted, bolted or welded.

506.2.7 *Design of lacing*

506.2.7.1 As far as practicable the lacing system shall not be varied throughout the length of the tension member.

506.2.7.2 Lacing bars shall be inclined at an angle of 40º to 70º to the axis of the member when a single intersection system is used and at an angle of 40º to 50º when a double intersection system is used.

506.2.7.3 Except for tie as specified in Clause **506.2.7.7** double intersection lacing systems shall not be combined with members or diaphragms perpendicular to the longitudinal axis of the member, unless all forces resulting from deformation of the member are calculated and provided for in the lacing and its fastenings.

506.2.7.4 Lacing bars shall be so connected that there is no appreciable interruption of the triangulation of the system.

506.2.7.5 The required section of the lacing bar shall be determined in accordance with the design provisions of lacings of compression members given in Clause **507.8**. The slenderness ratio of the lacing shall not exceed 140. For this purpose the effective length shall be taken as follows :

i) In riveted or bolted construction, the length between the inner end rivets or bolts of the lacing bar in single intersection lacing and 0.7 times this length for double intersection lacing effectively connected at intersection.

ii) In welded construction, the distance between the inner ends of effective lengths of welds connecting the bars to the components for single intersection lacing and 0.7 times this length for double intersection lacing effectively connected at intersection.

506.2.7.6 Riveting, bolting or welding of lacing bars to the main members shall be sufficient to transmit the load to the bars. Where welded lacing bars overlap the main components, the amount of lap shall not be less than four times the thickness of the bar or four times the mean thickness of the flange of the component to which the bars are attached, whichever is less. The welding shall be provided along each side of the bar for the full length of the lap and returned along the ends of the bar for a length equal to at least four times the thickness of the bar or width of the bar whichever is less.

Where lacing bars are fitted between main components, they shall be connected to each component by fillet welds on both sides of the bar or by full penetration butt welds.

506.2.7.7 Laced tension members shall be provided with tie plates at the ends of the lacing systems, at points where the lacing systems are interrupted and where the member is connected to another member.

506.2.7.8 The length of end tie plate parallel to the axis of the member shall not be less than the perpendicular distance between the centroids of the main components and the length of the intermediate tie plates shall not be less than 3/4 of this distance.

506.2.7.9 The thickness of all tie plates shall be not less than 1/60 of the distance between the innermost lines of rivets, bolts or welds attaching them to the main components, except when effectively stiffened at the edges, in which case the minimum thickness may be 8 mm; for this purpose the edge stiffeners shall have a slenderness ratio not less than 170.

506.2.7.10 When angles, channels etc. are used instead of end tie plates or are provided where the lacing system is interrupted, these shall be designed by the same method as for battens. The end batten or the intermediate batten and its fastenings shall be capable of carrying the forces for which the lacing has been designed. The slenderness ratio shall not exceed 140.

506.2.8 *Design of battens*

Battened tension members shall comply with the following requirements.

506.2.8.1 The spacing of battens, measured as the distance between the centres of adjacent end pitches of rivets or bolts or, for welded construction, the clear distance between the battens, shall be such that the maximum ratio of slenderness of each element is not greater than that specified for main members in Clause **506.2.1**

506.2.8.2 The effective length of the batten, parallel to the axis of the member, shall be taken as the longitudinal distance between end fastenings.

End battens shall have an effective length of not less than the perpendicular distance between centroids of the main components and the length of the intermediate battens shall have an effective length of not less than one-half of this distance.

506.2.8.3 Batten plates shall have a thickness of not less than 1/60 of the minimum distance between the connecting rivet or bolts groups or welds except where they are stiffened at their edges.

506.2.8.4 Where battens are attached by rivets or bolts, not less than two rivets or bolts shall be used in each connection. Where battens are attached by welds, the length of welds connecting each longitudinal edge of the batten plate to the component shall, in the aggregate, be not less than half the length of the batten plate, and at least one-third of the weld shall be placed at each end of the longitudinal edge. In addition, welding shall be returned along the ends of the plate for a length of atleast four times the thickness of the plate.

Where the tie or batten plates are fitted between main components they shall be connected to each member either by fillet welds on each side of the plate, atleast equal in length to that specified in the preceding paragraph or by full penetration butt weld.

506.2.9 *Splices*

Splices in tension members shall have a sectional area 5 percent more than that required to develop the load in the member and, whenever practicable, the cover material shall be disposed to suit the distribution of stress in the various parts of the cross section of the member. Both surfaces of the parts to be spliced shall be covered wherever possible. Rivets, bolts or welds shall develop the full strength of the cover material as defined above.

507 DESIGN OF COMPRESSION MEMBERS

507.1 **Design Strength**

507.1.1 Common hot rolled and built up steel members, used for carrying axial compression, usually fail by flexural buckling. The buckling strength of these members is affected by residual stresses, initial bow and accidental eccentricities of load. To account for all these factors, the strength of members subjected to axial compression is defined by buckling class a,b,c or d as given in **Table 3 and 4**.

507.1.2 The factored design compression P in the member due to external loads shall satisfy the following requirement:

$$P < P_d$$

where

P_d = design strength of the member as given below :

= $A_e \, f_{cd}$

where

A_e = effective sectional area as defined in Clause **507.3.2**

f_{cd} = design compressive stress obtained as per Clause **507.1.2.1**

507.1.2.1 The design compressive stress, f_{cd} of axially loaded compression members shall be calculated using the following equation:

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}} = \chi f_y / \gamma_{m0} \le f_y / \gamma_{m0}$$

where

ϕ = 0.5 [1 + α (λ - 0.2) + λ^2]

λ = non-dimensional effective slenderness ratio = $\sqrt{f_y / f_{cc}} = \sqrt{f_y \left(KL/r\right)^2 / \pi^2 E}$

f_{cc} = Euler buckling stress = $\pi^2 E / (KL/r)^2$

where

KL/r = Effective slenderness ratio or ratio of effective length KL, to appropriate radius of gyration, r

α = Imperfection factor given in **Table 3**

χ = Stress reduction factor (given in **Table 5**) for different buckling class,

slenderness ratio and yield stress = $\left[1 \Big/ \left[\phi + \left(\phi^2 - \lambda^2\right)^{0.5} \right] \right]$

γ_{m0} = Partial safety factor for material strength

Calculated values of design compressive stress f_{cd} for different buckling classes are given in **Table 6.**

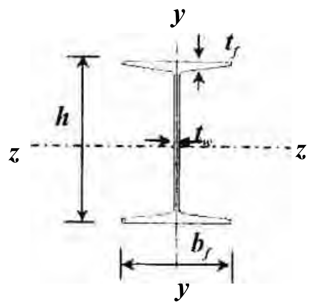
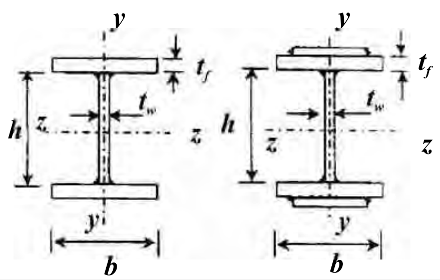
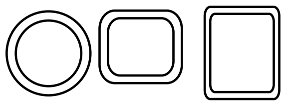
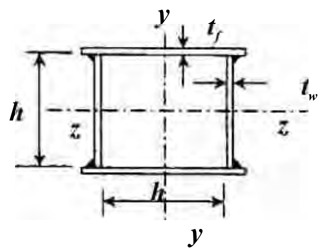
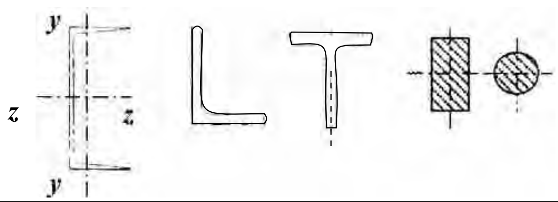
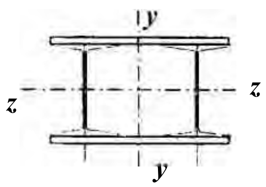
Table 3 Imperfection Factor, α

(Clauses **507.1.1** and **507.1.2.1**)

Buckling Class	**a**	**b**	**c**	**d**
α	0.21	0.34	0.49	0.76

507.1.2.2 The classification of different sections under different buckling class a, b, c or d, is given in **Table 4**. The stress reduction factor χ, and the design compressive stress, f_{cd} for different buckling class, yield stress, and effective slenderness ratio is given in **Table 5** for convenience. The curves corresponding to different buckling class are presented in non-dimensional form, in **Fig. 5**.

Table 4 Buckling Class of Cross-Sections
(Clause **507.1.2.2**)

Cross Section	Limits	Buckling about axis	Buckling Class
Rolled I-Sections 	$h/b_f > 1.2$: $t_f < 40$ mm	z-z y-y	a b
	40 mm$< t_f \leq 100$ mm	z-z y-y	b c
	$h/b_f \leq 1.2$: $t_f \leq 100$ mm	z-z y-y	b c
	$t_f > 100$ mm	z-z y-y	d d
Welded I-Sections 	$t_f \leq 40$ mm	z-z y-y	b c
	$t_f > 40$ mm	z-z y-y	c d
Hollow-Sections 	Hot rolled	Any	a
	Cold formed	Any	b
Welded Box Sections 	Generally (Except as below)	Any	b
	Thick welds and $b/t_f < 30$	z-z	c
	$h/t_w < 30$	y-y	c
Channel, Angle, T and Solid Sections 		Any	c
Built-up Member 		Any	c

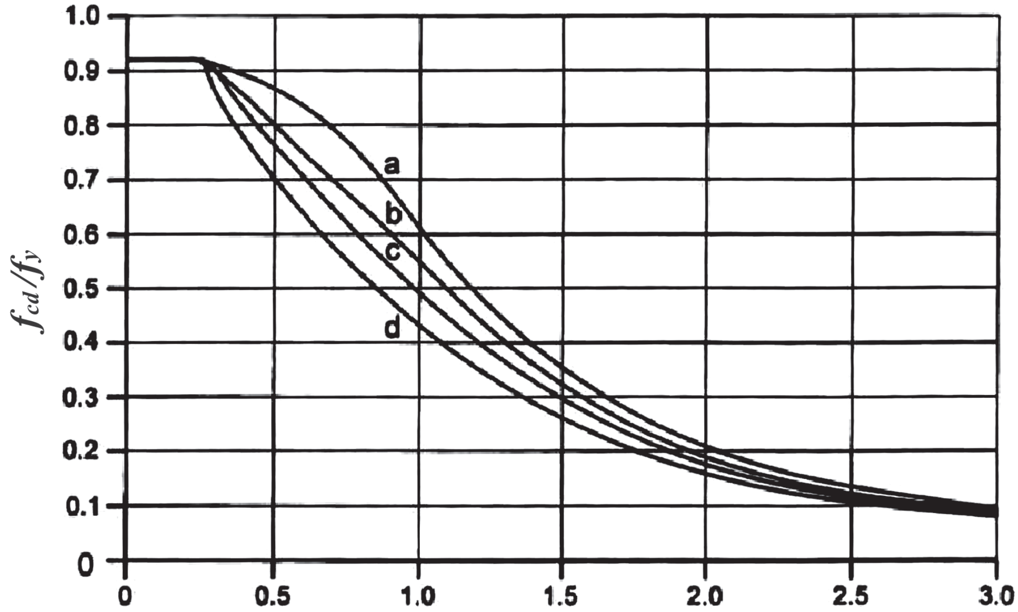


Fig. 5 Buckling Curves for Compression Members

507.2 Effective Length

507.2.1 The effective length, KL is calculated from the member length, L, of the member, considering the rotational and relative translational boundary conditions at the ends. The member length shall be taken as the length from centre to centre of its intersections with the supporting members in the plane of the buckling deformation. In the case of a member with a free end, the free standing length from the centre of the intersecting member at the supported end, shall be taken as the member length.

507.2.2 Where the boundary conditions in the plane of buckling can be assessed, the effective length KL, can be calculated on the basis of **Table 7.**

507.2.3 In the case of bolted, riveted or welded trusses and braced frames, the effective length, KL of the compression members should generally follow Clause **508** under "Design of Trusses or Open-web Girders" of this code. In the case of members of trusses for buckling in the plane perpendicular to the plane of the truss, the effective length, KL shall be taken as the distance between the centres of intersection. The design of angle struts shall be as specified in Clause **507.5**.

507.3 Design Details

507.3.1 Thickness of plate elements - The classification of members on the basis of thickness of constituent plate elements shall satisfy the width-thickness ratio requirements specified in **Table 2**.

Table 5a Stress Reduction Factor, χ, for Buckling Class a
(Clause **507.1.2.1** and **507.1.2.2**)

KL/r ↓	Yield Stress, f_y (MPa)																		
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
20	1.000	0.999	0.998	0.997	0.995	0.994	0.993	0.993	0.990	0.988	0.986	0.984	0.983	0.981	0.979	0.977	0.975	0.972	0.970
30	0.977	0.975	0.974	0.972	0.970	0.969	0.967	0.965	0.961	0.957	0.954	0.951	0.948	0.946	0.943	0.938	0.934	0.930	0.925
40	0.952	0.949	0.947	0.944	0.942	0.939	0.937	0.934	0.926	0.921	0.916	0.911	0.906	0.901	0.896	0.888	0.881	0.873	0.865
50	0.923	0.919	0.915	0.911	0.908	0.904	0.900	0.896	0.884	0.876	0.867	0.859	0.851	0.842	0.834	0.820	0.807	0.794	0.780
60	0.888	0.883	0.877	0.871	0.865	0.859	0.853	0.847	0.828	0.816	0.803	0.790	0.777	0.763	0.750	0.730	0.710	0.690	0.671
70	0.846	0.837	0.829	0.820	0.811	0.803	0.794	0.785	0.758	0.740	0.722	0.703	0.686	0.668	0.651	0.626	0.602	0.579	0.557
80	0.793	0.781	0.769	0.757	0.746	0.734	0.722	0.710	0.675	0.653	0.631	0.610	0.589	0.570	0.551	0.525	0.501	0.478	0.458
90	0.730	0.715	0.700	0.685	0.671	0.657	0.643	0.628	0.590	0.565	0.542	0.520	0.500	0.481	0.463	0.439	0.416	0.396	0.377
100	0.661	0.644	0.627	0.610	0.594	0.579	0.564	0.549	0.510	0.486	0.463	0.443	0.424	0.407	0.390	0.368	0.348	0.331	0.314
110	0.591	0.573	0.555	0.538	0.522	0.507	0.492	0.478	0.440	0.418	0.397	0.379	0.362	0.346	0.332	0.312	0.295	0.279	0.265
120	0.525	0.507	0.489	0.473	0.458	0.443	0.429	0.416	0.381	0.361	0.343	0.326	0.311	0.297	0.284	0.267	0.252	0.238	0.226
130	0.466	0.448	0.432	0.416	0.402	0.388	0.376	0.364	0.332	0.314	0.298	0.283	0.269	0.257	0.246	0.231	0.217	0.206	0.195
140	0.413	0.397	0.382	0.368	0.355	0.342	0.331	0.320	0.291	0.275	0.260	0.247	0.235	0.224	0.214	0.201	0.189	0.179	0.170
150	0.368	0.353	0.339	0.326	0.314	0.303	0.293	0.283	0.257	0.243	0.229	0.218	0.207	0.197	0.189	0.177	0.166	0.157	0.149
160	0.329	0.316	0.303	0.291	0.280	0.270	0.261	0.252	0.229	0.215	0.204	0.193	0.184	0.175	0.167	0.157	0.147	0.139	0.132
170	0.296	0.283	0.272	0.261	0.251	0.242	0.233	0.225	0.204	0.192	0.182	0.172	0.164	0.156	0.149	0.140	0.131	0.124	0.117
180	0.267	0.255	0.245	0.235	0.226	0.218	0.210	0.203	0.184	0.173	0.163	0.155	0.147	0.140	0.134	0.125	0.118	0.111	0.105
190	0.242	0.231	0.222	0.213	0.205	0.197	0.190	0.183	0.166	0.156	0.147	0.140	0.133	0.126	0.121	0.113	0.106	0.100	0.095
200	0.220	0.210	0.202	0.193	0.186	0.179	0.172	0.166	0.151	0.142	0.134	0.127	0.120	0.115	0.109	0.102	0.096	0.091	0.086
210	0.201	0.192	0.184	0.177	0.170	0.163	0.157	0.152	0.137	0.129	0.122	0.115	0.110	0.104	0.099	0.093	0.087	0.083	0.078
220	0.184	0.176	0.169	0.162	0.155	0.149	0.144	0.139	0.126	0.118	0.111	0.106	0.100	0.095	0.091	0.085	0.080	0.075	0.071
230	0.170	0.162	0.155	0.149	0.143	0.137	0.132	0.128	0.115	0.108	0.102	0.097	0.092	0.088	0.083	0.078	0.073	0.069	0.065
240	0.157	0.149	0.143	0.137	0.132	0.127	0.122	0.118	0.106	0.100	0.094	0.089	0.085	0.081	0.077	0.072	0.068	0.064	0.060
250	0.145	0.138	0.132	0.127	0.122	0.117	0.113	0.109	0.098	0.092	0.087	0.082	0.078	0.074	0.071	0.066	0.062	0.059	0.056

Table 5b Stress Reduction Factor, χ, for Buckling Class b
(Clause **507.1.2.1** and **507.1.2.2**)

	Yield stress, f_y, (MPa)																		
KL/r ↓	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
20	1.000	0.998	0.996	0.994	0.993	0.991	0.990	0.986	0.983	0.981	0.978	0.975	0.972	0.970	0.967	0.963	0.960	0.956	0.953
30	0.963	0.961	0.958	0.955	0.953	0.950	0.948	0.943	0.938	0.933	0.929	0.924	0.920	0.915	0.911	0.904	0.898	0.892	0.886
40	0.925	0.921	0.917	0.913	0.909	0.906	0.902	0.895	0.887	0.880	0.873	0.866	0.859	0.852	0.845	0.835	0.825	0.815	0.805
50	0.883	0.877	0.872	0.866	0.861	0.855	0.850	0.839	0.829	0.818	0.808	0.798	0.787	0.777	0.767	0.752	0.737	0.722	0.708
60	0.835	0.827	0.820	0.812	0.805	0.798	0.790	0.775	0.761	0.746	0.732	0.718	0.704	0.691	0.677	0.657	0.638	0.620	0.602
70	0.781	0.771	0.761	0.751	0.742	0.732	0.722	0.703	0.685	0.667	0.649	0.632	0.615	0.599	0.584	0.561	0.540	0.520	0.502
80	0.721	0.709	0.697	0.685	0.673	0.661	0.650	0.627	0.606	0.585	0.566	0.547	0.529	0.512	0.496	0.474	0.453	0.434	0.416
90	0.657	0.643	0.629	0.615	0.602	0.589	0.576	0.552	0.530	0.508	0.488	0.470	0.452	0.436	0.421	0.400	0.380	0.363	0.346
100	0.593	0.577	0.562	0.548	0.534	0.520	0.507	0.483	0.461	0.440	0.421	0.403	0.387	0.372	0.358	0.339	0.321	0.306	0.291
110	0.531	0.515	0.500	0.485	0.471	0.458	0.445	0.422	0.401	0.381	0.364	0.348	0.333	0.319	0.306	0.289	0.274	0.260	0.247
120	0.474	0.458	0.443	0.429	0.416	0.403	0.391	0.370	0.350	0.332	0.316	0.301	0.288	0.276	0.265	0.249	0.236	0.223	0.212
130	0.423	0.408	0.394	0.380	0.368	0.356	0.345	0.325	0.307	0.291	0.276	0.263	0.251	0.240	0.230	0.217	0.204	0.194	0.184
140	0.378	0.364	0.350	0.338	0.327	0.316	0.306	0.287	0.271	0.256	0.243	0.231	0.221	0.211	0.202	0.190	0.179	0.169	0.161
150	0.339	0.325	0.313	0.302	0.291	0.281	0.272	0.255	0.241	0.227	0.215	0.205	0.195	0.186	0.178	0.167	0.158	0.149	0.142
160	0.305	0.292	0.281	0.271	0.261	0.252	0.243	0.228	0.215	0.203	0.192	0.182	0.174	0.166	0.158	0.149	0.140	0.133	0.126
170	0.275	0.264	0.253	0.244	0.235	0.227	0.219	0.205	0.193	0.182	0.172	0.163	0.155	0.148	0.142	0.133	0.125	0.118	0.112
180	0.249	0.239	0.229	0.220	0.212	0.205	0.198	0.185	0.174	0.164	0.155	0.147	0.140	0.133	0.128	0.120	0.113	0.106	0.101
190	0.227	0.217	0.208	0.200	0.193	0.186	0.179	0.168	0.157	0.148	0.140	0.133	0.127	0.121	0.115	0.108	0.102	0.096	0.091
200	0.207	0.198	0.190	0.183	0.176	0.169	0.163	0.153	0.143	0.135	0.128	0.121	0.115	0.110	0.105	0.098	0.092	0.087	0.083
210	0.190	0.182	0.174	0.167	0.161	0.155	0.149	0.140	0.131	0.123	0.117	0.110	0.105	0.100	0.096	0.090	0.084	0.080	0.075
220	0.174	0.167	0.160	0.154	0.148	0.142	0.137	0.128	0.120	0.113	0.107	0.101	0.096	0.092	0.088	0.082	0.077	0.073	0.069
230	0.161	0.154	0.147	0.141	0.136	0.131	0.126	0.118	0.111	0.104	0.098	0.093	0.088	0.084	0.080	0.075	0.071	0.067	0.063
240	0.149	0.142	0.136	0.131	0.126	0.121	0.117	0.109	0.102	0.096	0.091	0.086	0.082	0.078	0.074	0.070	0.065	0.062	0.058
250	0.138	0.132	0.126	0.121	0.117	0.112	0.108	0.101	0.095	0.089	0.084	0.080	0.076	0.072	0.069	0.064	0.060	0.057	0.054

Table 5c Stress Reduction Factor, χ, for Buckling Class c
(Clause **507.1.2.1** and **507.1.2.2**)

	Yield stress, f_y, (MPa)																		
KL/r ↓	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
20	0.999	0.997	0.994	0.992	0.990	0.987	0.985	0.981	0.976	0.972	0.968	0.964	0.961	0.957	0.953	0.948	0.943	0.938	0.933
30	0.948	0.944	0.941	0.937	0.933	0.930	0.926	0.920	0.913	0.907	0.901	0.895	0.889	0.883	0.877	0.869	0.861	0.853	0.845
40	0.896	0.891	0.885	0.880	0.875	0.870	0.866	0.856	0.847	0.838	0.829	0.820	0.812	0.803	0.795	0.783	0.771	0.760	0.748
50	0.841	0.834	0.827	0.821	0.814	0.807	0.801	0.788	0.776	0.763	0.752	0.740	0.729	0.717	0.706	0.690	0.675	0.660	0.645
60	0.783	0.774	0.765	0.757	0.748	0.740	0.732	0.716	0.700	0.685	0.670	0.656	0.642	0.628	0.615	0.596	0.578	0.561	0.544
70	0.722	0.711	0.700	0.690	0.680	0.670	0.660	0.641	0.623	0.605	0.588	0.572	0.557	0.542	0.528	0.508	0.489	0.471	0.454
80	0.659	0.646	0.634	0.622	0.611	0.600	0.589	0.568	0.548	0.529	0.512	0.495	0.479	0.464	0.450	0.430	0.412	0.395	0.379
90	0.596	0.583	0.569	0.557	0.544	0.533	0.521	0.499	0.479	0.460	0.443	0.426	0.411	0.397	0.383	0.365	0.348	0.332	0.318
100	0.536	0.522	0.508	0.495	0.483	0.471	0.459	0.438	0.418	0.400	0.384	0.368	0.354	0.341	0.328	0.311	0.296	0.282	0.269
110	0.480	0.466	0.453	0.440	0.428	0.416	0.405	0.385	0.366	0.349	0.333	0.319	0.306	0.294	0.283	0.268	0.254	0.242	0.230
120	0.430	0.416	0.403	0.391	0.379	0.368	0.358	0.339	0.321	0.306	0.291	0.278	0.267	0.256	0.246	0.232	0.220	0.209	0.199
130	0.385	0.372	0.360	0.348	0.337	0.327	0.317	0.299	0.283	0.269	0.256	0.244	0.234	0.224	0.215	0.203	0.192	0.182	0.173
140	0.346	0.333	0.322	0.311	0.301	0.291	0.282	0.266	0.251	0.238	0.227	0.216	0.206	0.197	0.189	0.178	0.168	0.160	0.152
150	0.311	0.300	0.289	0.279	0.269	0.261	0.252	0.237	0.224	0.212	0.202	0.192	0.183	0.175	0.168	0.158	0.149	0.141	0.134
160	0.281	0.270	0.260	0.251	0.242	0.234	0.227	0.213	0.201	0.190	0.180	0.172	0.164	0.156	0.150	0.141	0.133	0.126	0.120
170	0.255	0.245	0.236	0.227	0.219	0.212	0.205	0.192	0.181	0.171	0.162	0.154	0.147	0.140	0.134	0.126	0.119	0.113	0.107
180	0.232	0.223	0.214	0.206	0.199	0.192	0.186	0.174	0.164	0.155	0.147	0.139	0.133	0.127	0.121	0.114	0.107	0.102	0.096
190	0.212	0.203	0.195	0.188	0.181	0.175	0.169	0.158	0.149	0.140	0.133	0.126	0.120	0.115	0.110	0.103	0.097	0.092	0.087
200	0.194	0.186	0.179	0.172	0.166	0.160	0.154	0.144	0.136	0.128	0.121	0.115	0.110	0.105	0.100	0.094	0.089	0.084	0.079
210	0.178	0.171	0.164	0.158	0.152	0.146	0.141	0.132	0.124	0.117	0.111	0.105	0.100	0.096	0.092	0.086	0.081	0.076	0.072
220	0.164	0.157	0.151	0.145	0.140	0.135	0.130	0.122	0.114	0.108	0.102	0.097	0.092	0.088	0.084	0.079	0.074	0.070	0.066
230	0.152	0.145	0.140	0.134	0.129	0.124	0.120	0.112	0.105	0.099	0.094	0.089	0.085	0.081	0.077	0.073	0.068	0.065	0.061
240	0.141	0.135	0.129	0.124	0.120	0.115	0.111	0.104	0.098	0.092	0.087	0.082	0.078	0.075	0.071	0.067	0.063	0.060	0.056
250	0.131	0.125	0.120	0.115	0.111	0.107	0.103	0.096	0.090	0.085	0.081	0.076	0.073	0.069	0.066	0.062	0.058	0.055	0.052

Table 5d Stress Reduction Factor, χ, For Buckling Class d
(Clause **507.1.2.1** and **507.1.2.2**)

	Yield stress, f_y, (MPa)																		
KL/r ↓	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
20	0.999	0.995	0.991	0.988	0.984	0.980	0.977	0.970	0.964	0.958	0.952	0.946	0.940	0.935	0.930	0.922	0.915	0.908	0.901
30	0.922	0.916	0.911	0.906	0.901	0.896	0.891	0.881	0.872	0.863	0.855	0.847	0.839	0.831	0.823	0.813	0.802	0.792	0.782
40	0.848	0.841	0.834	0.828	0.821	0.815	0.808	0.796	0.784	0.773	0.762	0.751	0.741	0.731	0.721	0.707	0.694	0.681	0.668
50	0.777	0.768	0.760	0.752	0.744	0.736	0.728	0.713	0.699	0.685	0.672	0.659	0.647	0.635	0.624	0.608	0.592	0.577	0.563
60	0.707	0.697	0.687	0.678	0.668	0.659	0.651	0.634	0.617	0.602	0.587	0.573	0.560	0.547	0.535	0.517	0.501	0.486	0.471
70	0.640	0.629	0.618	0.607	0.597	0.587	0.578	0.559	0.542	0.526	0.510	0.496	0.482	0.469	0.456	0.439	0.423	0.408	0.394
80	0.576	0.564	0.553	0.542	0.531	0.521	0.511	0.492	0.474	0.458	0.442	0.428	0.414	0.402	0.390	0.373	0.358	0.344	0.330
90	0.517	0.505	0.493	0.482	0.471	0.461	0.451	0.432	0.415	0.399	0.384	0.370	0.357	0.345	0.334	0.319	0.304	0.292	0.280
100	0.464	0.451	0.440	0.428	0.418	0.408	0.398	0.380	0.363	0.348	0.334	0.321	0.309	0.298	0.288	0.274	0.261	0.249	0.239
110	0.416	0.404	0.392	0.381	0.371	0.361	0.352	0.335	0.319	0.305	0.292	0.281	0.270	0.259	0.250	0.237	0.226	0.215	0.206
120	0.373	0.361	0.350	0.340	0.330	0.321	0.313	0.297	0.282	0.269	0.257	0.246	0.236	0.227	0.219	0.207	0.197	0.187	0.179
130	0.336	0.325	0.314	0.305	0.295	0.287	0.279	0.264	0.251	0.239	0.228	0.218	0.209	0.200	0.193	0.182	0.173	0.164	0.157
140	0.303	0.292	0.283	0.274	0.265	0.257	0.250	0.236	0.224	0.213	0.203	0.194	0.185	0.178	0.171	0.161	0.153	0.145	0.138
150	0.274	0.264	0.255	0.247	0.239	0.231	0.224	0.212	0.201	0.190	0.181	0.173	0.165	0.159	0.152	0.144	0.136	0.129	0.123
160	0.249	0.240	0.231	0.223	0.216	0.209	0.203	0.191	0.181	0.171	0.163	0.155	0.149	0.142	0.137	0.129	0.122	0.116	0.110
170	0.227	0.218	0.210	0.203	0.196	0.190	0.184	0.173	0.164	0.155	0.147	0.140	0.134	0.128	0.123	0.116	0.110	0.104	0.099
180	0.207	0.199	0.192	0.185	0.179	0.173	0.167	0.157	0.149	0.141	0.134	0.127	0.122	0.116	0.111	0.105	0.099	0.094	0.089
190	0.190	0.183	0.176	0.169	0.164	0.158	0.153	0.144	0.136	0.128	0.122	0.116	0.111	0.106	0.101	0.095	0.090	0.085	0.081
200	0.175	0.168	0.162	0.156	0.150	0.145	0.140	0.132	0.124	0.118	0.112	0.106	0.101	0.097	0.093	0.087	0.082	0.078	0.074
210	0.161	0.155	0.149	0.143	0.138	0.134	0.129	0.121	0.114	0.108	0.102	0.097	0.093	0.089	0.085	0.080	0.075	0.071	0.068
220	0.149	0.143	0.138	0.133	0.128	0.123	0.119	0.112	0.105	0.100	0.094	0.090	0.086	0.082	0.078	0.074	0.069	0.066	0.062
230	0.138	0.133	0.128	0.123	0.118	0.114	0.110	0.104	0.097	0.092	0.087	0.083	0.079	0.075	0.072	0.068	0.064	0.061	0.058
240	0.129	0.123	0.119	0.114	0.110	0.106	0.103	0.096	0.090	0.085	0.081	0.077	0.073	0.070	0.067	0.063	0.059	0.056	0.053
250	0.120	0.115	0.110	0.106	0.102	0.099	0.095	0.089	0.084	0.079	0.075	0.071	0.068	0.065	0.062	0.058	0.055	0.052	0.049

Table 6a Design Compressive Stress, f_{cd} (MPa), For Buckling Class a
(Clause **507.1.2.1**)

	Yield stress, f_y, (MPa)																		
KL/r ↓	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	182	191	200	213	218	227	236	255	273	291	309	327	345	364	382	409	436	464	491
20	182	191	200	208	217	226	235	252	270	287	305	322	339	357	374	400	425	451	476
30	178	186	195	203	212	220	229	245	262	279	295	311	328	344	360	384	408	431	454
40	173	181	189	197	205	213	221	237	253	268	283	298	313	328	342	363	384	405	425
50	168	176	183	191	198	205	213	227	241	255	268	281	294	306	318	336	352	368	383
60	162	169	175	182	189	195	202	214	226	237	248	258	268	278	286	299	310	320	329
70	154	160	166	171	177	182	188	197	207	215	223	230	237	243	249	256	263	268	274
80	144	149	154	158	163	167	171	178	184	190	195	199	204	207	210	215	219	222	225
90	133	137	140	143	146	149	152	157	161	164	168	170	173	175	177	179	182	184	185
100	120	123	125	128	130	132	133	136	139	141	143	145	146	148	149	151	152	153	154
110	107	109	111	112	114	115	116	118	120	121	123	124	125	126	127	128	129	129	130
120	95.5	96.7	97.9	98.9	100	101	101	103	104	105	106	107	107	108	109	109	110	110	111
130	84.6	85.5	86.3	87	87.7	88.3	88.8	89.8	90.6	91.3	92.0	92.5	93.0	93.5	93.9	94.4	94.9	95.3	95.7
140	75.2	75.8	76.4	76.9	77.4	77.8	78.2	78.9	79.5	80.0	80.5	80.9	81.3	81.6	81.9	82.3	82.6	83.0	83.2
150	67.0	67.4	67.9	68.2	68.6	68.9	69.2	69.7	70.2	70.6	70.9	71.2	71.5	71.8	72.0	72.3	72.6	72.9	73.1
160	59.9	60.3	60.6	60.9	61.1	61.4	61.6	62.0	62.4	62.7	62.9	63.2	63.4	63.6	63.8	64.0	64.3	64.5	64.6
170	53.8	54.1	54.3	54.6	54.8	55.0	55.1	55.5	55.7	56.0	56.2	56.4	56.6	56.7	56.9	57.1	57.3	57.4	57.6
180	48.6	48.8	49.0	49.2	49.3	49.5	49.6	49.9	50.1	50.3	50.5	50.6	50.8	50.9	51.0	51.2	51.3	51.5	51.6
190	44.0	44.2	44.3	44.5	44.6	44.7	44.9	45.1	45.3	45.4	45.6	45.7	45.8	45.9	46.0	46.2	46.3	46.4	46.5
200	40.0	40.2	40.3	40.4	40.5	40.7	40.7	40.9	41.1	41.2	41.3	41.4	41.5	41.6	41.7	41.8	41.9	42.0	42.1
210	36.6	36.7	36.8	36.9	37.0	37.1	37.2	37.3	37.4	37.6	37.7	37.8	37.8	37.9	38.0	38.1	38.2	38.3	38.3
220	33.5	33.6	33.7	33.8	33.9	34.0	34.0	34.2	34.3	34.4	34.5	34.5	34.6	34.7	34.7	34.8	34.9	35.0	35.0
230	30.8	30.9	31.0	31.1	31.2	31.2	31.3	31.4	31.5	31.6	31.6	31.7	31.8	31.8	31.9	31.9	32.0	32.1	32.1
240	28.5	28.5	28.6	28.7	28.7	28.8	28.8	28.9	29.0	29.1	29.1	29.2	29.3	29.3	29.4	29.4	29.5	29.5	29.6
250	26.3	26.4	26.5	26.5	26.6	26.6	26.7	26.7	26.8	26.9	26.9	27.0	27.0	27.1	27.1	27.2	27.2	27.3	27.3

Table 6b Design Compressive Stress, f_{cd} (MPa), For Buckling Class b
(Clause **507.1.2.1**)

	Yield stress, f_y, (MPa)																		
***KL*/r** ↓	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	182	191	200	209	218	227	236	255	273	291	309	327	345	364	382	409	436	464	491
20	182	190	199	208	217	225	234	251	268	285	302	319	336	353	369	394	419	443	468
30	175	183	192	200	208	216	224	240	256	271	287	302	318	333	348	370	392	414	435
40	168	176	183	191	198	206	213	228	242	256	270	283	297	310	323	342	360	378	395
50	161	167	174	181	188	194	201	214	226	238	250	261	272	283	293	308	322	335	347
60	152	158	164	170	176	181	187	197	207	217	226	235	243	251	259	269	279	287	295
70	142	147	152	157	162	166	171	179	187	194	201	207	213	218	223	230	236	241	246
80	131	135	139	143	147	150	154	160	165	170	175	179	183	186	190	194	198	201	204
90	120	123	126	129	131	134	136	141	144	148	151	154	156	159	161	163	166	168	170
100	108	110	112	114	116	118	120	123	126	128	130	132	134	135	137	139	140	142	143
110	96.5	98.3	100	101	103	104	105	107	109	111	112	114	115	116	117	118	119	121	121
120	86.2	87.5	88.6	89.7	90.7	91.7	92.5	94.1	95.4	96.6	97.7	98.6	100	100	101	102	103	104	104
130	76.9	77.8	78.7	79.5	80.3	81.0	81.6	82.7	83.7	84.6	85.4	86.1	86.8	87.3	87.9	88.6	89.2	89.8	90.3
140	68.7	69.4	70.1	70.7	71.3	71.8	72.3	73.1	73.9	74.6	75.2	75.7	76.2	76.6	77.1	77.6	78.1	78.5	78.9
150	61.6	62.1	62.6	63.1	63.6	64.0	64.3	65.0	65.6	66.1	66.6	67.0	67.4	67.7	68.1	68.5	68.9	69.2	69.5
160	55.4	55.8	56.2	56.6	56.9	57.3	57.5	58.1	58.5	59.0	59.3	59.7	60.0	60.3	60.5	60.9	61.2	61.5	61.7
170	50.0	50.3	50.7	51.0	51.2	51.5	51.7	52.2	52.5	52.9	53.2	53.5	53.7	53.9	54.1	54.4	54.7	54.9	55.1
180	45.3	45.6	45.9	46.1	46.3	46.5	46.7	47.1	47.4	47.7	47.9	48.1	48.3	48.5	48.7	48.9	49.2	49.3	49.5
190	41.2	41.5	41.7	41.9	42.1	42.2	42.4	42.7	42.9	43.2	43.4	43.6	43.7	43.9	44.0	44.2	44.4	44.6	44.7
200	37.6	37.8	38.0	38.2	38.3	38.5	38.6	38.9	39.1	39.3	39.5	39.6	39.8	39.9	40.0	40.2	40.3	40.5	40.6
210	34.5	34.7	34.8	35.0	35.1	35.2	35.3	35.5	35.7	35.9	36.0	36.2	36.3	36.4	36.5	36.6	36.8	36.9	37.0
220	31.7	31.9	32.0	32.1	32.2	32.3	32.4	32.6	32.8	32.9	33.0	33.1	33.2	33.3	33.4	33.6	33.7	33.8	33.9
230	29.2	29.4	29.5	29.6	29.7	29.8	29.9	30.0	30.1	30.3	30.4	30.5	30.6	30.7	30.7	30.8	30.9	31.0	31.1
240	27.1	27.2	27.3	27.3	27.4	27.5	27.6	27.7	27.8	27.9	28.0	28.1	28.2	28.3	28.3	28.4	28.5	28.6	28.7
250	25.1	25.2	25.3	25.3	25.4	25.5	25.6	25.7	25.8	25.9	26.0	26.0	26.1	26.2	26.2	26.3	26.4	26.5	26.5

Table 6c Design Compressive Stress, f_{cd} (MPa), For Buckling Class c
(Clause **507.1.2.1**)

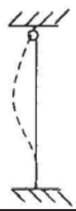
	Yield stress, f_y, (MPa)																		
KL/r ↓	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	182	191	200	209	218	227	236	255	273	291	309	327	345	364	382	409	436	464	491
20	182	190	199	207	216	224	233	250	266	283	299	316	332	348	364	388	412	435	458
30	172	180	188	196	204	211	219	234	249	264	278	293	307	321	335	355	376	395	415
40	163	170	177	184	191	198	205	218	231	244	256	268	280	292	304	320	337	352	367
50	153	159	165	172	178	183	189	201	212	222	232	242	252	261	270	282	295	306	317
60	142	148	153	158	163	168	173	182	191	199	207	215	222	228	235	244	252	260	267
70	131	136	140	144	148	152	156	163	170	176	182	187	192	197	202	208	213	218	223
80	120	123	127	130	133	136	139	145	149	154	158	162	165	169	172	176	180	183	186
90	108	111	114	116	119	121	123	127	131	134	137	140	142	144	146	149	152	154	156
100	97.5	100	102	104	105	107	109	112	114	116	119	120	122	124	125	127	129	131	132
110	87.3	89.0	90.5	92.0	93.3	94.6	95.7	97.9	100	102	103	104	106	107	108	110	111	112	113
120	78.2	79.4	80.6	81.7	82.7	83.7	84.6	86.2	87.6	88.9	90.1	91.1	92.1	93.0	93.8	94.9	95.9	96.8	97.6
130	70.0	71.0	71.9	72.8	73.5	74.3	75.0	76.2	77.3	78.3	79.2	80.0	80.7	81.4	82.0	82.9	83.6	84.3	84.9
140	62.9	63.6	64.4	65.0	65.6	66.2	66.7	67.7	68.6	69.3	70.0	70.7	71.2	71.8	72.3	72.9	73.5	74.1	74.6
150	56.6	57.2	57.8	58.3	58.8	59.2	59.7	60.4	61.1	61.7	62.3	62.8	63.3	63.7	64.1	64.6	65.1	65.5	65.9
160	51.1	51.6	52.1	52.5	52.9	53.3	53.6	54.2	54.8	55.3	55.7	56.1	56.5	56.9	57.2	57.6	58.0	58.4	58.7
170	46.4	46.8	47.1	47.5	47.8	48.1	48.4	48.9	49.3	49.8	50.1	50.5	50.8	51.1	51.3	51.7	52.0	52.3	52.6
180	42.2	42.5	42.8	43.1	43.4	43.6	43.9	44.3	44.7	45.0	45.3	45.6	45.8	46.1	46.3	46.6	46.9	47.1	47.3
190	38.5	38.8	39.0	39.3	39.5	39.7	39.9	40.3	40.6	40.9	41.1	41.4	41.6	41.8	42.0	42.2	42.5	42.7	42.9
200	35.3	35.5	35.7	35.9	36.1	36.3	36.5	36.8	37.0	37.3	37.5	37.7	37.9	38.1	38.2	38.4	38.6	38.8	39.0
210	32.4	32.6	32.8	33.0	33.1	33.3	33.4	33.7	33.9	34.1	34.3	34.5	34.7	34.8	34.9	35.1	35.3	35.4	35.6
220	29.9	30.1	30.2	30.4	30.5	30.6	30.8	31.0	31.2	31.4	31.5	31.7	31.8	31.9	32.1	32.2	32.4	32.5	32.6
230	27.6	27.8	27.9	28.0	28.2	28.3	28.4	28.6	28.8	28.9	29.1	29.2	29.3	29.4	29.5	29.7	29.8	29.9	30.0
240	25.6	25.7	25.9	26.0	26.1	26.2	26.3	26.4	26.6	26.7	26.9	27.0	27.1	27.2	27.3	27.4	27.5	27.6	27.7
250	23.8	23.9	24.0	24.1	24.2	24.3	24.4	24.5	24.7	24.8	24.9	25.0	25.1	25.2	25.3	25.4	25.5	25.6	25.7

Table 6d Design Compressive Stress, f_{cd} (MPa), For Buckling Class d
(Clause **507.1.2.1**)

	Yield stress, f_y, (MPa)																		
KL/r ↓	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10	182	191	200	209	218	227	236	255	273	291	309	327	345	364	382	409	436	464	491
20	182	190	198	206	215	223	231	247	263	279	294	310	325	340	355	377	399	421	442
30	168	175	182	189	197	204	211	224	238	251	264	277	290	302	314	332	350	367	384
40	154	161	167	173	179	185	191	203	214	225	235	246	256	266	275	289	303	316	328
50	141	147	152	157	162	167	172	182	191	199	208	216	224	231	238	249	258	268	277
60	129	133	137	142	146	150	154	161	168	175	182	188	193	199	204	212	219	225	231
70	116	120	124	127	130	133	137	142	148	153	158	162	167	171	174	180	184	189	193
80	105	108	111	113	116	118	121	125	129	133	137	140	143	146	149	153	156	159	162
90	94.1	96.4	98.6	101	103	105	107	110	113	116	119	121	123	126	128	130	133	135	137
100	84.3	86.2	87.9	89.6	91.1	92.6	94.0	96.7	99.1	101	103	105	107	108	110	112	114	116	117
110	75.6	77.0	78.4	79.7	81.0	82.1	83.2	85.3	87.1	88.8	90.4	91.8	93.1	94.4	95.5	97.1	98.5	100	101
120	67.8	69.0	70.1	71.1	72.1	73.0	73.9	75.5	77.0	78.3	79.5	80.6	81.7	82.6	83.5	84.7	85.8	86.9	87.8
130	61.0	62.0	62.8	63.7	64.5	65.2	65.9	67.2	68.3	69.4	70.4	71.2	72.1	72.8	73.5	74.5	75.4	76.2	76.9
140	55.0	55.8	56.5	57.2	57.8	58.4	59.0	60.0	61.0	61.8	62.6	63.3	64.0	64.6	65.2	66.0	66.7	67.3	67.9
150	49.8	50.4	51.0	51.6	52.1	52.6	53.1	53.9	54.7	55.4	56.0	56.6	57.2	57.7	58.1	58.8	59.3	59.9	60.4
160	45.2	45.7	46.2	46.7	47.1	47.5	47.9	48.6	49.3	49.9	50.4	50.9	51.3	51.7	52.1	52.7	53.1	53.6	54.0
170	41.2	41.6	42.1	42.4	42.8	43.1	43.5	44.1	44.6	45.1	45.5	45.9	46.3	46.7	47.0	47.4	47.8	48.2	48.6
180	37.7	38.0	38.4	38.7	39.0	39.3	39.6	40.1	40.5	41.0	41.3	41.7	42.0	42.3	42.6	43.0	43.3	43.6	43.9
190	34.5	34.9	35.2	35.4	35.7	35.9	36.2	36.6	37.0	37.4	37.7	38.0	38.2	38.5	38.7	39.1	39.4	39.6	39.9
200	31.8	32.0	32.3	32.5	32.8	33.0	33.2	33.6	33.9	34.2	34.5	34.7	35.0	35.2	35.4	35.7	35.9	36.2	36.4
210	29.3	29.6	29.8	30.0	30.2	30.4	30.5	30.9	31.2	31.4	31.7	31.9	32.1	32.3	32.5	32.7	32.9	33.1	33.3
220	27.1	27.3	27.5	27.7	27.9	28.0	28.2	28.5	28.7	29.0	29.2	29.4	29.6	29.7	29.9	30.1	30.3	30.5	30.6
230	25.2	25.3	25.5	25.7	25.8	26.0	26.1	26.4	26.6	26.8	27.0	27.1	27.3	27.5	27.6	27.8	27.9	28.1	28.2
240	23.4	23.6	23.7	23.9	24.0	24.1	24.2	24.5	24.7	24.8	25.0	25.2	25.3	25.4	25.5	25.7	25.9	26.0	26.1
250	21.8	22.0	22.1	22.2	22.3	22.5	22.6	22.8	22.9	23.1	23.2	23.4	23.5	23.6	23.7	23.9	24.0	24.1	24.2

Table 7 Effective Length of Prismatic Compression Members

(Clause **507.2.2**)

Boundary Conditions				Schematic Representation	Effective Length
At one end		At the other end			
Translation	Rotation	Translation	Rotation		
Restrained	Restrained	Free	Free		2.0*L*
Free	Restrained	Restrained	Free		
Restrained	Free	Restrained	Free		1.0*L*
Restrained	Restrained	Free	Restrained		1.2*L*
Restrained	Restrained	Restrained	Free		0.8*L*
Restrained	Restrained	Restrained	Restrained		0.65*L*

NOTE *: L is the unsupported length of the compression member* ***(7.2.1)***

507.3.2 Effective sectional area A_e - Except for Class 4 (slender section) in Clause **503.7.2** the gross sectional area shall be taken as the effective sectional area for all compression members fabricated by welding, bolting and riveting so long as the section is semi-compact or better. Holes not filled with rivets, bolts or pins shall be deducted from gross area to calculate effective sectional area.

507.3.3 *Eccentricity for columns*

For the purpose of determining the stress in the section of a column, the reactions from the connecting member or similar loads shall be assumed to be applied at an eccentricity of 100 mm from the face of the section or at the centre of bearing whichever dimension gives the greater eccentricity, and with the exception of the following two cases :

a) In the case of cap connection, the load shall be assumed to be applied at the face of the column section or at the edge of packing, if used towards the span of the beam.

b) In the case of a truss bearing on a cap, no eccentricity need to be taken for simple bearings without connections, capable of developing any appreciable moment. In case of web member connection with face, actual eccentricity is to be considered.

507.3.4 *Splices*

507.3.4.1 Where the ends of compression members are prepared for bearing over the whole area, they shall be spliced to hold the connected members accurately in position, and to resist bending or tension, if present. Such splices should maintain the intended member stiffness about each axis. Splices should be located as close to the point of inflection as possible. Otherwise their capacity should be adequate to carry moment (see *Clause* **510.3.2.2**). The ends of compression members faced for bearing shall invariably be machined to ensure perfect contact of surfaces in bearing.

507.3.4.2 Where such members are not faced for complete bearing, the splices shall be designed to transmit all the forces to which the members are subjected.

507.3.4.3 Wherever possible, splices shall be proportioned and arranged so that the centroidal axis of the splice coincides as nearly as possible with the centroidal axes of the members being jointed, in order to avoid eccentricity: but where eccentricity is present in the joint, the resulting stress shall be accounted for.

Wherever possible both surfaces of the parts spliced shall be covered or other means taken to maintain the alignment of the abutting ends.

507.3.4.4 Splices in compression members located at or near effectively braced panel points shall be designed to transmit design strength of the member. All other splices in compression members shall have a sectional area 5 percent more than that required to develop the design strength in the member. All cover materials shall, as far as practicable, be so disposed with respect to the cross-section of the member so as to transmit the proportional load of the respective parts of the section. Rivets, bolts or welds shall develop the full strength in the cover material as defined above.

507.3.4.5 Where flexural tension may occur in the member, the cover material shall be designed to resist such tension.

507.4 Base Plates

507.4.1 *General*

507.4.1.1 Base plates of compression members should have sufficient stiffness and strength to transmit axial force, bending moments and shear forces at the base of the compression members to their foundation without exceeding the load carrying capacity of the supports. Anchor bolts and shear keys should be provided wherever necessary. Shear resistance at the proper contact surface between steel base and concrete/grout and bearing pressure between the plate and support shall be determined and computed as per provisions of IRC:78.

507.4.1.2 If the size of the base plate is larger than that required to limit the bearing pressure on the base support, an equal projection, c, of the base plate beyond the face of the compression member and gusset may be taken as effective in transferring the column load as given in **Fig. 6**, such that beam pressure at the effective area does not exceed bearing capacity of concrete base.

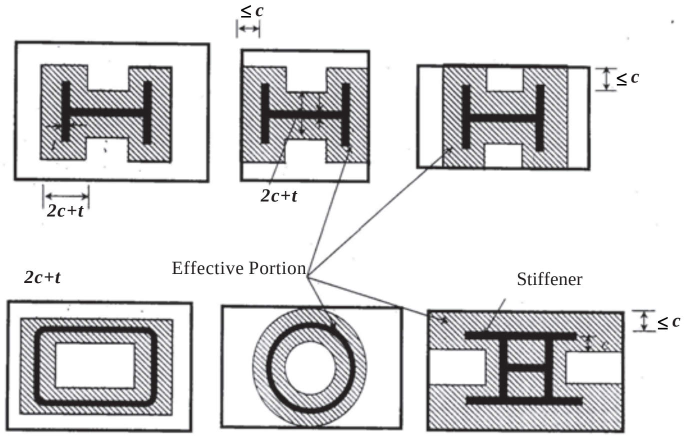


Fig. 6 Effective Area of Base Plate

507.4.2 *Gusseted bases*

In gusseted bases, the gusset plates, angle cleats, stiffeners, fastenings, etc. in combination with the bearing area of the shaft, shall be sufficient to take the loads, bending moments and reactions to the base plate without exceeding specified strength. All the bearing surfaces should be machined to ensure perfect contact.

507.4.2.1 Where the ends of the compression member and the gusset plates are not faced for complete bearing, the welding, fastenings connecting them to the base plate shall be sufficient to transmit all the forces to which the base is subjected.

507.4.2.2 Compression member and base plate connections-Where the end of the compression member is connected directly to the base plate by means of full penetration butt welds, the connection shall be deemed to transmit to the base all the forces and moments to which the compression member is subjected.

507.4.3 *Slab bases*

Compression members with slab bases need not be provided with gussets, but sufficient fastenings shall be provided to retain the parts securely in place and to resist all moments and forces, other than direct compression, including those arising during transit, unloading and erection.

507.4.3.1 The minimum thickness, t_s of rectangular slab bases, supporting members under axial compression shall be

$$t_s = \sqrt{2.5w\left(a^2 - 0.3b^2\right)\gamma_{mo} / f_y} > t_f$$

where

w = uniform pressure from below on the slab base under the factored load axial compression

a,b = larger and smaller projection of the slab base beyond the rectangle circumscribing the compression member respectively

t_f = flange thickness of compression member

When only the effective area of the base plate is used as in Clause **507.4.1.1**, c^2 may be used in the above equation (See **Fig. 6**) instead of $(a^2 - 0.3b^2)$.

507.4.3.2 When the slab base does not distribute the load uniformly, due to eccentricity of the load etc., special calculation shall be made to show that the base is adequate to resist the moment due to the non-uniform pressure from below.

507.4.3.3 Bases for bearing upon concrete or masonry need not be machined on the underside.

507.4.3.4 In cases where the cap or base is fillet welded directly to the end of the compression member without boring and shouldering, the contact surfaces shall be machined to give a perfect bearing and the welding shall be sufficient to transmit the forces as required in Clause **507.4.3**. Where full strength but welds are provided, machining of contact surfaces is not required.

507.5 Angle Struts

507.5.1 *Single angle struts* - The compression in single angles may be transferred either concentrically to its centroid through end gusset or eccentrically by connecting one of its legs to a gusset or adjacent member.

507.5.1.1 *Concentric loading* - When a single angle is concentrically loaded in compression the design strength may be evaluated using Clause **507.1.2**.

507.5.1.2 *Loaded through one leg* - The flexural torsional buckling strength of single angle loaded in compression through one of its legs may be evaluated using the equivalent slenderness ratio, λ_e, as given below:

$$\lambda_e = \sqrt{k_1 + k_2 \boldsymbol{\lambda}_{vv}^{\ 2} + k_3 \boldsymbol{\lambda}_{\phi}^{\ 2}}$$

where

k_1, k_2, k_3 = constants depending upon the end condition, as given in **Table 8**

$$\lambda_{vv} = \frac{(l/r_{vv})}{\varepsilon\sqrt{\pi^2 \varepsilon / 250}} \quad \text{and} \quad \lambda_\phi = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\pi^2 \varepsilon / 250}}$$

where

l = centre-centre length of the supporting member

r_{vv} = radius of gyration about the minor axis

b_1, b_2 = width of the two legs of the angle

t = thickness of the leg

ε = yield stress ratio $(250/f_y)^{0.5}$

Table 8 Constants k_1, k_2 and k_3
(Clause **507.5.1.2**)

No. of bolts at each end connection	Gusset/Connecting member Fixity *	k_1	k_2	k_3
≥ 2	Fixed	0.20	0.35	20
	Hinged	0.70	0.60	5
1	Fixed	0.75	0.35	20
	Hinged	1.25	0.50	60

* *Stiffness of in-plane rotational restraint provided by the gusset/connecting member. For partial restraint, the λ_e can be interpolated between the λ_e results for fixed and hinged cases.*

507.5.2 *Double angle struts*

507.5.2.1 For double angle discontinuous struts, connected back to back, on opposite sides of the gusset or a section, by not less than two bolts or rivets in line along the angles at each end, or by the equivalent in welding, the load may be regarded as applied axially. The effective length, KL, in the plane of end gusset shall be taken as between 0.7 and 0.85 times the distance between intersections, depending on the degree of the restraints provided. The effective length, KL, in the plane perpendicular to that of the end gusset, shall be taken as equal to the distance between centres of intersections. The calculated average compressive stress shall not exceed the values based on Clause **507.1.2**. The angles shall be connected together over their lengths so as to satisfy the requirements of Clauses **507.6** and **512**.

507.5.2.2 Double angle discontinuous struts connected back-to-back, to one side of a gusset or section by one or more bolts or rivets in each angle, or by the equivalent in welding, shall be designed in accordance with Clause **507.5.2** and the angles shall be connected together over their lengths so as to satisfy the requirements of Clauses **507.6** and **512**.

507.5.3 *Continuous members:* Double angle continuous struts such as those forming the chords or ties of trusses or trussed girders, or the legs of towers shall be designed as axially loaded compression members, and the effective length shall be taken in accordance with Clause **508**.

507.5.4 *Combined stresses:* In addition to axial loads, if the struts carry loads, which cause transverse bending, the combined bending and axial stresses shall be checked in accordance with Clause **510** of this code.

507.6 Compression Members Composed of Two Components Back-to-Back

507.6.1 Compression members composed of two angles, channels or tees back to back in contact or separated by a distance not exceeding 50 mm shall be connected together by riveting, bolting or welding, so that maximum ratio of the slenderness of each component of the member between such connections is not greater than 40 or 0.6 times the maximum ratio of slenderness of the member as a whole, whichever is less.

The number of connections shall be such that the member is divided into not less than three approximately equal parts.

507.6.2 Where the members are separated back-to-back the rivets or bolts in these connections shall pass through solid washers or packings, and where the connected angles, legs or tables of tees are 125 mm wide or over or where webs of channels are 150 mm wide or over, not less than two rivets or bolts shall be used in each connection, one on the line of each gauge mark.

507.6.3 Where these connections are made by welding, solid packings shall be used to effect the jointing unless the members are sufficiently close together to permit butt welding, and the members shall be connected by welding along both pairs of edges of the main components.

507.6.4 The rivets, bolts or welds in these connections shall be sufficient to carry the shear forces and the moments specified for battened struts and in no case shall the rivets or bolts be less than 16 mm. dia for members upto and including 10 mm thick; 20 mm dia for members upto and including 16 mm thick; and 22 mm dia for members over 16 mm thick.

507.6.5 Compression members connected by such riveting, bolting or welding shall not be subjected to transverse loading in a plane perpendicular to the riveted, bolted or welded surfaces.

507.6.6 Where components are in contact back-to-back riveting, bolting or intermittent welding shall be done in accordance with applicable clauses.

507.7 Lacing and Battening

507.7.1 The open sides of built-up compression members of channel or beam sections shall be connected by lacing or battening where the length of the outstand towards the open side exceeds 16 times the mean thickness of the outstand for steel conforming to IS 2062, upto Grade E 250 (Fe 410W) and 14 times the mean thickness of the outstand for steel conforming to IS 2062, Grade E300 (Fe 440W) and above.

507.7.2 Lacing and battening plates shall be designed in accordance with Clauses **507.8** and **507.9** and shall be proportioned to resist a total transverse shear force V_t at any point in the length of the member equal to at least 2.5 percent of the axial force in the member together with all shear due to external forces, if any, in the plane of lacing. The shear force V_t shall be considered as divided equally among all transverse system and plating in parallel planes.

507.7.3 Compression members composed of two or more components connected as described in Clauses **507.6, 507.8** and **507.9** may be designed as homogeneous members.

507.8 Design of Lacings

507.8.1 As far as practicable, the lacing system shall not be varied throughout the length of the compression member.

507.8.2 Lacing bars shall be inclined at an angle of 40^0 to 70^0 to the axis of the member where a single intersection system is used, and at an angle of 40^0 to 50^0 where a double intersection system is used.

507.8.3 Except for tie plates as specified in Clause **507.8.8** below, double intersection lacing systems shall not be combined with members of diaphragms perpendicular to the longitudinal axis of the main member, unless all forces resulting from deformation are calculated and provided for in the lacing and its fastenings.

507.8.4 Lacing bars shall be so connected that there in no appreciable interruption of the triangulation of the system.

507.8.5 The maximum spacing of lacing bars whether by welding, riveting or bolting shall be such that the effective slenderness ratio of the components of the compression member between consecutive connections of the lacing bars to one component is not greater than 50 or 0.7 times the maximum ratio of slenderness of the member as a whole whichever is lesser.

507.8.6 The lacing shall be proportioned to resist a total transverse shear, V_t at any point in the member in the manner defined in Clause **507.7.2**.

507.8.7 For members carrying calculated bending stress due to eccentricity of loading, applied end moments and/or lateral loading, the lacing shall be proportioned to resist the actual shear due to bending, in addition to that specified in Clause **507.8.6**.

507.8.8 The effective slenderness ratio KL/r of the lacing bars shall not exceed 140. For this purpose the effective length KL shall be taken as follows :

a) *In riveted or bolted construction* - the length between the inner ends of rivets or bolts of the lacing bar in single intersection lacing and 0.7 times this length for double intersection lacing effectively connected at intersections.

b) *In welded construction* - the distance between the inner ends of effective lengths of welds connecting the bars to the components in single intersection lacings and 0.7 times this length for double intersection lacing effectively connected at intersections.

507.8.9 Lacing bars shall be connected to the main members either by riveting or bolting by one or more rivets or bolts, in line along the lacings or by welding at each end sufficient to transmit the load to the bars. Any eccentricity of the connection with respect to the centroid of the lacing bar may be ignored and the lacing designed as an axially loaded strut. Where welded lacing bars overlap the main component, the amount of lap shall be not less than four times the thickness of the bar or four times the mean thickness of the flange of the component to which the bars are attached, whichever is less . Welding shall be provided at least along each side of the bar for the full length of the lap and returned along the ends of the bar for a length equal to at least four times the thickness of the bar or width of the bar whichever is less.

Where lacing bars are fitted between the main components they shall be connected to each component by fillet welds on both sides of the bar or by full penetration butt welds.

507.8.10 Laced compression member shall be provided with tie plate at the ends of the lacing systems, at points where the lacing systems are interrupted and where the member is connected to another member.

507.8.11 The length of end tie plates measured between end fastenings along the longitudinal axis of the member shall not be less than (a) the perpendicular distance between the lines of rivets connecting them to the flanges or (b) the distance between vertical side plates of the main chords whichever is greater and shall be at least equal to (c) the depth of the cross girders where these are directly attached to the struts and the length of intermediate tie plates shall be not less than three-quarter of (a) above.

507.8.12 The thickness of tie plates shall not be less than 1/50 of the distance between the innermost lines of rivets, bolts or welds except when effectively stiffened at the free edges in which case the minimum thickness may be 8 mm. For this purpose the edge stiffener shall have a slenderness ratio not greater than 170.

507.8.13 Tie plates and their fastenings (calculated in accordance with the method described for battens) shall be capable of carrying the forces for which the lacing system is designed.

507.8.14 When angles, channels, etc, are used instead of end tie plates or are provided where the lacing system is interrupted, these shall be designed by the same method as for battens. The end batten or the intermediate batten and its fastenings shall be capable of carrying the forces for which the lacing has been designed. The effective slenderness ratio shall not exceed 140.

507.9 Design of Battens

Battened compression members shall comply with the following requirements:

507.9.1 The battens shall be placed opposite each other at each end of the member and at points where the member stayed in its length and shall, as far as practicable, be spaced and proportioned uniformly throughout. The number of battens shall be such that the member is divided into not less than three bays within its actual length between centre to centre of connections.

507.9.2 In battened compression members when the effective slenderness ratio about the axis perpendicular to the battens is not more than 0.8 times the ratio about the axis parallel to the battens, the spacing of battens between centre to centre of end fastenings shall be such that the slenderness ratio of the lesser main component over this distance shall not be greater than 50 or 0.7 times the ratio of slenderness of the member as a whole about its axis parallel to the battens. In case it is more than 0.8 times the ratio about the axis parallel to the battens, the spacing of battens between centre to centre of end fastenings shall be such that slenderness ratio of the lesser main component over this distance shall not be greater than 50 or 0.7 times of slenderness ratio of the member as a whole about it weaker axis.

507.9.3 Battens shall be plates, channels or I sections and shall be riveted, bolted or welded to the main components, Battens and their connections shall be so designed that they resist simultaneously a longitudinal shear force equal to $V_t D/na$ and a moment equal to $V_t D/2n$ where

D = the longitudinal distance between centre-to-centre of battens.

a = the minimum transverse distance between the centroids of rivet or bolt groups or welding.

V_t = the transverse shear force as defined in Clause **507.7.2**

n = the number of parallel planes of battens

507.9.4 The effective length of a batten parallel to the axis of a member shall be taken as the longitudinal distance between the end fastenings. End battens shall have an effective length of not less than (a) the perpendicular distance between the lines of rivets connecting them to the components, or (b) the distance between the vertical side plates of the main chords whichever is greater and shall be at least equal to (c) the depth of the cross girders where these are directly attached to the struts; and intermediate battens shall have an effective length of not less than 3/4 of (a) above, but in no case shall the length (of any batten) be less than twice the width of the smaller component in the plane of the battens.

507.9.5 The thickness of batten plates shall not be less than 1/50 of the minimum distance between the innermost lines of connecting rivet, bolts or welds, except when effectively stiffened at the free edges, in which case the minimum thickness may be 8 mm; for the purpose the edge stiffeners shall have a slenderness ratio not greater than 170.

507.9.6 Battened compression members not complying with these requirements, or those subjected to bending moments in the plane of the battens shall be designed according to the theory of elastic stability, or empirically with verification by tests, so that they have a load factor of not less than 1.7.

507.9.7 Battened compression member composed of two angles forming a cruciform cross-section shall conform to the above requirements except as follows :

a) the battens shall be in pairs placed in contact one against the other, unless these are welded to form cruciform battens.

b) a transverse shear force $\frac{V_t}{\sqrt{2}}$ shall be taken as occurring separately about each rectangular axis of the whole member.

c) a longitudinal shear force of $\frac{V_t D}{a\sqrt{2}}$ and the moment $\frac{V_t D}{2\sqrt{2}}$ shall be taken in respect of each batten in each of the two planes, except where the effective slenderness ratio can occur about a rectangular axis, in which case each battens shall be designed to resist a shear force of 2.5 percent of the total axial force.

NOTE : V_t , D and a are as defined in Clause **507.9.3**

508 DESIGN OF TRUSSES OR OPEN-WEB GIRDERS

508.1 General

Trusses or open web girders are defined as triangulated skeletal girders. The design of individual members and connections should be made in accordance with this Clause in conjunction with Clauses **506, 507,** and **512** as appropriate.

508.2 Analysis

For analysis of trusses the following assumption may be made unless rigorous rigid frame analysis is adopted :

a) All members are frictionless pin jointed.

b) All members are straight and free to rotate at the joints.

c) All loads including self weight of members are applied at the joints

Stipulations made in this section are not applicable for design of stiffening trusses of suspension bridges.

508.3 Intersection at Joints

For triangulated trusses designed on the assumption of frictionless pin jointed connections, members meeting at a joint should, where practicable, have their centroidal axes meeting at a point, and wherever practicable the centre of resistance of a connection shall lie on the line of action of the load so as to avoid an eccentricity moment on the connections.

If, at a joint, the centroidal axes of the adjacent members do not meet at a single point, the resulting flexural stresses in the members should be taken into account as secondary stress.

Where loads are not applied at truss joints, account should be taken of the following:

a) resulting stresses where load is applied to a member in the plane of a truss other than at a joint

b) torsion and lateral flexure effects when the applied load is not in the plane of the truss. Where the load is applied to a cross-member, the effect of interaction between the cross-member so loaded and the truss and adjacent cross-member should be taken into account.

508.4 Effective Length of Compression Members

508.4.1 In riveted, bolted or welded trusses the compression members act in a complex manner and the effective lengths *KL* of such members shall be taken as given in **Table 9** for computing their design strengths. For battened compression members, all values given in Table 9 shall be increased by 10 percent.

Table 9 Effective Length of Compression Members

Member		Effective length *KL* of member		
		For buckling in the plane of the truss	For buckling normal to the plane of the truss	
			Compression chord or (compression) member effectively braced by lateral system	Compression chord or (compression) member unbraced
Chords		0.85 x distance between centres of intersection with the web members	0.85 x distance between centres of intersection with the lateral bracing members or rigidly connected cross girders	See Clause **508.4.4**
Web	Single triangulated system	0.70 x distance between centres of intersection with the main chords	0.85 x distance between centres of intersections	Distance between centres of intersection
	Multiple intersection system where adequate connections are provided at all points of intersection	0.85 x greatest distance between centres of any two adjacent intersection	0.70 x distance between centres of intersection with the main chords	0.85 x distance between centres of intersection with the main chords

NOTE : The intersection referred to are those of the centroidal axis of the members.

508.4.2 For single angle discontinuous strut connected to gussets or to a section either by riveting or bolting by not less than two rivets or bolts in line along the angle at each end, or by their equivalent in welding, the eccentricity of the connection with respect to the centroid of the strut may be ignored and the strut designed as an axially loaded member provided that the calculated average stress does not exceed the design stress f_{cd} given in Clause **507.1.2** in which effective length "KL" is the length of the strut, centre-to-centre of fastenings at each end and 'r' is the minimum radius of gyration.

508.4.3 For single angle discontinuous struts intersected by, and effectively connected to another angle in cross bracing, the effective length in the plane of bracings shall be taken as in **Table 9** and normal to the plane of bracing the effective lengths shall be taken as the distance along the bracing members between the points of intersection and the centroids of the main member. In calculating the slenderness ratio, the radius of gyration about the appropriate rectangular axis shall be taken for buckling normal to the plane of the bracing and the least radius of gyration for buckling in the plane of the bracing.

508.4.4 *Effective length of unbraced compression chords*

For simply supported trusses with ends restrained at the bearings against torsion, the effective length of the compression chord for buckling normal to the plane of the truss shall be taken as follows :

a) With no lateral support to compression chord; where there is no lateral bracing between compression chords and no cross frames:

$$KL = \text{span}$$

b) With compression chords supported by U frames, where there is no lateral bracing of the compression chord but where cross-members and verticals forming U frames provide lateral restraints:

$$KL = 2.5 \times \sqrt[4]{EIa\delta} \text{ but not less than "}a\text{"}$$

where

E = Young's Modulus

I = maximum moment of inertia of compression chord about the axis lying in the plane of the truss.

a = distance between frames, and

δ = the virtual lateral displacement of the compression chord at the frame nearest mid span of the truss, taken as the horizontal deflection. This deflection shall be computed assuming that the cross-member is free to deflect vertically and that the tangent to the deflection curve at the centre of its span remains parallel to the neutral axis of the unrestrained cross-member.

When δ is not greater than $\frac{a^3}{40El}$

$$KL = a$$

In case of symmetrical U frames, where cross-member and verticals are each of constant moment of inertia throughout their own length; it may be assumed that :

$$\delta = \frac{(d_1)^3}{3EI_1} + \frac{(d_2)^2 b}{EI_2}$$

where

d_1 = distance of the centroid of the compression chord from the top of the cross-member,

d_2 = distance of the centroid of the compression chord from the neutral axis of the cross-member,

b = half the distance between centres of the main trusses,

E = Young's Modulus,

I_1 = moment of inertia of the vertical in its plane of bending, and

I_2 = moment of inertia of the cross-member in its plane of bending

U frames shall have rigid connections and shall be designed to resist, in addition to the effects of wind and other applied forces, the effects of a horizontal force F acting normal to the compression chord of the truss at the level of the centroid of this chord where:

$$F = \frac{1.4 \times 10^{-3} KL}{\delta \left\{ \frac{f_{cc}}{f_a} - 1.7 \right\}}$$

In the above formula :

$$KL = 2.5 \ x \ \sqrt[4]{EIa\delta}$$

δ = the deflection of the chord under the action of unit horizontal force *F*

f_{cc} = Euler buckling stress in chord = $\frac{\pi^2 E}{(KL/r)^2}$

where

E = Young's Modulus

r = radius of gyration

f_a = calculated stress in the chord.

c) with compression chord supporting continuous deck

A compression chord continuously supporting a reinforced concrete or steel deck shall be deemed to be effectively restrained laterally throughout its length (e.g., KL = 0) if the friction or positive connection of the deck to the chord is capable of resisting a lateral force, distributed uniformly along its length of 2.5 percent of the maximum force in the chord, in addition to other lateral forces.

508.5 Effective Slenderness Ratio of Compression Members

Effective slenderness ratio KL/r of a compression member shall not exceed 120 for main members and 140 for wind bracings and subsidiary members.

508.6 Connections at Intersection

508.6.1 Connections of members at an intersection shall develop at least the design loads and moments transmitted by the members. Due regard to the nature and distribution of stress over the cross-section of the members shall be given in determining the distribution of the fastenings. All members shall, where possible, be so connected that the load is appropriately distributed over the cross-section; otherwise, consideration shall be given to the distribution of stress through the material to those parts of the section not directly connected, and for this purpose the angle of distribution may be taken as 45^0.

508.6.2 Gusset shall be capable of sustaining the design loads and moments transmitted by the members.

508.6.3 Gusset plates shall be so shaped and connectors so arranged as to avoid stress concentrations.

508.6.4 Rivet, bolt and welding groups shall be as compact as practicable.

508.7 Lug Angles

508.7.1 Lug angles connecting a channel or similar member shall, as far as possible, be disposed symmetrically with respect to the section of the member.

508.7.2 In the case of angle members the lug angles, and their connection to the gusset or other supporting member, shall be capable of developing a strength not less than 20 percent in excess of the force in the outstanding leg of the angle and the attachment of the lug angles to the angle member shall be capable of developing a strength 40 percent in excess of that force.

508.7.3 In the case of channel or similar members, the lug angles, and their connection to the gusset or other supporting member, shall be capable of developing a strength not less than 10 percent in excess of the force not accounted for by the direct connection of the member and the attachment of the lug angles to the member shall be capable of developing a strength 20 percent in excess of that force.

508.7.4 In no case shall less than two bolts or rivets be used for attaching the lug angle to the gusset or other supporting member.

508.7.5 The effective connection of the lug angle shall, as far as possible terminate at the end of the member connected, and the fastening of the lug angle to the member shall preferably start in advance of the direct connection of the member to the gusset etc.

508.8 Section at Pin Holes in Tension Members

In Pin-connected tension members (generally used for erection purpose) the longitudinal net section beyond the pin hole parallel with the axis of the member shall be not less than the required net section of the member. The net section through the pin hole transverse to the axis of the member shall be at least 33 percent greater than the required net section of the member. In the case of members without stiffened edges the ratio of the net width of the members (through the pin hole transverse to the axis of the member) to its thickness shall not be more than 16. Where the thickness of the main material is not sufficient to resist the load from the pin in bearing, or where the net section through the pinhole requires reinforcement, pin plates (see Clause **508.9** below) shall be provided and the total thickness shall comply with the above requirements.

508.9 Pin Plates

Pin plates shall be of sufficient thickness to make-up the required bearing or cross-sectional area and shall be so arranged as to reduce the eccentricity to a minimum. Their length measured from the centre of the pin to the end (on the reaction side) shall be at least equal to their width and at least one plate on each side shall be as wide as the dimensions of the member will allow. Pin plates shall be connected with enough rivets, bolts or welds to transmit the bearing pressure on them and shall be so arranged as to distribute it uniformly over the full section of the member.

508.10 Diaphragms in Members

In addition to diaphragms required for the proper functioning of the structure, diaphragms shall be provided as necessary for fabrication, transport and erection.

508.11 Lateral Bracings

508.11.1 Girders shall be provided with a lateral bracing system extending from end-to-end of sufficient strength designed to transmit the effect of wind, seismic and centrifugal forces, if any to the bearings. Bracing system need not be provided if alternative system for lateral load transfer has been catered for e.g., by rigid deck.

508.11.2 The bracing on the loaded chord shall be so designed as to transmit to the main girders the longitudinal loads due to tractive effort and/or braking effect in order to relieve the cross girders of horizontal bending stress.

508.11.3 Where the depth permits, lateral diagonal bracings shall be fixed between the top chords of main girders of through span, of sufficient rigidity to maintain the chords in line and of sufficient strength to transmit the wind or seismic forces to the portal bracing between end posts.

The floor system may be taken as part of the bracing system provided it is designed for that purpose.

508.11.4 The lateral bracings between compression chords shall be designed to resist a transverse shear at any section equal to 2.5 percent of the total compressive force carried by both the chords at the section under consideration. This force should be considered in addition to the wind, and centrifugal forces.

508.11.5 *Sway bracings*

Wherever the depth of girder allows, the intermediate cross bracings or sway bracings between vertical web members shall be proportioned to transmit to the chord supported on bearings through the web members at least 50 percent of the panel lateral load and the vertical members shall be designed to resist the resulting bending moment. The sway bracing so provided shall not be taken as affording any relief to the lateral bracing system or portal system.

508.11.6 *Portal bracings*

Through truss spans shall be provided with suitably designed portal system, as deep as the clearance will allow. The portal system shall be designed to take the full end reaction of the top chord lateral system and the end posts of the portal shall be designed to transfer this reaction to the bearings. In addition, the portal system shall be designed to resist a lateral shear equal to 1.25 percent of the total compressive force in the end posts or in the top chords in the end panel whichever is greater.

509 DESIGN OF BEAMS AND PLATE GIRDERS

509.1 General

509.1.1 Beams are defined as members with solid webs (or with openings in accordance with Clause **509.1.4**), including members of rolled and hollow section, and plate girders subjected primarily to bending.

509.1.2 Beams shall satisfy the deflection limitation presented in Clause **504.5** of this Code.

509.1.3 The effective span of a beam shall be taken as the distance between the centres of the supports.

509.1.4 *Openings*

509.1.4.1 Any openings in webs or compression flanges should be framed and the stiffened section designed for local load effects, including secondary bending. Alternatively, openings in webs may be unstiffened provided that they meet the provisions of Clause **509.2**.

All corners should be rounded with a radius of at least one-quarter of the least dimension of the hole.

509.1.4.2 Openings in a web may be unstiffened provided that:

a) the overall greatest internal dimension does not exceed one-tenth of the depth of the web, nor, for longitudinally, stiffened webs, one-third of the depth of the panel containing the opening;

b) the longitudinal distance between the boundaries of adjacent openings is atleast three times the maximum internal dimension;

c) not more than one opening is provided at any cross-section.

509.2 Design Strength in Bending (Flexure)

The design bending strength of beam, adequately supported against lateral torsional buckling (laterally supported beam) is governed by the yield stress (Clause **509.2.1**). When a beam is not adequately supported against lateral buckling (laterally unsupported beams) the design bending strength may be governed by lateral torsional buckling strength (Clause **509.2.2**).

The factored design moment, M at any section in a beam due to external loads shall satisfy the following requirement :

$$M < M_d$$

where, M_d = design bending strength of the section, calculated as given in Clause **509.2.1.2.**

509.2.1 *Laterally supported beam*

A beam may be assumed to be adequately supported provided the compression flange has full lateral restraint and nominal torsional restraint at supports suitably imparted by web cleats, partial depth end plates, fin plates or continuity with the adjacent span. Full lateral restraint to compression flange may be assumed to exist, if the frictional or other positive restraint of a structural connection to the compression flange of the member is capable of resisting a lateral force not less than 2.5 percent of the maximum force in the compression flange of the member. This may be considered to be uniformly distributed along the flange, provided gravity loads constitute the dominant loading on the member and the deck construction is capable of resisting this lateral force.

The design bending strength of a section which is not susceptible to web buckling under shear before yielding (where $d/t_w \leq 67\varepsilon$) shall be determined according to Clause **509.2.1.2**.

509.2.1.1 Section with web susceptible to shear buckling before yielding - When the flanges are plastic, compact or semi-compact but the web is susceptible to shear buckling before yielding (i.e. $d/t_w \leq 67\varepsilon$), the design bending strength shall be calculated using one of the following methods:

a) The bending moment and axial force acting on the section may be assumed to be resisted by flanges only and the web is designed only to resist shear (Clause **509.4**).

b) The bending moment and axial force acting on the section may be assumed to be resisted the whole section. In such a case, the web shall be designed for combined shear and normal stresses using simple elastic theory in case of semi-compact webs and simple plastic theory in the case of compact and plastic webs.

509.2.1.2 When the factored design shear force does not exceed 0.6 V_d, where V_d is the design shear strength of the cross-section (Clause **509.4**), the design bending strength, M_d, shall be taken as

$$M_d = \beta_b Z_\rho f_y / \gamma_{m0}$$

To avoid irreversible deformation under serviceability loads, M_d shall be less than $1.2\ Z_e f_y / \gamma_{m0}$ in case of simply supported and $1.5\ Z_e f_y / \gamma_{m0}$ in cantilever beams.

where

β_b = 1.0 for plastic and compact sections.

β_b = Z_e / Z_ρ for semi-compact sections

Z_ρ, Z_e = plastic and elastic section moduli of the cross-section, respectively

f_y = yield stress of the material

γ_{m0} = partial safety factor (Clause **503.4**)

509.2.1.3 When the design shear force (factored), V, exceeds 0.6 V_d , where V_d is the design shear strength of the cross-section (Clause **509.4**), the design bending strength, M_d shall be taken as :

$$M_d = M_{dv}$$

where M_{dv} = design bending strength under high shear as defined in Clause **510.2**.

509.2.1.4 Holes in the tension zone

a) The effect of holes in the tension flange, on the design bending strength need not be considered if

$$(A_{nf}/A_{gf}) \geq (f_y/f_u)\ (\gamma_{ml} / \gamma_{m0})/0.9$$

where

A_{nf} / A_{gf} = ratio of net to gross area of the flange in tension

f_y/f_u = ratio of yield and ultimate stress of the material

$\gamma_{ml} / \gamma_{m0}$ = ratio of partial safety factors against ultimate to yield stress (Clause **503.4**)

When A_{nf}/A_{gf} does not satisfy the above requirement, the reduced effective flange area, A_{ef} satisfying the above equation may be taken as the effective flange area in tension, instead of A_{gf}.

b) The effect of holes in the tension region of the web on the design flexural strength need not be considered, if the limit given in (a) above is satisfied for the complete tension zone of the cross-section, comprising the tension flange and tension region of the web.

c) Fastener holes in the compression zone of the cross-section need not be considered in design bending strength calculation, except for oversize and slotted holes, or holes without any fastener.

509.2.1.5 Shear lag Effects - The shear lag effects in flanges may be disregarded provided:

a) For outstand elements (supported along one edge) , $b_0 \leq L_o/20$

b) For internal elements (supported along two edges), $b_i \leq L_o/10$

where

L_o = length between points of zero moment (inflection) in the span

b_0 = width of the flange with outstand

b_i = width of the flange as an internal element

When these limits are exceeded, the effective width of flange for design strength may be calculated using specialist literature, or conservatively taken as the value satisfying the limit given above.

509.2.2 *Laterally unsupported beams*

Resistance to lateral tortional buckling need not be checked separately (member may be treated as laterally supported (Clause **509.2.1**) in the following cases:

a) Bending is about the minor axis of the section;

b) Section is hollow (rectangular/tubular) or solid bars;

c) In case of major axis bending, λ_{LT} (as defined below) is less than 0.4

The design bending strength of laterally unsupported beam as governed by lateral torsional buckling is given by :

$$M_d = \beta_b Z_p f_{bd}$$

where

β_b = 1.0 for plastic and compact sections.

= Z_e / Z_ρ for semi-compact sections

Z_ρ, Z_e = plastic section modulus and elastic section modulus with respect to extreme compression fibre.

f_{bd} = design bending compressive stress, obtained as given in **Tables 10a and 10b**

f_{bd} = $\chi_{LT} f_y / \gamma_{mo}$

χ_{LT} = bending stress reduction factor to account for lateral torsional buckling, given by:

$$\chi_{LT} = \frac{1}{\left\{\phi_{LT} + \left[\phi_{LT}^2 - \lambda_{LT}^2\right]^{0.5}\right\}} \leq 1.0$$

$$\phi_{LT} = 0.5\left(1 + \alpha_{LT}\left(\lambda_{LT} - 0.2\right) + \lambda_{LT}^2\right)$$

α_{LT} the imperfection parameter is given by :

α_{LT} = 0.21 for rolled steel section

α_{LT} = 0.49 for welded steel section

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

$$\lambda_{LT} = \sqrt{\beta_b Z_\rho f_y / M_{cr}} \leq \sqrt{1.2 Z_c f_y / M_{cr}}$$

$$= \sqrt{f_y / f_{cr,b}}$$

where

M_{cr} = elastic critical moment calculated in accordance with Clause **509.2.2.1** and

$f_{cr,\,b}$ = extreme fibre bending compressive stress corresponding to elastic lateral buckling moment (Clause **509.2.2.1**) **Table 11**.

Table 10a Design Bending Compressive Stress Corresponding to Lateral Buckling, (f_{bd}, α_{LT} = 0.21)

(Clause **509.2.2**)

$f_{cr,b}$	f_y																		
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
8 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
6 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
4 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
2 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
1 000	169.1	179.5	186	196.5	202.9	209.1	219.8	229.1	245.5	261.8	275.1	291.3	300.5	323.6	332.2	355.9	370.9	384.8	412.4
900	169.1	179.5	186	194.5	200.7	204.5	215.1	231.6	242.7	258.9	272	291.3	300.5	316.4	328.4	339.5	366.5	380.2	392.7
800	167.3	177.5	184	190.3	196.4	206.8	212.7	224	240	258.9	268.9	284.7	293.6	301.8	324.5	335.5	349.1	370.9	387.8
700	163.6	171.8	182	188.2	192	202.3	208	226.5	237.3	250.2	259.6	278.2	286.7	294.5	305.5	327.3	340.4	352.4	363.3
600	161.8	168	176	181.9	194.2	197.7	203.3	218.9	226.4	244.4	253.5	261.8	276.4	287.3	294	306.8	322.9	333.8	343.6
500	161.8	166.1	172	179.8	185.5	188.6	200.9	208.7	218.2	232.7	244.2	248.7	259.1	269.1	274.9	286.4	296.7	301.4	314.2
450	158.2	164.2	168	173.5	183.3	186.4	191.5	206.2	215.5	224	231.8	242.2	248.7	258.2	263.5	274.1	279.3	292.1	294.5
400	150.9	162.3	166	169.4	174.5	184.1	186.7	196	204.5	215.3	222.5	229.1	238.4	243.6	248.2	257.7	261.8	264.3	274.9
350	147.3	152.7	162	165.2	170.2	172.7	179.6	188.4	193.6	200.7	210.2	212.7	221.1	225.5	229.1	233.2	240	241.1	245.5
300	143.6	147	152	154.7	161.5	163.6	167.8	175.6	182.7	186.2	194.7	196.4	196.9	203.6	206.2	212.7	213.8	217.9	220.9
250	134.5	137.5	142	144.3	148.4	152.3	153.6	160.4	163.6	165.8	170	173.5	179.6	178.2	179.5	184.1	183.3	185.5	191.5
200	121.8	124.1	126	129.6	130.9	134.1	134.7	137.5	141.8	142.5	145.3	147.3	148.5	149.1	152.7	151.4	152.7	153	157.1
150	101.8	103.1	104	104.5	106.9	106.8	108.7	109.5	111.8	113.5	114.4	114.5	117.5	116.4	118.4	118.6	117.8	120.5	122.7
100	74.5	76.4	76	77.4	76.4	77.3	78	78.9	79.1	78.5	80.4	81.8	79.5	80	80.2	81.8	82.9	83.5	83.5
90	67.3	68.7	70	69	69.8	70.5	70.9	71.3	70.9	72.7	74.2	72	72.5	72.7	72.5	73.6	74.2	74.2	73.6
80	61.8	63	62	62.7	63.3	63.6	63.8	63.6	65.5	64	64.9	65.5	65.6	65.5	64.9	65.5	65.5	64.9	68.7
70	54.5	55.4	56	56.5	56.7	56.8	56.7	56	57.3	58.2	58.7	58.9	58.7	58.2	57.3	57.3	56.7	60.3	58.9
60	47.3	47.7	48	48.1	48	50	49.6	48.4	49.1	49.5	49.5	49.1	48.4	50.9	49.6	49.1	52.4	51	49.1
50	40	40.1	40	41.8	41.5	40.9	40.2	40.7	40.9	40.7	43.3	42.5	41.5	43.6	42	40.9	43.6	41.7	44.2
40	32.7	32.5	32	33.5	32.7	34.1	33.1	33.1	32.7	34.9	34	32.7	34.5	32.7	34.4	32.7	34.9	32.5	34.4
30	25.5	24.8	26	25.1	26.2	25	26	25.5	24.5	26.2	24.7	26.2	24.2	25.5	26.7	24.5	26.2	27.8	24.5
20	16.4	17.2	18	16.7	17.5	18.2	16.5	17.8	16.4	17.5	18.5	16.4	17.3	18.2	19.1	16.4	17.5	18.5	19.6
10	9.1	9.5	8	8.4	8.7	9.1	9.5	7.6	8.2	8.7	9.3	9.8	10.4	7.3	7.6	8.2	8.7	9.3	9.8

Table 10b Design Bending Compressive Stress Corresponding to Lateral Buckling, (f_{bd}, α_{LT} = *0.49*)

(Clause **509.2.2**)

$f_{cr,b}$	f_y																		
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450	480	510	540
10 000	181.8	190.9	200.0	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
8 000	181.8	190.9	200.0	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
6 000	181.8	190.9	200.0	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
4 000	181.8	190.9	200.0	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
2 000	181.8	190.9	200.0	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.1	436.4	463.6	490.9
1 000	160.0	164.2	170.0	179.8	185.5	190.9	196.2	211.3	220.9	235.6	247.3	255.3	266.0	280	290.2	302.7	318.5	329.2	343.6
900	154.5	164.2	170.0	173.5	183.3	188.6	193.8	203.6	218.2	226.9	238.0	252.0	262.5	269.1	282.5	290.5	305.5	319.9	333.8
800	152.7	158.5	168.0	171.5	176.7	181.8	191.5	201.1	210.0	224.0	234.9	242.2	252.2	258.2	271.1	282.3	296.7	306	319.1
700	150.9	154.6	160.0	169.4	172.4	177.3	182.0	196	207.3	215.3	222.5	232.4	238.4	247.3	259.6	270	279.3	292.1	304.4
600	145.5	150.8	154.0	161.0	168.0	172.7	177.3	188.4	193.6	203.6	213.3	222.5	228	236.4	244.4	253.6	261.8	273.5	274.9
500	140.0	145.1	150.0	154.7	159.3	161.4	167.8	175.6	185.5	192	200.9	206.2	214.2	218.2	225.3	229.1	240	245.7	250.4
450	134.5	141.3	144.0	148.5	152.7	156.8	160.7	168	177.3	186.2	191.6	196.4	203.8	210.9	213.8	220.9	231.3	236.5	235.6
400	129.1	135.5	138.0	142.2	148.4	150	153.6	162.9	169.1	174.5	182.4	183.3	193.5	196.4	202.4	208.6	209.5	217.9	220.9
350	123.6	129.8	132.0	135.9	139.6	143.2	148.9	152.7	158.2	162.9	170	173.5	176.2	181.8	183.3	192.3	196.4	199.4	206.2
300	118.2	122.2	126.0	129.6	130.9	134.1	137.1	142.5	147.3	154.2	157.6	157.1	162.4	167.3	168	175.9	178.9	180.8	181.6
250	109.1	112.6	116.0	117.1	120.0	122.7	125.3	129.8	130.9	136.7	139.1	140.7	145.1	149.1	148.9	151.4	152.7	157.6	157.1
200	98.2	101.2	102.0	104.5	104.7	109.1	108.7	112	117.3	119.3	120.5	121.1	124.4	127.3	126	130.9	130.9	129.8	132.5
150	83.6	84.0	86.0	87.8	89.5	88.6	89.8	91.6	95.5	96.0	95.8	98.2	100.2	101.8	103.1	102.3	104.7	106.6	103.1
100	63.6	63.0	64.0	64.8	65.5	65.9	66.2	68.7	68.2	69.8	71.1	68.7	69.1	72.7	72.5	73.6	74.2	74.2	73.6
90	58.2	57.3	60.0	58.5	61.1	61.4	61.5	61.1	62.7	64.0	64.9	65.5	65.6	65.5	64.9	65.5	65.5	64.9	68.7
80	52.7	53.5	54.0	54.4	54.5	54.5	54.4	56	57.3	58.2	58.7	58.9	58.7	58.2	61.1	61.4	61.1	60.3	58.9
70	47.3	47.7	48.0	48.1	48.0	50.0	49.6	50.9	49.1	49.5	52.5	52.4	51.8	50.9	53.5	53.2	52.4	55.6	54.0
60	41.8	42.0	42.0	41.8	43.6	43.2	42.5	43.3	43.6	43.6	43.3	45.8	44.9	47.3	45.8	45.0	48.0	46.4	49.1
50	36.4	36.3	36.0	35.5	37.1	36.4	37.8	38.2	38.2	37.8	37.1	39.3	38.0	40.0	38.2	40.9	39.3	37.1	39.3
40	29.1	30.5	30.0	29.3	30.5	29.5	30.7	30.5	30.0	32.0	30.9	32.7	31.1	32.7	30.5	32.7	30.5	32.5	34.4
30	23.6	22.9	22.0	23.0	24.0	22.7	23.6	22.9	24.5	23.3	24.7	22.9	24.2	25.5	22.9	24.5	26.2	23.2	24.5
20	16.4	15.3	16.0	16.7	15.3	15.9	16.5	15.3	16.4	17.5	15.5	16.4	17.3	18.2	15.3	16.4	17.5	18.5	14.7
10	9.1	7.6	8.0	8.4	8.7	9.1	9.5	7.6	8.2	8.7	9.3	9.8	6.9	7.3	7.6	8.2	8.7	9.3	9.8

Table 11 Critical Stress, $f_{cr,b}$

(Clause **509.2.2.1**)

KL/r	h/t_f 8	10	12	14	16	18	20	25	30	35	40	50	60	80	100
10	22 551.2	22 255.1	22 092.6	21 994.1	21 929.8	21 885.7	21 854.0	21 805.4	21 779.0	21 763.1	21 752.7	21 740.5	21 733.8	21 727.2	21 724.2
20	6 220.5	5 947.9	5 794.5	5 700.0	5 637.8	5 594.7	5 563.8	5 515.8	5 489.7	5 473.8	5 463.5	5 451.4	5 444.8	5 438.2	5 435.1
30	3 149.3	2 905.9	2 764.6	2 676.0	2 616.7	2 575.3	2 545.3	2 498.5	2 472.8	2 457.1	2 447.0	2 434.9	2 428.3	2 421.7	2 418.6
40	2 036.1	1 821.2	1 693.0	1 610.8	1 555.1	1 515.8	1 487.0	1 441.7	1 416.5	1 401.1	1 391.0	1 379.0	1 372.5	1 365.9	1 362.8
50	1 492.9	1 303.2	1 187.3	1 111.8	1 059.9	1 022.7	995.3	951.7	927.1	912.0	902.0	890.2	883.7	877.1	874.2
60	1 178.0	1 009.5	905.0	835.6	787.4	752.4	726.4	684.6	660.9	646.1	636.4	624.7	618.2	611.7	608.7
70	973.9	823.2	728.5	664.8	620.1	587.4	562.9	522.9	500.0	485.5	476.0	464.4	458.0	451.7	448.7
80	831.3	695.4	609.2	550.7	509.1	478.4	455.3	417.2	395.1	381.2	371.8	360.5	354.1	347.7	344.7
90	725.9	602.6	523.6	469.5	430.9	402.2	380.4	344.2	322.9	309.3	300.2	289.1	282.8	276.5	273.5
100	644.7	532.0	459.3	409.3	373.2	346.4	325.8	291.4	270.9	257.7	248.8	237.9	231.8	225.5	222.5
110	580.4	476.6	409.3	362.9	329.2	303.9	284.5	251.8	232.1	219.3	210.8	200.1	194.0	187.8	184.8
120	527.9	431.9	369.5	326.0	294.5	270.7	252.3	221.2	202.4	190.1	181.6	171.2	165.2	159.1	156.2
130	484.3	395.0	336.8	296.1	266.5	244.1	226.7	197.1	179.0	167.1	158.8	148.6	142.8	136.7	133.9
140	447.6	364.2	309.5	271.5	243.4	222.3	205.8	177.5	160.2	148.7	140.7	130.8	125.0	119.0	116.2
150	416.0	337.8	286.6	250.6	224.2	204.2	188.4	161.5	144.8	133.7	126.0	116.3	110.6	104.7	101.9
160	388.7	315.2	266.8	232.8	207.8	188.8	173.9	148.2	132.0	121.3	113.9	104.3	98.8	93.0	90.1
170	364.9	295.4	249.6	217.5	193.7	175.6	161.4	136.7	121.3	111.0	103.6	94.4	89.0	83.2	80.4
180	343.9	278.0	234.6	204.1	181.5	164.2	150.6	127.1	112.2	102.2	95.2	86.0	80.7	75.0	72.3
190	325.2	262.6	221.3	192.3	170.7	154.2	141.2	118.6	104.3	94.6	87.8	79.0	73.7	68.1	65.3
200	308.3	248.8	209.6	181.7	161.2	145.4	133.0	111.3	97.5	88.1	81.4	72.8	67.8	62.2	59.5
210	293.3	236.5	198.9	172.4	152.7	137.6	125.7	104.8	91.5	82.4	75.9	67.5	62.6	57.1	54.5
220	279.5	225.3	189.3	163.9	145.1	130.6	119.1	99.0	86.2	77.4	71.2	62.9	58.1	52.7	50.1
230	267.1	215.2	180.7	156.3	138.2	124.3	113.3	93.9	81.5	72.9	66.9	58.9	54.1	48.8	46.2
240	255.8	205.8	172.8	149.4	132.0	118.6	108.0	89.3	77.2	69.0	63.1	55.3	50.6	45.4	42.8
250	245.3	197.3	165.6	143.0	126.3	113.4	103.2	85.1	73.5	65.5	59.7	52.1	47.5	42.4	39.8
260	235.7	189.5	159.0	137.3	121.1	108.7	98.8	81.3	70.1	62.3	56.7	49.3	44.8	39.7	37.2
270	226.8	182.3	152.8	131.9	116.3	104.3	94.7	77.9	67.0	59.4	53.9	46.8	42.2	37.3	34.8
280	218.6	175.7	147.2	126.9	111.9	100.2	91.1	74.7	64.1	56.8	51.5	44.4	40.0	35.2	32.7
290	210.9	169.4	141.9	122.3	107.8	96.6	87.7	71.8	61.5	54.3	49.2	42.2	38.1	33.2	30.8
300	203.8	163.7	137.1	118.1	104.1	93.2	84.5	69.1	59.1	52.1	47.1	40.4	36.2	31.5	29.0

509.2.2.1 Elastic lateral torsional buckling moment - In case of simply supported, prismatic members with symmetric cross-section, the elastic lateral buckling moment, M_{cr} can be determined from:

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

$f_{cr,}$ b of non-slender rolled steel sections in the above equation may be approximately calculated from the values given in **Table 11** which has been prepared using the following equation :

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

The following simplified 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 lateral buckling moment, M_{cr} **(Table 11)**.

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

where

I_t = torsional constant = $\sum b_i t_i^3/3$ for open section

I_w = warping constant

$I_y\ r_y$ = moment of inertia, radius of gyration about the weak axis, respectively

L_{LT} = effective length for lateral torsional buckling (Clause **509.3**)

h_f = centre-to-centre distance between flanges

t_f = thickness of the flange

M_{cr} for different beam sections, considering loading, support condition and non-symmetric section, shall be calculated using the method given in **Annex-C**.

509.3 Effective Length for Lateral Torsional Buckling

509.3.1 For simply supported beams and girders of span length, L, where no lateral restraint to the compression flanges is provided, but where each end of the beam is restrained against torsion, the effective length L_{LT} for the lateral buckling to be used in Clause **509.2.2.1** shall be taken as in **Table 12**.

Table 12 Effective Length for Simply Supported Beams, L_{LT}
(Clause **509.3.1**)

Conditions of restraint at supports		Loading condition	
Torsional restraint	**Warping restraint**	**Normal**	**Destabilising**
Fully restrained	Both flanges fully restrained	0.70 L	0.85 L
Fully restrained	Compression flange fully restrained	0.75 L	0.90 L
Fully restrained	Both flanges fully restrained	0.80 L	0.95 L
Fully restrained	Compression flange partially restrained	0.85 L	1.00 L
Fully restrained	Warping not restrained in both flanges	1.00 L	1.20 L
Partially restrained by bottom flange support connection	Warping not restrained in both flanges	1.0 L + 2D	1.2 L + 2D
Partially restrained by bottom flange bearing support	Warping not restrained in both flanges	1.2 L + 2D	1.4 L + 2D

1) Torsional restraint prevents rotation about the longitudinal axis
2) Warping restraint prevents rotation of the flange in its plane
3) D is the overall depth of the beam

In simply supported beams with intermediate lateral restraints against lateral torsional buckling, the effective length for lateral torsional buckling to be used in Clause **509.2.2.1**, L_{LT}, shall be taken as the length of the relevant segment in between the lateral restraints. The effective length shall be equal to 1.2 times the length of the relevant segment in between the lateral restraints.

Restraints against torsional rotation at supports in these beams may be provided by:

a) web or flange cleats, or

b) bearing stiffeners acting in conjunction with the bearing of the beam, or

c) lateral end frames or external supports providing lateral restraint to the compression flanges at the ends

509.3.2 For beams, which are provided with members giving effective lateral restraint to the compression flange at intervals along the span, in addition to the end torsional restraint required in Clause **509.3.1**, the effective length for lateral torsional buckling shall be taken as the distance, centre-to-centre, of the restraint members in the relevant segment under normal loading condition and 1.2 times the distance, where the load is not acting on the beam at the shear centre and is acting towards the shear centre so as to have destabilizing effect during lateral torsional buckling deformation.

509.3.3 For cantilever beams of projecting length, L, the effective length L_{LT} to be used in Clause **509.2.2.1** shall be taken as in **Table 13** for different support conditions.

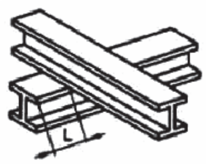
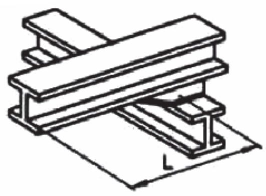
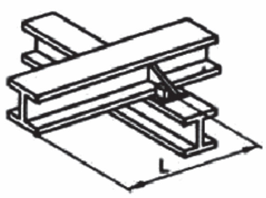
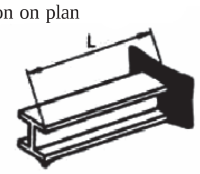
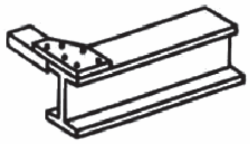
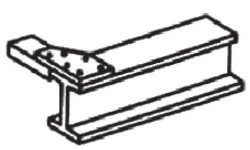
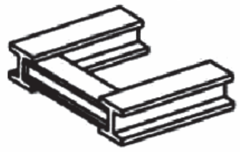
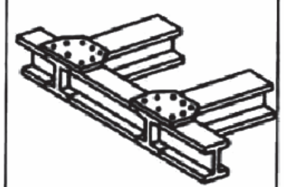
509.3.4 Where a member is provided with intermediate lateral supports to improve the lateral buckling strength, these restraints should have sufficient strength and stiffness to prevent lateral movement of the compression flange at the point, relative to the end supports. The intermediate lateral restraints should be either connected to an appropriate bracing system capable of transferring the restraint force to the effective lateral support at the ends of the member, or should be connected to an independent robust part of the structure capable of transferring the restraint force. Two or more parallel members requiring such lateral restraint shall not be simply connected together assuming mutual dependence for the lateral restraint.

The intermediate lateral restraints should be connected to the member as close to the compression flange as practicable. Such restraints should be closer to the shear centre of the compression flange than to the shear centre of the section. However, if torsional restraint preventing relative rotation between the two flanges is provided, the intermediate lateral restraint may be connected at any appropriate level.

For beams which are provided with members giving effective lateral restraint at intervals along the span, the effective lateral restraint shall be capable of resisting a force of 2.5 percent of the maximum force in the compression flange taken as divided equally between the points at which the restraint members are provided. Further, each restraint point should be capable of resisting 1 percent of the maximum force in the compression flange.

Table 13 Effective Length, L_{LT}, For Cantilever of Length L

(Clause **509.3.3**)

Restraint Condition		**Loading Condition**	
At Support	**At Top**	**Normal**	**Destabilizine**
(1)	**(2)**	**(3)**	**(4)**
a) Continuous, with lateral restraint to top flange 	i) Free ii) Lateral restraint to top flange iii) Torsional restraint iv) Lateral and torsional restraint	3.0L 2.7L 2.4L 2.1L	7.5L 7.5L 4.5L 3.6L
b) Continuous, with partial torsinal restraint 	i) Free ii) Lateral restraint to top flange iii) Torsional restraint iv) Lateral and torsional restraint	2.0L 1.8L 1.6L 1.4L	5.0L 5.0L 3.0L 2.4L
c) Continuous, with lateral and torsional restraint 	i) Free ii) Lateral restraint to top flange iii) Torsional restraint iv) Lateral and torsional restraint	1.0L 0.9L 0.8L 0.7L	2.5L 2.5L 1.5L 1.2L
d) Restrained laterally, torsionally and against rotation on plan 	i) Free ii) Lateral restraint to top flange iii) Torsional restraint iv) Lateral and torsional restraint	0.8L 0.7L 0.6L 0.5L	1.4L 1.4L 0.6L 0.5L
Top restraint consitions			
i) Free 	ii) Lateral restraint to top flange 	iii) Torsional restraint 	iv) Lateral and torsional restraint 

509.3.4.1 In a series of such beams, with solid webs, which are connected together by the same system of restraint members, the sum of the restraining forces required shall be taken as 2.5 percent of the maximum flange force in one beam only.

509.3.4.2 In the case of a series of latticed beams or girders, which are connected together by the same system of restraint members, the sum of restraining forces required shall be taken as 2.5 percent of the maximum force in the compression flange plus 1.25 percent of the force for every member of the series other than the first, upto a maximum total of 7.5 percent.

509.3.5 For simply supported beams where there is no lateral bracing of the compression flanges but where cross members and stiffeners forming U-Frames provide lateral restraint.

$$KL = 2.5 \text{ x } \sqrt[4]{(EI_c a\delta)} \text{ but not less than "}a\text{"}$$

where

E = Young's Modulus

I_c = Maximum moment of inertia of compression flange about its centroidal axis parallel to the web of the girder.

a = distance between frames

δ = the lateral deflection which would occur in the U-Frame at the level of the centroid of the flange being considered when a unit force acts laterally to the U-Frame only at this point and simultaneously at each corresponding point on the other flange or flanges connecting to the same U-Frame. The direction of each unit force should be such as to produce the maximum aggregate value of δ. The U-Frame should be taken as fixed in position at each point or intersection between the cross member and a vertical as free and unconnected at all other points.

when δ is not greater than $a^3/(40\ EI_c)$

$$KL = a$$

In cases of symmetrical U-Frames where cross-members and stiffeners are each of constant moment of inertia throughout their own length.

$$\delta = \frac{(d_1)^3}{3EI_1} + \frac{(d_2)^2 b}{EI_2}$$

where

d_1 = distance of the centroid of the compression flange from the top of the cross-member

d_2 = distance of the centroid of the compression flange from the neutral axis of the cross-member

b = half the distance between centres of the main girders.

I_1 = the moment of inertia of a pair of stiffeners about the centre of the web, or a single stiffener about the face of the web. A width of web plate upto 16 times the web thickness may be included on each side of centerline of connection.

I_2 = Moment of inertia of the cross member in its plane of bending

509.4 Shear

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

$$V \leq V_d$$

where

V_d = design strength

= V_n / γ_{m0}

where

γ_{m0} = partial safety factor against shear failure(Clause **503.4**)

The nominal shear strength of a cross section, V_n, may be governed by plastic shear resistance (Clause **509.4.1**) or strength of the web as governed by shear buckling (Clause **509.4.2**).

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

$$V_n = V_p$$

where $V_p = \dfrac{A_v f_{yw}}{\sqrt{3}}$

A_v = shear area

f_{yw} = yield strength of the web

509.4.1.1 The shear area may be calculated as given below

I and channel sections :

Major Axis Bending :

Hot Rolled : $h\, t_w$

Welded : $d\, t_w$

Minor Axis Bending

Hot rolled or welded : $2b\, t_f$

Rectangular hollow sections of uniform thickness :

Loaded parallel to depth (*h*) : $Ah/(b + h)$

Loaded parallel to width (*b*) : $Ab/(b + h)$

Circular hollow tubes of uniform thickness : $2A/\pi$

Plates and solid bars : A

where

A = cross-section area

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

d = clear depth of the web between flanges

h = overall depth of the section

t_f = thickness of the flange

t_w = thickness of the web

NOTE : Fastener holes need not be accounted for in plastic design shear strength calculation provided that :

$$A_{vn} \geq (f_y / f_u)(\gamma_{m1} / \gamma_{m0})\, A_v / 0.9$$

If A_{vn} does not satisfy the above condition, the effective shear area may be taken as that satisfying the above limit. Block shear failure criteria may be verified at the end connections. Clause **510** may be referred to for design strength under combined high shear and bending

509.4.2 *Resistance to shear buckling*

509.4.2.1 Resistance to Shear buckling shall be verified as specified when

$d/t_w > 67\,\varepsilon$ for a web without stiffeners and

$> 67\,\varepsilon\sqrt{\dfrac{K_v}{5.35}}$ for a web with stiffeners

where

K_v = shear buckling coefficient (Clause **509.4.2.2**)

ε = $\sqrt{250/f_y}$

509.4.2.2 *Shear buckling design methods*

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

= $A_v\ \tau_b$

where

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

a) When $\lambda_w \leq 0.8$

$\tau_b = f_{yw} / \sqrt{3}$

b) When $0.8 < \lambda_w < 1.2$

$\tau_b = [1 - 0.8\,(\lambda_w - 0.8)]\,(f_{yw}/\sqrt{3}\,)$

c) When $\lambda_w \geq 1.2$

$$\tau_b = f_{yw} / (\sqrt{3}\lambda_w^2)$$

where

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

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

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

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

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, 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 if c/d $\geq$ 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, V_n, is given by

$$V_n = V_{tf}$$

where

V_{tf} = $[A_v \tau_b + 0.9 w_{tf} t_w f_v \sin\phi] \leq V_p$

where

τ_b = buckling strength, as obtained from Clause **509.4.2.2 (a)**

f_v = yield strength of the tension field

$= [f_{yw}^2 - 3\,\tau_b^2 + \Psi^2]^{0.5} - \Psi$

Ψ = $1.5\,\tau_b \sin 2\phi$

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

w_{tf} = width of the tension field

= $d\cos\phi + (c - s_c - s_t)\sin\phi$

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 (Clause **509.4.2.2 (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} = $0.25\, b_f\, t_f^2\, f_{yf}\, [1 - \{N_f / (b_f\, t_f\, f_{yf} / \gamma_{m0})\}^2]$

where

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

f_{yf} = Yield stress of the flange

509.5 Stiffened Web Panels

509.5.1 *End panels design* ***(Fig. 7)*** -The design of end panels in girders in which the interior panel (panel A) is designed using tension field action shall be carried out in accordance with the provisions given herein. In this case the end panel should be designed using only Simple Post Critical Method, according to Clause **509.4.2.2 (a)**.

Additionally, the end panel along with the stiffeners should be checked as a beam spanning between the flanges to resist a shear force, R_{tf}, and a moment, M_{tf}, due to tension field forces as given in Clause **509.5.3**. Further, end stiffener should be capable of resisting the reaction plus a compressive force due to moment, equal to M_{tf} , **(Fig. 7)**

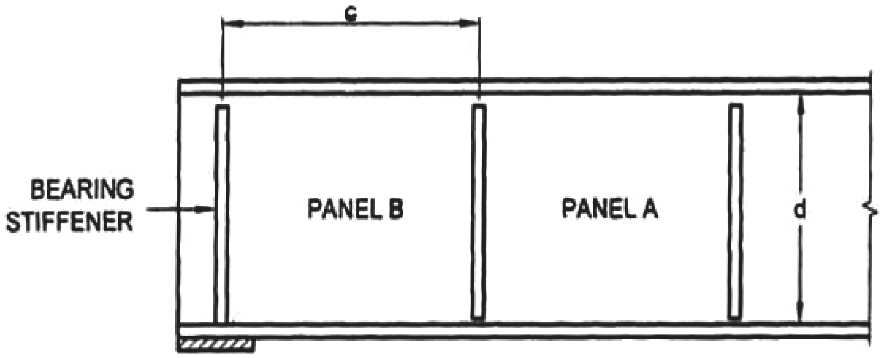


Fig. 7 End Panel Designed Not Using Tension Field Action

NOTES:

1) Panel A is designed utilizing tension field action, as given in Clause **509.4.2.2(b)**
2) Panel B is designed without utilizing tension filed action, as given in Clause **509.4.2.2(a)**.
3) Bearing stiffener is designed for the compressive force due to bearing plus comprehensive force due to the moment M_{tf} as given in Clause **509.5.3**.

509.5.2 *End panels designed using tension field action (**Figs. 8 and 9**)* - The design of end panels in girders which ar designed using tension field action shall be carried out in accordance with the provisions mentioned herein. In this case, the end panel (Panel B) shall be designed according to Clause **9.4.2.2(b).**

Additionally it should be provided with an end post consisting of a single or double stiffener, **(Figs. 8 and 9)**, satisfying the following :

a) *Single stiffener (**Fig. 8**)* - The top of the end post should be rigidly connected to the flange using full strength welds.

   The end post should be capable of resisting the reaction plus a moment from the anchor forces equal to 2/3 M_{tf} due to tension field forces, where M_{tf} is obtained from Clause **509.5.3**. The width and thickness of the end post are not to exceed the width and thickness of the flange.

b) *Double stiffener (**Fig. 9**)* - The end post should be checked as a beam spanning between the flanges of the girder and capable of resisting a shear force R_{tf} and a moment, M_{tf} due to the tension field forces as given in Clause **509.5.3**.

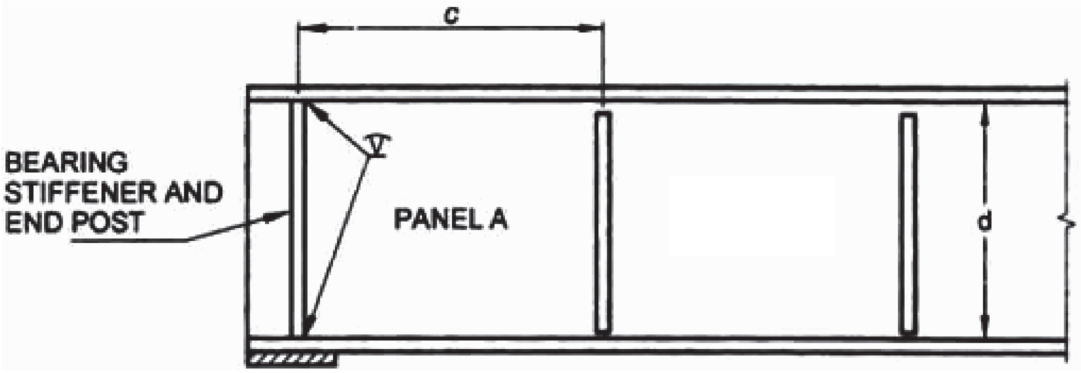


Fig. 8 End Panel Designed Using Tension Field Action (Single Stiffener)

NOTES:

1) Panel A is designed utilizing tension field action as given in Clause **509.4.2.2(b)**

2) Bearing stiffener and end post is designed for combination of compressive loads due to bearing and a moment equal to 2/3 M_{tf} as given in **509.5.3**.

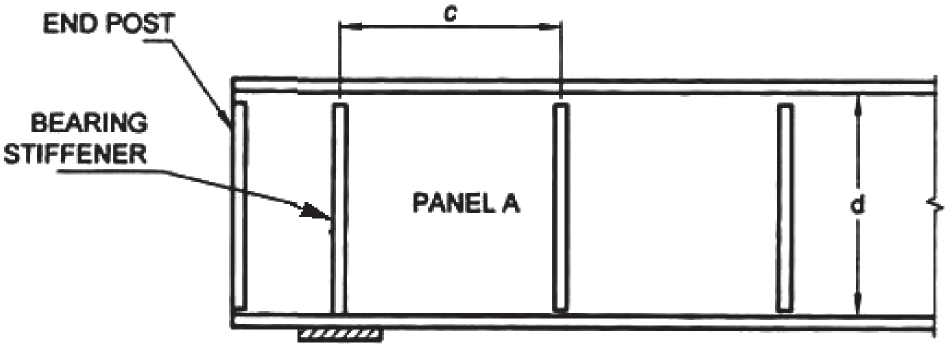


Fig. 9 End Panel Designed Using Tension Field Action (Double Stiffener)

NOTES:

1) Panel A is designed utilizing tension field action, as given in Clause **509.4.2.2(b)**

2) Bearing stiffener is designed for compressive force due to bearing as given in Clause **509.4.2.2(a)**.

3) End post is designed for horizontal shear R_{tf} and moment M_{tf} as given in Clause **509.5.3**.

509.5.3 *Anchor forces* - The resultant longitudinal shear, R_{tf} and a moment M_{tf} from the anchor of tension field forces are evaluated as given below :

$$R_{if} = H_q/2 \qquad \text{and} \qquad M_{tf} = H_q d / 10$$

where $H_q = 1.25\, V_p \left(1 - \frac{V_{cr}}{V_p}\right)^{1/2}$; $V_p = \frac{dt\, f_y}{\sqrt{3}}$; d = web depth

If the actual factored shear force, V in the panel designed using tension field approach is less than shear strength, V_{tf} (Clause **509.4.2.2b**), then the values of H_q may be reduced by the ratio;

$$\frac{V - V_{cr}}{V_{tf} - V_{cr}}$$

where

V_{tf} = the basic shear strength for the panel utilising tension field action as given in Clause **509.4.2.2b**

V_{cr} = critical shear strength for the panel designed utilising tension field action as given in Clause **509.4.2.2a**

509.5.4 *Panels with openings* - Panels with opening of dimension greater than 10 percent of the minimum panel dimension should be designed without using tension field action as given in Clause **509.4.2.2(b)**. The adjacent panels should be designed as an end panel as given in Clause **509.5.1** or Clause **509.5.2** as appropriate.

509.6 Design of Beams and Plate Girders

509.6.1 *Minimum web thickness* - The thickness of the web in a section shall satisfy the following requirements:

509.6.1.1 Serviceability requirement

a) When transverse stiffeners are not provided,

$\frac{d}{t_w} \leq 200\,\varepsilon$ (web connected to flanges along both longitudinal edges

$\frac{d}{t_w} \leq 90\,\varepsilon$ (web connected to flanges along one longitudinal edge only)

b) when only transverse stiffeners are provided (in webs connected to flanges along both longitudinal edges).

i) when $3d \geq c \geq d$,

$$\frac{d}{t_w} \leq 200\ \varepsilon_w$$

ii) when $0.74d \leq c < d$,

$$\frac{c}{t_w} \leq 200\ \varepsilon_w$$

iii) when $c < d$

$$\frac{d}{t_w} \leq 270\ \varepsilon_w$$

iv) when $c > 3d$, the web shall be considered as unstiffened.

c) When transverse stiffeners and longitudinal stiffeners at one level only are provided (0.2 d from compression flange) as per Clause **509.7.12(a)**

i) when $2.4d \geq c \geq d$,

$$\frac{d}{t_w} \leq 250\ \varepsilon_w$$

ii) when $0.74\ d \leq c \leq d$

$$\frac{c}{t_w} \leq 250\ \varepsilon_w$$

iii) when $c < 0.74\ d$,

$$\frac{d}{t_w} \leq 340\ \varepsilon_w$$

d) when a second longitudinal stiffener (located at neutral axis) is provided.

$$\frac{d}{t_w} \leq 400\ \varepsilon_w$$

where

d = depth of the web

t_w = thickness of the web

c = spacing of transverse stiffener (Fig. 7, Fig. 8)

ε_w = $\sqrt{250 / f_{yw}}$

f_{yw} = yield stress of the web

509.6.1.2 *Compression flange buckling requirement* - In order to avoid buckling of the compression flange into the web, the web thickness shall satisfy the following:

a) when transverse stiffners are not provided.

$$\frac{d}{t_w} \leq 345\ \varepsilon_f^2$$

b) when transverse stiffeners are provided and

i) when $c \geq 1.5\ d$,

$$\frac{d}{t_w} \leq 345\ \varepsilon_f^2$$

ii) when $c < 1.5\ d$

$$\frac{d}{t_w} \leq 345\ \varepsilon_f$$

where

d = depth of the web

t_w = thickness of the web

c = spacing of transverse stiffener **(Figs. 7 & 8)**

$\varepsilon_f = \sqrt{250 / f_{yf}}$

f_{yf} = yield stress of compression flange

509.6.2 *Sectional properties*

509.6.2.1 The effective sectional area of compression flanges shall be the gross area with deductions for excessive width of plates as specified for compression members (Clause **507**) and for open holes occurring in a plane perpendicular to the direction of stress at the section being considered (Clause **509.2.1.4**).

The effective sectional area of tension flanges shall be the gross sectional area with deductions for holes as specified in Clause **509.2.1.4**.

The effective sectional area for parts in shear shall be taken as specified in Clause **509.4.1.1**.

509.6.3 *Flanges*

509.6.3.1 In riveted or bolted construction, flange angles shall form as large a part of the area of the flange as practicable (preferably not less than one-third) and the number of flange plates shall be kept to a minimum.

In exposed situations, where flange angles are used, at least one plate of the top flange shall extend over the full length of the girder, unless the top edge of the web is machined flush with the flange angles. Where two or more flange plates are used, tacking rivets shall be provided, if necessary to comply with the requirements of Clause **512**.

Each flange plate shall extend beyond its theoretical cut-off point, and the extension shall contain sufficient rivets, bolts or welds to develop in the plate, the load calculated for the bending moment on the girder section (taken to include the curtailed plate) at the theoretical cut-off point.

The outstand of flange plates, that is the projection beyond the outer line of connections to flange angles, channel or joist flanges, or, in the case of welded constructions, their projection beyond the face of the web or tongue plate, shall not exceed the values given in Clause **503.7.2 (Table 2)**.

509.6.3.2 In welded construction, the use of curtailed flange plates shall be avoided as far as possible, local strengthening being provided by other means such as inserting by butt welding a thicker and or wider plate. The heavier section plate shall be suitably tapered to the lighter plate. If, in welded construction the use of curtailed flange plates cannot be avoided, the end of the plate shall be tapered in plan to a rounded end and all welds shall be continuous round the ends.

509.6.3.3 Flange splices

Flange splices preferably should not be located at points of maximum stress. Where splice plates are used, their area shall not be less than 5 percent in excess of the area of the flange element spliced; their centre of gravity shall coincide, as nearly possible, with that of the element spliced. There shall be enough bolts, rivets or welds on each side of the splice to develop the load in the element spliced plus 5 percent but in no case should the strength developed be less than 50 percent of the effective strength of the material spliced. In welded construction, flange plates shall be joined by complete penetration butt welds, wherever possible. These butt welds shall develop the full strength of the plates.

509.6.3.4 Connection of flanges to web

The flanges of plate girders shall be connected to the web by sufficient rivets, bolts or welds to transmit the maximum horizontal shear force resulting from the bending moment gradient in the girder, combined with any vertical loads which are directly applied to the flange. If the web is designed using tension field method as given in Clause **509.4.2.2(b)**, then the weld should be able to transfer the tension field stress, f_{yw}, acting on the web.

509.6.3.5 Bolted/riveted construction

For girders in exposed situations and which do not have flange plates for their entire length, the top edge of the web plate shall be flush with or above the angles, and the bottom edge of the web plate shall be flush with or set back from the angles.

509.6.3.6 Welded construction

The gap between the web plates and flange plates shall be kept to a minimum and for fillet welds, shall not exceed 1 mm at any point before welding.

509.6.4 *Webs*

509.6.4.1 Effective sectional area of web of plate girder - The effective cross-sectional area shall be taken as the full depth of the web plate multiplied by the thickness.

NOTE : Where webs are varied in thickness in the depth of the section by the use of tongue plates or the like, or where the proportion of the web included in the flange area is 25 percent or more of the overall depth, the above approximation is not permissible and the maximum shear stress shall be computed based on theory.

509.6.4.2 Splices in webs

Splices and cut-outs for service ducts in the webs preferably should not be located at points of maximum shear force and heavy concentrated loads.

Splices in the webs of the plate girders and rolled sections shall be designed to resist the shears and moments at the spliced section.

In riveted or bolted construction, splice plate shall be provided on each side of the web. In welded construction, web splices shall preferably be made with complete penetration butt welds. Where this is not possible, splice plates on both sides should be used (Refer to Clause **512**).

509.6.4.3 Where additional plates are required to augment the strength of the web, they shall be placed on each side of the web and shall be equal in thickness. The proportion of shear force, assumed to be resisted by these plates shall be limited by the amount of horizontal shear which they can transmit to the flanges through their fastenings, and such reinforcing plates and their fastenings shall be carried up to the points at which the flange without the additional plates is adequate (Refer to Clause **512**).

509.7 Design of Stiffeners

509.7.1 *General*

509.7.1.1 When the web of a member acting alone (that is without stiffeners) proves inadequate, stiffeners for meeting the following requirements should be provided.

a) *Intermediate transverse web stiffeners* - To improve the buckling strength of a slender web due to shear (Clause **509.7.2**).

b) *Load carrying stiffener* - To prevent local buckling of the web due to concentrated loading (Clauses **509.7.3** and **509.7.5**).

c) *Bearing stiffener* - To prevent local crushing of the web due to concentrated loading (Clauses **509.7.4** and **509.7.6**).

d) *Diagonal stiffener* - To provide local reinforcement to a web under shear and bearing (Clause **509.7.7**).

e) *Tension stiffener* - To transmit tensile forces applied to a web through a flange (Clause **509.7.8**).

f) *Torsion stiffener* - To provide torsional restraint to beams and girders at supports (Clause **509.7.9**).

The same stiffeners may perform more than one function and their design should comply with the requirements of all the functions designed for.

509.7.1.2 *Outstand of web stiffeners* - Unless the outer edge is continuously stiffened, the outstand from the face of the web should not exceed $20t_q\varepsilon$.

When the outstand is between $14t_q\varepsilon$ and $20t_q\varepsilon$, then the stiffener design should be on the basis of a core section with an outstand of $14t_q\varepsilon$.

where

t_q = thickness of the stiffener

509.7.1.3 Stiff bearing length - The stiff bearing length of any element b_1, is that length which cannot deform appreciably in bending. To determine b_1 the dispersion of load through a steel bearing element should be taken as 45^0 through solid material, such as bearing plates, flange plates etc. (**Fig. 10**)

509.7.1.4 Eccentricity - Where a load or reaction is applied eccentric to the centre line of the web or where the centroid of the stiffener does not lie on the centre line of the web, the resulting eccentricity of loading should be accounted for in the design of the stiffener.

509.7.1.5 Buckling resistance of stiffener - The buckling resistance F_{qd}, should be based on the design compressive stress f_{cd} (Clause **507.1.2.1**) of a strut (curve c), the radius of gyration being taken about the axis parallel to the web. The effective section is the full area or core area of the stiffener (Clause **509.7.1.2**) together with an effective length of web on each side of the centre line of the stiffeners, limited to 20 times the web thickness. The design strength used should be the minimum value obtained for buckling about the web or the stiffener.

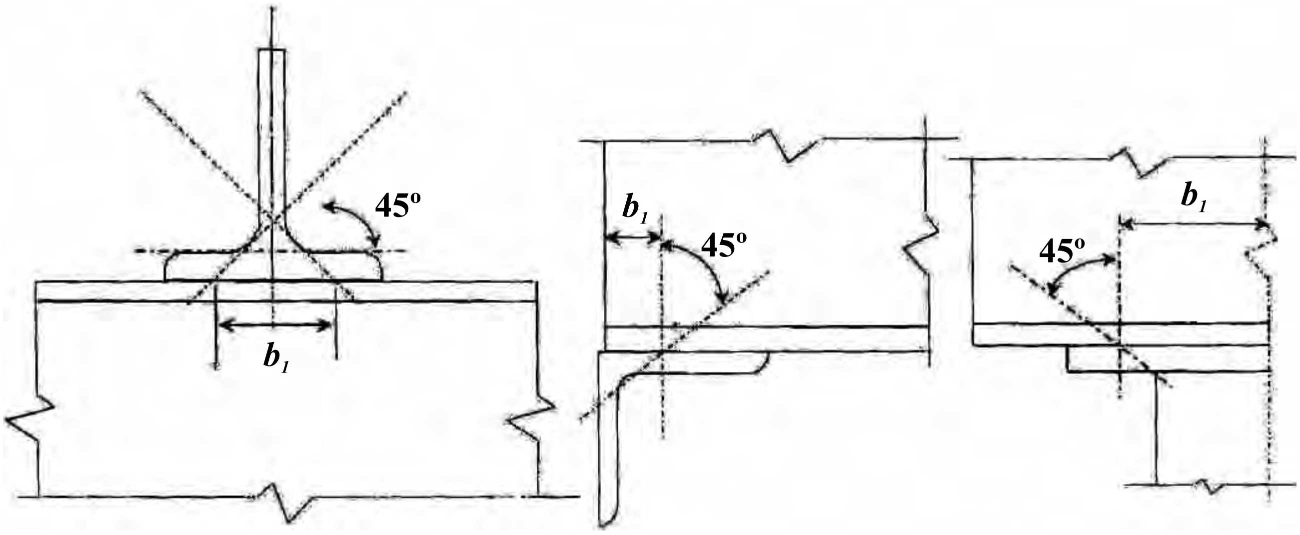


Fig. 10 Stiff Bearing Length, b_1

The effective length of intermediate transverse stiffeners used in calculating the buckling resistance, F_{qd} should be taken as 0.7 times the length, L, of the stiffener.

The effective length for load carrying web stiffeners used in calculating the buckling resistance, F_{xd}, assumes that the flange through which the load or reaction is applied is effectively restrained against lateral movement relative to the other flange, and length should be taken as:

a) $KL = 0.7\ L$ when flange restrained against rotation in the plane of the stiffener (by other structural elements)

b) $KL = L$ when flange not so restrained

where

L = length of the stiffener

If the load or reaction is applied to the flange by a compression member, then unless effective lateral restraint is provided at that point, the stiffener should be designed as part of the compression member applying the load, and the connection between the compression member and the beam flange shall be checked for the effects of the strut action.

509.7.2 *Design of intermediate transverse web stiffeners*

509.7.2.1 *General* - Intermediate transverse stiffeners may be provided on one or both sides of the web.

509.7.2.2 *Spacing* - Spacing of intermediate stiffeners, where they are provided, shall comply with Clause **509.6.1** depending on the thickness of the web.

509.7.2.3 *Outstand of Stifferers* - The outstand of the stiffeners should comply with Clause **509.7.1.2**.

509.7.2.4 *Minimum stiffeners* - Transverse web stiffeners not subject to external loads or moments should have a second moment of area, I_s about the centreline of the web (if stiffeners are on both sides of the web) and about the face of the web (if single stiffener on only one side of the web is used) such that:

if $c/d \geq \sqrt{2}$ $\quad I_s \geq 0.75\, dt_w^{\,3}$

and if $c/d < \sqrt{2}$ $\quad I_s \geq \dfrac{1.5d^3 t_w^3}{c^2}$

where

d = depth of the web

t_w = minimum required web thickness for spacing using tension field action, as given in Clause **509.4.2.1**.

c = actual stiffener spacing

509.7.2.5 *Buckling check on intermediate transverse web stiffeners* - Stiffeners not subjected to external loads of moments should be checked for a stiffener force:

$$F_q = V - V_{cr} / \gamma_{m0} \leq F_{qd}$$

where

F_{qd} = design resistance of the intermediate stiffeners

V = factored shear force adjacent to the stiffener

V_{cr} = shear buckling resistance of the web panel designed without using tension field action Clause **509.4.2.2(a)**.

Stiffeners subject to external loads and moments should meet the conditions for load carrying web stiffeners in Clause **509.7.3**. In addition they should satisfy the following interaction expression.

$$\frac{F_q - F_x}{F_{qd}} + \frac{F_x}{F_{xd}} + \frac{M_q}{M_{yq}} \leq 1$$

If $F_q < F_x$ then $(F_q – F_x)$ should be taken as zero.

where

F_q = stiffener force given above

F_{qd} = design resistance of an intermediate web stiffener corresponding to buckling about an axis parallel to the web (Clause **509.7.1.5**).

F_x = external load or reaction at the stiffener

F_{xd} = design resistance of a load carrying stiffener corresponding to buckling about axis parallel to the web (Clause **509.7.1.5**).

M_q = moment on the stiffener due to eccentrically applied load and transverse load, if any.

M_{yq} = yield moment capacity of the stiffener based on its elastic modulus about its centroidal axis parallel to the web.

509.7.2.6 Connection of intermediate stiffeners to web - Intermediate transverse stiffeners not subject to external loading should be connected to the web so as to withstand a shear between each component of the stiffener and the web (in kN/mm) of not less than

$$t^2_w / (5b_s)$$

where

t_w = web thickness (in mm)

b_s = outstand width of the stiffener (in mm)

For stiffeners subject to external loading, the shear between the web and the stiffener due to such loading has to be added to the above value.

Stiffeners not subject to external loads or moments may terminate clear of the tension flange and in such a situation the distance cut short from the line of the weld should not be more than $4t_w$.

509.7.3 *Load carrying stiffeners*

509.7.3.1 *Web Check* - Load carrying web stiffeners should be provided where compressive forces applied through a flange by loads or reactions exceed the buckling strength, F_{cdw}, of the unstiffened web, calculated using the following:

The effective length of the web for evaluating the slenderness ratios is calculated as in Clause **509.7.1.5**. The area of cross section is taken as

$$(b_1 + n_1)t_w$$

where

b_1 = width of stiff bearing on the flange (Clause **509.7.1.3**).

n_1 = dispersion of the load through the web at 45^0, to the level of half the depth of the cross section.

The buckling strength of this web about axis parallel to the web is calculated as given in Clause **507.1.2.1** using curve 'c'.

509.7.4 *Bearing stiffeners*

Bearing stiffeners should be provided for webs where forces applied through a flange by loads or reactions exceed the local capacity of the web at its connection to the flange, F_w as given below:

$$F_w = (b_1 + n_2)\, t_w f_{yw} / \gamma_{m0}$$

where

b_1 = stiff bearing length (Clause **509.7.1.3**)

n_2 = length obtained by dispersion through the flange to the web junction at a slope of 1:2.5 to the plane of the flange.

t_w = thickness of the web

f_{yw} = yield stress of the web

509.7.5 *Design of load carrying stiffeners*

509.7.5.1 *Buckling check* - The external load or reaction, F_x, on a stiffener should not exceed the buckling resistance, F_{xd}, of the stiffener as given in Clause **509.7.1.5**.

Where the stiffener also acts as an intermediate stiffener it should be checked for the effect of combined loads in accordance with Clause **509.7.2.5**.

509.7.5.2 *Bearing check* - Load carrying web stiffeners should also be of sufficient size that the bearing strength of the stiffener, F_{psd}, given below is not less than the load transferred, F_x.

$$F_{psd}, = A_q\, f_{yq} / (0.8\, \gamma_{m0}) \geq F_x$$

where

F_x = external load or reaction

A_q = area of the stiffener in contact with the flange

f_{yq} = yield stress of the stiffener

509.7.6 *Design of bearing stiffeners*

Bearing stiffeners should be designed for the applied load or reaction less the local capacity of the web as given in Clause **509.7.4**. Where the web and the stiffener material are of different strengths the lesser value should be assumed to calculate the capacity of the web and the stiffener. Bearing stiffeners should project nearly as much as the overhang of the flange through which load is transferred.

509.7.7 *Design of diagonal stiffeners*

Diagonal stiffeners should be designed to carry the portion of the applied shear and bearing that exceeds the capacity of the web.

Where the web and the stiffener are of different strengths, the value for design should be taken as given in Clause **509.7.6**.

509.7.8 *Design of tension stiffeners*

Tension stiffeners should be designed to carry the portion of the applied load or reaction less the capacity of the web as given in Clause **509.7.4** for bearing stiffeners.

Where the web and the stiffener are of different strengths, the value for design should be taken as given in Clause **509.7.6.**

509.7.9 *Torsional stiffeners*

Where bearing stiffeners are required to provide torsional restraint at the supports of the beam, they should meet the following criteria:

a) Conditions of Clause **509.7.4**

b) Second moment of area of the stiffener section about the centreline of the web, I_s, should be such that :

$$I_s \geq 0.34\ \alpha_s D^3 T_{cf}$$

where

α_s = 0.006 for $L_{LT}/r_y \leq 50$

= $0.3/(L_{LT}/r_y)$ for $50 < L_{LT}/r_y = 100$

= $30/(L_{LT}/r_y)^2$ for $L_{LT}/r_y > 100$

D = overall depth of beam at support

T_{cf} = maximum thickness of compression flange in the span under consideration

KL = laterally unsupported effective length of the compression flange of the beam

r_y = radius of gyration of the beam about the minor axis

509.7.10 *Connection of load carrying and bearing stiffeners to web* - Stiffeners, which resist loads or reactions applied through a flange, should be connected to web by sufficient welds or fasteners to transmit a design force equal to the lesser of:

a) The tension capacity of the stiffener

b) The sum, of the forces applied at the two ends of the stiffener when they act in the same direction or the larger of the forces when they act in opposite directions.

Stiffeners, which do not extend right across the web, should be of such length that the shear stress in the web due to the design force transmitted by the stiffener does not exceed the shear strength of the web. In addition, the capacity of the web beyond the end of the stiffener should be sufficient to resist the applied force.

509.7.11 *Connection to flanges*

509.7.11.1 *In tension* - Stiffeners required to resist tension should be connected to the flange transmitting the load by continuous welds or non-slip fasteners.

509.7.11.2 *In compression* - Stiffeners required to resist compression should either be fitted against the loaded flange or connected by continuous welds or non-slip fasteners.

The stiffener should be fitted against or connected to both flanges when:

a) a load is applied directly over a support; or

b) it forms the end stiffener of a stiffened web; or

c) it acts as a torsion stiffener

509.7.12 *Hollow sections* - Where concentrated loads are applied to hollow sections, consideration should be given to local stresses and deformations and the section reinforced as necessary.

509.7.13 *Longitudinal stiffeners* - Where longitudinal stiffeners are used in addition to transverse stiffeners, they shall be as follows :

a) One longitudinal stiffener shall be placed on the web at a distance from the compression flange equal to 1/5 of the distance from the compression flange angle, plate or tongue plate to the neutral axis when the thickness of the web is less than the limit specified in Clause **509.6.1**. The stiffener shall be designed so that I_s is not less that $4ct_w^3$ where I_s and t_w are as defined in Clause **509.7.2.4** and c is the actual distance between the transverse stiffeners.

b) A second longitudinal stiffener (single or double) shall be placed at the neutral axis of the girder when the thickness of the web is less than the limit specified in Clause **509.6.1**. This stiffener shall be designed so that I_s is not less that $d_2t_w^3$ where I_s and t_w are as defined in Clause **509.7.2.4** and d_2 is twice the clear distance from the compression flange angles, plates or tongue plates to the neutral axis.

c) Longitudinal web stiffeners shall extend between vertical stiffeners, but need not be continuous over them.

d) Longitudinal stiffeners may be in pairs arranged on each side of the web, or single located on one side of the web.

509.7.14 *Detailing requirements*

a) Load bearing stiffeners should be in pairs (that is two legs of plates, angles etc.) placed symmetrically at both sides of the web. When the condition is not met the effect of the resulting eccentricity should be considered.

b) The ends of the load bearing stiffener should be closely fitted or adequately connected to both flanges. They should be shaped to allow space for any root fillet or weld connecting the web to the flange, with a clearance not exceeding five times the thickness of the web.

c) Load bearing stiffeners shall not be joggled and shall be solidly packed throughout.

d) Outstanding legs of each pair of load bearing stiffeners shall be so proportioned that the bearing stress on that part of their area in contact with the flange and clear of the root of the flange or flange angles or clear of the flange welds, does not exceed the design bearing strength.

e) Load bearing stiffeners consisting of two legs shall be designed as struts assuming the section to consist of the pair of stiffeners together with a length of web on each side of the centre line of the stiffeners equal to twenty times the web thickness (but limited to the edge distance of the web and half the distance of the adjacent stiffener).

   In case of bearing stiffeners consisting of four or more legs, the effective stiffener section should be taken to comprise the stiffeners, the web plate between the two outer legs and a portion of web plate not exceeding the length of the web as specified for single leg stiffeners on the outer sides of the outer legs.

f) The load bearing stiffeners shall be provided with sufficient rivets, bolts or welds to transmit to the web the whole of the load in the stiffeners.

g) In no case shall the greater unsupported clear dimension of a web panel exceed 270 t nor the lesser unsupported clear dimension of the same panel exceed 180 t where t is the thickness of the web plate.

h) Where transverse stiffeners are required, they shall be provided throughout the length of the girder at a distance apart not greater than 1.5 d_1 and not less than 0.33 d_1, where d_1 is the depth as defined in clause **509.8**. Where longitudinal stiffeners are provided d_1 shall be taken as the clear distance between the horizontal stiffener and the farthest flange ignoring fillets.

NOTE : If the thickness of the web is made greater, or the spacing of stiffener made smaller than that required by the standard, the moment of inertia of the stiffener need not be correspondingly increased.

Intermediate transverse stiffeners, when not acting as load bearing stiffeners, may be joggled and may be single or in pairs placed one on each side of the web. Where single stiffeners are used, they should preferably be placed alternatively on opposite sides of the web. The stiffeners shall extend from flange to flange. They can be connected or fitted to, or kept well clear of the flanges.

509.8 Lateral Bracings

All spans shall be provided with a lateral bracing system extending from end to end of sufficient strength to transmit to the bearings all lateral forces due to wind, seismic effect etc, as applicable.

509.9 Expansion and Contraction

In all bridges, provision shall be made in the design to resist thermal stresses induced, or means shall be provided for movement caused by temperature changes. Provision shall also be made for changes in length of span resulting from live loads.

510 MEMBERS SUBJECTED TO COMBINED FORCES

510.1 General

This clause governs the design of members subjected to combined forces such as shear and bending, axial force and bending, or shear force, axial force and bending.

510.2 Combined Shear and Bending

510.2.1 No reduction in moment capacity of the section is necessary as long as the cross-section is not subjected to high shear force (factored value of applied shear force is less than or equal to 60 percent of the shear strength of the section as given in Clause **509.4**. The moment capacity may be taken as, M_d (Clause **509.2**) without any reduction.

510.2.2 When the factored value of the applied shear force is high (exceeds the limit in Clause **510.2.1**) the factored moment of the section should be less than the moment capacity of the section under higher shear force M_{dv} calculated as given below:

a) Plastic or compact section

$$M_{dv} = M_d - \beta (M_d - M_{fd}) \leq 1.2\, Z_e f_y / \gamma_{m0}$$

where

β = $(2V/V_d - 1)^2$

M_d = plastic design moment of the whole section disregarding high shear force effect (Clause **509.2.1.2**) considering web buckling effects (Clause **509.2.1.1**).

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

V_d = design shear strength as governed by web yielding or web buckling (Clause **509.4.1 or 509.4.2**)

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

Z_e = elastic section modulus of the whole section

b) Semi-compact Section

$$M_{dv} = Z_e f_y / \gamma_{m0}$$

510.3 Combined Axial Force and Bending Moment

Under combined axial force and bending moment section strength as governed by material failure and member strength as governed by buckling failure shall to be checked in accordance with Clauses **510.3.1** and **510.3.2**., respectively.

510.3.1 *Section strength*

510.3.1.1 Plastic and compact sections - In the design of members subjected to combined axial force (tension or compression) and bending moment, the following should be satisfied.

$$\left(\frac{M_y}{M_{ndy}}\right)^{a_1} + \left(\frac{M_z}{M_{ndz}}\right)^{a_2} \leq 1.0$$

Conservatively, the following equation may be used under combined axial force and bending moment.

$$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1.0$$

where

M_y, M_z	=	factored applied moments about the minor and major axis of the cross section, respectively
M_{ndy}, M_{ndz}	=	design reduced flexural strength under combined axial force and the respective uniaxial moment acting alone, (Clause **510.3.1.2**)
N	=	factored applied axial force (Tension T, or Compression P)
N_d	=	design strength in tension (T_d) as obtained from Clause **506** or in compression due to yielding given by : $N_d = A_g f_y / \gamma_{m0}$
M_{dy}, M_{dz}	=	design strength under corresponding moment acting alone, (Clause **509.2**)
A_g	=	gross area of the cross section
α_1, α_2	=	constants as given in **Table 14**
Y_{mo}	=	partial factor of safety in yielding

Table 14 Constants α_1 and α_2

Section	α_1	α_2
I and Channel	$5n \geq 1$	2
Circular tubes	2	2
Rectangular tubes	$1.66 / (1 = 1.13n^2) \leq 6$	$1.66 / (1 = 1.13n^2) \leq 6$
Solid rectangles	$1.73 + 1.8n^3$	$1.73 + 1.8n^3$

510.3.1.2 For plastic and compact sections without bolt holes, the following approximations may be used for evaluating M_{ndy} and M_{ndz}

a) *Plates*

$$M_{nd} = M_d\,(1 - n^2)$$

b) *Welded I or H section*

$$M_{ndy} = M_{dy}\left[1 - \left(\frac{n-a}{1-a}\right)^2\right] \leq M_{dy} \text{ where } n \geq a$$

$$M_{ndz} = M_{dz}\,(1 - n) / (1 - 0.5a) \leq M_{dz}$$

where $n = N/N_d$ and $a = (A - 2\,b\,t_f) / A \leq 0.5$

c) For standard I or H sections

for $n \leq 0.2$ $\quad M_{ndy} = M_{dy}$

for $n > 0.2$ $\quad M_{ndy} = 1.56\,M_{dy}\,(1 - n)(n + 0.6)$

$$M_{ndz} = 1.11\,M_{dz}\,(1 - n) \leq M_{dz}$$

d) *For rectangular hollow sections and welded box sections* - When the section is symmetric about both axes and without bolt holes

$$M_{ndy} = M_{dy}\,(1 - n) / (1 - 0.5a_f) \leq M_{dy}$$

$$M_{ndz} = M_{dz}\,(1 - n) / (1 - 0.5a_w) \leq M_{dz}$$

where $\quad a_w = (A - 2\,b\,t_f) / A \leq 0.5$

$$a_f = (A - 2\,h\,t_w) / A \leq 0.5$$

e) *Circular hollow tubes without bolt holes*

$$M_{nd} = 1.04\ M_d\ (1 - n^{1.7}) \leq M_d$$

510.3.1.3 *Semi-compact section* - In the absence of high shear force (Clause **510.2.1**) semi-compact section design is satisfactory under combined axial force and bending, if the maximum longitudinal stress under combined axial force and bending, f_x, satisfies the following criteria.

$$f_x \leq f_y / \gamma_{m0}$$

For cross section without holes, the above criteria reduces to

$$\frac{N}{N_d} + \frac{M_y}{M_{dy}} + \frac{M_z}{M_{dz}} \leq 1.0$$

where

N_d, M_{dy}, M_{dz} are defined in Clause **510.3.1.1**

510.3.2 *Overall member strength*

Members subjected to combined axial force and bending moment shall be checked for overall buckling failure as given in this section.

510.3.2.1 *Bending and axial tension* - The reduced effective moment, M_{eff}, under tension and bending calculated as given below, should not exceed the bending strength due to lateral torsional buckling, M_d (Clause **509.2.2**)

$$M_{eff} = [M - \psi\ T\ Z_{ec} / A] \leq M_d$$

where

M, T = factored applied moment and tension, respectively

A = area of cross-section

Z_{ec} = elastic section modulus of the section with respect to extreme compression fibre

ψ = 0.8 if *T* and *M* vary independently or otherwise

= 1.0

510.3.2.2 *Bending and axial compression* - Members subjected to combined axial compression and biaxial bending shall satisfy the following interaction relationships.

$$\frac{P}{P_{dy}} + k_y \frac{C_{my} M_y}{M_{dy}} + k_{LT} \frac{M_z}{M_{dz}} \leq 1.0$$

$$\frac{P}{P_{dz}} + 0.6 k_y \frac{C_{my} M_y}{M_{dy}} + k_z \frac{C_{mz} M_z}{M_{dz}} \leq 1.0$$

where

C_{my}, C_{mz} = equivalent uniform moment factor as per **Table 15**

P = applied axial compression under factored load

M_y , M_z = maximum factored applied bending moments about y and z-axis of the member, respectively.

P_{dy} , P_{dz} = design strength under axial compression as governed by buckling about minor (y) and major (z) axis respectively.

M_{dy} , M_{dz} = design bending strength about y (minor) or z (major) axis of the cross section (Clause **509**)

$$k_y = 1 + (\lambda_y - 0.2)\, n_y \leq 1 + 0.8\, n_y$$

$$k_z = 1 + (\lambda_z - 0.2)\, n_z \leq 1 + 0.8\, n_z$$

$$k_{LT} = 1 - \frac{0.1 \lambda_{LT} n_y}{(C_{mLT} - 0.25)} \geq 1 - \frac{0.1 n_y}{(C_{mLT} - 0.25)}$$

where

n_y , n_z = ratio of actual applied axial force to the design axial strength for buckling about the y and z axis, respectively and

C_{mLT} = equivalent uniform moment factor for lateral torsional buckling as per Table 15 corresponding to the actual moment gradient between lateral supports against torsional deformation in the critical region under consideration.

511 FATIGUE

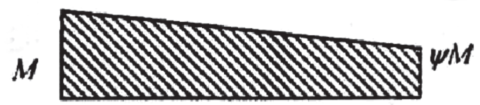
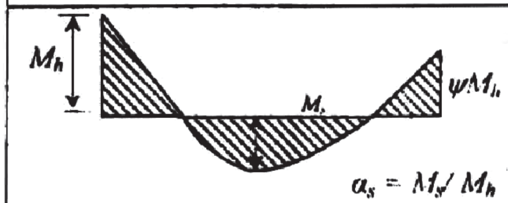
511.1 General

This section applies to the design of structures and structural elements subject to loading which could lead to fatigue failure. This shall, however, not cover the following :

a) Corrosion fatigue

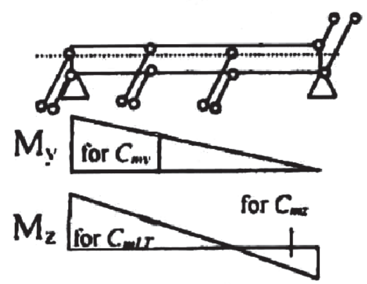
Table 15 Equivalent Uniform Moment Factor

(Clause **510.3.2.2**)

Bending moment diagram	Range		C_{my}, C_{mz}, C_{mLT}	
			Uniform loading	*Concentrated load*
	$-1 \le \psi \le 1$		$0.6 + 0.4\,\psi \ge 0.4$	
	$0 \le \alpha_s \le 1$	$-1 \le \psi \le 1$	$0.2 + 0.8\,\alpha_s \ge 0.4$	$0.2 + 0.8\,\alpha_s \ge 0.4$
	$-1 \le \alpha_s \le 0$	$0 \le \psi \le 1$	$0.1 - 0.8\,\alpha_s \ge 0.4$	$-0.8\,\alpha_s \ge 0.4$
		$-1 \le \psi \le 0$	$0.1(1-\psi) - 0.8\,\alpha_s \ge 0.4$	$0.2(1-\psi) - 0.8\,\alpha_s \ge 0.4$
	$0 \le \alpha_h \le 1$	$-1 \le \psi \le 1$	$0.095 - 0.05\,\alpha_h$	$0.90 + 0.10\,\alpha_h$
	$-1 \le \alpha_h \le 0$	$0 \le \psi \le 1$	$0.095 + 0.05\,\alpha_h$	$0.90 + 0.10\,\alpha_h$
		$-1 \le \psi \le 0$	$0.95 + 0.05\,\alpha_h\,(1+2\,\psi)$	$0.90 + 0.05\,\alpha_h\,(1+2\,\psi)$
For members with sway buckling mode the equivalent uniform moment factor $C_{my} = C_{mz} = 0.9$.				

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

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



b) Low cycle (high stress) fatigue

c) Thermal fatigue

d) Stress corrosion cracking

e) Effects of high temperature (>150^{o}C)

f) Effects of low temperature (< brittle transition temperature)

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

511.2 Design

511.2.1 *Reference design conditions*

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 transverse fillet or butt weld connects plates of thickness not greater than 25 mm.

g) As far as possible, holes should preferably be avoided in members and connections subjected to fatigue.

Fatigue need not be investigated if condition in Clauses **511.2.2.3, 511.5.1, or 511.6** is satisfied.

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, as 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 non-destructive testing.

511.2.2 *Design spectrum*

511.2.2.1 *Stress evaluation* - The design stress shall be determined by elastic analysis of the structure to obtain stress resultants and the local stresses may be obtained by a conventional stress analysis method. The normal and shear stresses shall be determined considering all design loads on the members, but excluding stress concentration due to the geometry of the detail. The stress concentration effect is accounted for in detail category classification (**Table 19**). The stress concentration, however, not characteristic of the detail shall be accounted for separately in the stress calculation.

In the fatigue design of trusses made of members with open sections in which the end connections are not pinned, the stresses due to secondary bending moments shall be taken into account unless the slenderness ratio, (KL/r), of the member is greater than 40.

In the determination of stress range at the end connections between hollow sections, the effect of connection stiffness and eccentricities may be disregarded, provided:

a) The calculated stress range is multiplied by appropriate factor given in **Table 16** in the case of circular hollow section connections and **Table 17** in the case of rectangular hollow section connections.

b) The design throat thickness of fillet welds in the joints is greater than the wall thickness of the connected member.

511.2.2.2 *Design stress spectrum* - In the case of loading events producing non-uniform stress range cycle, the stress spectrum may be obtained by a rational method.

Table 16 Multiplying Factors for Calculated Stress Range (Circular Hollow Sections)

(Clause **511.2.2.1**)

Types of Connection		Chords	Verticals	Diagonals
Gap connections	K type	1.5	1.0	1.3
	N type	1.5	1.8	1.4
Overlap connections	K type	1.5	1.0	1.2
	N type	1.5	1.65	1.25

Table 17 Multiplying Factors for Calculated Stress Range (Rectangular Hollow Sections)

(Clause **511.2.2.1**)

Types of Joint		Chords	Verticals	Diagonals
Gap connections	K type	1.5	1.0	1.5
	N type	1.5	2.2	1.6
Overlap connections	K type	1.5	1.0	1.3
	N type	1.5	2.0	1.4

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

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

Or if the actual number of stress cycles, N_{sc}, satisfies

$$N_{SC} < 5 \text{ X } 10^6 \left(\frac{27/\gamma_{mft}}{\gamma_{fft} f} \right)^3$$

where

γ_{mft}, γ_{fft} = partial safety factors for strength and load, respectively (See Clause **511.2.3**)

f = actual fatigue stress range for the detail

511.2.3 *Partial safety factors*

511.2.3.1 *Partial safety factor for loads and their effects (γ_{fft})* - Unless and otherwise the uncertainty in the estimation of the applied loads and their effects demand a higher value, the partial safety factor for loads in the evaluation of stress range in fatigue design shall be taken as 1.0.

511.2.3.2 *Partial safety factor for fatigue strength (γ_{mft})* - Partial safety factor for strength is influenced by consequences of fatigue damage and level of inspection capabilities.

511.2.3.3 Based on consequences of fatigue failure, component details have been classified as given in the **Table 18** and the corresponding partial safety factor for fatigue strength shall be used.

a) 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 structure due to availability of alternative load path (redundant system).

b) Non-fail-safe structural component/detail is the one where local failure of one component leads rapidly to failure of the structure due to its non-redundant nature.

Table 18 Partial Safety Factors for Fatigue Strength (γ_{mft})

(Clause **511.2.3.3**)

Inspection and Access	**Consequence of failure**	
	Fail-safe	**Non-fail-safe**
Periodic inspection, maintenance and accessibility to detail is good	1.00	1.25
Periodic inspection, maintenance and accessibility to detail is poor	1.15	1.35

511.3 **Detail Category**

Tables 19 (a) to (d) indicate the classification of different details into various categories for the purpose of assessing fatigue strength. Details not classified in the table may be treated as the lowest detail category of a similar detail, unless superior fatigue strength is proved by testing and/or analysis.

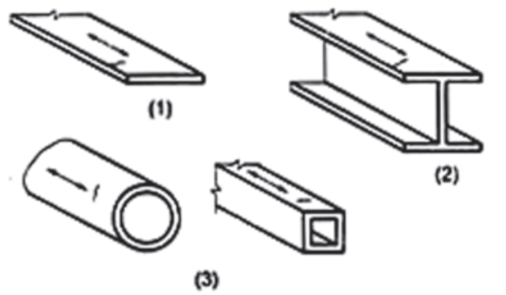
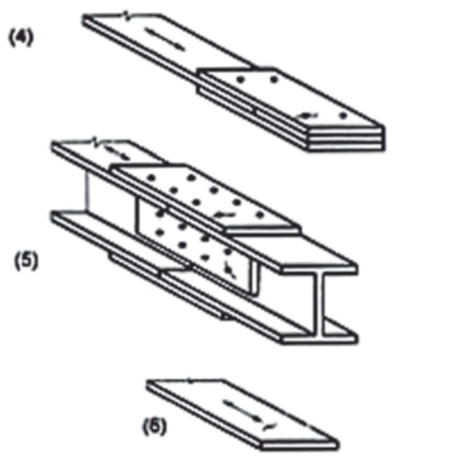
Holes in members and connections subjected to fatigue loading shall not be made :

a) Using punching in plates having thickness greater than 12 mm unless the holes are sub-punched and subsequently reamed to remove the affected material around the punched hole, and

b) Using gas cutting unless the holes are reamed to remove the material in the heat affected zone.

Table 19 (a) Detail Category Classification Group 1 Non-welded Details

(Clauses **511.2.2.1** and **511.3**)

Sl No.	Detail Category	Constructional Details	
		Illustration (see Note)	Description
(1)	(2)	(3)	(4)
i)	118		**Rolled and extruded products** i) Plates and flats (1) ii) Rolled sections (2) iii) Scamless tubes (3) Sharp edges, surface and rolling flaws to be removed by grinding in the direction of applied stress.
ii)	103		**Bolted connections** (4) and (5): 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** (6): All hardened material and visible signs of edge discontinuities to be removed by machining or grinding in the direction of applied stress.
iii)	92		**Material with machine gas-cut edges with draglines or manual gas-cut material** (7) : Corners and visible signs of edge discontinuities to be removed by grinding in the direction of the applied stress.

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 19 (b) Detail Category Classification Group 2 Welded Details-not in Hollow Sections

(Clauses **511.2.2.1** and **511.3**)

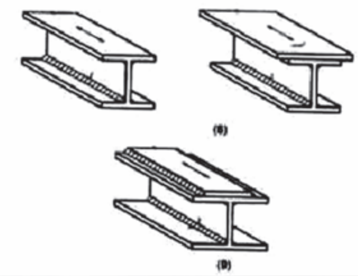
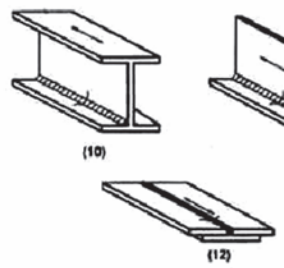
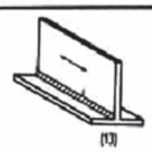
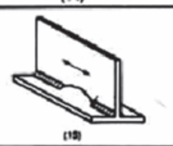
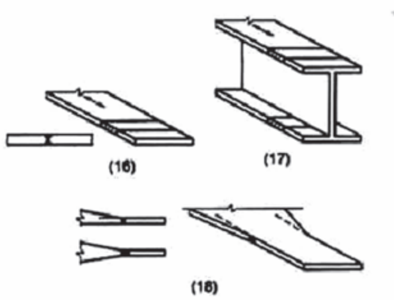
Sl No.	Detail Category	Constructional Details: Illustration (*see* Note)	Constructional Details: Description
(1)	(2)	(3)	(4)
i)	92		**Welded plate I-section and box girders with continuous longitudinal welds** (8) & (9) : Zones of continuous automatic longitudinal fillet or butt welds carried out from both sides and all welds not having un-repaired stop-start positions.
ii)	83		**Welded plate I-section and box girders with continuous longitudinal welds** (10) & (11) : 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 postitions. (12) : Zones of continuous longitudinal fillet or butt welds carried out from both sides but containing stop-start positions. For continuous manual longitudinal fillet or but welds carried out from both sides, use Detail Category 92.
iii)	66		**Welded plate I-section and box girders with continuous longitudinal welds** (13) : Zones of continuous longitudinal welds carried out from one side only, with or without stop-start positions.
iv)	59		**Intermittient longitudinal welds** (14) : Zones of intermittent longitudinal welds
v)	52		**Intermittient longitudinal welds** (15) : Zones containing cope holes in longitudinally welded T-joints. Cope hole not to be filled with weld.
vi)	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. (16) : 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. (17) : Plate girders welded as in (16) before assembly. (18) : Transverse splices as in (16) with reduced or tapered transition with taper ≤1:4

Table 19 (b) (continued)

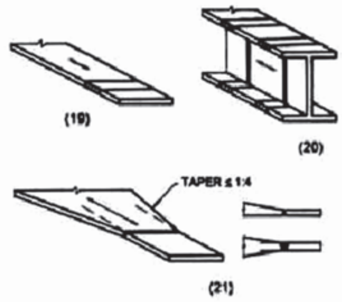
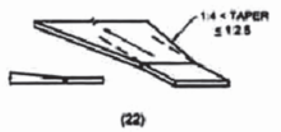
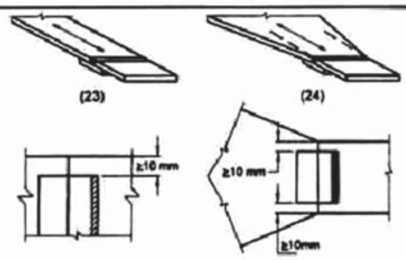
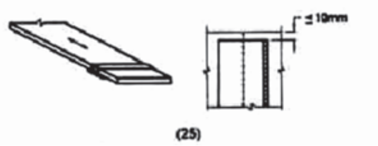
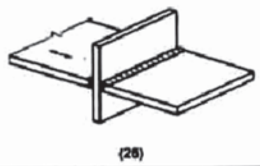
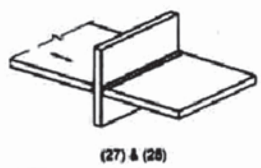
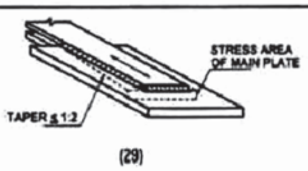
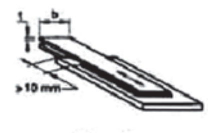
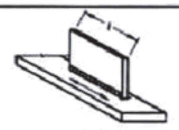
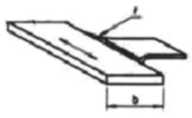
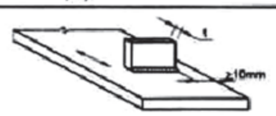
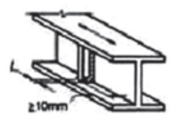
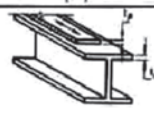
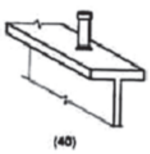
Sl No. (1)	Detail Category (2)	Constructional Details: Illustration (*see* Note) (3)	Constructional Details: Description (4)
vii)	66		**Transverse butt welds (complete penetration)** welds run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from two sides. (19) : Transverse splices of plates, rolled sections or plate girders. (20) : Transverse splice of rolled section or welded plate girders, without cope hole. With cope hole use Detail Category 52, as in (15). (21) : Transverse splices in plates or flats being tapered in width or in thickness where the taper is ≤1:4
viii)	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. (22) : Transverse splices as in (21) with taper in width or thickness >1:4 but ≤1:2:5.
ix)	52		**Transverse butt welds (complete penetration)** (23) : Transverse butt-welded splices made on a backing bar. The end of the fillet weld of the backing strip shall stop short by more than 10 mm from the edges of the stressed plate. (24) : Transverse butt welds as per (23) with taper on width or thickness <1:2:5.
x)	37		**Transverse butt welds (complete penetration)** (23) : Transverse butt welds as in (23) where fillet welds end closer than 10 mm to plate edge.
xi)	52		**Cruciform joints with load-carrying welds** (26) : 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.
xii)	41 (27) 27 (28)		(27) : Partial penetration or fillet welds with stress range calculated on plate area. (28) : Partial penetration or fillet welds with stress range calculated on throat area of weld.
xiii)	46		**Overlapped welded joints** (29) : 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 illustraton.

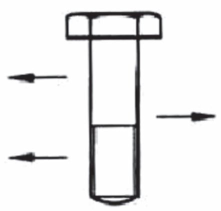
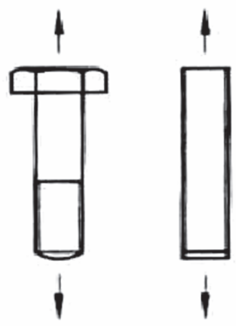
Table 19 (b) (continued)

Sl No. (1)	Detail Category (2)			Constructional Details: Illustration (*see* Note) (3)	Constructional Details: Description (4)
xiv)	41	(30)			**Overlapped weld joints** (30) : Fillet welded lap joint, with welds and main plate both having a design capacity greater than the overlapping elements.
xv)	33	(31)		(30) & (31)	(31) : Fillet welded lap joint, with main plate and overlapping elements both having a design capacity greater than the weld.
xvi)		(32)	(33)	 (32)  (33)	**Welded attachments (non-load carrying welds)–Longitudinal welds** (32) : Longitudinal fillet welds. Class of detail varies according to the length of attachment weld as noted. (33) : Gussct 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.
	66	—	$1/3 \leq r/b$		
	59	$l \leq 50$ mm	—		
	52	$50 < l \leq 100$ mm	$1/6 \leq r/b < 1/3$		
	37	100 mm $< l$	—		
	33	—	$r/b < 1/6$		
xvii)	59			 (34)	**Welded attachments** (34) : Shear connectors on base material (failure in base material).
xviii)	59	$t \leq 12$ mm		 (35)  (36)  (37)	**Transverse welds** (35) : Transverse fillet welds with the end of the weld ≥ 10 mm from the edge of the plate. (36) : 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. (37) : Diaphragms of box girders welded to the flange or web by continuous or intermittent welds.
	52	$t > 12$ mm			
xix)	37	t_f or t_p ≤25 mm		 (38)	**Cover plates in beams and plate girders** (38) : 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 t_p >25 mm			
xx)	67			 (40)  (39)	**Welds loaded in shear** (39) : Fillet welds transmitting shear. Stress range to be calculated on weld throat area. (40) : 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).

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 19 (c) Detail Category Classification Group 3 Bolts

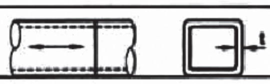
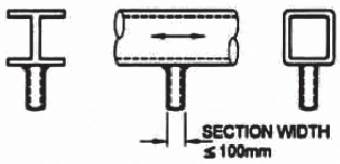
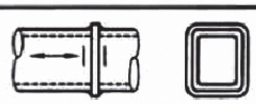
(Clauses **511.2.2.1** and **511.3**)

Sl No.	Detail Category	Constructional Details	
		Illustration (*see* Note)	Description
(1)	(2)	(3)	(4)
i)	83	 **(41)**	**Bolts in shear (8.8/TB bolting category only)** (41) : Shear stress range calculated on the minor diameter area of the bolt (*Ac*). 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.
ii)	27	 **(42)**	**Bolts and threaded rods in tension (tensile stress to be calculated on the tensile stress area, A_1)** (42) : 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 stress acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

Table 19 (d) Detail Category Classification Group : 4 Welded Details - In Hollow Sections

(Clauses **511.2.2.1** and **511.3**)

Sl No.	Detail Category	Constructional Details	
		Illustration (*see* Note)	Description
(1)	(2)	(3)	(4)
i)	103	 (43)	**Continuous automatic longitudinal welds** (43) : No stop-starts, or as manufactured, proven free to detachable discontinuities.
ii)	66 ($t \geq 8$ mm) 52 ($t < 8$ mm)	 (44)	**Transverse butt welds** (44) : Butt-welded end-to-end connection of circular hollow sections. NOTE — Height of the weld reinforcement less than 10 percent of weld with smooth transition to the plate surface. Welds made in flat position and proven free to detachable discontinuities.
iii)	52 ($t \geq 8$ mm) 41 ($t < 8$ mm)	 (45)	(45) : Butt-welded end-to-end connection of rectangular hollow sections
iv)	41 ($t \geq 8$ mm) 37 ($t < 8$ mm)	 (46)	**Butt welds to intermediate plate** (46) : Circular hollow sections, end-to-end butt-welded with an intermediate plate.
v)	37 ($t \geq 8$ mm) 30 ($t < 8$ mm)	 (47)	(47) Rectangular hollow sections, end-to-end butt welded with an intermediate plate
vi)	52	 (48)	**Welded attachments (non-load-carrying)** (48) : Circular or rectangular hollow section, fillet welded to another section. Section width parallel to stress direction ≤100 mm.
vii)	33 ($t < 8$ mm) 29 ($t < 8$ mm)	 (49)	**Fillet welds to intermediate plate** (49) : Circular hollow sections, end-to-end fillet welded with an intermediate plate.
viii)	29 ($t \geq 8$ mm) 27 ($t < 8$ mm)	 (50)	(50) : Rectangular hollow sections, end-to-end fillet welded with an intermediate plate.

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.

511.4 Fatigue Strength

The fatigue strength of the standard detail for the normal or shear fatigue stress range, not corrected for effects discussed in Clause **511.2.1** is given below (**Figs. 11 & 12**)

a) *Normal stress range*

when $N_{SC} \leq 5 \times 10^6$

$$f_f = f_{fn} \sqrt[3]{5x10^6 / N_{SC}}$$

when $5 \times 10^6 \leq N_{SC} \leq 10^8$

$$f_f = f_{fn} \sqrt[5]{5x10^6 / N_{SC}}$$

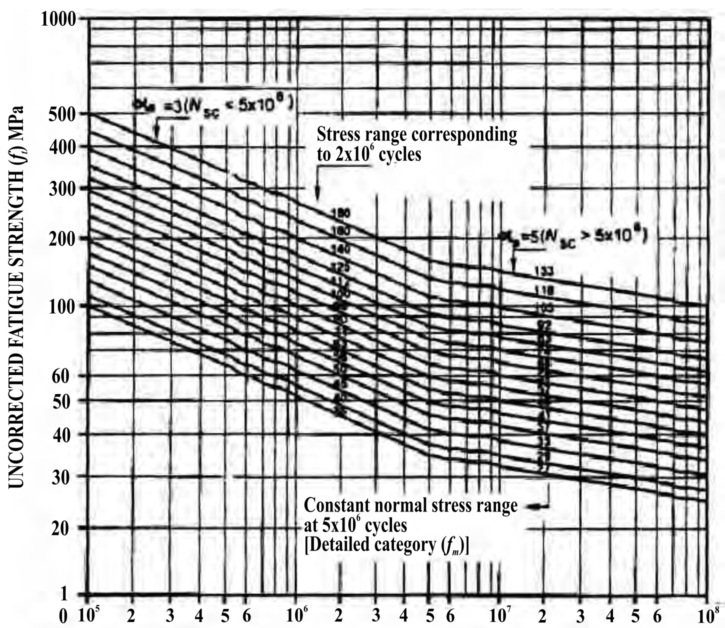
b) *Shear stress*

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

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} = normal and shear fatigue strength of the detail for 5 x 10^6 cycles, for the detail category (**Table 19**)



Number Of Stress Cycles (N_{SC})

Fig. 11 S-N Curve for Normal Stress

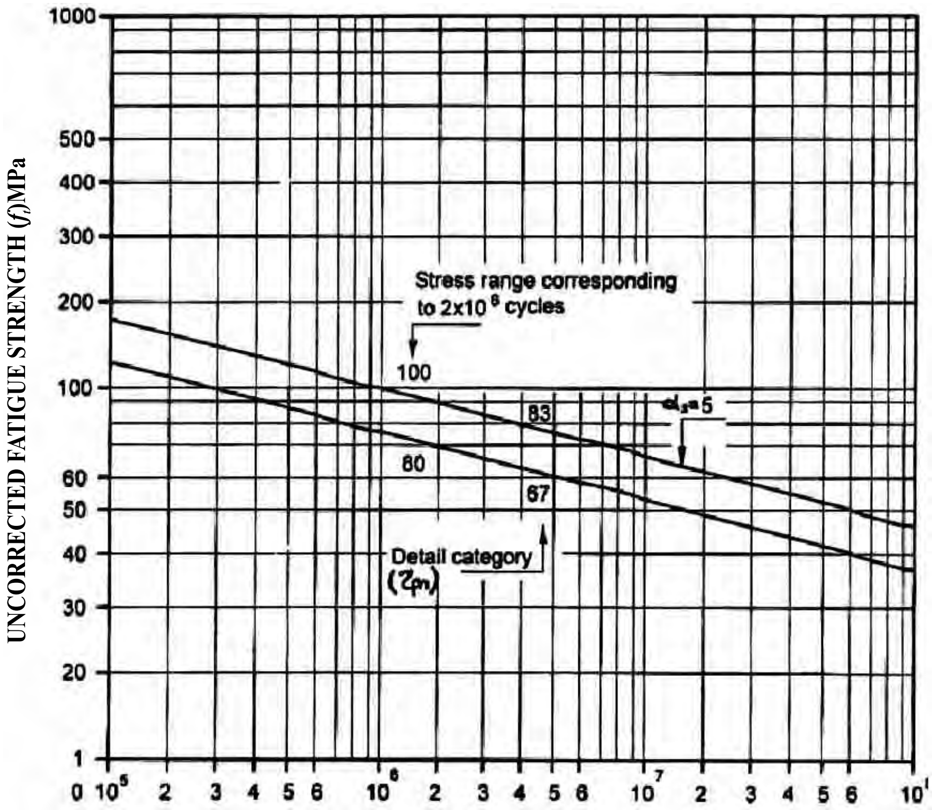


Fig. 12 S-N Curve for Shear Stress

511.5 Fatigue Assessment

The design fatigue strength for N_{SC} life cycles (f_{fd} , τ_{fd}) may be obtained from the standard fatigue strength for N_{SC} cycles by multiplying with correction factor, μ_r, for inspection level and thickness, as discussed in Clause **511.2.1** and dividing by partial safety factor given in **Table 18**.

511.5.1 *Exemptions* - At any point in a structure if the actual normal and shear stress range f, are less than the design fatigue strength range corresponding to 5 x 10^6 cycles, with appropriate partial safety factor, no further assessment for fatigue is necessary at that point.

511.5.2 *Stress limitations*

511.5.2.1 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.

511.5.2.2 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.

511.5.2.3 *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.

$$f \le f_{fd} = \mu_r f_f / \gamma_{mft}$$

$$\tau \le \tau_{fd} = \mu_r \tau_f / \gamma_{mft}$$

where

μ_r = correction factor (Clause **511.2.1**)

γ_{mft} = partial safety factor against fatigue failure, given in **Table 18**

f_f, τ_f = normal and shear fatigue strength ranges for the actual life cycle, N_{SC}, obtained from Clause **511.4**.

511.5 .2.4 *Variable stress range* - Fatigue assessment at any point in a structure, wherein variable stress ranges f_{ft} or τ_{ft} for n_i number of cycles (i = 1 to r) are encountered, shall satisfy the following:

a) For normal stress (f)

$$\frac{\sum_{i=1}^{r5} n_i f_i^3}{5\times10^6(\mu f_{fn}/\gamma_{mft})^3} + \frac{\sum_{j=y5}^{r} f_j^5}{5\times10^6(\mu f_{fn}/\gamma_{mft})^5} \le 1.0$$

b) For shear stresses (τ)

$$\sum_{j=1}^{r} n_i \tau_{ft}^{\ 5} \le 5\times10^6(\mu\tau_{fn}/\gamma_{mft})^5$$

where γ_5 is the summation upper limit of all the normal stress ranges (f_i) having magnitude lesser than ($\mu_c\ f_{fn}/\gamma_{mft}$) for that detail and the lower limit of all the normal stress ranges (f_j) having magnitude greater than ($\mu_c f_{fn}/\gamma_{mft}$) for the detail. In the above summation all normal stress ranges, f_i and τ_i having magnitude less than 0.55 $\mu_c f_{fn}$ and 0.55 $\mu_c \tau_{fn}$ may be disregarded.

511.6 Necessity for Fatigue Assessment

No fatigue assessment is necessary if any of the following conditions is satisfied.

a) The highest normal stress range f_{fmax} satisfies

$$f_{fmax} \le 27\,\mu_c/\gamma_{mft}$$

b) The highest shear stress range τ_{fmax} satisfies

$$\tau_{fmax} \leq 67 \ \mu_c / \gamma_{mft}$$

c) The total number of actual stress cycles N_{SC}, satisfies

$$N_{SC} \leq 5 \times 10^6 \left(\frac{27 \mu_c}{\gamma_{mft} f_{feq}} \right)^3$$

Where f_{feq} = equivalent constant amplitude stress range in Mpa given by

$$f_{feq} = \left[\frac{\sum_{i=1}^{\gamma_5} n_i f_{ft}^3 + \sum_{j=\gamma_5}^{r} n_i f_{ft}^5}{n} \right]^{1/3}$$

where

$$n = \sum_{j=1}^{\gamma} n_i$$

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

512 CONNECTIONS

512.1 General

512.1.1 The term "connection" applies to all joints between different components of a structural member, joints between separate structural members and splices in members.

The term "fasteners" applies to bolts, rivets and pins.

512.2 Basis of Design

512.2.1 All connections should satisfy the provisions of Clause **512** for the ultimate limit state.

512.2.2 The fatigue consideration should be in accordance with the recommendations of Clause **511** of this Code.

512.2.3 The connections in a structure shall be designed so as to be consistent with the assumptions made in the analysis of the structure and comply with the requirement specified in this Clause. Connections shall be capable of transmitting the calculated design loads and moments communicated by the members.

512.2.4 Where members are connected to the surface of a web or flange of a section, the ability of the web or flange to transfer the applied forces locally should be checked and local stiffening provided where necessary.

512.2.5 Ease of fabrication and erection, as also subsequent inspection and maintenance should be considered in the design of connections.

512.2.6 In general, use of different forms of fasteners to transfer the same force shall be avoided. However, in any connection which is made with more than one type of fastening, only rivets and turned and fitted bolts may be considered as acting together to share the load. In all other connections sufficient number of one type of fastening shall be provided to transmit the entire load for which the connection is designed.

512.2.7 The partial safety factor in the evaluation of design strength of connections shall be taken as given in **Table 1**.

512.3 Alignment of Members

The centroidal axes of members meeting at a joint or at a splice should preferably meet at a point. When this is not the case, the moment on the connection due to any eccentricity should be taken into account.

512.4 Welded Connections

512.4.1 Welds shall conform to IS 816 and IS 9595 as appropriate.

512.4.2 *Types of welds*

The following types of welds can be used :

a) Continuous full penetration or partial penetration butt welds.

b) Continuous or intermittent fillet welds,

c) Plug welds

Intermittent butt welds shall not be used.

Partial penetration butt welds shall not be used for transmitting tensile forces or bending moments along longitudinal axis of the welds. Plug welds shall not be used for transmitting loads or moments and shall be used only to prevent the buckling or separation of lapped parts or to joint components of built-up members.

512.4.3 *Strength of weld*

512.4.3.1 *Butt weld*

The strength of a full penetration butt weld shall be taken as equal to the strength of the weaker of the parts joined provided the yield stress of the weld metal is atleast equal to that of the parent metal.

The strength of a partial penetration butt weld together with its reinforcing fillet weld, if any, shall be calculated as for a full penetration fillet weld. The throat thickness shall be taken as

a) the depth of weld preparation where this is of the J or U type.

b) the depth of weld preparation minus 3 mm where the preparation is the V or bevel type.

512.4.3.2 *Fillet weld*

The strength of a fillet weld shall be based on the effective throat thickness and the effective length.

The effective throat thickness shall be considered as the height of a triangle that can be inscribed within the weld and measured perpendicular to its outer side.

The effective length shall be considered as the actual length minus twice the leg length. In case of fillet welds with end returns as per Clause **512.4.4.1** the effective length shall be considered as the actual length.

512.4.4 *General requirements of welds*

512.4.4.1 *Fillet welds*

Maximum leg length of a fillet weld shall be 1 mm less than the thickness of the connected parts at the edge.

Minimum leg length of a fillet weld shall be in accordance with IS 9595 Intermittent fillet welds.

Intermittent fillet weld should not be used at locations where they could result in the possible formation of rust pockets. Where the connection is protected from weather, e.g. in the interior of box sections, intermittent fillet welds are permitted.

The clear unconnected gap between the ends of the welds whether in line or staggered shall not be more than 200 mm and also shall not be more than -

a) 12 times the thickness of the thinner part when the part is in compression

b) 16 times the thickness of the thinner part when the part is in tension

c) One-quarter of the distance between stiffeners when used to connect stiffeners to a plate or other part subject to compression or shear.

In a line of intermittent welds, there shall be a weld at each end of the part connected.

In built-up members in which plates are connected by intermittent welds, continuous side fillet welds shall be provided at the ends of each side of the plate for a length at least equal to three quarter of the width of the narrower plate concerned. In exceptional cases, where this is not possible, the intermittent plug or slot weld shall be provided to prevent separation.

End returns

The fillet weld shall be returned continuously around the corner at the end of the side of a part for a length beyond the corner of not less than twice the leg length of the weld.

End connections by side fillets

If side fillets alone are used in end connections, both sides of the part shall be welded and the length of the weld on each side shall not be less than the distance between the welds nor less than 4 times the thickness of the thinnest part connected. Where the distance between the welds exceeds 16 times the thickness of thinnest part connected, intermediate plug or slot welds shall be used to prevent separation.

End connections by transverse welds

The overlap between the connected parts shall not be less than four times thickness of the thinnest part and the parts shall be connected by two transverse lines of welds. Where the distance between the weld exceeds 16 times the thickness of the thinnest part connected intermediate slot or plug welds shall be used to prevent separation.

Welds with packings

Where two parts connected by welding are separated by packing having thickness less than the leg length of a weld necessary to transmit the force, the required leg length will be increased by thickness of the packing. The packing shall be trimmed flush with the edge of the part which is to be welded. Where two parts connected by welding are separated by packing having a thickness equal to or greater than the leg length of weld necessary to transmit force, each of the parts shall be connected to the packing by a weld capable of transmitting the design force.

Welds in holes and slots

Fillets welds in holes or slots may be used to transmit shear in lap joints or to prevent the buckling or separation of the lapped parts or to join components of built-up members.

512.4.4.2 T butt joints

Butt welds in T joints shall be completed by means of fillet welds each having a size of not less than 25 percent of the thickness of the outstanding part.

512.4.4.3 Plug welds

The entire area of the hole or slot shall be filled with weld metal having a thickness

a) equal to the thickness of the holed or slotted part where it is 16 mm or less.

b) In other cases, not less than any of the following :

   1) 16 mm

   2) 0.45 times the diameter of the hole or the width of the slot.

   3) One-tenth of the length of slot but not greater than the thickness of the holed or slotted part.

The diameter of the hole or the width of a slot shall not be less than the thickness of the hole or slotted parts plus 8 mm.

The distance between centres of holes or between the centre lines of slots shall not be less than four times the diameter of the hole or the width of the slot. The distance between the centres of the slots measured in the direction of their length shall not be less than double the length of the slot.

The ends of the slot shall be semicircular except where the slot terminates at the edges of the part where it can be square.

512.4.4.4 Welding procedure

The welding procedure and details shall be in accordance with IS 9595 unless otherwise stipulated in this Clause.

512.4.5 *Design stresses in welds*

512.4.5.1 *Shop welds*

512.4.5.1.1 *Fillet welds* - Design strength of a fillet weld, f_{wd} shall be based on its throat area.

$$f_{wd} = f_{wn} / \gamma_{mw}$$

where

f_{wn} = $f_u / \sqrt{3}$

f_u = smaller of the ultimate stress of the weld or the parent metal

γ_{mw} = partial safety factor (**Table 1**)

512.4.5.1.2 *Butt welds* - Butt welds shall be treated as parent metal with a thickness equal to the throat thickness, and the stresses shall not exceed those permitted in the parent metal.

512.4.5.1.3 *Slot or plug welds* - The design shear stress of slot or plug welds shall be as per Clause **512.4.5.1.1.**

512.4.5.2 *Site welds* - The design strength in shear and tension for site welds made during erection of structural members shall be calculated as per Clause **512.4.5.1**, using appropriate partial safety factor as per **Table 1**.

512.4.5.3 *Long joints* - When the length of the welded joint, l_j , of a splice or end connection in a compression or tension element is greater than 150 t_t , the design capacity of weld (Clause **512.4.5.1.1**), f_{wd}, shall be reduced by the factor

$$\beta_{tw} = 1.2 - \frac{0.2 l_f}{150 t_t} \leq 1.0$$

where

l_f = length of the joint in the direction of the force transfer

t_t = throat size of the weld

512.4.6 *Fillet weld applied to the edge of a plate or section*

512.4.6.1 Where a fillet weld is applied to the square edge of a part, the specified size of the weld should generally be at least 1.5 mm less than the edge thickness in order to avoid washing down of the exposed areas (**Fig. 13a**).

512.4.6.2 Where a fillet weld is applied to the rounded toe of a rolled section, the specified size of the weld should generally not exceed ¾ of the thickness of the section at the toe (**Fig. 13b**).

512.4.6.3 Where the size specified for a fillet weld is such that the parent metal will not project beyond the weld, no melting of the outer cover or covers shall be allowed to occur to such an extent as to reduce the throat thickness (**Fig. 14**).

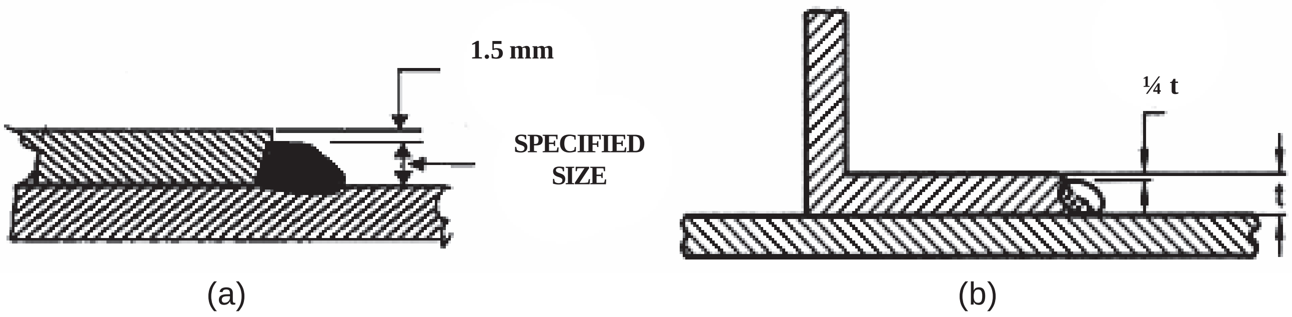


Fig. 13 Fillet Welds on Square Edge of Plate or Round Toe of Rolled Section

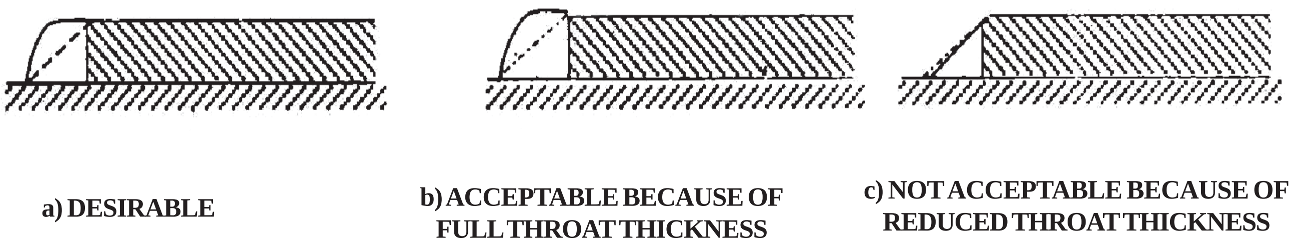
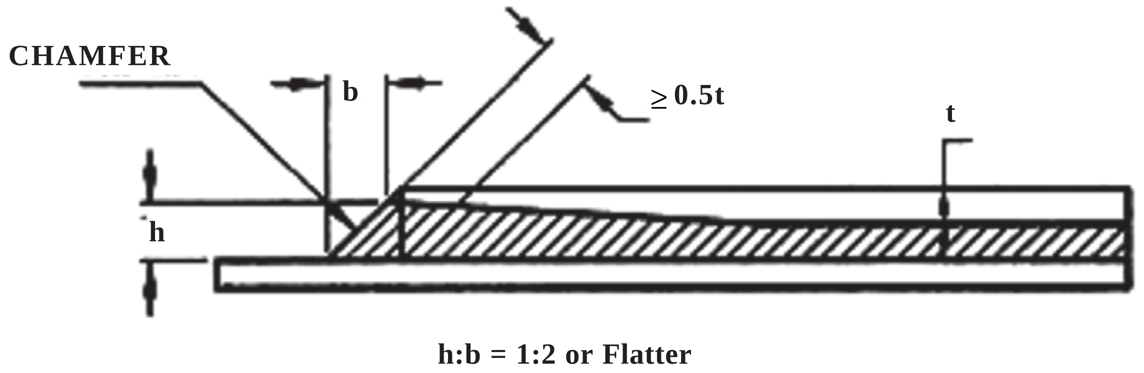


Fig. 14 Full Size Fillet Welds Applied to the Edge of a Plate or Section

512.4.6.4 When fillet welds are applied to the edges of a plate or section in members subject to dynamic loading, the fillet weld shall be of full size, that is, with its leg length equal to the thickness of the plate or section, with the limitations specified in Clause **512.4.6.3**.

512.4.6.5 End fillet weld normal to the direction of force shall be of unequal size with a throat thickness not less than 0.5t where t is the thickness of the part as shown in **Fig. 15**. The difference in thickness of the welds shall be negotiated at a uniform slope.



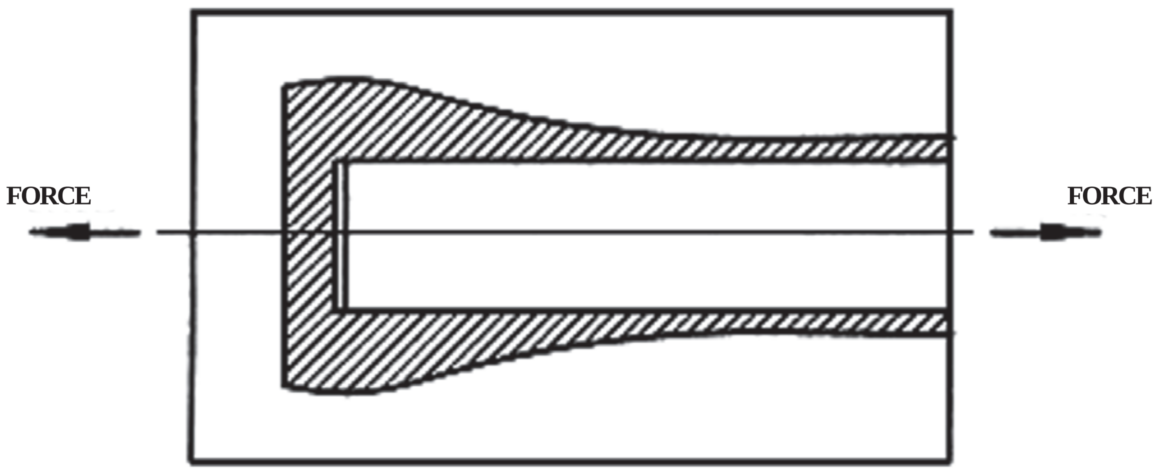


Fig. 15 End Fillet Welds Normal to Direction of Force

512.4.7 *Stresses due to individual forces* - When subjected to either compressive or tensile or shear force alone, the stress in the weld is given by :

$$f_a \text{ or } q = \frac{P}{t_t l_w}$$

where

f_a = calculated normal stress due to axial force in N/mm^2

q = shear stress in N/mm^2

P = force transmitted (axial force N or the shear force Q)

t_t = effective throat thickness of weld in mm

l_w = effective length of weld in mm

512.4.8 *Combination of stresses*

512.4.8.1 *Fillet welds*

512.4.8.1.1 When subjected to a combination of normal and shear stress, the equivalent stress fe shall satisfy following

$$f_e = \sqrt{f_a^2 + 3q^2} \leq \frac{f_u}{\sqrt{3}\gamma_{mw}}$$

where

f_a = normal stresses, compression or tension, due to axial force or bending moment (Clause **512.4.7**)

q = shear stress due to shear force or tension (Clause **512.4.7**)

512.4.8.1.2 Check for the combination of stresses need not be done for :

a) Side fillet welds joining cover plates and flange plates, and

b) Fillet welds where sum of normal and shear stresses does not exceed f_{wd} (Clause **512.4.5.1.1**)

512.4.8.2 *Butt welds*

512.4.8.2.1 Check for the combination of stresses in butt welds need not be carried out provided that :

a) Butt welds are axially loaded, and

b) In single and double bevel welds the sum of normal and shear stresses does not exceed the design normal stress, and the shear stress does not exceed 50 percent of the design shear stress.

512.4.8.2.2 *Combined bearing, bending and shear* - Where bearing stress, f_{br}, is combined with bending (tensile or compressive), f_b, and shear stresses, q, under the most unfavorable conditions of loading in butt welds, the equivalent stress, f_e, as obtained from the following formula shall not exceed the values allowed for the parent metal.

$$f_e = \sqrt{f_b^2 + f_{br}^2 + f_b f_{br} + 3q^2}$$

where

f_e = equivalent stress

f_b = calculated stress due to bending in N/mm^2

f_{br} = calculated stress due to bearing in N/mm^2

q = shear stress in N/mm^2

512.5 Connections made with Bearing Type Bolts, Rivets or Pins

512.5.1 *General*

Connections and splices in all members may be made by the use of fasteners (bolts, rivets or pins). The arrangement of plates, rolled sections and other connecting elements shall be such as to make proper provision for all axial, flexural, shear and/or torsional forces in the members being connected.

Bolted or riveted splices in all compression members shall be located as near as practicable to points of effective lateral support.

A member carrying a calculated stress shall not have a splice or connection with a single bolt or rivet.

Connections and splices for minor members, such as light bracing members, railings, etc. may have single bolted or riveted connections.

Minimum dia of fasteners used in load bearing members shall be 16 mm diameter.

512.5.2 *Connections and splices in flexural members*

a) The connection between a flange and a web of a built-up girder shall be designed to transmit the longitudinal shear force in the flange combined with any vertical loads which are directly applied to the flanges.

b) Flange splices

1) General

Flange splices to join flange components are to be made from the same grade of steel but may be of different cross-sections.

2) Bolted or riveted splices

Where bolted or riveted splice plates are used to obtain a splice in a flange the sum of their areas shall be at least equal to the area of the flanges as spliced. The centres of gravity of the sections on either side of the splice shall coincide as far as practicable. The splice plates and connections on each side of the splice shall be capable of transmitting at least the greater of -

i) 1.10 times the force in the flange at the splice point computed from factored loads.

ii) 0.80 times the maximum capacity of the weaker flange, considering appropriate safety factor **(Table 1)**, the net section being used for tension flanges and the gross section for compression flanges.

c) Web splice

A splice in the web of a plate girder or rolled section used as a beam shall be designed to resist the shearing forces and the portion of the design moment in the web, and for the moment due to the eccentricity of shear introduced by the splice connection, computed from factored loads.

Web plates shall be spliced symmetrically by plates on each side. The splice plates shall extend as far as practicable for the full depth of the web. There shall not be less than at least two rows of rivets or bolts on each side of the joint.

512.5.3 *Connections in triangulated structures*

a) Eccentric connections :

Axially stressed members meeting at a joint shall have their gravity axes intersect at a point if practicable; if not, provision shall be made for bending stresses due to the eccentricity

b) Connections at intersections :

Connection of members at an intersection shall develop at least 1.10 times the design loads and moments in the members computed from factored loads. Due regard to the nature and distribution of stress over the cross-section of the members shall be given in determining the distribution of the fasteners.

All members shall, where possible, be so connected that the load is appropriately distributed over their cross-section.

If this is impracticable, consideration shall be given to the way in which the stresses at the joint are distributed to those parts of the cross-section of the member which are not directly connected at the joint. For this purpose the angle of distribution of stress may be taken as 45^0.

Gusset plates shall be capable of sustaining 1.05 times the design loads and moments transmitted by the members. If an unsupported edge of a gusset plate is in compression and if the length of such edge exceeds 50 times the thickness of the gusset plate, the edge shall be suitably stiffened.

c) Splices in tension members and compression members of non bearing type

Such splices shall be made symmetrical about the gravity axes of the members as far as is practicable.

Bolted/riveted splices shall be designed for any applied moment computed from factored loads and the greater of

1) 1.10 times the computed forces in the member, and

2) 0.80 times the capacity of the member, considering appropriate partial safety factor (**Table 1**).

The ends of the members need not be in close contact.

d) Splices in compression members of bearing type

In bearing type splices in a compression member, the ends of the members shall be machined and assembled to be in close contact with each other. For a bearing splice it may be assumed that the machined faces transmit 50 percent of the compressive force in the member. The splice plates and connection shall be however designed to transmit 60 percent of the factored compressive force in the member and the factored moment, if any.

NOTE : Before specifying bearing splices the designer shall, however, satisfy himself that such facilities for machining are available in the particular project.

512.5.4 *Details of bolted and riveted connections*

a) Diameter of bolt and rivet holes

The diameter of a bolt hole shall generally be taken as the nominal diameter of the bolt plus 1.5 mm unless otherwise specified

The diameter holes of a rivet of upto 25 mm nominal diameter shall be taken as 1.5 mm larger than the nominal diameter of the rivet and 2.0 mm larger than the nominal rivet diameter in case of larger diameter rivets.

b) Edge distances

1) In case of rolled, machine flame cut, sawn or plane edges the distance between the centre of the rivet or bolt hole to such edge shall not be less than 1.5 times the diameter of the hole.

2) In case of sheared or hand flame cut edges the edge distance shall be 1.75 times the diameter of the hole.

3) The maximum edge distance to the nearest line of fasteners from an edge of any unstiffened part should not exceed 12*t* where *t* is the thickness of the thinner outer plate (This rule does not apply to fasteners interconnecting the components of back-to-back tension members). Where the members are exposed to corrosive influences the maximum edge distance shall not exceed 40 mm plus 4t where t is the thickness of thinner connected plate.

c) Pitch of bolts or rivets

1) The minimum distance between the centres of any two adjacent bolts or rivets shall not be less than 2.5 times the diameter of the shank of the bolt or rivet.

2) The maximum distance between the centres of any two adjacent bolts or rivets connecting members either in tension or in compression shall not exceed either 32*t* or 300 mm, where *t* is thickness of the thinner outside element.

3) The distance between centres of two consecutive bolts or rivets in a line along the direction of stress shall not exceed 16*t* or 200 mm in tension members, and 12*t* or 200 mm in compression members. In the case of compression members transferring forces through butting faces the pitch shall not exceed 4.5 times the diameter of bolt or rivet from the abutting faces. This pitch will apply for a distance equal to 1.5 times the width of the member.

4) When bolts or rivets are staggered at equal intervals and the gauge does not exceed 75 mm, the distance as specified in (2) and (3) above between centre of adjacent connectors may be increased by 50 percent.

5) Except as noted in (4) above, the distance between centre of two consecutive bolts or rivets in a line adjacent and parallel to an edge of an outside connected part should not be greater than (100 mm +4t) or 200 mm, whichever is lesser, where t is the thickness of the thinner outside plate.

d) Rivets with counter sunk head

For countersunk rivets, half of the depth of the counter sinking shall be neglected in calculating the length of the rivet in bearing. As far as possible rivets in tension shall be avoided. However, when rivets with counter sunk heads are in tension, the tensile value of the rivets shall be reduced by $33^1/_3$ percent. No reduction is needed in shear

e) Long rivets

The grip of rivets carrying calculated loads shall not exceed 8 times the diameter of the holes. Where the grip exceeds 6 times the diameter of the hole, the number of rivets required by normal calculations shall be increased by not less than half percent for each additional millmetre of length of grip above 6 times the hole diamter.

f) Rivets or bolts through packing

Number of rivets or bolts carrying calculated shear through a packing shall be increased above the number required by normal calculations by 2.5 percent for each 2 mm thickness of packing, except that, for packing having a thickness of upto 6 mm, no increase need be made. For double shear connections packed on both sides, the number of additional rivets or bolts required shall be determined from the thickness of the thicker packing. The additional rivets or bolts shall be placed in an extension of the packing.

512.5.5 *Bearing type bolts*

512.5.5.1 *Effective areas of bolts*

Since threads can occur in the shear plane, the effective area A_{eb} for resisting shear should normally be taken as the net tensile stress area, A_{nb} of the bolts. For bolts where the net tensile stress area is not defined, A_{nb} shall be taken as the area at the root of the threads.

Where it can be shown that the threads do not occur in the shear plane. A_{eb} may be taken as the cross section area, A_{sb} at the shank.

- The net sectional area of a bolt or screwed tension rod A_{nb} shall be taken as the tension area for the particular diameter of bolt as given in the table below :

Nominal Bolt Diameter (mm)	12	14	16	18	20	22	24	27	30	33
Nominal Stress Area (mm^2)	84	115	157	192	245	303	353	459	561	694

512.5.5.2 A bolt subjected to a factored shear force (V_{sb}) shall satisfy the conditon

$$V_{sb} \leq V_{db}$$

Where V_{db} is the design strength of the bolt taken as the smaller of the value as governed by shear, V_{dsb} (Clause **512.5.5.3**) and bearing, V_{dpb} (Clause **512.5.5.4**)

512.5.5.3 Shear capacity of bolt – The design strength of the bolt, V_{dsb}, as governed shear strength is given by

$$V_{dsb} = V_{nsb} \,/\, \gamma_{mb}$$

where

V_{nsb} = nominal shear capacity of a bolt, calculated as follows :

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})$$

where

f_{ub} = ultimate tensile strength of a bolt

n_n = number of shear planes with threads intercepting the shear plane

n_s = number of shear planes without threads intercepting the shear plane

A_{sb} = cross-sectional area of the bolt at the shank

A_{nb} = net shear area of the bolt taken as the area corresponding to root diameter at the thread.

512.5.5.4 Bearing capacity of bolt – The design strength of a bolt on any plate, V_{dpb}, as governed by bearing is given by

$$V_{dpb} = V_{npb} / \gamma_{mb}$$

where

V_{npb} = nominal bearing strength of a bolt, calculated as follows :

= $2.5\ k_b\ d\ t\ f^1_u$

where

k_b is smallest of $e/3d_0$; $p/3d_0$ - 0.25; f_{ub}/f_u ; 1.0

e, p = end and pitch distances of the fastener along bearing direction

d_o = diameter of the hole

f^1_u = smaller of f_{ub}, f_u

f_{ub}, f_u = ultimate tensile stress of the bolt and of the plate respectively

d = nominal diameter of the bolt

t = summation of the thicknesses of the connected plates experiencing bearing stress in the same direction, or, if the bolts are countersunk, the thickness of the plate minus one half of the depth of countersinking

NOTE : The block shear of the edge distance due to bearing force may be checked as given in Clause **506.1.3**

512.5.5.5 Tension capacity of bolt - A bolt subjected to a factored tensile force (T_b) shall satisfy

$$T_b \leq T_{db}$$

where

$$T_{db} = T_{nb}/ \gamma_{mb}$$

and T_{nb} = nominal tensile capacity of the bolt, calculated as follows :

$$T_{nb} = 0.90\ f_{ub}\ A_{nb} < f_{yb}\ A_{sb}\ (\gamma_{mb}/\gamma_{mo})$$

where

f_{ub} = ultimate tensile stress of the bolt

f_{yb} = yield stress of the bolt

A_{nb} = net tensile stress area as specified in Clause **512.5.1**

A_{sb} = shank area of the bolt

512.5.6 *Rivets*

512.5.6.1 *Effective areas of rivets*

Gross area of a rivet. A_r shall be taken as the cross-section area of the rivet hole.

512.5.6.2 A rivet subjected to a factored shear force (V_{sr}) shall satisfy

$$V_{sr} \leq V_{dr}$$

Where V_{dr} is the design strength of the rivet taken as the smaller of the values as governed by shear V_{dsr} (Clause **512.5.6.3**) and bearing V_{dpr} (Clause **512.5.6.4**)

512.5.6.3 *Shear capacity of rivet*

The design strength of the rivet as governed by shear strength is given by :

$$V_{dsr} = V_{nsr} / \gamma_{mr}$$

where

V_{nsr} = nominal shear capacity of rivet calculated as flows :

$$V_{nsr} = \frac{f_{ur}}{\sqrt{3}} \text{ x } (n.A_r)$$

where

f_{ur} = ultimate tensile strength of a rivet

n = number of shear planes

A_r = area of the rivet hole

512.5.6.4 Bearing capacity of rivet

The design strength of a rivet on any plate as governed by bearing is given by :

$$V_{dpr} = V_{npr} / \gamma_{mr}$$

where

V_{npr} = nominal bearing strength of a rivet calculated as follows :

$= 2.5\, k_r.\, d\, .t.\, f'_u$

where

k_r = smaller of $\frac{e}{3d_e};\frac{p}{3d_0}-0.25;\frac{f_{ur}}{f_u};1.0$

e,p = end and pitch distances of the fastener along bearing direction

d_o = diameter of the hole

f'_u = smaller of the f_{ur}, f_u

f_{ur}, f_u = ultimate tensile stress of the rivet and of the plate, respectively.

d = diameter of the rivet hole

t = summation of the thicknesses of the connected plates experiencing bearing stress in the same direction, or if the rivets are countersunk, the thickness of the plate minus one half of the depth of countersinking.

NOTE : The block shear of the edge distance due to bearing force may be checked as given in Clause **506.1.3**.

512.5.6.5 Rivets subjected to axial tension

The use of rivets in tension should be avoided wherever possible. When unavoidable the factored tensile force (T_r) shall satisfy

$$T_r \leq T_{dr}$$

where

$$T_{dr} = T_{nr} / y_{mr}$$

and T_{nr} = nominal tensile capacity of the rivet calculated as follows :

$$T_{nr} = 0.90\, f_{ur}\, A_r < f_{yr}.A_r\, (\gamma_{mr} / \gamma_{mo})$$

where

f_{ur} = ultimate tensile stress of the rivet

f_{yr} = yield stress of the rivet

A_r = area of the rivet hole

512.5.7 Bolt and rivets subjected to combined shear and tension – A bolt or rivet required to resist both design shear force (V_{sb}) and design tensile force (T_b) at the same time shall satisfy

$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1.0$$

where

V_{sb} = factored shear force acting on the bolt/rivet

V_{db} = design shear capacity (Clauses **512.5.5.3 & 512.5.6.3**)

T_b = Factored tensile force acting on the bolt/rivet

T_{db} = design tension capacity (Clauses **512.5.5.5 & 512.5.6.5**)

512.5.8 *Long joints* - When the length of the joint, l_j, of a splice or end connection in a compression or tension element containing more than two fasteners (i.e. the distance between the first and last rows of fasteners in the joint, measured in the direction of the load transfer) exceeds 15d in the direction of load, the nominal shear capacity of the fastener (Clauses **512.5.5.2 & 512.5.6.2**) shall be reduced by the factor, β_{lj}, given by

$$\beta_{lj} = 1.075 - l_j/(200d) \quad \text{but } 0.75 \leq \beta l_j \leq 1.0$$
$$= 1.075 - 0.005(l_j/d)$$

where

d = nominal diameter of the fastener

NOTE: This provision does not apply when the distribution of shear over the length of joint is uniform as in the connection of web of a section to the flanges.

512.5.9 *Large grip lengths* - When the grip length, l_g (equal to the total thickness of the connected plates) exceeds 5 times the diameter, d, of the fastener the design shear capacity shall be reduced by a factor β_{lg}, given by

$$\beta_{lg} = 8\,d / (3d + l_g)$$

$$= 8/(3 + l_g/\text{d})$$

β_{lg} shall not be more than β_{ij} given in Clause **512.5.8**. The grip length, l_g in no case shall be greater than 8d

Packing Plates The design shear capacity of bolts carrying shear through a packing plate in excess of 6 mm shall be decreased by a factor, β_{pk} given by

$$\beta_{pk} = (1 - 0.0125\, t_{pk})$$

where

t_{pk} = thickness of the thicker packing in mm

512.6 Connections made with High Strength Friction Grip (HSFG) Bolts

512.6.1 In high strength friction grip bolting, initial pretension in bolt develops clamping force at the interfaces of elements being joined. The frictional resistance to slip between the plate surfaces subjected to clamping force opposes slip due to externally applied shear. Friction grip type bolts and nuts shall conform to IS 3757. Their installation procedures shall conform to IS 4000.

512.6.2 *Long joints* - The provision for the long joints in Clause **512.5.8** shall apply to friction grip connections also.

512.6.3 *Capacity after slipping* - When friction type bolts are designed not to slip only under service loads, the design capacity at ultimate load may be calculated as per bearing type connection (Clause **512.5.5**).

The block shear resistance of the edge distance due to bearing force may be checked as given in Clause **506.1.3.**

512.6.4 *Tension resistance* – A friction bolt subjected to a factored tension force (T_f) shall satisfy

$$T_f \leq T_{df}$$

where

T_{df} = T_{nf} / γ_{mf}

T_{nf} = nominal tensile strength of the friction bolt, calculated as follows:

$$T_{nf} = 0.9\, f_{ub}\, A_n \leq f_{yb}\, A_{sb}(y_{mf} / y_m)$$

where

f_{ub} = ultimate tensile stress of the bolt

A_n = net tensile stress area as specified in IS 1367. (For bolts where the tensile stress area is not defined, A_n shall be taken as the area at the root of the threads)

A_{sb} = shank area of the bolt

γ_{mf} = partial factor of safety

512.6.5 Combined shear and tension Bolts in a connection for which slip in the serviceability limit state shall be limited and which are subjected to a tension force, T, and shear force, V, shall satisfy

$$\left(\frac{V_{sf}}{V_{df}}\right)^2 + \left(\frac{T_f}{T_{df}}\right)^2 \leq 1.0$$

where

V_{sf} = applied factored shear at design load

V_{df} = design shear strength

T_f = externally applied factored tension at design load

T_{df} = design tension strength

512.7 Prying Forces

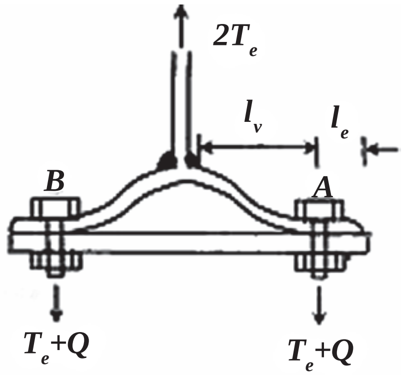
Where prying force, Q, is significant, prying force shall be calculated as given below and added to the tension in the bolt.

$$Q = \frac{l_v}{2l_e}\left[T_e - \frac{\beta\eta\ f_0\ b_e\ t^4}{27\ l_e\ l_v^{\ 2}}\right]$$

where

l_v = distance from the bolt centreline to the toe of the fillet weld or to half the root radius for a rolled section;

l_e = distance between prying force and bolt centerline and is the minimum of, either the end distance or the value given by



$$l_e = 1.1\, t \sqrt{\frac{\beta f_0}{f_y}}$$

β = 2 for non pre-tensioned bolt and 1 for pre-tensioned bolt

η = 1.5

b_e = effective width of flange per pair of bolts

f_o = proof stress in consistent units

t = thickness of the end plate

513 FABRICATION AND INSPECTION

513.1 General

All work shall be in accordance with the drawings and clauses of this code unless otherwise agreed.

513.2 Laminations in Plates

The following areas of plate shall not have laminations exceeding the prescribed limits :

a) Steel plate and sections in which tension stresses are transmitted through thickness of plate or in region in which lamination could affect the buckling behaviour under compression and bending compression.

b) On each side of welded bearing diaphragm, strip of flange and web plate having width equal to 25 times of their thickness.

c) The strip of web plate having 25 times thickness on each side of single sided bearing stiffener welded to web.

d) For welded cruciform joints transmitting tensile stress through the plate thickness on strip having width four times the thickness of plate on each side of attachment

e) For edges of plates where corner welds are provided on to the surface of such plates.

Other areas of plate, section specified by the Engineer shall not have lamination exceeding prescribed limits.

513.3 Storage of Materials

All material, consumable, including raw steel or fabricated material shall be stored specificationwise and sizewise above the ground upon platforms, skids or other supports. These shall be kept free from dirt and other foreign matter and shall be protected from corrosion and distortion. The electrodes shall be stored specificationwise and shall be kept in dry warm condition in properly designed racks. The bolts, nuts, washers and other fasteners shall be stored on racks above the ground with protective oil coating in gunny bags. The paint shall be stored under cover in air tight containers.

513.4 Straightening, Bending and Pressing

513.4.1 Straightening and flattening of steel shall be done by methods that will not injure the metal. Hammering shall not be permitted.

Straightening by heating shall be done under controlled procedure. Temperature of the steel shall not be more than 650^{0}C; heating and cooling rate shall be appropriate to the particular type of steel and shall be agreed by the authorities. Accelerated cooling shall not be used without the approval of Engineer.

513.4.2 *Bending and curving*

513.4.2.1 Steel having yield stress more than 360 MPa shall not be heat curved.

513.4.2.2 Heating procedure – Rolled beams and girders may be curved by either continuous or V-type heating as approved by Engineer.

a) For the continuous method, a strip of sufficient width along the edge of top and bottom flange shall be heated simultaneously to desired temperature to obtain required curve.

b) For V-type of heating, the top and bottom flanges shall be heated in truncated triangular or wedge-shaped areas having their base along the flange edge and spaced at regular intervals along each flange. The truncated triangular pattern shall have an angle 15 to 30 degrees with base not more than 250 mm long. The spacing and temperature shall be as required to obtain the required curvature and heating shall be at approximately same rate along the top and bottom flanges.

   For flange thickness of 32 mm or more, both inside and outside surfaces shall be heated concurrently.

513.4.2.3 Temperature

The heat bending shall be conducted so that the temperature of steel does not exceed 620^{0}C. The girder shall not be artificially cooled until temperature comes down to 315^{0}C by natural cooling. The method of artificial cooling has to be approved by Engineer.

513.4.2.4 Camber

Camber for rolled beams may be obtained by heat curving methods approved by Engineer. For camber in plate girders, the web shall be cut to prescribed camber with suitable allowance for shrinkage due to cutting, welding and heat curving.

513.5 Workmanship

513.5.1 Fabricator has to submit a Quality Assurance Plan according to the nature of fabrication work such as welded fabrication or rivetted fabrication and the same should be approved by the client. Quality Assurance Plan should elaborate Nodal point checking and inspection during the stages of fabrication and also the materials.

513.5.2 Fabrication work shall be taken up only after receipt of approved fabrication/ working drawing.

513.5.3 All members shall carry mark number and item number and, if required, serial number. Method of marking shall be commensurate with the process of manufacture and shall ensure retention of identity at all stages.

513.5.4 *Preparation of edges, ends and surfaces*

Material shall be cleaned and any burring. scales or abnormal irregularities shall be removed.

513.5.4.1 Edge end planing and cutting

End/edge planing and cutting shall be done by any one of the following prescribed methods or left as rolled.

a) Shearing, cropping, sawing, machining, machine flame cutting.

b) Hand flame cutting with subsequent grinding to a smooth edge.

c) Sheared edges of plate not more than 16 mm thick with subsequent grinding to smooth profile, which are for secondary use such as stiffeners and gussets.

   If ends of stiffeners are required to be fitted they shall be ground so that the maximum gap over 60 percent of the contact area does not exceed 0.25 mm.

513.5.4.2 Shearing and flame cutting

Where flame cutting or shearing is used as specified in Clause **513.5.4.1** atleast one of following requirements shall be satisfied.

a) The cut edge is not subjected to applied stress.

b) The edge is incorporated in weld.

c) The hardness of cut edge does not exceed 350 HV 30.

d) The material is removed from edge to the extent of 2 mm or minimum necessary, so that hardness is less than 350 HV 30.

e) Edge is suitably heat-treated by approved method to the satisfaction of Engineer and shown that cracks had not developed by dye penetrant or magnetic particle test.

f) Thickness of plate is less than 40 mm for machine flame cutting for materials conforming to IS 2062 upto Grade E250 (Fe 410w). The requirement of hardness below 350 HV 30 of flame cut edges should be specified by Engineer. Wherever specified by the Engineer the flame cut edges shall be ground or machined over and above requirement (a) to (f).

513.5.4.3 Where machining for edge preparation in butt joint is specified, the ends shall be machined after the members have been fabricated.

513.5.4.4 Outside edges of plate and section, which are prone to corrosion shall be smoothened by grinding or filing.

513.5.5 *Rivet and bolt holes*

513.5.5.1 Holes for rivets, black bolts, high strength bolts and countersunk bolts/rivets (Excluding close tolerance and turn fitted bolts) – All holes for rivets or bolts shall be either punched or drilled. The diameter of holes shall be 1.5 mm larger for bolts/rivets upto 25 mm dia and 2.0 mm for more than or equal to 25 mm.

All holes shall be drilled except for secondary members such as, floor plate, handrails, etc. and members which do not carry the main load can be punched subject to the thickness of member does not exceed 12 mm for material conforming to IS 2062 upto Grade E 250 (Fe 410w).

Holes through more than one thickness of material or when any of the main material thickness exceeds 20mm for steel to IS 2062 upto Grade E 250 (Fe 410w) or 16 mm for IS 2062, Grade E 300 (Fe 440) and above , shall either be subdrilled or subpunched, less than 3 mm diameter than required size and reamed to full diameter. The reaming of material more than one thickness shall be done after assembly.

513.5.5.2 Holes for close tolerance and turn fitted bolts - The diameter of the holes shall be equal to + 0.15 mm to - 0.0 mm, of the bolt shank.

The members to be connected with close tolerance or turn fitted bolts shall be firmly held together by service bolts or clamped and drilled through all thickness in one operation and subsequently reamed to required size within specified limit of accuracy.

The holes not drilled through all thicknesses at one operation shall be drilled to smaller size and reamed after assembly.

513.5.5.3 Holes for high strength friction grip bolts - All holes shall be drilled after removal of burrs. Where the number of plies in the grip does not exceed three, the diameters of holes shall be 1.5 mm larger than those of bolts and for more than three plies in grip, the diameters of hole in outer plies shall be as above and diameter of holes in inner plies shall be not less than 1.5 mm and not more than 3.0 mm larger than those in bolts, unless otherwise specified by Engineer.

513.5.6 *Bolted construction*

513.5.6.1 All joint surface for bolted connection including bolts, nuts, washers, shall be free of scale, dirts, burrs other foreign material and other defects that would prevent solid seating of parts. The slope of surface of bolted parts in contact with bolt head & nuts shall not exceed 1:20, plane normal to bolts axis, otherwise suitable tapered washer shall be used.

All fasteners shall have a washer under nut or bolt head, whichever is turned in tightening.

Each fastener of joint shall be tightened to specified value or equal to 70 percent of specified minimum tensile strength by hand wrenches (turn of nut method) or calibrated wrenches or manual torque wrenches. Impact wrench or any other method specified by Engineer.

When turn of nut method is used for tightening the bolts in joint first all bolts shall be brought to "snug tight" condition, that is tightening by full effort of man using ordinary wrench or by few impacts of any impact wrench. All bolts in the joint shall be then tightened additionally by applicable amount of nut rotation specified below for guidance:

Bolt length (from underside of head to edge)	Disposition of outer faces of bolted parts	
	Bolt face normal to bolt axis	**One face Normal to bolt axis and other face sloped less than 1:20**
Upto and including 4 dia	1/3 turn	1/2
Over 4 dia but less than 8 dia	1/2 turn	2/3
Over 8 dia but less than 12 dia	2/3 turn	5/6

513.5.6.2 *High strength friction grip bolts and bolted connections*

The general requirement shall be as per relevant IS specifications mentioned in Clause **502.4** (Fasteners) of this code. Unless otherwise specified by Engineer, bolted connections of structural joints using high tensile friction grip bolts shall comply with requirements mentioned in IS 4000.

513.5.7 *Riveted construction*

513.5.7.1 Assembled riveted joint surfaces including those adjacent to the rivet head shall be free of scale, dirt, loose scale, burrs, other foreign material and defects that would prevent solid seating of parts.

513.5.7.2 The part/members to be riveted shall be firmly drawn together with bolts, clamps or tack weld. Every third hole of the joint shall have assembly bolts till riveted. Drift shall be used only for matching of holes of the parts/members, but not to the extent as to distort the holes. Drift of larger size than the normal diameter of the holes shall not be used.

513.5.7.3 Rivets shall be heated uniformly to a "light cherry red colour" between 650^{0}C to 700^{0}C for hydraulic riveting and "Orange colour" for pneumatic riveting of mild steel rivets. High tensile steel rivets shall be heated up to 1100^{0}C. Any rivet, which is heated more than the prescribed limit, shall not be driven.

513.5.7.4 Rivet shall be driven in hole when hot so as to fill the hole as completely as possible and shall be of sufficient length to form a head of the standard dimension. When countersunk head is required the head shall fill the countersunk hole. Projection after countersinking shall be ground off wherever necessary.

513.5.7.5 The riveting shall be done by hydraulic or pneumatic machine unless otherwise specified by Engineer.

513.5.7.6 Any defective rivet due to defect in head size or head driven off the centre shall be removed and replaced.

513.5.7.7 The parts not completely riveted in the shop shall be secured by bolts to prevent damage during transport and handling.

513.5.8 *Welded construction*

513.5.8.1 Surfaces and edges to be welded shall be smooth, uniform and free from fins, tears, cracks and other discontinuities. Surface shall also be free from loose or thick scale, slag rust, moisture oil and other foreign materials.

513.5.8.2 The general welding procedures including particulars of the preparation of fusion faces for metal arc welding shall be carried out in accordance with IS 9595.

513.5.8.3 The welding procedures for shop and site welds including edge preparation of fusion faces shall be submitted in writing in accordance with Clause 22 of IS 9595 for the approval of the Engineer before commencing fabrication, and shall also be as per details shown on drawings. Any deviation for above has to be approved by Engineer.

513.5.8.4 Electrodes to be used for metal arc welding shall comply with relevant IS specifications mentioned in Clause **502.5** of this code. Procedure test shall be carried out as per IS 3613 to find out suitable wire-flux combination for welded joint.

513.5.8.5 Assembly of parts for welding shall be accordance with Clause 14 to 16 of IS 9595.

513.5.8.6 The welded temporary attachment should be avoided as far as possible, otherwise the method of making any temporary attachment shall be approved by Engineer. Any scars from temporary attachment shall be removed by cutting, chipping and surface shall be finished smooth by grinding to the satisfaction of Engineer.

513.5.8.7 For welding of any particular type of joint, welders shall qualify to the satisfaction of Engineer in accordance with appropriate welders qualification test as prescribed in any of the Indian standards IS 817, IS 1393, IS 7307 (Part-I), IS 7310 (Part-I) and IS 7318 (Part I) as relevant.

513.5.8.8 In assembling and joining parts of a structure or of built-up members, the procedure and sequence of welding shall be such as to avoid distortion and minimise shrinkage stress.

513.5.8.9 All requirements regarding preheating of present material and interpass temperature shall be in accordance with provisions of IS 9595.

513.5.8.10 Peening of weld shall be carried out wherever specified by Engineer.

a) If specified, peening may be employed to be effective on each weld layer except first.

b) The peening should be carried out after weld has cooled by light blows from a power hammer, using a round nose tool. Care shall be taken to prevent scaling or flecking of weld and base metal from overpeening.

513.5.8.11 Where the Engineer has specified the butt welds are to be ground flush, the loss of parent metal shall not be greater than that allowed for minor surface defects.

513.5.8.12 The joints and welds listed below are prohibited , since these do not perform well under cyclic loading.

a) Butt joints not fully welded throughout their cross section.

b) Butt welds made from one side only without any backing strip.

c) Intermittent butt welds

d) Intermittent fillet welds

e) Bevel and J-preparations in butt joints for other than horizontal position.

f) Plug and slot welds

513.5.8.13 The run-on and run-off plate, extension shall be used providing full throat thickness at the end of butt welded joints. These plates shall comply with following requirements.

a) One pair of "run-on" and one pair of "run-off" plates prepared from same thickness and profile as the parent metal shall be attached to start and finish of all butt welds preferably by clamps.

b) When "run-on and "run-off plates shall be removed by flame cutting, it should be cut at more than 3 mm from parent metal and remaining metal shall be removed by grinding or by any other method approved by Engineer.

513.5.8.14 *Welding of stud shear connectors*

a) The stud shear connectors shall be welded in accordance with the manufacturer's instructions including preheating.

b) The stud and the surface to which studs are welded shall be free from scale, moisture, rust and other foreign material. The stud base shall not be painted, galvanised or cadmium-plated prior to welding.

c) Welding shall not be carried out when temperature is below 0^0C or surface is wet.

d) The welds shall be visually free from cracks and lack of fusion and shall be capable of developing at least the nominal ultimate strength of studs.

e) The procedural trial for welding the stud shall be carried out when specified by Engineer.

513.5.9 *Annealing and stress relieving*

The members which are indicated in the contract or specified by Engineer, to be annealed or stress relieved shall have finish machining, boring, etc., done subsequent to heat treatment. The stress relief treatment shall conform to the following unless specified by Engineer.

a) The temperature of the furnace shall not be more than 300^{0}C at the time welded assembly is placed in.

b) The rate of heating shall not be more than 220ºC per hour divided by *max.* metal thickness subject to maximum 220ºC per hour.

c) After maximum temperature of 600ºC is reached, the assembly shall be held within specified limit of time based on weld thickness. The temperature shall be maintained uniformly throughout the furnace during holding period such that temperature at no two points on the member will differ by more than 80ºC.

d) The cooling shall be done in closed furnace when temperature is above 300^{0}C at the maximum rate of 260ºC per hour divided by maximum metal thickness. The local stress relieving shall be carried out if specified and procedure approved by Engineer.

513.5.10 *Pins and pin holes*

The pins shall be of required length, parallel throughout and of smooth surface free from flaws. The pin holes shall be bored smooth, straight and true to gauge and right angles to the axis of the member. Boring shall be done only after member is finally riveted, bolted or welded unless otherwise approved by Engineer. To facilitate insertion and extraction, pins may be chamfered beyond the required length and provided with suitable holes in the chamfered portion.

513.5.11 *Rectification of surface defects and edge laminations*

The surface defects revealed during fabrication or cleaning shall be repaired as specified. The repair by welding on any surface defect or exposed edge lamination shall be carried out only with approval of Engineer.

513.5.12 *Shop assembly*

The steel work shall be temporarily assembled at place of fabrication. Assembly shall be full truss or girder, unless progressive truss or girder assembly, full chord assembly, progressive chord assembly or special, complete structure assembly is specified by Engineer.

The field connections of main members of trusses, arches, continuous beam spans, bents, plate girders and rigid frame assembled, aligned, accuracy of holes, camber shall be checked by Engineer and then only reaming of subsize holes to specified size shall be taken up.

The assembly will be dismantled after final drilling of holes and approval of Engineer.

The camber diagram showing camber at each panel point, and method of shop assembly and any other relevant detail shall be submitted to Engineer for approval.

513.5.13 *Fabrication tolerances*

513.5.13.1 In general all parts in an assembly shall fit together accurately within tolerances specified in **Table 20**, unless otherwise specified by the Engineer and agreed in contract.

Table 20 Fabrication Tolerances

(Clause **513.5.13**)

Individual Components		
1) Length		
	a) Member with both ends finished for contact bearing	± 1mm
	b) Individual components of members with end plate connection	0 mm - 2 mm
	c) Other members -	
	i) Upto & including 12 M	± 2 mm
	ii) Over 12 M	± 3.5 mm
2) Width		
	a) Width of built-up girders	± 3 mm
	b) Deviation in the width of members required to be inserted in other members	0 mm - 3 mm
3) Depth		
	Deviation in the depths of solid web and open web	3 mm -2 mm
4) Straightness		
	a) Deviation from straightness of columns	*L*/300 subject to a maximum of 15 mm
	1) In elevation	+ 5 mm
	2) in plan	0 mm

5)		Deviation of centre line of web from centre line of flanges in built-up members at contact surfaces	3 mm
6)		Deviation from flatness of plate webs of built-up members in a length equal to the depth of the member	0.005 d to a *max.* 2 mm
7)		Tilt of flange of plate girders	
	a)	at splices & stiffeners, at supports, at the top flanges of plate girders, at bearings.	0.005 b to a *max.* of 2 mm
	b)	at other places	0.015 b to a *max.* of 4 mm
8)		Deviation from squareness of flange to web of columns and box girders	*L*/1000, where *L* is the nominal length of the diagonal
9)		Deviation from squareness of fixed base plate (not machined) to axis of column. This dimension shall be measured parallel to the longitudinal axis of the column at points where the outer surfaces of the column sections make contact with the base plate.	*D*/500, where *D* is the distance from the column axis to the point under consideration on the base plate.
10)		Deviation from squareness of machined ends to axis of columns	*D*/1000, where *D* is as defined in 9 above.
11)		Deviation from squareness of machined ends to axis of beam or girders	*D*/1000, where *D* is as defined in 9 above.
12)		Ends of members abutting at joints through cleats or end plates, permissible deviation from squareness of ends.	1/600 of depth of member subject to a *max.* of 1.5 mm.

513.5.13.2 A machined bearing surface, where specified by the Engineer, shall be machined within a deviation of 0.25 mm for surfaces that can be inscribed within a square of side 0.5 m.

513.5.14 Alignment at splice and butt joints

513.5.14.1 Bolted splice shall be provided with steel packing plates where necessary to ensure that the sum of any unintended steps between adjacent surfaces does not exceed 1 mm for HSFG bolted joints and 2 mm for other joints.

513.5.14.2 In welded butt joints, misalignment of parts to be joined shall not exceed the lesser of 0.15 times the thickness of thinner parts or 3 mm. However, if due either to different thicknesses arising from rolling tolerances or a combination of rolling tolerances with above permitted misalignment, this deviation is more than 3 mm, it shall be smoothened by a slope not steeper than 1:4.

513.6 Inspection and Testing

513.6.1 *General*

No protective treatment shall be applied to the work until the appropriate inspection & testing has been carried out. The stage inspection shall be carried out for all operations so as to ensure the correctness of fabrication and good quality.

513.6.2 *Testing of material*

513.6.2.1 Structural steel shall be tested for mechanical and chemical properties as per various Indian Standards as may be applicable and shall conform to requirements specified in IS 2062, IS 11587.

513.6.2.2 Rivets, bolts, nuts, washers, welding consumables, steel forging, casting and stainless steel shall be tested for mechanical and chemical properties as applicable and shall conform to requirements, as specified in the appropriate Indian Standard.

513.6.3 Rolling and cutting tolerance shall be as per IS 1852. The thickness tolerance check measurements for the plates and rolled sections shall be taken at not less than 15 mm from edge.

513.6.4 Laminations in plates shall be carried out for areas specified in Clause **513.2** by ultrasonic testing or any other specified methods. Flame cut edges without visual signs of laminations need not be tested for compliance with Clause **513.2** unless specified otherwise by Engineer.

513.6.5 Steelwork shall be inspected for surface defects and exposed edge laminations during fabrication and blast cleaning. Significant edge laminations found shall be reported to Engineer for his decision.

Chipping, grinding, machining or ultrasonic testing shall be used to determine depth of imperfection.

For dynamically loaded structures recommended criteria for allowable discontinuities for edge defects and the repair procedure shall be as given in **Table 21** until and unless specified otherwise. The weld procedure shall be as appropriate to the material.

Table 21 Discontinuity of Edge

S. No.	Discontinuity	Repairs required
1)	Discontinuities of maximum 3 mm in depth, any length for material thickness upto 200 mm.	None
2)	Discontinuities of 3 mm to 6 mm in depth and over 25 mm in length for thickness upto 100 mm and 6 mm to 12 mm depth, over 50 mm in length for thickness 100 mm to 200 mm.	Remove. Need not be welded
3)	Discontinuities of 6 mm to 25 mm in depth, over 25 mm in length for thickness upto 100 mm and 12 mm to 25 mm in depth, over 25 mm in length for thickness over 100 to 200 mm.	Remove and weld. No single repair shall exceed 20 percent of edge being repaired.
4)	Discontinuities over 25 mm in depth, any length for thickness 100 to 200 mm.	With approval of Engineer remove to depth of 25 mm and repair by weld block
5)	On edges cut in fabrication, discontinuities of 12 mm maximum depth any length	None

513.6.6 *Measurement of curvature and camber*

Horizontal curvature and vertical camber shall not be measured for final acceptance before all welding and heating operations are completed and flanges have cooled to uniform temperature. Horizontal curvature shall be checked with girder in the vertical position by measuring offsets from a string line or wire attached to both flanges or by any other suitable means. Camber shall be checked by adequate means.

513.6.7 *Tolerance for drilled and reamed holes*

Acceptable deviation in holes drilled and reamed for mild steel and high strength rivets, bolts of normal accuracy and also for high strength friction grip bolts should be as per appropriate Indian Standard.

513.6.8 *Bolted connections*

513.6.8.1 Bolted connection joints with black bolts and high strength bolts shall be inspected for compliance of requirements mentioned in Clauses **513.5.5** and **513.5.6.**

The Engineer shall observe the installation and tightening of bolts so as correct tightening procedure is used and shall determine that all bolts are tightened. Regardless of tightening method used, tightening of bolts in a joint should commence at the most rigidly fixed or stiffest point and progress towards the free edges, both in initial snug tightening and in final tightening.

The tightness of bolts in connection shall be checked by inspection wrench, which can be torque wrench, power wrench or calibrated wrench.

Tightness of 10 percent bolts, but not less than two bolts, selected at random in each connection shall be checked by applying inspection torque. If no nut or bolt head is turned by this application connection can be accepted as properly tightened, but if any nut or head has turned all bolts shall be checked and if necessary retightened.

513.6.8.2 Bolts, and bolted connection joints with high strength friction grip bolts shall be inspected and tested according to IS 4000.

513.6.9 Riveted connections

Rivets and riveted connection shall be inspected and tested for compliance or requirements mentioned in Clause **513.5.7.**

The firmness of the joint shall be checked by 0.2 mm filler gauge, which shall not go inside under the rivet head by more than 3 mm. There shall not be any gap between members to be riveted.

Driven rivets shall be checked with rivet testing hammer. When struck sharply on head with rivet testing hammer, rivet shall be free from movement and vibration.

All loose rivets and rivets with cracked, badly formed or deficient heads or with heads which are unduly eccentric with shanks, shall be cut out and replaced.

513.6.10 *Alignment of joints*

The alignment of plates at all bolted splice joint and welded butt joints shall be checked for compliance with requirements of Clause **513.5.14**.

513.6.11 Testing of flame cut and sheared edges is to be done where the hardness criteria of Clause **513.5.4.2 (a) to (d)** are adopted. Hardness testing shall be carried out on six specimens.

513.6.12 *Welding and welding connection*

513.6.12.1 Welders qualification test shall be carried out as per requirements laid down in IS 7318 (Part 1), for respective approved welding procedure, they shall satisfy relevant requirements of IS 7310 (Part 1).

Welding procedure, welded connection and testing shall be in compliance of requirements mentioned in Clause **513.5.8**.

513.6.12.2 All facilities necessary for stage inspection during welding and on completion shall be provided to Engineer or their inspecting authority by manufacturer.

513.6.12.3 Adequate means of identification either by an identification mark or other record shall be provided to enable each weld to be traced to the welder (s) by whom it was carried out.

513.6.12.4 All metal arc welding shall be in compliance with the provision of IS 9595.

513.6.12.5 The method of inspection shall be according to IS 822 and extent of inspection and testing shall be in accordance with the relevant standards or in the absence of such a standard, as agreed with the Engineer.

513.6.12.6 *Procedure tests*

The Destructive and Non-Destructive test of weld shall be carried out according to IS 7307 (Part-I).

513.6.12.7 *Non-destructive testing of welds*

One or more of following methods may be applied for inspection or testing of weld.

513.6.12.7.1 *Visual inspection*

All welds shall be visually inspected, which should cover all defects of weld such as size, porosity, crack in the weld or in the HAZ (Heat affected zone) etc. Suitable magnifying glass may be used for visual inspection. A weld shall be acceptable by visual inspection if it shows that :

a) The weld has no crack

b) Through fusion exist between weld and base metal and between adjacent layers of weld metal.

c) Weld profiles are in accordance with requisite clauses of IS 9595 or as agreed with Engineer.

d) The weld shall be of full cross section, except for the ends of intermittent fillet welds out side of their effective length.

e) When weld in transverse to the primary stress, undercut shall not be more than 0.25 mm deep in the part that is under cut and shall not be more than 0.8 mm deep when the weld is parallel to the primary stress in the part that is undercut.

f) The fillet weld in any single continuous weld shall be permitted to underrun the nominal fillet weld size specified by 1.6 mm without correction provided that undersize portion of the weld does not exceed 10 percent of the length of the weld. On the web-to-flange welds on girders, no underrun is permitted at the ends for a length equal to twice the width of the flange.

g) The piping porosity in fillet welds shall not exceed one in each 100 mm of weld length and the maximum diameter shall not exceed 2.4 mm, except for fillet welds connecting stiffeners to web where the sum of diameters of piping porosity shall not exceed 9.5 mm in any 25 mm length of weld and shall not exceed 19 mm in any 300 mm length of weld.

h) The full penetration groove weld in butt joints transverse to the direction of computed tensile stress shall have no piping porosity. For all other groove welds, the piping porosity shall not exceed one in 100 mm of length and the maximum diameter shall not exceed 2.4 mm.

513.6.12.7.2 Magnetic particle and radiographic inspection

Welds that are subject to radiographic or magnetic particle testing in addition to visual Inspection shall have no crack.

Magnetic particle test shall be carried out for detection of crack and other discontinuity in the weld according to IS 5334.

Radiographic test shall be carried out for detection of internal flaws in the weld such as crack, piping porosity, inclusion, lack of fusion, incomplete penetration etc. This test may be carried out as per IS 1182 and IS 4853.

513.6.12.7.3 Ultrasonic inspection

The Ultrasonic testing in addition to visual inspection shall be carried out for detection of internal flaws in the weld such as cracks, piping porosity inclusion, lack of fusion, incomplete penetration etc. Acceptance criteria shall be as per IS 4260 or any other relevant IS Specification and as agreed by Engineer.

513.6.12.7.4 Liquid penetrant inspection

The liquid penetrant test shall be carried out for detection of surface defect in the weld, as per IS 3658, in addition to visual inspection.

513.6.12.7.5 The non-destructive testing of following welds be carried out using one or more of the methods described in Clause **513.6.12.7.2 to 4** as may be agreed by Engineer.

a) All transverse butt weld in tension flange.

b) 10 percent of length of longitudinal butt welds in tension flange

c) 5 percent of the length of longitudinal and transverse butt welds in compression flanges

d) All transverse butt welds in webs adjacent to tension flanges as specified by the Engineer.

The particular length of welds to be tested shall be agreed with the Engineer, in case of (b) and (c).

Where specified by the Engineer, bearing stiffeners or bearing diaphragms adjacent to welds, flange plates adjacent to web/flange welds, plates at cruciform welds, plates in box girder construction adjacent to corner welds or other details shall be ultrasonically tested after fabrication.

Any lamination, lamellar tearing or other defect found shall be recorded and reported to Engineer for his decision.

513.6.12.8 Testing of welding for cast steel

The testing of weld for cast steel shall be carried out as may be agreed by the Engineer.

513.6.12.9 Stud shear connectors

Stud shear connectors shall be subject to the following tests

a) The fixing of studs after being welded in position shall be tested by striking the side of the head of the stud with 2 kg hammer, to the satisfaction of the Engineer.

b) The selected stud head stroked with 6 kg hammer shall be capable of lateral displacement of approximately 0.25 height of the stud from its original position. The stud weld shall not show any signs of cracks or lack of fusion.

The studs whose welds have failed the tests given in (a) and (b) shall be replaced.

513.6.12.10 Inspection of members and components

513.6.12.10.1 Inspection requirement

The fabricated member/ component made out of rolled and built-up section shall be checked for compliance of the tolerances given in **Table 20** Inspection of member/components for compliance with tolerances, the check for deviations shall be made over the full length.

During checking the inspection requirement shall be placed in such a manner that local surface irregularities do not influence the results.

For plate, out-of-plane deviation shall be checked at right angle to the surface over the full area of plate.

The relative cross girder or cross frame deviation shall be checked over the middle third of length of cross girder or frame between each pair of webs and for cantilever at the end of member.

The web of rolled beam or channel section shall be checked for out-of-plane deviation in longitudinal direction equal to the depth of the section.

During inspection, the component/ member shall not have any load or external restraint.

513.6.12.10.2 Inspection stages

The inspection to be carried out for compliance of tolerances shall include but not be limited to the following stages :

a) For completed parts, component/ members on completion of fabrication and before any subsequent operation such as surface preparation, painting transportation, erection.

b) For webs of plate and box girder, longitudinal compression flange stiffeners in box girders, and orthotropic decks and all web stiffeners at site joints, on completion of site joint.

c) For cross girders and frames, cantilevers in orthotropic decks and other parts in which deviations have apparently increased on completion of site assembly.

513.6.12.10.3 Where, on checking member/component for the deviations in respect of out-of-plane or out-of-straightness at right angles to the plate surface, and any other instances, exceed tolerance, the maximum deviation shall be measured and recorded. The recorded measurements shall be submitted to the Engineer, who will determine whether the component/ member may be accepted without rectification, with rectification, or rejected.

514 TRANSPORTATION, HANDLING AND ERECTION

514.1 General

514.1.1 This clause lays down guidelines of general nature for handling, transportation and erection of bridges and their components.

514.1.2 It deals with the actions to be taken for various operations in handling, transportation in shop floor and in transit, as also in the erection site.

514.2 Transportation and Handling

514.2.1 Engineer should plan the transportation and mention the mode of transportation, packing, placement, fastening of components or materials to ensure carriage free from damages or undue distortion. When deciding the mode, the route should be surveyed and local restriction in terms DO's/DON'T's statement for proper handling/ transportation to be issued.

514.2.2 All transportable consignments should carry dispatch advice/ challan as per directions to party concerned. Depending on "Target Factor" requirement of materials to be adjusted.

514.2.3 Loose assembled or sub-assembled items should have clear match mark number of the erection drawing. Critical items should be given special care.

514.2.4 Protruded members to be specially protected during transit. Threaded and machined portion of fabricated structures should be carefully handled against damage.

514.2.5 Small items e.g. nuts, bolts, washers, packing plates rivets electrodes shall be despatched in containers and details fully listed to ensure proper receipt and storage. Underloaded consignments should be normally avoided.

514.2.6 In case of heavy and unusual structures availability of the transportation medium should be checked in advance and arrangements tied up. Stability of the members shall be checked during loading or transportation. Necessary safety measures shall be ensured.

514.2.7 For access to the erection site it may be necessary to erect temporary road bridges which can allow safe movement of the fabricated materials & equipment.

514.3 Storage

514.3.1 Suitable area for storage of structures and components shall be located near the site of work. The access road should be free from water logging during the working period and the storage area should be on a level and firm ground.

514.3.2 The store should be provided with adequate handling equipment e.g. road mobile crane, gantries, derricks, chain-pulley blocks, winch of capacity as required. Stacking area should be planned and have racks, sleeper stands, access tracks and be properly lighted.

514.3.3 Storage should be planned to suit erection work sequence and avoid damage or distortion.

514.3.4 Fabricated materials are to be stored on non-corroding surfaces with erection marks visible such as not to come in contact with earth surface or water and should be accessible to handling equipment.

514.3.5 Small fittings, hand tools, etc., are to be kept in containers in covered stores.

514.3.6 IS 7293, and IS 7969 dealing with handling of materials and equipment for safe working should be followed. Safety nuts & bolts as directed are to be used while working.

514.4 Erection Scheme

514.4.1 Design of a bridge should take into consideration the method of erection. A detailed scheme must be prepared showing stagewise activities, with complete drawings and phasewise working instructions. This should be based on detailed stagewise calculation and take into account specifications and capacity of erection equipment machinery, tools, tackles to be used and temporary working loads as per codal provisions.

514.4.2 The scheme should be based on site conditions e.g. hydrology rainfall, flood timings and intensity, soil and subsoil conditions in the riverbed and banks, maximum water depth, temperature and climatic conditions, available working space, etc.

514.4.3 The scheme should indicate detail of materials required with specifications, quantities, type of storage required, etc.

514.4.4 The scheme should indicate precisely the type of temporary fasteners to be used as also the minimum percentage of permanent fasteners to be fitted during the stage erection. The working drawing should give clearly details of the temporary jigs, fixtures, clamps, spacers supports, etc. Adequate provision of spares of vulnerable items to be made.

514.4.5 Erection scheme of the bridge shall be checked to ensure the adequacy of the affected components of the bridge structure and safety of the bridge at all stages of erection.

514.5 Procedure of Erection

514.5.1 Prior to actual commencement of erection all equipment, machinery, tools, tackles, ropes, etc., need to be tested to ensure their efficient working. Frequent visual inspection is essential in vulnerable areas to detect displacements, distress, damages, etc.

514.5.2 Deflection and vibratory tests shall be conducted in respect of supporting structures, launching truss as also the structure under erection and unusual observations reviewed looseness of fittings are to be noted.

514.5.3 For welded structures, welders' qualification and skill are to be checked as per standard norms. Non-destructive tests of joints as per designer's directives are to be carried out.

514.5.4 Precision non-destructive testing instruments available in the market should be used for noting various important parameters of the structures frequently and systematic record is to be kept.

514.5.5 Safety requirements should conform to IS 7205 as applicable and should be a consideration of safety economy & rapidity.

514.5.6 Erection work should start with complete resources mobilized as per latest approved drawings and after a thorough survey of foundations and other related structural work. In case of work of magnitude, maximum mechanization is to be adopted.

514.5.7 The structure should be divided into modules, as per the scheme. This should be pre-assembled in a suitable yard/platform and its matching with members of the adjacent module checked by trial assembly before erection. Such assembled Girders may be tested with simulated loads in case of erection on difficult terrain.

514.5.8 The structure shall be set out to the required lines and levels. The steelwork should be erected, adjusted and completed in the required position to the specified line and levels with sufficient drifts and bolts. Packing materials are to be available to maintain this condition. Organized "Quality Surveillance" checks need to be exercised frequently.

514.5.9 The method of erection, as also the drawing of temporary work and the use of erection equipment, shall be subject to the approval by the Engineer.

514.5.10 *Joints*

Any connection to be riveted or bolted shall be secured in close contact by service bolts or specified No. of permanent bolts before final connection. Service bolts are to be fully tightened and kept as such by torque wrenches when the joint is assembled. Joints shall be made by filling not less than 50 percent of the holes with service bolts and drifts in the ratio of 4:1. Connections are to be completed by close tolerance bolts or as specified.

514.5.11 Any connection to be site welded shall be securely held in position by approved means to ensure accurate alignment, camber and position before welding is commenced.

ANNEX-A

(Clause **501.2**)

LIMITATIONS

A1. The following special steel bridge structures have not been covered in the present code:

a) Curved bridges

For curved bridges, rigorous analysis should be made and detailing must follow the needs of the curvature effects. Having taken into consideration the above aspects, provisions of this code can be applied to curved bridges as appropriate.

b) Cable stayed bridges and

c) Suspension bridges

These are special types of bridges calling for specialised treatment both for analysis and design. Also, erection conditions need to be thoroughly analysed.

d) Temporary bridges and

e) Pedestrian bridges

Because of their nature of use, certain provisions of the code (such as permissible deflection, live load etc.) can be relaxed subject to the approval of the Engineer

f) Swing bridges and

g) Bascule bridges

These types of bridges involve mechanical equipment for which relevant Codes need to be referred. For structural portion of analysis and design, provisions of the code as appropriate can be applied.

h) Box girder bridges

i) Prestressed steel bridges

j) Arch bridges

For these bridges special analysis and detailing are warranted. For structural details provisions of the code as appropriate can be used.

A2. Also aspects concerning Ratings of steel road bridges have not been covered in this Code. For this aspect, IRC:SP:37 "Guidelines for Evaluation for Load Carrying Capacity of Bridges" may be followed.

ANNEX-B

[Clause **504.6.2 (b)**]

RULES FOR CAMBERING OPEN WEB GIRDER SPANS

B1. Preparation of Camber Diagram

B1.1 Contract drawings are dimensioned for the main girder without camber and in order to ensure that its fabrication and erection shall be such as to eliminate deformation stresses in the loaded span, a camber diagram shall be prepared on which shall be clearly indicated the amounts by which the nominal lengths (i.e. the lengths which will not give camber) of members shall be increased or decreased in order that the outline of the girder under full load (dead load & 75 percent live load without impact), shall be the nominal outline. A further change as indicated in clause **B1.4** may be made when the outline of the girder shall be normal outline, enlarged (1 + *K*) times in the case of a through span and reduced (1 - *K*) times in the case of a deck span (See clause **B1.4** below for definition of *K*).

B1.2 The stress camber change in each member shall be equal to the change of length of member due to the above loading, but of opposite sign.

B1.3 For the purpose of calculating the change in length of members under stress, the modulus of elasticity for both high tensile and mild steel shall be taken as specified in Clause **502.2.4.1**. The effective length shall be taken between the theoretical intersection points of adjacent members.

B1.4 To ensure that the length of the floor system of a span shall be constructed to its nominal dimensions, i.e., to avoid changes in lengths of floor and loaded chord lateral system a further change in length shall be made in the lengths of all members equal to :

$$\frac{\text{Loaded chord extension or contraction}}{\text{Loaded chord length}} \times \text{length of member} = K \times L$$

In through spans this change will be an increase in the lengths of all members while in the case of deck spans it will be a decrease in the lengths of all members.

B1.5 The nominal girder lengths altered in accordance with clauses B1.1 and B1.4 give a girder correctly stress cambered but with the loaded chord length identical with that shown on the contract drawings, thus requiring no modifications to floor and loaded chord lateral systems.

B1.6 The nominal lengths and camber lengths shall be rounded off to the nearest half a millimeter.

B1.7 The difference between nominal lengths and camber lengths thus modified is the practical camber changes.

B1.8 The ordinates corresponding to the required camber at nodes may be obtained either by drawing a Williot Mohr Diagram or any other acceptable method.

B1.9 Adjustments of the lengths shall be made to top lateral bracing members to suit camber lengths of the top chords in the case of through girder spans and to the bottom lateral bracing members in the case of deck spans. The average value of the cambered length of top or bottom lateral member, as the case may be, shall be adopted throughout.

B2. Fabrication

B2.1 The actual manufactured lengths of the members are to be the lengths "with camber" given on the camber diagram.

B2.2 The positions and angular setting out lines of all connection holes in the main gussets and also the positions of the connection holes in the chord joints and the machining of the ends shall be exactly as shown on the contract drawings. This will permit the butts in the chord segments to be exactly as shown on the contract drawings.

B2.3 The groups of connection holes at the ends of all the members are to be as shown on the contract drawings, i.e., without any allowance for camber but the distance between the groups at the ends of each member shall be altered by the amount of the camber allowance in the member.

B3. Erection

B3.1 The joints of the chords shall be drifted, bolted and preferably riveted to their geometric outline.

B3.2 All other members are to be elastically strained into position by external forces, so that as many holes as possible are fair when filled with rivets.

B3.3 Drifting of joints shall be avoided as far as possible, and when necessary, should be done with great care and under close expert supervision. Hammers not exceeding one kg. in weight should be used with turned barrel drifts and a number of holes drifted simultaneously, the effect of the drifting being checked by observation of adjacent unfilled holes.

B3.4 The first procedure during erection consists of placing camber jacks in position on which to support the structure. The camber jacks should be set with their tops level and with sufficient run out to allow for lowering of panel points except the centre by the necessary amounts to produce the required camber in the main girders. It is essential that the camber is accurately maintained throughout the process of erection and it should be constantly checked. The jacks shall be spaced so that they will support the ends of the main girders and the panel points. The bottom chord members shall then be placed on the camber jacks, carefully leveled and checked for straightness and the joints made and riveted up.

B3.5 The vertical and diagonal web members, except the posts, shall then be erected in their proper positions on the bottom chords. It is recommended that temporary top gussets, the positions of the holes in which are corrected for the camber change of length in the members, should be used to connect the top ends of the members; this will ensure that the angles between the members at the bottom joints are as given by the nominal outline of the girders. The vertical and diagonal shall then be riveted to the lower chords.

B3.6 All panel points, except the centre, shall now be lowered by amounts to produce the correct camber in the main girders as shown on the camber diagram.

B3.7 The top chord should be erected piece by piece working symmetrically from the centre outwards, and the joint made by straining the members meeting at the joint and bringing the holes into correct registration.

B3.8 The temporary gussets, if used, shall be replaced by the permanent gussets in the same sequence as the erection of the top boom members.

B3.9 The end posts shall be erected last. The upper end connection should preferably be made first and if there is no splice in the end raker, the final closure made at the bottom end connection. If there is a splice, the final closure should be made at the splice.

B3.10 When cantilever method of erection is used, the above procedure does not apply.

ANNEX-C

(Clause **509.2.2.1**)

ELASTIC LATERAL TORSIONAL BUCKLING MOMENT

C1. Elastic Critical Moment

C1.1 General

The elastic critical moment is affected by

a) Moment gradient in the unsupported length

b) Boundary conditions at the lateral support points

c) Non-symmetric and non-prismatic nature of the member

d) Location of transverse load with respect to shear centre

The boundary conditions at the lateral supports have two components:

a) Torsional restaint - where the cross section is prevented from rotation about the shear centre

b) Warping restraint - where the flanges are prevented from rotating in their own plane about an axis perpendicular to the flange.

The elastic critical moment corresponding to lateral torsional buckling of a doubly symmetric prismatic beam subjected to uniform moment in the unsupported length and torsionally restraining lateral supports is given by

$$M_{cr} = \frac{\pi^2 EI_y}{(L_{LT})^2} \left[\frac{I_w}{I_y} + \frac{GI_t \, (L_{LT})^2}{\pi^2 EI_y} \right]^{0.5}$$

where

I_y, I_w, I_t = Moment of inertia about the minor axis, warping constant and St. Venants torsion constant of the cross section, respectively.

G = modulus of rigidity

L_{LT} = effective length against lateral torsional buckling (Clause **509.3**)

This equation in simplified form for I section has been presented in Clause **509.2.2.1**.

While the simplified equation is generally on the safe side, there are many situations where this may be very conservative. More accurate calculation of the elastic critical moment for general case of unsymmetrical sections, loading away from shear centre and beams with moment gradient can be obtained from specialist literature, by using an appropriate computer programme or equations given below.

C1.2 Elastic critical moment of a section symmetrical about minor axis

In case of a beam, which is symmetrical only about the minor axis, and bending about major axis, the elastic critical moment for lateral torsional buckling is given by the general equation,

$$M_{cr} = c_1 \frac{\pi^2 EI_y}{(L_{LT})} \left\{ \left[\left(\frac{K}{K_w} \right)^2 \frac{I_w}{I_y} + \frac{GI_t (L_{LT})^2}{\pi^2 EI_y} + \left(c_2 y_g - c_3 y_j \right)^2 \right]^{0.5} - \left(c_2 y_g - c_3 y_j \right) \right\}$$

where

c_1, c_2, c_3 = factors depending upon the loading and end restraint conditions (**Table C.1**).

K, = effective length factors of the unsupported length accounting for boundary conditions at the end lateral supports. The effective length factors K varies from 0.5 for complete restraint against rotation about weak axis to 1.0 for free rotation about weak axis, with 0.7 for the case of one end fixed and other end free. It is analogous to the effective length factors for compression members with end rotational restraint.

K_w = Warping restraint factor. Unless special provisions to restrain warping of the section at the end lateral supports are made K_w should be taken as 1.0.

y_g = y distance between the point of application of the load and the shear centre of the cross section and is positive when the load is acting towards the shear centre from the point of application.

$y_j = y_s - 0.5 \int_A \left(z^2 - y^2 \right) ydA/I_z$

y_s = Coordinate of the shear centre with respect to centroid, positive when the shear centre is on the compression side of the centroid

y, z = Coordinates of the elemental area with respect to centroid of the section

The y_j can be calculated by using the following approximation.

a) Plain flanges

$y_j = 0.8\,(2\beta_f - 1)\, h_y / 2.0$ (when $\beta_f > 0.5$)

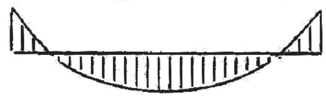
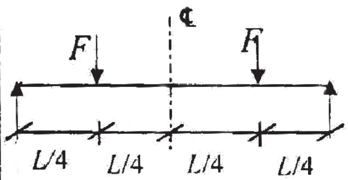
$y_j = 1.0\,(2\beta_f - 1)\, h_y / 2.0$ (when $\beta_f \leq 0.5$)

Table C.1 Constants c_1 , c_2 and c_3

(Clause **C.1.2**)

t	Value of K	Constants		
		c_1	c_2	c_3
+1	1.0	1.000	---	1.000
	0.7	1.000		1.113
	0.5	1.000		1.144
- ¾	1.0	1.141	---	0.998
	0.7	1.270		1.565
	0.5	1.305		2.283
- ½	1.0	1.323	---	0.992
	0.7	1.473		1.556
	0.5	1.514		2.271
- ¼	1.0	1.563	---	0.977
	0.7	1.739		1.531
	0.5	1.788		2.235
= 0	1.0	1.879	---	0.939
	0.7	2.092		1.473
	0.5	2.150		2.150
- ¼	1.0	2.281	---	0.855
	0.7	2.538		1.340
	0.5	2.609		1.957
- ½	1.0	2.704	---	0.676
	0.7	3.009		1.059
	0.5	3.093		1.546
- ¾	1.0	2.927	---	0.366
	0.7	3.009		0.575
	0.5	3.093		0.837
- 1	1.0	2.752	---	0.000
	0.7	3.063		0.000
	0.5	3.149		0.000

Table C.1 (Continued ....)

Loading and Support Conditions	Bending Moment Diagram	Value of K	Constants c_1	c_2	c_3
		1.0 0.5	1.132 0.972	0.459 0.304	0.525 0.980
		1.0 0.5	1.285 0.712	1.562 0.652	0.753 1.070
		1.0 0.5	1.365 1.070	0.553 0.432	1.780 3.050
		1.0 0.5	1.565 0.938	1.257 0.715	2.640 4.800
		1.0 0.5	1.046 1.010	0.430 0.410	1.120 1.390

b) Lipped flanges

$$y_j = 0.8\,(2\beta_f - 1)\,(1 + h_L / h)\,h_y / 2 \qquad \text{(when } \beta_f > 0.5)$$

$$y_j = (2\beta_f - 1)\,(1 + h_L / h)\,h_y / 2 \qquad \text{(when } \beta_f \leq 0.5)$$

where

h_L = height of the lip

h = overall height of the section

h_y = distance between shear centre of the two flanges of the cross section

I_t = The torsion constant given by

I_t = $\Sigma\, b_t\,(t_i)^3 / 3$ for open section

= $4A_e^2 / \Sigma\,(b / t)$ for hollow section

where

A_e = area enclosed by the section

b, t = breadth and thickness of the elements of the section respectively.

The warping constant, I_w, is given by

I_w = $(1 - \beta_f)\, \beta_f I_y h_y^2$ for I sections mono-symmetric about weak axis

= 0 for angle, Tee, narrow rectangle section and approximately for hollow sections.

β_f = $I_{fc} / (I_{fc} + I_{ft})$ where I_{fc}, I_{ft} are the moments of inertia of the compression and tension flanges, respectively, about the minor axis of the entire section

ANNEX-D

DURABILITY

D1. General

A durable steel structure is one that performs satisfactorily the desired function in the working environment under the anticipated exposure condition during its service life, without deterioration of the cross sectional area and loss of the strength due to corrosion. The material used, the detailing, fabrication, erection and surface protection measures should all address the corrosion reduction and durability requirements.

D2. Requirements for Durability

D2.1 Shape, size, orientation of members, connections and details

The design, fabrication and erection details of exposed structures should be such that good drainage of water is ensured. Standing pool of water, moisture accumulation and rundown of water for extended duration are to be avoided.

The details of connections should ensure that

a) All exposed surfaces are easily accessible for inspection and maintenance.

b) All surfaces not so easily accessible are completely sealed against ingress of moisture.

D2.2 Exposure condition

D2.2.1 General environment - The general environment, to which steel structure is exposed during its working life, is classified into five levels of severity, as given in **Table D.1**.

D2.2.2 Abrasion - Specialist literature may be referred for durability of surfaces exposed to abrasive action.

D2.2.3 Exposure to sulphate attack

Appropriate coatings may be used when surfaces of structural steel are exposed to concentration of sulphates (SO_3) in soil, ground water etc.

When exposed to very high sulphate concentrations, of more than 2 percent in soil and 5 percent in water, some form of lining such as polyethylene, polychloroprene sheet, or surface coating based on asphalt, chlorinated rubber, epoxy or polymethane material should be used to completely avoid access of the solution to the steel surface.

Table D.1 Environmental Exposure Conditions

(Clause **D 2.2.1**)

Environmental Classifications	**Exposure conditions**
Mild	Surfaces normally protected against exposure to weather or aggressive condition, except when located in coastal areas
Moderate	Structural steel surfaces: i) exposed to condensation and rain ii) continuously under water iii) exposed to non-aggressive soil/ groundwater iv) sheltered from saturated salt air in coastal areas
Severe	Structural steel surfaces: i) exposed to severe frequent rain ii) exposed to alternate wetting and drying iii) exposed to severe condensation iv) completely immersed in sea water v) exposed to saturated salt air in coastal area
Very severe	Structural steel surface exposed to i) sea water spray ii) corrosive fumes iii) aggressive sub soil or ground water
Extreme	Structural steel surfaces exposed to i) tidal zones and splash zones in the sea ii) aggressive liquid or solid chemicals

D2.3 Corrosion protection methods - The methods of corrosion protection are governed by actual environmental conditions as per IS 9077 and IS 9172. The main corrosion protection methods are given below:

a) Controlling the electrode potential (Cathodic protection)

b) Inhibitors

c) Inorganic/metal coatings or organic/paint systems

D2.4. Surface Protection

D2.4.1 In the case of mild exposure, a coat of primer after removal of any loose mill scale may be adequate. As the exposure condition becomes more critical more elaborate surface preparations and coatings become necessary. In case of extreme environmental classification, protection shall be as per specialist literature. **Table D-2** gives guidance to protection of steelwork for different desired lives.

D2.4.2 Steel surfaces shall be provided with atleast one coat of primer immediately after its surface preparations, such as sand blasting to remove all mill scale and rust to expose steel.

D2.4.3 Steel without protective coating shall not be stored for long duration in out door environment.

D2.4.4 Surfaces to transfer forces by friction as in HSFG connections shall not be painted. However it shall be ensured that moisture is not trapped on such surfaces after pretensioning of bolts by proper protective measures.

D2.4.5 Member to be assembled by welding shall not be prepainted at the adjacent to the location of such welds. However, after welding appropriate protective coatings shall be applied in the region as required by the exposure conditions. If the contact surfaces cannot be properly protected against ingress of moisture by surface coating, they may be completely sealed by appropriate welds.

D2.4.6 Prepainted members shall be protected against abrasion of the coating during transportation, handling and erection.

D2.5 Special steels - Steels with special alloying elements and production process to obtain better corrosion resistance may be used as per Specialist literature.

Table D-2 Protection Guide for Steel Work Application

(Clause **D 2.4.1**)

a) Desired Life of coating system in different environments

Atmospheric Condition	Coating system					
	1	**2**	**3**	**4**	**5**	**6**
Normal Inland (rural and urban areas)	12 years	18 years	20 years	About 20 years	About 20 years	Above 20 years
Polluted Inland (high airborne sulphur dioxide), moderate	10 years	15 years	12 years	About 18 years	15 - 20 years	Above 20 years
Normal coastal (as normal inland plus high airborne salt levels), severe	10 years	12 years	20 years	About 20 years	About 20 years	Above 20 years
Polluted coastal (as polluted inland plus high airborne salt levels), very severe or extreme	8 years	10 years	10 years	About 15 years	15 - 20 years	Above 20 years

Table D-2 (Continued…)

b) Specification for different coating system

Protection	Coating system					
	1	**2**	**3**	**4**	**5**	**6**
Surface Preparation	Blast clean	Blast clean	Blast clean	Blast clean	Grit blast	Blast clean
Pre-fabrication primer	Zinc phosphate epoxy 20 µm	2 pack zinc-rich epoxy 20 µm	-	2 pack zinc-rich epoxy 20 µm	-	Ethyl zinc silicate 20 µm
Post-fabrication primer	High-build zinc phosphate modified alkyd 60 µm	2 pack zinc-rich epoxy 20 µm	Hot dip galvanised 85 µm	2 pack zinc-rich epoxy 25 µm	Sprayed zinc or sprayed aluminium	Ethyl zinc silicate 60 µm
Intermediate coat	-	High-build zinc phosphate 25 µm	-	2 pack Epoxy micaceous iron oxide	Sealer	Chlorinated rubber alkyd 35 µm
Top coat	-	-	-	2 pack epoxy micaceous Iron oxide 85 µm	Sealer	-

Table D-2 (Continued...)

b) Specification for different coating system

(Site applied treatments)

Protection	Coating system					
	1	2	3	4	5	6
Surface Preparation	As necessary	As necessary	No site treatment	As necessary	No site treatment	As necessary
Primer	Touch in	Touch in	-	-	-	Touch in
Intermediate coat	-	Modified alkyd Micaceous iron oxide 50 μm	-	Touch In	-	High-build micaceous iron oxide Chlorinated rubber micaceous 75 μm
Top Coat	High-build Alkyd finish 60 μm	Modified Alkyd Micaceous iron oxide 50 μm	-	High-build chlorinated rubber	-	High-build iron oxide Chlorinated Rubber 75 μm

ANNEX-E

POST - CONSTRUCTION INSPECTION AND PREVENTIVE MAINTENANCE GUIDELINES

E1. General

Bridge structures, permanently exposed to atmosphere are subjected to effect of adverse environmental conditions. Investment made in the structural facility can be protected by well programmed, monitored-inspection and maintenance schedule adopted for its designed life. Such systems ensure structural safety by recording the state of the structure periodically and providing feed back information to designers while identifying actual and potential sources of trouble and taking remedial measures in time.

This section lays down the desired inspection procedure for determining physical condition and programming maintenance needs of the bridges. Systematic periodical inspection required for various elements and the responsibilities of the inspection group have been specified. The scope of maintenance work involved does not include correcting measures for known deviations introduced during construction stage and no attempt has been made in this direction. It is necessary to understand that, for proper inspection of various components of the bridge structure in-built facilities should be developed at the detailing and construction stage, for accessibility to important areas.

E2. Inspection

Bridge inspection is done by use of well tried and established techniques required for assessing the physical condition of the structure. IRC Special Publication 35 : "Guidelines for inspection and maintenance of bridges" may be referred in this connection.

E2.1 Personnel

E2.1.1 The in-charge for bridge inspection and reporting shall posses the following qualifications :

a) Be a qualified engineer or equivalent with adequate experience in Bridge Inspection or

b) Have a minimum of 10 years experience in bridge inspection assignments in a responsible capacity.

E2.1.2 He/she shall be responsible for a methodical and thorough field inspection, the detailed analysis of all observations recorded, arrive at findings to recommend rectification of defects, imposition of speed restriction or load limitations and any other measures as necessary.

E2.1.3 The problems encountered in this work are varied and complex; as such matured judgment is often required for evaluation of the recordings.

E2.1.4 He/She must be thoroughly familiar with design and construction features of the bridge so as to make a correct interpretation and be capable of determining the safe load carrying capacity of the existing structure. He should be capable of recognising any deficiency in the structure, assess the seriousness and suggest appropriate remedial measures to ensure safety. His experience and knowledge to recognise problem areas (actual and potential) and to ensure preventive maintenance is an important requisite.

E2.1.5 He/She should be able to utilise the expert knowledge and skills of associate engineers in respect of structural design, construction methods, material, hydro dynamics, equipment, soil technology, maintenance methods for permanent and emergency measures etc. and should have access to resources and expert systems.

E2.1.6 Definite guidelines should be given to ensure availability of technical assistance from other agencies/ where regular staff is not available. In case of specialised structure consultation with expert bodies is a necessity.

E3. Training

Bridge management requires extensive team work involving various levels of responsibility and skills. Training programmes need be framed in order to develop expertise. Training facilities may be set up at central level for training of trainees and at local level for actual imparting training to field staff. Workshop on topical interest may be held regularly to acquaint the concerned people with accepted technical methods and their correct application.

E4. Frequency of Inspection

E4.1 Detailed inspection

E4.1.1 The details and frequency to which Bridges are to be inspected will depend on such factors as age, traffic characteristics, state of the structure and vulnerability and known history of deteriorating condition. Evaluation of these factors will be the responsibility of the individual in-charge of the inspection programme.

E4.1.2 Each bridge has to be inspected in detail at regular intervals not exceeding 5 years.

E4.2 Periodic routine inspection

Certain items in each bridge have to be inspected at definite intervals of time, at least once a year, irrespective of whether anything alarming has taken place or not.

E4.3 Special inspection

Such Inspections are required for any bridge with known deficiencies like restriction on weight/ speed, loss of camber and are considered necessary on the basis of routine inspection or unusual occurrence.

Recommended frequencies of various inspection item are shown in **Table E-1**. These however, are for guidance only.

Table E-1 Bridge Inspection Record Sheet

(Clause **E 4.3**)

Sl. No.	**Inspection Items**		**5 Year**	**1 Year**	**6 Months**
1)	Main Bridge Structure	a) Steel Girders and Stringers	√ √	√ √	
		b) Trusses	√	√	
2)	Bearing	a) Wearing surface	√	√	
3)	Decks	a) Drainage System	√	√	As situation demands
		b) Steel Deck	√	√	
		c) Curbs	√	√	
		d) Foot Path	√	√	
4)	Expansion Joints		√	√	
5)	Railing & Crash Barriers		√	√	
6)	Signs	-	√		
7)	Services & Utilities		√		

NOTE : In the case of distressed Bridge Special instructions to be issued by competent authority.

E5. Inspection Procedure

E5.1 General

E5.1.1 The field inspection of a bridge should be conducted in a systematic and organised manner and observation recorded on a format to ensure that no item is overlooked. Notes must be clear and detailed to the extent that they can be interpreted at a later date when report is prepared. Sketches and photographs should be included in an effort to record actual conditions.

E5.1.2 As far as possible the inspecting officer should schedule bridge inspection in those periods of the year which offer the most favourable conditions. Inspections during temperature extremes should be made at bearing, joints etc. Inspection should not be confined only to search for defects which exist, but for conditions of anticipatory nature and marking those zones. Preventive maintenance is equally important to corrective ones.

E5.2 Inspection items

Inspection of all items such as approaches, waterways, basic floor conditions, substructure which may affect the safety of the steel superstructure need be done like other types of bridges.

E6. Main Structure

E6.1 Steel girders and stringers in the deck structure should be examined for signs of corrosion, cracks along the flanges around rivet or bolt heads, its contact surfaces and where water enters and stands or debris may collect at the ends.

E6.2 Flanges and webs shall be checked for any damage or misalignment. Web-stiffeners are to be examined for signs of deformation due to buckling. Unusual vibration or excessive deflection under passage of heavy loads should be noted and cause investigated.

E6.3 All end connections should be inspected to make sure that they are secure.

E6.4 Weld areas should be inspected to check crack. Special care should be exercised to inspect corners, curved, sections and areas where there is an abrupt change in the size of metal or in configuration which may produce an area of concentrated stress or in areas where vibration or movement could produce stress concentration. Damages or deformation caused to the main-structural members due to vehicular impact should be particularly watched. Fatigue failure in welded joints being a cause of concern in bridge structures with age such structures need more careful check and watch.

E6.5 Creep - The longitudinal movement of a girder is termed as creep. This point should be checked and girders pulled back if necessary to the proper position.

E6.6 Distortion- With variation of temperature, the girder is likely to have longitudinal movement due to expansion and contraction. Absolute freedom of movement is impracticable and there remains a residual force which develops internal stresses, causing tendency to distort.

E6.7 Lateral bracing

Normally a span of a bridge consists of two or more girders braced together with lateral bracings. These bracings should be thoroughly checked for corrosion on loose rivets and deformity viz. bracing distortion etc.

E6.8 Loose rivets

Rivets are to be examined for looseness. This is caused due to running traffic and consequent vibratory effect and corrosion around the rivets. A joint with loose rivets should not be touched unless more than 20 percent rivets in the joint are loose.

E6.9 Bearings

E6.9.1 All bearing devices should be examined to ascertain that they are functioning properly. Changes in other parts of the structure such as piers/abutments settlement and tilt may be reflected in the bearings. Bearings should be seated properly on their bed plates and provided with gaps at both ends. Bearing assembly should be checked for possible cracks by magnifying glass after removal of paint cover in doubtful cases. In case of roller bearing, the relative position of the top castings, bottom castings and rollers and variation due to the temperature shall be noted. Longitudinal movement of the free-end shall be recorded under moving load. After unusual occurrences bearings and support pads must be examined for cracks, etc. Lateral shear keys in skew bridges also need special check.

E6.9.2 Lubrication of bearings

Oiling and greasing of the bearings is done periodically once in 3 years. Improper and failure in timely lubrication may lead to corrosion of bearings, resulting in reduction in strength and consequent damages.

Where bearings are encased in oil baths and stay submerged in recommended brand of lubricating oil the level of oil should be maintained by checking every year. It should be ensured that the oil baths are always sealed.

E6.9.3 Elastomeric bearings

The physical conditions of elastomeric bearing pads should be inspected for observing any abnormal flattening, bulging or splitting which may indicate overloading or excessive unequal distribution of loading, Shifting from original position should be checked particularly.

E6.9.4 Condition of bed-block & H.D. Bolts

Bed blocks receive the full load from the bearings of the bridge and distribute and transmit the same to the masonry below. Restriction of free movement in superstructure may result in:

1) Development of transverse cracks in piers/abutments

2) Failure of bed blocks joints leading to shaking bed blocks

3) Shearing of holding down bolts (Particular importance due to introduction of greater longitudinal forces)

It must be ensured that the anchor bolts are well secured.

E7. Trusses

E7.1 Camber of the trusses should be checked and the ambient temperature recorded at the time of detailed inspection. A camber diagram should be made in the inspection register. Loss of camber may be assessed from comparative readings.

E7.2 All truss members should be checked. The compression members should be checked for straightness absence of kink or bows and the connections are undisturbed. (Tension members should not show signs of cracking).

E7.3.1 The truss should be checked against damage due to collision with vehicular traffic; portal bracings and sway bracings are usually the most restrictive to overload movements and consequently susceptible to damage.

E7.3.2 The condition of pins at the connections and rivets, bolts should be checked to see that none are loose, worn-out or sheared. Particular care to be given to following locations :

a) Connections of stringers to cross girders

b) Connections of cross girder to main girder

c) End connection of bracings

d) Chord joints & web-member connection

E8. Corrosion and Painting

Steel structure is sensitive to the atmospheric moisture and vehicular smoke and therefore should be protected by paints or anti-corrosive measures. The condition of the members should be examined and the extent of corrosion recorded. The portions of steel work where water is likely to stagnate or which are subjected to alternate wetting or drying need special care. Deformation in riveted or bolted multiple sections should be examined to check if moisture has entered and corroded the contact surfaces of the plates causing them to be pushed apart. The exact location and area of the affected portion should be recorded. This area should be got cleaned, thoroughly scraped, old paint, rust, scaling removed and repainted and appropriate remedial measures taken up immediately.

E9. Decks

E9.1 Steel decks should be checked for corrosion and unsound welds. It is important to maintain an impervious surface over a steel plate deck to protect against corrosion in aggressive environmental condition.

E9.2 It is necessary to have effective drain holes to prevent collection of water on the deck.

E10. Expansion Joints

E10.1 Maintenance of these joints need special attention and should be carefully examined. The joint should be clear of debris and be able to have free thermal expansion as designed.

E10.2 Finger type joints and sliding plate joints should be checked for loose anchorage, cracking or separation of welds or other defects. Such defects cause structural deformation and is hazardous to traffic. Deck adjacent to expansion joints should be carefully examined for voids and cracks. Underside of expansion joints also need careful inspection. Systematic documentation of the movement of expansion joints need to be kept to judge proper functioning of the bridge structure.

E11. Railings and Safety Barriers

E11.1 Handrails are to be examined for unusual damage, deformation, corrosion and paintings. The embedment of posts to be checked for rust stains, which are signs of rusting. Extent of corrosion need to be checked when signs exist on the surface.

E11.2 All handrails are to be checked for any damage for traffic. The vertical and horizontal alignment are to be maintained.

E12. Services

The number and types of utilities, such as pipelines, cables etc. must be inspected and observations kept for record with the details suitably displayed. Special care need be kept for hazardous utilities, Regular joint inspection in such case with suitable guidelines is a necessity.

E13. Special Structures

E13.1 Moveable bridges

The most common type of moveable bridge are the swing span, vertical lift Bascule

(Single or double leaf). Inspection of the trusses, floor system, and other structural elements will require inspection procedures suitably modified as per guidelines mentioned in the Code. Ensuring proper seating of the girder after operation is an absolute necessity.

E13.2 In case of other structures like suspension Bridges, Cable stayed Bridges detailed inspection manuals should be prepared and staff trained to observe the same.

E14. Documentation

E14.1 The most important function of bridge maintenance unit is to prepare a complete, methodical and current record for each bridge on the system. Much of the usefulness of the information obtained from field investigation depend upon its reliability and availability on a concise format. The record must be preserved systematically and readily available.

E14.2 Records should provide a full history of the structure including all recommendations for strengthening and restoration works undertaken and the behaviour of the structure thereafter. This record should indicate clearly the load carrying capacity of the structure with supporting document showing calculations.

E14.3 Complete record in an usable format is vital for the continued service ability of the bridge. It is essential that computerised data system is introduced as soon as practicable.

E14.4 A sample record sheet is shown in **Table E-1.**

E15. Standard Tools

A list of standard tools required for inspection is given in **Table E-2** as guide.

E16. Signs

E16.1 All signs required to indicate restrictive load limit, reduced speed or impaired clearance should be inspected to ascertain they are visible and located in proper places. This inspection is to include sign at or on the structure and any advance warning sign. Examination should include that indications are legible and sign posts are well secured.

E16.2 For bridges over navigable, channels, it is necessary to inspect if the navigational signs for water-traffic are in place and secured. Navigational lights and serial obstructional lights should be inspected often to ensure that these are operating efficiently.

E17. For more information please refer to IRC:SP-35 : Guidelines for Inspection and Maintenance of Bridges.

Table E-2 Standard Tools and Equipment

A. STANDARD TOOLS

1) Clip board, chalk, markers clamps etc,
2) Pocket tapes, folding rules, tapes (10 m to 50 m) feeler gauges, callipers, micrometer gauges,
3) Straight edge, plumb bob, protector, spirit level.
4) Thermometer, inspection mirror, binoculars, magnifying glass, camera.
5) Scrapers, emery paper, portable torque wrenches, light hammer, piano-wire, portable ladder, rope.
6) Flash light, pocket knife, wire brush, chipping hammer, thin steel rod (for use as probe) (8 mm to 20 mm dia)
7) Hydraulic jacks, pulley blocks, wire-ropes, chains, slings etc., of appropriate capacity.
8) Safety equipment for inspecting staff.

B. ASSESSMENT POSSIBLE

Steel	
Cracks	Ultrasonic, radiographic
Cable/wire failure	Electric half cell potential
Corrosion	Electrical resistivity meter
Global behaviour	
Movements	Modern surveying Instruments dial gauges
Extensometric measurements	Strain gauges, and extensometer
Pressures, Forces	Pressures transducers, or load cells
Miscellaneous	
Thickness of coating	Paint film gauge (digital electrometer)
Water proofing membranes	Electric resistance
Vibration	Accelerometer
Widening of Cracks	Glass cell tabs
Metal Thickness	Ultrasonic metal thickness measuring

ANNEX-F

DESIGN ASSISTED BY TESTING

F1. Need for Testing

Testing of structures, members or components of structures is not required when designed in accordance with this standard. Test may be accepted as an alternative to calculation or may become necessary in special circumstances.

Testing on of a structural system, member or component may be required to assist the design in the following cases:

a) When the calculation methods available are not adequate for the design of a particular structure, member or component, testing shall be undertaken in place of design by calculation or to supplement the design by calculation;

b) Where rules or methods for design by calculation would lead to uneconomical design, experimental verification may be undertaken to avoid conservative design;

c) When the design or construction is not entirely in accordance with sections of this standard, experimental verification is recommended;

d) When confirmation is required on the consistency of production of material, components, members or structures originally designed by calculations or testing;

e) When the actual performance of an existing structure capacity is in question, testing shall be used to confirm it;

F1.1 Testing of structural system, member or component shall be of the following categories :

a) Proof testing - The application of test loads to a structure, sub-structure member or connection to ascertain the structural characteristics of only that specific unit.

b) Prototype testing - Testing of structures, sub-structures, members or connections is done to ascertain the structural characteristics of a class of such structures, sub-structures members or connection, which are nominally identical to the units tested.

F2. Types of Test

F2.1 Acceptance test - This is intended as a non-destructive test for confirming structural performance. It should be recognized that the loading applied to certain structures might cause permanent distortions. Such effects do not necessarily indicate structural failure in acceptance test. However, the possibility of their occurrence should be agreed to before testing.

The load for acceptance test $F_{test,a}$ shall be determined from $F_{test,a}$ = (1.0 x self weight) + (1.15 x remainder of the permanent load) + (1.25 x variable load).

The assembly shall satisfy the following criteria:

a) It shall demonstrate substantially linear behaviour under test loading.

b) On removal of the test load, the residual deflection should not exceed 20 percent of the maximum-recorded deflection.

If the above criteria are not satisfied the test may be repeated one more time only, when the assembly shall satisfy the following criteria :

a) It shall demonstrate substantially linear behaviour on this second application of test loading.

b) The corresponding recorded residual deflection in the second test shall not exceed 10 percent of the maximum deflection during the test.

F2.2 Strength test - Strength test is used to confirm the calculated resistance of a structure or component. Where a number of items are to be constructed to a common design, and one or more prototypes are tested to confirm their strength, the others may be accepted without any additional test, provided they are similar in all relevant respects to the prototype.

Before carrying out the strength test, the specimen should first be subjected to and satisfy the acceptance test. Since the resistance of the assembly under test depends on the material properties, the actual yield strength of all the steel materials in the assembly shall be determined from coupon tests (test piece as defined in IS 1608). The mean value of the yield strength, f_{ym}, taken from such tests shall be determined with due regard to the importance of each element in the assembly.

The strength test load $F_{test,s}$ (including self weight) shall be determined from

$$F_{test,\,s} = \gamma_{mi} F_d \,(f_{ym} / f_y)$$

where

f_y = characteristic yield stress of the material as assumed in the design

F_d = factored design load for the ultimate limit state, and

γ_{mi} = partial safety factor for the type of failure, as prescribed in this Standard Specification

At this load there shall be no failure by buckling or rupture of any part of the Standard structure or component tested. On removal of the test load, the deflection should decrease by at least 20 percent of the maximum deflection at $F_{test.s}$.

F2.3 Test to failure (ultimate strength test) - The objective of a test to failure is to determine the design resistance from the ultimate resistance. In this situation it is still desirable to carry out the acceptance and strength tests, before test to failure.

Not less than three test shall be carried out on nominally identical specimens An estimate should be made of the anticipated ultimate resistance as a basis for such tests. During a test to failure, the loading shall first be applied in increments upto the strength test load. Subsequent load increments shall then be determined from consideration of the principal load deflection plot. The test load resistance, $F_{tests\ R}$ shall be determined as that load at which the specimen is unable to sustain any further increase in load. At this load, gross permanent distortion is likely to have occurred and in some cases such large gross deformation may define the test limit. If the deviation of any individual test result exceeds 10 percent, of the mean value obtained for all the three tests, atleast three more tests shall be carried out. When the deviation from the mean does not exceed 10 percent of the mean, the design resistance may be evaluated as given below:

a) When the failure is ductile, the design resistance, F_d, may be determined from

$$F_d = 0.9\ F_{test.min}\ (f_y / f_{ym}) / \gamma_{m0}$$

where

$F_{test.min}$ = minimum test result from the tests to failure

f_{ym} = average yield strength as obtained from the material tests

f_y = characteristic yield stress of the grade of steel

b) In the case of a sudden (brittle) rupture type failure, the design resistance may be determined from :

$$F_d = 0.9\ F_{test.min}\ (f_u / f_{um}) / \gamma_{m0}$$

where

f_u = characteristic ultimate stress of the grade of steel used

f_{um} = average ultimate tensile strength of the material obtained from tests

c) In the case of a sudden ("brittle") buckling type failure, the design resistance shall be determined from :

$$F_d = 0.75\, F_{test.min}\, (f_y / f_{ym}) / \gamma_{m0}$$

d) In ductile buckling type failure in which the relevant slenderness can be reliably assessed, the design resistance may be determined from:

$$F_d = 0.9\, F_{test.min}\, (\chi f_y / \chi_m f_{ym}) / \gamma_{m0}$$

where

χ = reduction factor for the relevant buckling curve

χ_m = value of χ when the yield strength is f_{ym}

F2.4 Check tests - Where a component or assembly is designed on the basis of strength tests or tests to failure and a production run is carried out of such items, an appropriate number of samples (not less than two) shall be selected from each production batch at random for check tests.

F2.4.1 The samples should be carefully examined to ensure that they are similar in all respects to the prototype tests, particular attention being given to the following:

a) Dimensions of components and connections

b) Tolerance and workmanship

c) Quality of steel used, checked with reference to mill certificates

F2.4.2 Where it is not possible to determine either the variations or the effect of variations from the prototype, an acceptance test shall be carried out as a check test.

F 2.4.3 In this check test, the deflections shall be measured at the same positions as in the acceptance test of the prototype. The maximum measured deflection shall not exceed 120 percent of the deflection recorded during the acceptance test on the prototype and the residual deflection should not be more than 105 percent of that recorded for the prototype.

F3. Test Conditions

a) Loading and measuring devices shall be calibrated in advance

b) The design of the test rig shall be such that

i) The loading system adequately simulates the magnitude and distribution of the loading.

ii) It allows the specimen to perform in a manner representative of service conditions,

iii) Lateral and torsional restraint, if any should be representative of those in service,

iv) The specimen should be free to deflect under load as per the service condition,

v) The loading system shall be able to follow the movements of the specimen without interruption or abnormal restraints.

vi) Inadvertent eccentricities at the point of application of the test loads and at the supports are avoided

c) The test load shall be applied to the unit at a rate as uniform as practicable.

d) Deflections should be measured at sufficient points of high movements to ensure that the maximum value is determined.

e) If the magnitude of stresses in a specimen is to be determined, the strain at the desired location may be measured and the corresponding stress calculated.

f) Prior to any test, preliminary loading (not exceeding the characteristic values of the relevant loads) may be applied and then removed, in order to set the test specimen on to the test rig.

F4. Test Loading

F4.1 Where the self-weight of the specimen is not representative of the actual permanent load in service, allowance for the difference shall be made in the calculation of test loads to be applied.

F4.2 On the attainment of maximum load for either acceptance or strength tests, this load shall be maintained for at least 1 hour. Reading of load and deflection shall be taken at intervals of 15 minutes and the loading shall be maintained constant until there is no significant increase in deflection during a 15 minutes period or until at least 1 hour has elapsed.

F4.3 The test load shall be equal to the design load for the relevant limit state in proof testing.

F4.4 The test load in prototype testing shall be equal to the design load for the relevant limit state as multiplied by the appropriate factor given in **Table F.1**.

F5. Criteria for Acceptance

F5.1 Acceptance for strength - The test structure, sub-structure, member or connection shall be deemed to comply with the requirements for strength if it is able to sustain the strength test load for atleast 15 minutes.

Table F.1 Factors to Alow for Variability of Structural Units

(Clause **F 4.4**)

No. of similar Units to be tested	Strength limit state	Serviceability Limit State
1	1.5	1.2
2	1.4	1.2
3	1.3	1.2
4	1.3	1.1
5	1.3	1.1
10	1.2	1.1

It shall then be inspected to determine the nature and extent of any damage incurred during the test. The effects of the damage shall be considered and if necessary appropriate repairs to the damaged parts carried out.

F5.2 Acceptance for serviceability - The maximum deformation of the structure or member under the serviceability limit state test load shall be within the serviceability limit values appropriate to the structure.

ANNEX-G

WORKING STRESS DESIGN

(Based on IRC:24-2001)

G1. General

In this method structural members are designed so that the unit stress caused by the design loads does not exceed a pre-defined permissible stress. The permissible stress is defined as the yield stress multiplied by a factor of safety. This factor of safety caters for the uncertainties in loads, material strength, behaviour of the structure, corrosion, fatigue etc.

In the working stress method, apart from structural strength, serviceability requirements such as deflection etc also need to be considered and properly checked.

G2. Loads and Stresses

G2.1 Combinations

G2.1.1 Main effects

For the purpose of computing stresses, the classifications (column 1) and combinations (column 2) as given in **Table G.1** will be followed. For legend of symbols under Combination (column 2) refer to Clause **202.1** of IRC:6.

G2.1.2 Other effects

G2.1.2.1 Secondary Effects (F_s) shall include, where applicable, the effects due to creep and shrinkage of concrete for composite deck and warping for box girder sections.

G2.1.2.2 Erection effects shall include the loads and forces arising out of construction equipment and the effects of wind/ seismic.

G2.2 Permissible increase in stress

G2.2.1 Increase

The permissible increase (percent) in stress in the various members covered by this code due to combination stated in Clause **G2.1** shall be as given under Increase (column 3) of **Table G.1**.

G2.2.2 Limitation

The above permissible increase in stress, shall however be limited to 90 percent of yield stress.

Table G.1

Classification (1)	Combination (2)	Increase (3)
I	$G + Q$ or $G_s + Q_{im} + F_{wc} + F_f + G_b + F_{cf} + F_{ep} + G_c$	Nil
II	(I) + $F_s + F_d + F_{te}$	15 percent
III	(II) + $W + F_{wp}$	25 percent
IV	(II) + $F_{eq} + F_{wp}$	40 percent
V	(II) + $F_{im} + W$	25 percent
VI	$G + F_{wc} + G_b + F_{ep} + F_{er} + F_f + W + G_c$	30 percent
VII	(VI) + F_{eq} - W	40 percent

G2.3 Worst effect

Subject to the provision of other clauses, all forces shall be considered as applied and all loaded lengths chosen in such a manner that the worst adverse effect is caused on the member under consideration.

G2.4 Working stresses

G2.4.1 Basic permissible stresses

The basic permissible stresses for steelwork are given in **Table G.2**.

Table G.2 Basic Permissible Stresses

1)	Axial tension on net area	0.6 f_y
2)	Axial compression on effective section	0.6 f_y
3)	Bending In plates, flats, tubes and similar sections In girders and rolled sections	 0.66 f_y 0.62 f_y
4)	Shear Stress Maximum Average For yield stress $f_y \leq 250$ MPa For $f_y > 250$ MPa	 0.43 f_y 0.38 f_y 0.35 f_y
5)	Bearing stress on flat surface	0.8 f_y

However, the permissible stresses in axial or flexural compression shall not exceed those as per relevant clauses considering the effect of buckling.

G2.4.2 Equivalent stress

G2.4.2.1 When a member is subjected to a combination of stresses, the equivalent stress $\sigma_{e,\ cal}$ due to combination of shear stress $\tau_{v,\ cal}$ bearing stress $\sigma_{p,\ cal}$ and bending stress $\sigma_{bt,\ cal}$ tensile or $\sigma_{bc,\ cal}$ compressive is calculated from

$$\sigma_{e,\ cal} = \sqrt{(\sigma_{bt,cal})^2 + (\sigma_{p,cal})^2 + (\sigma_{bt,cal})(\sigma_{p,cal}) + 3(\tau_{v,cal})^2}$$

or $$\sigma_{e,\ cal} = \sqrt{(\sigma_{bc,cal})^2 + (\sigma_{p,cal})^2 - (\sigma_{bc,cal})(\sigma_{p,cal}) + 3(\tau_{v,cal})^2}$$

$\sigma_{e,\ cal}$ shall not exceed the permissible stresses as indicated in relevant sections under different combination of stresses.

G2.4.2.2 Irrespective of the permissible increase of stress in other clauses, the equivalent stress $\sigma_{e,\ cal}$ calculated in Clause **G2.4.2.1** above shall not exceed 92 percent of yield stress.

G2.5 Permissible Stresses in Bolts, Rivets & Tension Rods

G2.5.1 Fasteners

All fasteners would be in accordance with Indian Standards. For bolts the yield stress used for calculating the permissible stress would be derived from the property class chosen as per relevant Indian Standards. The nut should be of matching property class. For hot rolled and high tensile rivets the yield stress would be in accordance with the relevant Indian Standards

G2.5.2 Calculation of stresses

In calculating shear and bearing stresses the effective diameter of a rivet shall be taken as the hole diameter and that of bolt shall be taken as its nominal diameter. In calculating the axial tensile stress in a rivet the gross area shall be used and in calculating the axial tensile stress in a bolt or screwed tension rod the net area shall be used.

G2.5.3 Gross and net area

G2.5.3.1 The gross area of a rivet shall be taken as the cross sectional area of the rivet hole. The nominal diameter of rivet shall be the diameter (cold) before driving. The nominal area of a rivet shall be based on the nominal diameter.

G2.5.3.2 The net sectional area of a bolt or a screwed tension rod shall be taken as the area of the root of the threaded part or cross sectional area of the unthreaded part whichever is lesser. The nominal diameter of a bolt shall be the diameter of the shank of the bolt. The nominal area of a bolt shall be based on the nominal diameter.

G2.5.4 Basic permissible stresses

The basic permissible stresses for rivets, bolts, tension rods are given in **Table G.3**.

G2.5.5 Combined tensile & shear stresses

Rivets and bolts subject to shear and externally applied tensile forces shall be so proportioned that the quantity.

$$\left[(\sigma_{tf,cal}/\sigma_{tf})^2 + (\tau_{vf,cal}/\tau_{vf})^2\right] \leq 1$$

where

$\sigma_{tf,cal}$ = actual tensile stress in the rivet or bolt

σ_{tf} = permissible tensile stress in the rivet or bolt as given in **Table G.3**

$\tau_{vf,cal}$ = actual shear stress in the rivet or bolt,

τ_{vf} = permissible shear stress in the rivet or bolt as given in **Table G.3**

G2.5.6 HSFG bolts

High strength friction grip bolts shall be used in conformity with IS 4000-1992.

G2.6 Permissible Stresses in Welds

G2.6.1 Basic permissible stresses

The basic permissible stresses in weld shall be as per Indian Standards namely IS 816-1969 and as modified in IS 1024-1979.

G2.6.2 Shop welds

G2.6.2.1 Butt welds

But weld shall be treated as parent metal with a thickness equal to the throat thickness, and the stress shall not exceed those permitted in the parent metal.

Table G.3 Basic Permissible Stresses for Rivets, Bolts and Tension Rods

1)	In tension	
	Axial stress on nominal area of rivet and on net area of bolts and tension roads :	
	Power driven shop rivets	0.33 f_y
	Power driven field rivets	0.27 f_y
	Bolts over 38 mm dia	0.53 f_y
	Bolts 20 mm up to 38 mm dia	0.40 f_y
	Bolts less than 20 mm dia	0.33 f_y
	Tension Rods	0.53 f_y
2)	In shear	
	Shear stress on gross area of rivets and nominal area of bolts :	
	Power driven shop rivets	0.43 f_y
	Power driven field rivets	0.40 f_y
	Hand driven rivets	0.33 f_y
	Turned and fitted bolts (IS 3640)	0.43 f_y
	Black bolts (IS 1363)	0.37 f_y
3)	In bearing	
	Bearing stress on gross diameter of rivets and nominal diameter bolts :	
	Power driven shop rivets	1.00 f_y
	Power driven field rivets	0.90 f_y
	Hand driven rivets	0.67 f_y
	Turned and fitted bolts (IS 3640)	1.00 f_y
	Black bolts (IS 1363)	0.87 f_y

G2.6.2.2 Fillet welds

The basic permissible stress in fillet welds shall not exceed the permissible shear stress as follows :

Steel Conforming	Electrode Designation as per IS 815-1974	Shear Stress Mpa
IS 2062 upto Grade E250	EXXX-43X	108
IS 2062 Grade E300 and above	EXXX-51X	131

G2.6.2.3 Plug welds

The permissible shear stress in plug welds will not exceed those given for fillet welds as above.

G2.6.3 Site welds

The permissible stresses for shear and tension for site welds made during erection of structural members shall be reduced to 80 percent of those given in Clause **G2.6.2** above. Site welding should be proposed only if quality welds can be ensured at site including facilities for testing the welds as per codal requirements. The percentage of site welds to be tested should be 100 percent as given under Clauses **513.6.12.7.2 to 4**.

G2.6.4 Combined stresses in a weld

G2.6.4.1 When a weld is subjected to a combination of stresses, the equivalent stress $\sigma_{e,cal}$ due to combination of shear stress $\tau_{v,cal}$ bearing stress $\sigma_{p\,cal}$ and bending stress $\sigma_{bt,cal}$ tensile or $\sigma_{bc,cal}$ compressive is calculated from

$$\sigma_{e,cal} = \sqrt{(\sigma_{bt,cal})^2 + (\sigma_{p,cal})^2 + (\sigma_{bt,cal})(\sigma_{p,cal}) + 3(\tau_{v,cal})^2}$$

$$\text{or } \sigma_{e,cal} = \sqrt{(\sigma_{bc,cal})^2 + (\sigma_{p,cal})^2 - (\sigma_{bc,cal})(\sigma_{p,cal}) + 3(\tau_{v,cal})^2}$$

$\sigma_{e,cal}$ shall not exceed the permissible stresses as indicated in relevant sections under different combination of stresses.

G2.6.4.2 Irrespective of the permissible increase of stress in other clauses, the equivalent stress $\sigma_{e\,cal}$ calculated in Clause **G2.6.4.1** above shall not exceed 92 percent of yield stress f_y.

G2.7 Stress Analysis

G2.7.1 General

The global analysis of the structure should be done using an elastic method. For structures in which the load effects are not proportional to the loads and/or the secondary effects due to deformation are significant, the method of analysis should be suitable for treatment of non-linear behaviour.

G2.7.2 Sectional properties

The sectional properties to be used in global analysis should generally be calculated for the gross section assuming the specified sizes. For beams or trusses on flexible supports account should however be taken of its influence of shear lag on their stiffenesses. The effect of shear lag should also be taken into account in analysis of conditions during erection of continuous girders of box construction or with integral decks.

G2.7.3 Longitudinal stresses in beams

The distribution of longitudinal stress between the flanges and web or webs of a beam may be calculated on the assumption that plane section remains plane, but using effective widths of the flanges and the effective thickness of a deep web in accordance with the provisions of Clause G4, no further account need be taken of deformation of the plate out of its plane.

G2.7.4 Shear stress

The design values of shear stress in webs of rolled or fabricated I, box or channel sections may be calculated in accordance with the provisions of Clause G4. Shear stresses in other sections should be computed from the whole cross-section having regard to the distribution of flexural stress across the section.

G2.8 Stresses

G2.8.1 Primary stresses

In the design of triangulated structures, axial stresses in members are usually calculated on the assumption that :

- all members are straight and free to rotate at the joints;
- all joints lie at the intersection of the centroidal axes of the members
- all loads, including the weight of the members, are applied at the joints.

These stresses are defined as primary stresses.

G2.8.2 Secondary stresses

In practice these assumptions are not realised and consequently members are subjected not only to axial stress but also to bending and shear stresses. These stresses are defined as secondary stresses and fall into two groups :

i) Stresses which are the result of eccentricity of connections and off-joint loading generally (i.e. loads rolling directly on chords, self weight of member and wind loads on member)

ii) Stresses which are the result of the elastic deformation of the structure and the rigidity of the joints. These are known as deformation stresses.

G2.8.2.1 Structures shall be designed, fabricated and erected in such a manner as to minimise as far as possible secondary stresses.

G2.8.2.2 Secondary stresses which are the result of eccentricity of connections and of off-joint loading [under Clause **G2.8.2 (i)** above], shall be computed and combined with the coexistent axial stresses in accordance with appropriate clause, but secondary stress due to the self weight and wind on the member shall be ignored in this case.

NOTE : In computing the secondary stress due to loads being carried direct by a chord, the chord may be assumed to be a continuous girder supported at the panel points, the resulting bending moments, both at the centre and at the supports being taken as equal to 3/4 of the maximum bending moment in a simply supported beam of span equal to the panel length. Where desired, calculations may be made and the calculated bending moments may be taken. In computing such bending moments, the impact allowances shall be based on a loaded length equal to one panel length.

G2.8.2.3 Secondary stresses which are the result of the elastic deformation of the structure (under Clause **G2.8.2 (ii)** above) shall be either computed or assumed in accordance with Clause **G2.8.3** below and combined with the coexistent axial stresses.

G2.8.3 Deformation stresses

In order to minimize the deformation stresses in girder, the ratio of the width of the members in the plane of distortion to their length between centre of intersections shall preferably be not greater than 1/12 of the chord members and 1/24 of web members. In the absence of calculations the deformation stresses shall be assumed to be not less than $16\frac{2}{3}$ percent of the dead load and live load stresses including impact.

G2.8.3.1 All open web girders of effective spans greater than 50 m may properly be cambered. Recommended procedure for cambering such girders is given in **Annex–B**. For such girders, deformation stresses (under Clause **G2.8.3**) above may be ignored.

G3. General Design Considerations

G3.1 Effective Spans

Refer to Clause **504.1**

G3.2 Effective Depths

Refer to Clause **504.2**

G3.3 Spacing of Girders

Refer to Clause **504.3**

G3.4 Depth of Girders

Refer to Clause **504.4**

G3.5 Deflection of Girders

Refer to Clause **504.5**

G3.6 Camber

Refer to Clause **504.6**

G3.7 Minimum Sections

Refer to Clause **504.7**

G3.8 Sectional Area

G3.8.1 Gross sectional area

The gross sectional area shall be the area of the cross section as calculated from the specified sizes.

G3.8.2 Effective sectional area

G3.8.2.1 Tension members - The effective sectional area of the member shall be the gross sectional area with the following deductions as appropriate.

G3.8.2.1.1 Except as required in Clause **G3.8.2.1.2** the areas to be deducted shall be the sum of the sectional areas of the maximum number of holes in any cross-section at right angles to the direction of stress in the member.

G3.8.2.1.2 In the case of :

a) all axially loaded tension members

b) beams of structural steel conforming to IS 2062 upto Grade E250 and with d/t greater than 85

c) beams of structural steel conforming to IS 2062 Grade 300 and above and with d/t greater than 75

where

t = thickness of web, and

d = depth of beams to be taken as the clear distance between flanges ignoring fillets.

and where bolt or rivet holes are staggered, the area to be deducted shall be the greater of the following :

i) the maximum number of the holes in any cross-section at right angles to the direction of stress in the member

ii) the sum of the sectional areas of all holes in a chain of lines extending progressively across the member, less s^2 t/ 4g for each line extending between holes at other than right angles to the direction of stress, where, s,g and t are respectively the staggered pitch, gauge, and thickness associated with the line under consideration. The chain of lines shall be chosen to produce the maximum such deduction. For non-planer sections, such as angles with holes in both legs, the gauge, g, shall be the distance along the centre of the thickness of the section between hole centres.

NOTE : In a built-up member where the chains of holes considered in individual parts do not correspond with the critical chain of holes for the members as a whole, the value of any rivets or bolts joining the parts between such chains of holes shall be taken into account in determining the strength of the member.

G3.8.2.1.3 Angles and tees in tension

a) In the case of single angle connected through one leg the net effective sectional area shall be taken as :

$$A1 + A2 \text{ x } k$$

where

$A1$ = effective cross-sectional area of the connected leg

$A2$ = the gross cross-sectional area of the unconnected leg, and

k = $3A1 \div (3A1 + A2)$

where lug angles are used, the effective sectional area of the whole of the angle member shall be considered.

b) In the case of pair of angles back-to-back (or a single tee) connected by one leg of the angle (or by the flange of the tee) to the same side of a gusset, the net effective area shall be taken as

$A1 + A2 \times k$

where

A1 and A2 are as defined in **G3.8.2.1.3 (a)** and

$k = 5A1 \div (5A1 + A2)$

The angles shall be connected together along their length in accordance with the requirements as given in Clause **506.2.6.1**.

G3.8.2.2 Compression members

The gross sectional area shall be taken for all compression members subject to relevant clauses.

G3.8.2.3 Parts in shear

The effective sectional area for calculating average shear stress for parts in shear shall be as follows :

a) *Rolled beams and channels* - The product of the thickness of the web and the overall depth of the section.

b) *Plate girder* - The product of the thickness of the web and the full depth of the web plate.

NOTES:

1) Where webs are varied in thickness in the depth of the section by the use of tongue plates or the like and in the case of other sections, the maximum shear stress shall be computed from the whole area of cross-section having regard to the distribution of flexural stresses.

2) Webs which have openings larger than those used for rivets, bolts or other fastenings require special consideration and the provisions of this clause are not applicable.

G3.9 Skew Bridges

Refer to Clause **504.8**.

G4. Design of Beams

G4.1 General

G4.1.1 Beams shall be proportioned on the basis of the moment of inertia of the gross cross-section with the neutral axis taken at the centroid of that section. In computing the maximum stresses, the stresses calculated on this basis shall be increased in the ratio of the gross to the effective area of the flange section. For this purpose the flange sectional area in riveted or bolted construction shall be taken to be that of the flange plates, flange angles and the portion of the web and side plates, if any, between the flange angles. In welded construction, the flange sectional area shall be taken to be that of flange plates and of the tongue plates (i.e. the thick vertical plates connecting flange to web) if any, upto a depth of the tongue plate equal to eight times its thickness, which shall not be less than twice that of the web.

G4.2 Web Plates

G4.2.1 Minimum thickness

Refer to Clause **509.6**.

G4.3 Flanges

G4.3.1 The effective sectional area of compression flanges shall be the gross area with the specified deduction for excessive width of plates (Clause **G4.3.3, G4.3.4**) and maximum deduction for open holes and holes for bolts occurring in section perpendicular to the axis of the member.

G4.3.2 The effective sectional area of tension flanges shall be the gross sectional area with specified deduction for excessive width or projection of plates (Clause **G4.3.5**) and deduction of all holes as specified for rivet or bolt holes in tension members in Clause **G3.8.2.1.**

G4.3.3 In riveted or bolted construction, flange angles shall form as large a part of the area of the flange as practicable (preferably not, less than 1/3) and the number of flange plates shall be kept to a minimum. Where flange plates are used, they shall preferably of equal thickness and at least one plate of the top flange shall extend the full length of the girder, unless the top edge of the web is finished flush with the flange angles.

Compression flange plates unstiffened at their edges shall not project beyond the outer lines of connections to the flange angles by more than 16t' for steel conforming to IS 2062 upto

Grade E250 or 14t' for steel conforming to IS 2062 Grade E300 and above where t' is the thickness of the thinnest flange plate or the aggregate thickness of the two or more plates when projecting portions of these plates are adequately tacked together.

G4.3.4 In welded construction, compression flange plates unstiffened at their edges shall not project beyond the line of connection to the web or tongue plates by more than 12t'.

G4.3.5 In all cases tension flange plates, stiffened or unstiffened at their edges, shall not project beyond the outer line of connections to the flange angles (or where there are no flange angles, to the web or tongue plates) by more than 20t'.

G4.3.6 For the flanges of beams with vertical stiffeners only (see Clause **G4.11.2.2**), where d_1/t is greater than 130 in the case of steel conforming to IS 2062 upto Grade E250 or 110 in the case of steel conforming to IS 2062 Grade E300 and above and when the average shear stress in the web is greater than 0.6 of the permissible stress given for mild steel in Clause **G2.4.1**, the quantity $I/(b^3t)$ shall not be less than 2.5 x 10^{-4} in the case of steel conforming to IS 2062 upto Grade E250 and 3 x 10^{-4} in the case of steel conforming to IS 2062 Grade E300 and above.

where

I = the moment of inertia of the compression flange about its axis normal to the web, taken as that of the flange angles and plates and the enclosed portion of the web in the case of riveted construction and in the case of welded construction, as the flange plate together with a depth of web (adjacent to the flange plate) equal to 16 times the web thickness.

d_1 = effective depth of the girder as defined in Clause **509.6**

b = spacing of stiffeners

t = thickness of web

G4.4 Effective Length of Compression Flanges

The effective length of the compression flange for buckling normal to the plane of the girder shall be as given below.

G4.4.1 Simply supported beams with no intermediate lateral support to compression flange, but with each end restrained against torsion

G4.4.1.1 When there is no intermediate lateral restraint to a compression flange, effective length l should be taken as

$$l = k_1 L$$

where

L = Span of the beam (i.e between restraint at supports)

k_1 = 1.0 if the compression flange is free to rotate in plan at the points of support, or

= 0.85 if the compression flange is partially restrained against rotation in plan at one support and free to rotate in plan at the other, or

= 0.7 if the compression flange is fully restrained against rotation in plan at the points of support.

G4.4.1.2 Restraint against torsion at the supports can be provided by web or flange cleats, by bearing stiffeners, by end frames or by lateral supports to the compression flange. The restraint element shall be designed to resist, in addition to the effects of wind and other applied lateral forces, the effects of a horizontal force acting normal to the compression flange of the girder at the level of the centroid of this flange where

$$F = \frac{1.4\times10^{-3}\times l}{\delta(f_{cb}/f_{bc}-1.7)}$$

In the above formula :

l = has the value given in Clause **G4.4.1.1**

f_{cb} = the critical stress in the flange as defined in Clause **G4.6.2**

f_{bc} = the calculated working stress in flange

δ = the deflection of the flange under the action of unit horizontal force as defined in Clause **G4.4.2**

G4.4.2 Simply supported beams with compression flange laterally supported by U-Frames.

For simply supported girders where there is no lateral bracing of the compression flanges but where cross members and stiffeners forming U-Frames provide lateral restraint.

$$l = 2.5 \text{ x } \sqrt[4]{EI_c a\delta} \quad \text{but not less than "}a\text{"}$$

where

E = Young's Modulus

I_c = Maximum moment of inertia of compression flange about its centroidal axis parallel to the web of the girder.

a = distance between frames

δ = the lateral deflection which would occur in the U-Frame at the level of the centroid of the flange being considered when a unit force acts laterally to the U-Frame only at this point and simultaneously at each corresponding point on the other flange or flanges connecting to the same U-Frame. The direction of each unit force should be such as to produce the maximum aggregate value of δ. The U-Frame should be taken as fixed in position at each point or intersection between the cross member and a vertical as free and unconnected at all other points.

when δ is not greater than $a^3 / (40 E I_c)$

$l = a$

In cases of symmetrical U-Frames where cross members and stiffeners are each of constant moment of inertia throughout their own length.

$$\delta = \frac{(d_1)^3}{3EI_1} + \frac{(d_2)^2 b}{EI_2}$$

where

d_1 = distance of the centroid of the compression flange from the top of the cross member

d_2 = distance of the centroid of the compression flange from the neutral axis of the cross member

b = half the distance between centres of the main girders.

I_1 = the moment of inertia of a pair of stiffeners about the centre of the web, or a single stiffener about the face of the web. A width of web plate upto 16 times the web thickness may be included on each side of centre lines of connection.

I_2 = Moment of inertia of the cross member in its plane of bending

G4.4.3 Beams with laterally supported compression flanges

When a compression flange is provided with effective discrete lateral restraints effective length l should be taken as the greatest distance centre-to-centre of restraint members between a restraint and a support. Where such restraint is provided by interconnecting bracing, consideration should be given to the possibility of lateral instability of the combined cross-section.

G4.4.4 Cantilever beams without intermediate lateral support

When a cantilever beam is not provided with lateral support between its support and tip, l may be taken from **Table G.4** where L is the length of cantilever.

Table G.4 Effective Length *l* for A Cantilever Beam Without Intermediate Lateral Restraint

(Clause **G4.4.4**)

Restraint Conditions		**Position of Load**	
At support	At tip	On tension flange where there is no lateral restraint to load or flange	All other position
1) Built in	a) Free	1.4 L	0.8 L
	b) Tension flange held against displacement	1.4 L	0.7 L
	c) Both flanges held against lateral displacement	0.6 L	0.6 L
2) Continuous and both flanges held against lateral displacement	a) Free	2.5 L	1.0 L
	b) Tension flange held against displacement	2.5 L	0.9 L
	c) Both flanges held against lateral displacement	1.5 L	0.8 L
3) Continuous with tension flange held against lateral displacement	a) Free	7.5 L	3.0 L
	b) Tension flange held against displacement	7.5 L	2.7 L
	c) Both flanges held against lateral displacement	4.5 L	2.4 L

NOTE : L is the length of the cantilever

G4.4.5 Beams continuously restrained by deck at compression flange level

A compression flange continuously supporting a reinforced concrete or steel deck shall be deemed to be effectively restrained laterally throughout its length (i.e. l = 0) if the frictional or positive connection of the deck to the flange is capable of resisting a lateral force of 2.5 percent of the force in the flange at the point of maximum bending moment, distributed uniformly along length.

G4.5 Slenderness Ratio

The slenderness ratio λ $(i.e, l/r_{yy})$ of a beam shall not exceed 300 and it shall not exceed 150 for cantilevers.

where

l = effective length of the compression flange as specified in Clause **G4.4**

r_{yy} = the radius of gyration of the whole beam about its y-y axis based on the gross moment of inertia and the gross sectional area.

G4.6 Permissible Bending Stresses

G4.6.1 The tensile and compressive bending stresses calculated according to Clause **G4.1.2** shall not exceed the appropriate permissible stresses in **Table G.2**.

G4.6.2 For beams and plate girders with l_y smaller than l_x

where

I_y = moment of inertia of the whole section about the axis lying in the plane of bending (y-y axis)

I_x = moment of inertia of the whole section about the axis normal to the plane of bending (x-x axis)

The bending compression stress calculated according to Clause **G4.1** shall not exceed the permissible bending compressive stress σ_{bc} given in **Table G.5** corresponding to f_{cb}, (elastic critical stress), calculated as follows :

Table G.5 Values of σ_{Bc} Calculated from f_{cb} for Different Values of f_y

(Clause **G4.6.2**)

All Units in MPa

$f_y \rightarrow$ $f_{cb} \downarrow$	**250**	**340**	**400**
20	13	13	13
30	19	19	19
40	25	26	26
50	31	31	32
60	36	37	38
70	41	43	44
80	46	48	49
90	51	54	55
100	55	59	60
110	60	64	65
120	64	68	70
130	67	73	75
140	71	77	80
150	74	81	84
160	78	85	89
170	81	89	93
180	84	93	97
190	87	97	102
200	89	100	105
210	92	103	109
220	94	106	112
230	93	110	116
240	99	113	119
250	101	115	122
260	103	118	126
270	104	121	129
280	106	123	132

290	108	126	135
300	110	128	137
310	111	130	140
320	113	133	143
330	114	135	145
340	115	137	148
350	117	139	150
360	118	141	152
370	119	143	155
380	120	144	157
390	121	146	159
400	122	148	161
420	124	151	165
440	126	154	169
460	128	157	172
480	130	159	175
500	131	162	178
520	133	164	181
540	134	166	184
560	135	168	187
580	136	170	189
600	137	172	192
620	138	174	194
640	139	175	196
660	140	177	198
680	141	178	200
700	142	180	202
720	143	181	204
740	143	182	205
760	144	184	207
780	145	185	208

800	145	186	210
850	147	188	213
900	148	191	216
950	149	193	219
1000	150	195	222
1050	151	196	224
1100	152	198	226
1150	152	199	228
1200	153	200	230
1300	154	203	233
1400	155	205	236
1500	156	206	238
1600	157	208	240
1700	157	209	242
1800	128	210	243
1900	158	211	245
2000	159	212	246
2200	160	213	248
2400	160	215	250
2600	161	216	251
2800	161	216	252
3000	161	217	253
3500	162	218	255
4000	163	219	257
4500	163	220	258
5000	163	221	259
5500	163	221	259
6000	164	222	260

Elastic critical stress

The elastic critical stress f_{cb} for beams and plate girders with I_y smaller than I_x shall be calculated using the following formula :

$$f_{cb} = k_1 (X + k_2 Y) (c_2 / c_1)$$

where

X	=	$Y \sqrt{1 + (1/20)[(l T)/(r_y D)]^2}$ MPa
Y	=	$26.5 \times 10^5 / (l / r_y)^2$ MPa
c_1, c_2	=	respectively the lesser and greater distances from the section neutral axis to the extreme fibres
D	=	overall depth of the beam
T	=	mean thickness of the compression flange
l	=	effective length of compression flange
r_y	=	radius of gyration of the section about its axis of minimum strength (y-y axis)
k_1	=	a coefficient to allow for reduction in thickness or breadth of flanges between the points of effective lateral restraint and depends on ψ the ratio of total area of both flanges at the point of least bending moment to the corresponding area at the point of greatest bending moment between such points of restraint. Values of k_1 for different values of ψ are given in **Table G.6**.
k_2	=	a coefficient to allow for the inequality of flanges and depends on ω, the ratio of the moment of inertia of the compression flange alone to that of the sum of the moments of inertia of the flanges each calculated about its own axis parallel to the axis of the girder, at the point of maximum bending moment. The values of k_2, for different values of ω are given in **Table G.7** Values of X and Y for appropriate values of D/T and l/r_y are given in **Table G.8**.

Table G.6 Values of k_1 for Beams with Curtailed Flanges

(Clause **G4.6.2**)

ψ	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.0
k_1	1.0	1.0	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2

NOTE : Flanges should not be reduced in breadth to give a value of ψ lower than 0.25

Table G.7 Values of k_2 for Beams with Unequal Flanges

(Clause **G4..6.2**)

ω	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.0
k_2	0.5	0.4	0.3	0.2	0.1	0	–0.2	–0.4	–0.6	–0.8	–1.0

G4.6.2.1 Values of f_{cb} shall be increased by 20 percent when T/t is not greater than 2.0 and d_1/t is not greater than 1344 / $\sqrt{f_y}$ where d_1 is as defined in Clause **509.6** and t the thickness of the web and the value of f_y is expressed in MPa.

G4.6.3 Beams bent about the axis of minimum strength (y-y axis)

The maximum permissible bending stress in tension or in compression in beams bent about the axis of minimum strength shall not exceed the appropriate permissible stresses in **Table G.2**.

G4.6.4 Angle and tee shapes

The bending stress in the leg when loaded with the flange or table in compression shall not exceed the appropriate permissible stresses in **Table G.2**. When loaded with the leg in compression, the permissible bending stress shall be calculated from Clause **G4.6.2** with $k_2 = -1.0$ and T = thickness of leg.

G4.7 Permissible Shear Stress

G4.7.1 Maximum shear stress

The maximum shear stress in a member having regard to the distribution of stresses in conformity with the elastic behaviour of the member in flexure, shall not exceed the appropriate permissible stress in **Table G.2**.

Table G.8 Values of X and Y for Calculating f_{cb}
(Clause G4.6.2)

	X															Y
D/T → 1/ry ↓	8	10	12	14	16	18	20	25	30	35	40	50	60	80	100	
40	2 484 2	222 2	066 1	965 1	897 1	849 1	8141	759 1	728 1	709 1	697 1	683 1	675 1	6671	663	1 656
45	2 103 1	856 1	709 1	612 1	546 1	409 1	465 1	4111	380 1	362 1	349 1	335 1	327 1	319 1	315	1 309
60	1 822	1590 1	449 1	357 1	293 1	248 1	214 1	161 1	131 1	113 1	101 1	086 1	078 1	0701	067	1 060
55	1 607 1	389 1	234 1	166 1	105 1	060 1	028	976	947	929	917	902	894	886	883	876
60	1 437 1	232 1	104 1	020	961	915	886	835	806	755	776	762	754	746	743	736
65	1 301 1	107	985	904	847	806	775	726	697	679	657	653	645	637	634	627
70	1 188 1	005	889	811	757	717	687	638	610	592	581	567	559	551	547	541
75	1 094	920	810	735	682	644	615	567	540	522	G7	497	489	481	478	471
80	1 014	849	743	672	621	584	556	G5	482	465	454	440	432	424	421	414
85	945	788	687	618	570	533	G2	461	434	417	406	392	385	377	373	367
90	886	735	639	573	526	491	464	420	394	377	366	353	345	337	334	327
95	833	689	597	534	488	454	428	385	360	343	332	319	311	304	300	294
100	787	649	560	499	455	423	398	356	331	314	304	290	283	275	272	265
110	708	582	799	443	402	371	347	307	283	268	257	244	237	229	226	219
120	644	527	451	398	359	330	308	370	247	232	222	209	202	194	191	184
130	591	482	411	361	325	298	277	240	218	204	194	181	174	167	163	157
140	546	444	378	331	297	271	251	217	195	181	172	160	153	145	142	135
150	G4	412	350	306	274	249	230	197	177	163	154	142	135	145	124	118
160	474	385	326	284	254	230	212	181	161	148	139	127	121	113	110	104
170	445	360	306	265	236	214	197	167	148	135	126	115	109	102	95	92
180	420	339	286	249	221	200	184	155	137	125	116	105	98	92	88	82
190	397	320	270	235	208	188	172	145	2127	115	107	96	90	83	80	73
200	476	304	256	222	197	177	162	136	119	107	99	89	83	76	78	66
210	358	238	243	210	186	168	153	128	112	101	93	82	76	70	66	60
220	341	275	231	200	177	159	145	121	105	94	87	77	71	64	61	55
230	326	262	220	191	169	152	138	115	99	89	82	72	66	60	56	50
240	312	251	211	182	161	145	132	109	94	84	77	67	62	55	52	46
250	299	241	202	175	151	139	126	104	90	80	73	64	58	52	49	42
260	288	231	194	167	148	133	121	99	85	76	68	60	55	48	45	39
270	277	222	186	161	142	127	115	95	82	72	66	57	52	46	42	36
280	267	214	180	155	137	122	111	91	78	69	63	54	49	43	40	34
290	257	207	173	149	132	118	107	88	75	66	60	52	46	41	38	32
300	249	200	167	144	127	114	103	84	72	64	57	49	44	38	35	30

G4.7.2 Average shear stress

The average shear stress in a member calculated on the cross section of the web shall not exceed the maximum permissible average shear stress τ_{va} as follows :-

a) *For unstiffened webs* : appropriate permissible stress in **Table. G.2**

b) *For stiffended webs* : the values given in **Tables G.9, G.10** and **G.11** as appropriate yield stress values 250, 340 and 400 MPa respectively.

NOTE : The allowable stresses given in **Tables G.9, G.10** and **G 11** apply provided any reduction of the web cross-section is due only to rivet/bolt holes etc. where large apertures are cut in the web, a special analysis shall be made to ensure that the maximum permissible average shear stresses laid down in this standard are not exceeded.

G4.8 Curtailment of Flange Plates

Each flange plate shall be extended beyond its theoretical cut-off point adequately. The extension shall contain sufficient rivets, bolts and welds to withstand the forces developed at the theoretical cut off point.

In welded construction, the use of curtailed flange plates shall be avoided as far as possible, local strengthening being provided by other means such as inserting by butt welding a thicker and or wider plate. The heavier section plate shall be suitably tapered to the lighter plate. If, in welded construction the use of curtailed flange plates cannot be avoided, the end of the plate shall be tapered in plan to a rounded end and all welds shall be continuous round the ends.

G4.9 Connection of Flanges to Web

The flanges of plate girders shall be connected to the web by sufficient rivets, bolts or welds to transmit the horizontal shear force combined with any vertical loads which are directly applied to the flange. In welded construction, where the web is in close contact with the flange before welding, vertical loads causing compression may be deemed to be resisted by the bearing between the flange and the web.

G4.10 Dispersion of Load through Flange to Web

Where a load is directly applied to a flange, it shall be considered as dispersed uniformly through the flange and the web at an angle of 45º.

Table G.9 Permissible Average Shear Stress τ_{va} in Stiffened Webs of Steel with f_y = 250 Mpa

(Clause **G4.7.2**)

Stress τ_{va} (MPa) for different distances between stiffeners

d/t ↓	0.3d	0.4d	0.5d	0.6d	0.7d	0.8d	0.9d	1.0d	1.1d	1.2d	1.3d	1.4d	1.5d
90	100	100	100	100	100	100	100	100	100	100	100	100	100
95	100	100	100	100	100	100	100	100	100	100	100	100	99
100	100	100	100	100	100	100	100	100	100	100	99	99	98
105	100	100	100	100	100	100	100	100	100	99	98	97	96
110	100	100	100	100	100	100	100	100	99	98	96	95	94
115	100	100	100	100	100	100	100	100	98	96	95	94	93
120	100	100	100	100	100	100	100	98	96	95	93	92	91
125	100	100	100	100	100	100	98	97	95	93	92	91	90
130	100	100	100	100	100	99	97	96	94	92	90	89	88
135	100	100	100	100	100	98	96	94	92	90	89	87	86
140	100	100	100	100	99	96	95	93	91	89	97	86	85
150	100	100	100	100	97	94	92	90	88	85	84	83	81
160	100	100	100	97	94	92	89	88	85	83	81	80	78
170	100	100	100	96	92	89	87	85	82	80	78	76	75
180	100	100	98	94	90	87	84	82	80	77	75	73	72
190	100	100	97	92	88	84	82						
200	100	100	95	90	86	82	81						
210	100	99	93	88	83	81							
220	100	98	91	86	81	80							
230	100	96	90	84	79			Non-applicable zone.					
240	100	95	88	83	77								
250	100	93	86	82	74								
260	100	92	85	81									
270	100	90	84	81									

NOTE : Intermediate values may be obtained by linear interpolitan.

Table G.10 Permissible Average Shear Stress τ_{va} in Stiffened Webs of Steel With f_y = 340 MPa

(Clause **G4.7.2**)

Stress τ_{va} (MPa) for different distances between stiffeners

d/t↓	0.3d	0.4d	0.5d	0.6d	0.7d	0.8d	0.9d	1.0d	1.1d	1.2d	1.3d	1.4d	1.5d
75	136	136	136	136	136	136	136	136	136	136	136	136	136
80	136	136	136	136	136	136	136	136	136	136	136	136	136
85	136	136	136	136	136	136	136	136	136	136	136	134	133
90	136	136	136	136	136	136	136	136	136	135	133	132	131
95	136	136	136	136	136	136	136	136	135	133	131	129	128
100	136	136	136	136	136	136	136	136	132	130	128	127	126
105	136	136	136	136	136	136	135	133	130	128	126	124	123
110	136	136	136	136	136	135	133	131	128	126	124	122	120
115	136	136	136	136	136	133	131	129	126	123	121	119	118
120	136	136	136	136	135	131	129	127	124	121	119	117	115
125	136	136	136	136	133	129	127	125	121	119	116	114	113
130	136	136	136	135	131	127	125	122	119	116	114	112	110
135	136	136	136	134	129	126	123	120	117	114	111	109	108
140	136	136	136	132	127	124	121	118	115	112	109	107	105
150	136	136	136	129	124	120	117	114	110	107	104	102	100
160	136	136	132	126	120	116	113	110	106	102	99	97	95
170	136	136	129	123	117	112	109	106	101	98	95	92	90
180	136	135	127	119	113	108	105	102	97	93	90	87	84
190	136	133	124	116	110	105	100						
200	136	130	121	113	106	101	96						
210	136	128	118	110	103	97							
220	136	126	116	107	99	93							
230	136	123	113	103	96								
240	134	121	110	100	92								
250	132	119	107	97	89			(Non-applicable zone)					
260	130	116	104	94									
270	128	114	102	91									

NOTE : Intermediate values may be obtained by linear interpolation.

Table G.11 Permissible Average Shear Stress τ_{va} in Stiffened Webs of Steel with f_y = 400 MPa

(Clause **G4.7.2**)

Stress τ_{va} (MPa) for different distances between stiffeners

d/t↓	0.3d	0.4d	0.5d	0.6d	0.7d	0.8d	0.9d	1.0d	1.1d	1.2d	1.3d	1.4d	1.5d
70	160	160	160	160	160	160	160	160	160	160	160	160	160
75	160	160	160	160	160	160	160	160	160	160	160	160	159
80	160	160	160	160	160	160	160	160	160	160	159	157	156
85	160	160	160	160	160	160	160	160	160	158	156	154	152
90	160	160	160	160	160	160	160	160	157	155	152	151	149
95	160	160	160	160	160	160	159	157	154	152	149	147	146
100	160	160	160	160	160	160	157	155	151	149	146	144	143
105	160	160	160	160	160	157	154	152	149	146	143	141	139
110	160	160	160	160	159	155	152	149	146	143	140	138	136
115	160	160	160	160	156	152	149	147	143	140	137	135	133
120	160	160	160	159	154	150	147	144	140	137	134	132	129
125	160	160	160	127	152	147	144	141	137	134	131	128	126
130	160	160	160	155	150	145	141	139	131	131	128	125	123
135	160	160	160	153	147	143	139	136	132	128	125	122	120
140	160	160	158	151	145	140	136	133	129	125	122	119	116
150	160	160	155	147	141	135	131	128	123	119	115	112	110
160	160	160	151	143	136	130	126	123	117	113	109	106	103
170	160	153	148	139	132	126	121	117	112	107	103	100	97
180	160	155	144	135	127	121	116	112	106	101	97	93	90
190	160	152	140	131	123	116	111						
200	160	149	137	127	118	111	106						
210	160	146	133	123	114	106							
220	1577	143	130	119	109	101							
230	155	140	126	114	105								
240	153	137	123	110	100			(Non-applicable zone)					
250	151	134	119	106	96								
260	148	131	116	102									
270	146	128	112	98									

NOTE : Intermediate values may be obtained by linear interpolation.

G4.11 Web stiffeners

Web stiffeners shall be provided as follows :

G4.11.1 Load bearing stiffeners

G4.11.1.1 General

Webs of plate girders and rolled beams shall be provided with load bearing stiffeners at points of supports and at points of concentrated load where reaction or concentrated load exceeds the value of

$$\sigma_{ac} . t . B$$

where

σ_{ac} = maximum permissible axial stress for struts as given in Clause **G2.4.2.1** for a slenderness ratio of $(d_3 \sqrt{3})/t$

t = web thickness

d_3 = clear depth of web between root fillets

B = the length of the stiff portion of the bearing plus the additional length given by dispersion at 45^o to the level of the neutral axis. The stiff portion of a bearing is that length which cannot deform appreciably in bending and shall not be taken as greater than half the depth of the beams continuous over a bearing.

G4.11.1.2 Details and design

a) Load bearing stiffeners should be in pairs (i.e, two legs of plates, angles etc.) placed symmetrically at both sides of the web. When the condition is not met the effect of the resulting eccentricity should be considered.

b) The ends of the load bearing stiffener should be closely fitted or adequately connected to both flanges. They should be shaped to allow space for any root fillet or weld connecting the web to the flange, with a clearance not exceeding five times the thickness of the web.

c) Load bearing stiffeners shall not be joggled and shall be solidly packed throughout.

d) Outstanding legs or each pair of load bearing stiffeners shall be so proportioned that the bearing stress on that part of their area in contact with the flange and clear of the root of the flange or flange angles or clear of the flange welds, does not exceed the bearing stress specified in Clause **G2.4.1**.

e) Load bearing stiffeners consisting of two legs shall be designed as struts assuming the section to consist of the pair of stiffeners together with a length of web on each side of the centre line of the stiffeners equal to twenty times the web thickness (but limited to the edge distance of the web and half the distance of the adjacent stiffener).

In case of bearing stiffeners consisting of four or more legs, the effective stiffener section should be taken to comprise the stiffeners, the web plate between the two outer legs and a portion of web plate not exceeding the length of the web as specified for single leg stiffeners on the outer sides of the outer legs.

f) The radius of gyration shall be taken about the axis parallel to the web of the beam or girder, and the working stress shall be in accordance with appropriate allowable value for a strut, assuming the effective length equal to 0.7 times the length of the stiffener.

g) The load bearing stiffeners shall be provided with sufficient rivets, bolts or welds to transmit to the web the whole of the load in the stiffeners.

h) When load bearing stiffeners at supports are the sole means of providing restraint against torsion (see Clause **G4.4.1.2**) the moment of inertia I of the stiffener shall not be less than

$$(D^3 T_m / 250) \text{ x } (R/W)$$

where

I	=	moment of inertia of the pair of stiffeners about the centre line of the web plate
D	=	overall depth of the girder
T_m	=	maximum thickness of compression flange
R	=	reaction of the bearing
W	=	total load on girder

i) In addition, the base of the stiffeners in conjunction with the bearing of the girder shall be capable of resisting a moment due to horizontal force specified in Clause **G4.4.1.2**.

G4.11.2 Intermediate stiffeners

G4.11.2.1 General

When the thickness of the web is less than the limits specified in Clause **509.6(a)**, transverse stiffeners shall be provided throughout the length of the girder. When the thickness of the web is less than the limits specified in Clause **509.6(b)**, longitudinal stiffeners shall be provided in addition to the transverse stiffeners.

In no case shall the greater unsupported clear dimension of a web panel exceed 270 t nor the lesser unsupported clear dimension of the same panel exceed 180 t where t is the thickness of the web plate.

G4.11.2.2 Transverse stiffeners

Where transverse stiffeners are required, they shall be provided throughout the length of the girder at a distance apart not greater than $1.5\ d_1$ and not less than $0.33\ d_1$, where d_1 is the depth as defined in Clause **509.6**. Where horizontal stiffeners are provided d_1 shall be taken as the clear distance between the horizontal stiffener and the farthest flange ignoring fillets.

Transverse stiffeners shall be designed so that I is not less than :

$$1.5 \times (d_1^{3} \times t^3)/S^2$$

where

I = the moment of inertia of a pair of stiffeners about the centre of the web or a single stiffener about the face of the web,

t = minimum required thickness of the web

S = the maximum permitted clear distance between stiffener for thickness t

NOTE : If the thickness of the web is made greater, or the spacing of stiffener made smaller than that required by the standard, the moment of inertia of the stiffener need not be correspondingly increased.

Intermediate transverse stiffeners, when not acting as load bearing stiffeners, may be joggled and may be single or in pairs placed one on each side of the web. Where single stiffeners are used, they should preferably be placed alternatively on opposite sides of the web. The stiffeners shall extend from flange to flange. They can be connected or fitted to, or kept well clear of the flanges.

G4.11.2.3 Longitudinal stiffeners

Where longitudinal stiffeners are used in addition to vertical stiffeners they shall be as follows:

One longitudinal stiffener, on one or both sides of the web, shall be placed at a distance from the compression flange equal to two fifths of the distance from the compression flange to the neutral axis, when the thickness of the web is less than $d_2/200$ for steel conforming to IS 2062 upto Grade E250 and $d_2/180$ for steel conforming to IS 2062 Grade E300 and above where d_2 is the depth of web as defined in Clause **509.6**. This stiffener shall have a moment of inertia I not less than 4 $S_1\, t^3$ where I and t are as defined in Clause **G4.11.2.2** and S_1 is the actual distance between stiffeners.

A second longitudinal, on one or both sides of the web shall be placed on the neutral axis of the girder when the thickness of the web is less than $d_2/250$ for steel conforming to IS 2062 upto Grade E250 or $d_2/225$ for steel conforming to IS 2062 Grade E300 and above. The stiffener shall have a moment of inertia I not less than $d_2\, t^3$ where I and t are as defined in Clause **G4.11.2.2** and d_2 in Clause **509.8**.

Longitudinal stiffeners shall extend between vertical stiffeners but need not be continuous over them or connected to them.

G4.11.2.4 External forces on intermediate stiffeners

When vertical intermediate stiffeners are subject to bending moments and shears due to the eccentricity of vertical loads, or the action of transverse forces, the moment of inertia I of the stiffeners specified in Clause **G4.11.2.2** shall be increased as follows :

a) Bending moment on stiffener due to eccentricity of vertical loading with respect to the vertical axis of the web :

   Increase of $I = (1.5\, M\, D^2)/(E\, t)$

b) Lateral loading on stiffener

   Increase of $I = (3\, P\, D^3)/(E\, t)$

where

M = the applied bending moment

D = overall depth of girder

E = Young's modulus

t = thickness of web

P = lateral force to be taken by the stiffener and deemed to be applied at the compression flange of the girder.

G4.11.2.5 Connection of intermediate stiffeners to web

Intermediate transverse and longitudinal stiffeners not subjected to external loads shall be connected to the web by welds or rivets, in order to withstand a shearing force in kilograms per millimeter run between each component of stiffener and the web, of not less than $12.6.t^2/h$, where t equals web thickness in mm and h equals the projection in mm, of the stiffener component from the web.

G4.11.2.6 Outstand of all stiffeners

Unless the outer edge of each stiffener is continuously stiffened, the outstand from the web shall be not more than the following :

For sections : $16t$ for steel conforming to IS 2062 upto Grade E250

$14t$ for steel conforming to IS 2062 Grade E300 and above

For flats : $12t$ for all steels

Where t is the thickness of the section or flat

G4.12 Flange splices

Flange joints should preferably not be located at points of maximum stress. Where splice plates are used, their area shall not be less than 5 percent in excess of the area of the flange element spliced, their centre of gravity shall coincide, as nearly as possible, with that of the element spliced. There shall be enough rivets or welds on each side of the splice to develop the load in the element spliced plus 5 percent, but in no case should the strength developed be less than 50 percent of the effective strength of the material spliced.

In welded construction, flange plates or angles shall be joined by full penetration buttwelds wherever possible. These buttwelds shall develop the full strengths of plates or angles. Where this is not possible, splice plate should be used.

G4.13 Splices in webs

Splices in the webs of plate girders and rolled sections shall be designed to resist the shears and moments at the spliced section.

In riveted construction, splice plates shall be provided on each side of the web. In welded construction web plates shall be joined by full penetration buttwelds wherever possible. Where this is not possible, splice plate may be used on both sides.

G4.14 End connections

Connections at the ends of all beams designed as simply supported beams shall have flexibility.

G4.15 Lateral bracing

All spans shall be provided with a lateral bracing system extending from end to end of sufficient strength to transmit to the bearings all lateral forces due to wind, seismic effect etc., as applicable.

G4.16 Expansion and contraction

In all bridges, provision shall be made in the design to resist thermal stresses induced, or means shall be provided for movement caused by temperature changes. Provision shall also be made for changes in length of span resulting from live loads.

G5 DESIGN OF COMPRESSION MEMBERS

G5.1 General

Design of compression members should generally follow the considerations under Clause **G7.2** under "Trusses or open web Girders" of this Annex.

G5.2 Base Plate

G5.2.1 Base Plate is a structural part which serves as medium for uniformly transferring load from member/stanchion/column to foundation.

G5.2.2 Area of base plate should be such that at any point reactive pressure acting on it is less than allowable stress of concrete in compression.

$$A_F = N/\sigma_{cc}$$

where

A_F = Area of base plate

N = Load in the member

σ_{cc} = Allowable compressive stress of concrete

For Crushing value of concrete IS 456 may be referred for guidance.

G5.2.2.1 Width of base plate should be $B = b(\text{ or } h) + 2t_s + 2c$,

where

b and h = Size of member/stanchion/column,

t_s = Thickness of saddle plate, 8 mm - 10 mm

c = Cantilever portion restricted to 100 mm - 120 mm, but not less than 20 mm from outside member, stiffener to the edge of base plate.

Length of base plate $L = \dfrac{A_F}{B}$

A_F = Area of base plate

G5.2.2.2 Thickness of base plate should not be less than 20 mm.

G5.2.2.3 Thickness of plate is determined from its bending consideration due to reactive pressure of foundation on base plate.

$$P_F = N / A_F$$

where

P_F = Reactive pressure on base plate

Base plate area in general can be divided in four types depending upon boundary conditions of support (stiffeners).

i) Cantilever

ii) Supported on two sides perpendicular to each other

iii) Supported on three sides

iv) Supported on four sides.

G5.2.2.3.1 Bending moment in case of cantilever for 1 cm width of base plate (case-1):

$$M_1 = P_F c^2 / 2$$

G5.2.2.3.2 Maximum bending moment in centre of free B-side in cases of plate having support at three sides and also at two perpendicular sides:

$$M_2 = \alpha P_F b^2$$

where

α = Coefficient as per table below depending on a/b.

a = Length in the other perpendicular direction

b = Length of free shorter side of plate

a/b	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	2.0	More than 2
α	0.06	0.074	0.088	0.097	0.107	0.112	0.120	0.126	0.132	0.133

If $a/b \leq 0.5$ the support of plate on a-side does not have any effect, as such for bending moment on base plate formula for cantilever type should be used with $c = a$.

G5.2.2.3.3 Maximum Bending Moment in case of plate having support at four sides.

$$M_3 = \beta P_F (b_1)^2$$

where

β = Coefficient as per table below, depending on a_1/b_1.

a_1 = longer side length

b_1 is short side length,

a_1/b_1	1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0	More than 2
β	0.048	0.055	0.063	0.069	0.075	0.081	0.086	0.091	0.094	0.098	0.1	0.125

In case of $a_1 / b_1 > 2$, plate works as single span simply supported beam and bending moment,

$$M_3 = P_F (b_1)^2 / 8$$

G5.2.2.3.4 Thickness of the base plate $t_B = \sqrt{\dfrac{6M}{\sigma_{bc}}}$

where

M = Maximum bending moment considering all areas in which base plate is divided.

σ_{bc} = Maximum permissible bending stress in slab base.

G5.2.2.4 Section of stiffeners saddle element of base plate and its connections are designed for loads coming on them. Main stiffeners are designed as simply supported over hanging beam loaded with uniformly distributed load equal to $qs_1 = P_F \, x \, l_1$ and checked for bending and shear stresses.

G5.2.2.4.1 Secondary stiffeners (considering cantilever) are designed for load equal to $qs_2 = P_F \, x \, l_2$ and checked for bending and shear stress.

G5.2.2.5 Welded and riveted connections are designed to transfer the loads coming on stiffeners to main member and also to connect base plate with stiffeners.

G5.2.2.6 In case of heavy load transfer from member to the base plate machining of contact surface between base plate and member is recommended and the area of the base plate shall be sufficient to limit the stress in bearing for whole of the load. In such cases the weld or rivet connecting base plate with stiffeners and main member should be designed for 25 percent of total load coming on base plate (for resisting the unforeseen bending and shear) which should be resisted by total weld length or all rivets .

G5.2.2.7 Base plate for eccentrically loaded members - Action due to bending moment in base plate along with compression causes non uniform pressure on the foundation and value of max. and min. pressure can be computed as under:

$$P_{FMAX,\,Min} = \frac{N}{BL} \pm \frac{6Mx}{BL^2} \pm \frac{6My}{LB^2}$$

Where B&L are width and length of base plate. M_x and M_y are moments in the length and width direction of base plate respectively.

G5.2.2.8 Thickness of base plate is computed as per Clauses **G5.2.2.3.1 to G5.2.2.3.4** and bending moment is calculated based on maximum pressure acting on each area in which base plate is divided, neglecting non-uniform pressure from foundation on base plate on conservative side.

G5.2.2.9 Elements of base plate main and secondary stiffeners are designed as per Clauses **G5.2.2.4** and **G5.2.2.4.1**.

G5.3 Cap plate

Cap plate serves as medium for transferring the axial load from structure above (beam, girder) uniformly to the member/stanchion

G5.3.1 The thickness of cap plate should be preferably 16-25 mm and stiffener's thickness should not be less than

$\frac{I}{15}\sqrt{\frac{2600}{F_y}}$ times width of stiffener, where F_y is the yield stress of stiffener in kg/cm^2

G5.3.1.2 When the load from beam is transferred to stanchion member through bearing stiffeners extended beyond the beam, the cap plate serves as media to transmit this load to the stiffeners connected to web of stanchion/member, or to tie beam for lattice member by rivet or weld. The cap plate is designed for specified load.

G5.3.1.2.1 If the beam/girder is supported on stanchion in such a manner that loads are directly transmitted to the flange of stanchion or main element or lattice member, cap plate should be provided as per Clause **G5.3.1** without calculation

G5.3.1.3 The width of cap stiffeners is determined from bearing criteria $b_{sc} = \dfrac{N}{\sigma_{bg.tc}}$ and also shear stress should not exceed specified stress :

$$\frac{N}{l_{sc}.t_c} \leq \sigma_s$$

where

l_{sc} = Length of stiffener, to be sufficient for transmitting the load to web of stanchion by rivet or weld.

σ_{bg} = Basic permissible bearing stress

N = Load to be transmitted from girder/ beam

b_{sc} = Width of stiffener.

t_c = Thickness of the stiffener

G6. DESIGN OF TENSION MEMBERS

G6.1 Design of tension member should generally follow the considerations under Clause **G7.3** under "Trusses or open web Girders" of this Annex.

G7. DESIGN OF TRUSSES OR OPEN-WEB GIRDERS

G7.1 General

G7.1.1 Analysis

Refer to Clause **508.2**

G7.1.2 Intersection at joints

Refer to Clause **508.3**

G7.2 Compression Members

G7.2.1 General requirements

This clause covers the design of straight members of uniform cross section subjected to axial compression or to combined compression and bending.

G7.2.2 Effective sections

G7.2.2.1 The properties of the cross section should be computed from the effective sectional area. Where plates are provided solely for the purposes of lacing or battening these shall be ignored in computing the radius of gyration of the section.

G7.2.2.2 The effective sectional area shall be the gross area less the specified deduction for excessive widths of plate, (see Clauses **G7.2.2.3 & G7.2.2.4**) and the maximum deduction for open holes, including holes for pins and black bolts occurring in a section perpendicular to the axis of the member within the critical zone of the compression member.

G7.2.2.3 For members other than circular hollow section for calculating the effective cross sectional area of a member in compression the effective width be of a plate, in terms of its width b measured between adjacent lines of rivets, bolts or welds connecting it to other parts of the section, unless effectively stiffened shall be taken as :

i) For riveted, bolted or stress relieved welded members in steel conforming to IS 2062 upto Grade E250.

For b/t not above 45, $b_e = b$

For b/t above 45, $b_e = 45t$

with a maximum value of $b/t = 90$

ii) For riveted or bolted members in steel conforming to IS 2062 Grade E300 and above

For b/t not above 45, $b_e = b$

For b/t above 45, $b_e = 40t$

with a maximum value of $b/t = 80$

iii) For welded members (not stress relieved) in steel conforming to IS 2062 (all Grades)

For b/t not above 30, $b_e = b$

For b/t above 30, $b_e = 40t \times [(b/t - 18) / (b/t - 14)]$

with a maximum value $b/t = 80$

In the above, "t" is the thickness of a single plate or the aggregate thickness of two or more plate, provided these are adequately tacked together considering maximum allowable pitch and edge distance of rivets or bolts.

G7.2.2.4 The unsupported projection of any plate, measured from its edge to the line of rivets, bolts or weld connecting the plate to other parts of the section shall not exceed :

i) *16 t* for Steel conforming to IS 2062 upto Grade E250

ii) *14 t* for Steel conforming to IS 2062 Grade E300 and above

Where "t" is as defined above. However in case of compression flanges 't' is the thickness of the thinnest flange plate or the aggregate thickness of two or more plates when the projecting portions of these plates are adequately tacked together.

G7.2.3 Permissible stresses and slenderness ratio

G7.2.3.1 Permissible stress

Values of permissible stress in axial compression in MPa for some of the structural steels corresponding to various slenderness ratios are given in **Table G.12.**

G7.2.3.2 The ratio of the effective length to the least radius of gyration shall not exceed:

120 for main members, and

140 for wind bracings and subsidiary members.

G7.2.3.3 All values of permissible stress in axial compression in MPa for structural steel with Yield stress other than those shown in **Table G.12** may be calculated by using the following formula subject to the condition that σ_{ac} shall not exceed 0.6 f_y.

$$\sigma_{ac} = 0.6 \frac{f_{cc} \cdot f_y}{\left[(f_{cc})^n + (f_y)^n \right]^{1/n}}$$

where

σ_{ac}	=	permissible stress in axial compression, in MPa;
f_y	=	yield stress of steel, in MPa;
f_{cc}	=	elastic critical stress in compression, = $\dfrac{\pi^2 . E}{\lambda^2}$;
E	=	modulus of elasticity of steel; 2 x 10^5 MPa;
$\lambda = (l/r)$	=	slenderness ratio of the member, ratio of the effective length to appropriate radius of gyration; and
n	=	a factor assumed as 1.4

Table G.12 Permissible Stress σ_{ac} (MPa) in Axial Compression for Steels with Various Yield Stresses

(Clause **G7.2.3.1**)

	Yield stress (f_y) MPa		
$\lambda = l / r \downarrow$	250	340	400
10	150	204	239
20	148	201	235
30	145	194	225
40	139	183	210
50	132	168	190
60	122	152	168
70	112	135	147
80	101	118	127
90	90	103	109
100	80	90	94
110	72	79	82
130	64	69	71
130	57	61	62
140	51	54	55
150	45	48	49
160	41	43	43
170	37	38	39
180	33	34	35
190	30	31	31
200	28	28	28
210	25	26	26
220	23	24	24
230	21	22	22
240	20	20	20
250	18	18	19

Where l = effective length of the member & r = radius of gyration

G7.2.4 Effective length of compression members other than lacings

G7.2.4.1 In riveted, bolted or welded trusses, the compression members act in a complex manner and the effective length to be used in computing allowable working stresses for compression members shall be taken as given in **Table G.13** except that, for battened struts, all values given in table shall be increased by 10 percent.

Table G.13 Effective Length of Compression Members
(Clause **G7.2.4.1**)

Member		Effective length *l* of member		
		For buckling in the plane of the truss	For bucking normal to the plane of the truss: Compression chord or (compression) member effectively braced by lateral syustem	For bucking normal to the plane of the truss: Compression chord or (compression) member unbraced
Chords		0.85 x distance between centres of intersection with the web members	0.85 x distance between centres of intersection with the lateral bracing members or rigidly connected cross girders	See Clause **G7.2.4.1**
Web	Single triangulated system	0.70 x distance between centres of intersection with the main chords	0.85 x distance between centres of intersections	Distance between centres of intersection
Web	Multiple intersection system where adequate connections are provided at all points of intersection	0.85 x greatest distance between centres of any two adjacent intersections	0.70 x distance between centres of intersection with the main chords	0.85 x distance between centres of intersection with the main chords

NOTE : The intersection referred to are those of the centroidal axis of the members.

G7.2.4.2. For single angle discontinuous struts connected to gussets or to a section either by riveting or bolting by not less than two rivets or bolts in line along the angle at each end, or by their equivalent in welding, the eccentricity of the connection with respect to the centroid of the strut may be ignored and the strut designed as an axially loaded member provided that the calculated average stress does not exceed the allowable stresses given in **Table G.12** in which "l" is the length of the strut, centre to centre of fastenings at each end and 'r' is the minimum radius of gyration.

G7.2.4.3 For single angle discontinuous struts intersected by, and effectively connected to another angle in cross bracing, the effective length in the plane of bracings shall be taken as in **Table G.13** and normal to the plane of bracing the effective lengths shall be taken as the distance along the bracing members between the points of intersection and the centroids of the main member. In calculating the ratio of slenderness, the radius of gyration about the appropriate rectangular axis shall be taken for buckling normal to the plane of the bracing and the least radius of gyration for buckling in the plane of the bracing.

G7.2.4.4 Effective length of unbraced compression chords

For simply supported trusses with ends restrained at the bearings against torsion, the effective length l of the compression chord for buckling normal to the plane of the truss shall be taken as follows :

a) With no lateral support to compression chord; where there is no lateral bracing between compression chords and no cross frames:

$$l = \text{span}$$

b) With compression chords supported by U frames, where there is no lateral bracing of the compression chord but where cross members and verticals forming U frames provide lateral restraints:

$$l = 2.5 \times \sqrt[4]{EIa\delta} \qquad \text{but not less than "}a\text{"}$$

where

E = Young's modulus

I = maximum moment of inertia of compression chord about the axis lying in the plane of the truss.

a = distance between frames, and

δ = the virtual lateral displacement of the compression chord at the frame nearest mid span of the truss, taken as the horizontal deflection. This deflection shall be computed assuming that the cross member is free to deflect vertically and that the tangent to the deflection curve at the centre of its span remains parallel to the neutral axis of the unrestrained cross member.

When δ is not greater than $\dfrac{a^3}{40EI}$

$$I = a$$

In case of symmetrical U frames, where cross member and verticals are each of constant moment of inertia throughout their own length; it may be assumed that :

$$\delta = \frac{(d_1)^3}{3EI_1} + \frac{(d_2)^2 C}{EI_2}$$

where

d_1 = distance of the centroid of the compression chord from the top of the cross member,

d_2 = distance of the centroid of the compression chord from the neutral axis of the cross member,

C = half the distance between centres of the main trusses;

E = Young's Modulus;

I_1 = moment of inertia of the vertical in its plane of bending and

I_2 = moment of inertia of the cross member in its plane of bending

f = flexibility of the joint between the cross member and the stiffeners of U frame expressed in radian per unit moment

U frames shall have rigid connections and shall be designed to resist, in addition to the effects of wind and other applied forces, the effects of a horizontal force F acting normal to the compression chord of the truss at the level of the centroid of this chord where:

$$F = \frac{1.4 \times 10^{-3} l}{\delta \left\{ \frac{C_0}{f_a} - 1.7 \right\}}$$

In the above formula :

$$l = 2.5 \times \sqrt[4]{EIa\delta}$$

δ = the deflection of the chord under the action of unit horizontal force F

C_0 = Euler critical stress in chord = $\dfrac{\pi^2 E}{\left(\frac{l}{r}\right)^2}$

where

E = Young's Modulus

r = radius of gyration

f_a = calculated working stress in the chord.

c) With compression chord supporting continuous deck, a compression chord continuously supporting a reinforced concrete or steel deck shall be deemed to be effectively restrained laterally throughout its length (e.g., l = 0) if the friction or positive connection of the deck to the chord is capable of resisting a lateral force, distributed uniformly along its length of 2.5 percent of the maximum force in the chord, in addition to other lateral forces.

G7.2.5 Lacings and battening

For general design and detailing aspect refer to Clauses **507.6, 507.7, 507.8** and **507.9.**

G7.3 Tension members

G7.3.1 General requirements

Tension members should preferably be of solid cross section. However, when composed of two or more components these shall be connected as described in Clauses **506.2.6, 506.2.7** and **506.2.8**.

G7.3.2 Effective sectional area

The properties of the cross-section shall be computed from the effective sectional areas as given in Clause **G3.8.2**. When plates are provided solely for the purposes of lacing or battening, they shall be ignored in computing the radius of gyration of the section.

G7.3.3 Slenderness ratio

Refer to Clause **506.2.1**

G7.3.4 Lacing and battening

G.7.3.4.1 For design and detailing refer to Clauses **506.2.4, 506.2.5, 506.2.7** and **506.2.8.**

G7.3.4.2 Where battens are attached by welds, the length of welds connecting each longitudinal edge of the batten plate to the component shall, in the aggregate, be not less than half the length of the batten plate, and at least 1/3 of the weld shall be placed at each end of the longitudinal edge. In addition, welding shall be returned along the ends of the plate for a length at least four times the thickness of the plate.

G7.3.4.3 Where the tie or batten plates are fitted between main components they shall be connected to each member either by fillet welds on each side of the plate, at least equal in length to that specified in the preceding paragraph or by full penetration butt weld.

G7.4 Splices

G7.4.1 For compression member

G7.4.1.1 Splices in compression members located at or near effectively braced panel points shall be designed to transmit the full design load in the member. All other splices in compression members shall have a sectional area 5 percent more than that required to develop the load in the member at the average working stress of the member. All cover material shall, as far as practical, be so disposed with respect to the cross-section of the member so as to transmit the proportional load of the respective parts of the section.

G7.4.1.2 Wherever possible both surfaces of the parts spliced shall be covered or other means taken to maintain the alignment of the abutting ends.

G7.4.1.3 Where flexural tension may occur in the member, the cover material shall be designed to resist such tension.

G7.4.1.4 Rivets, bolts or welds shall develop the full load in the cover material as defined above calculated on the cover area.

G7.4.2 For tension members

G7.4.2.1 Splices in tension members shall have a sectional area 5 percent more than that required to develop the load in the member and, whenever practicable, the cover material shall be disposed to suit the distribution of stress in the various parts of the cross section of the member. Both surfaces of the parts splices shall be covered wherever possible.

G7.4.2.2 Rivets, bolts or welds shall develop the full load in the cover material as defined above, calculated on the cover area.

G7.5 Connections at intersection

Refer to Clause **508.6**

G7.6 Lug angles

Refer to Clause **508.7**

G7.7 Section at pin holes in tension members

Refer to Clause **508.8**

G7.8 Pin plates

Refer to Clause **508.9**

G7.9 Diaphragms in members

Refer to Clause **508.10**

G7.10 Lateral bracings

Refer to Clause **508.11**

G8. CONNECTIONS

G8.1 Composite connections

In any connection which takes a force directly communicated to it and which is made with more than one type of fastening, only rivets and turned and fitted bolts may be considered as acting together to share the load. In all other connections sufficient number of one type of fastening shall be provided to communicate the entire load for which the connection is designed.

G8.2 Welded connections

G8.2.1. Welds shall conform to IS 816 and IS 9595 as appropriate.

G8.2.2. Types of welds

Refer to Clause **512.4.2**

G8.2.3 Strength of weld

Refer to Clause **512.4.3**

G8.2.4. General requirements of welds

Refer to Clause **512.4.4**

G8.3 Connections made with bearing type bolts, rivets or pins

G8.3.1 General

Refer to Clause **512.5.1**

G8.3.2 Connections and splices in flexural members

Refer to Clause **512.5.2**

G8.3.3 Connections in triangulated structures

Refer to Clause **512.5.3**

G8.3.4 Rivets and bolts

a) Gross and Net Areas of Rivets and Bolts -

The gross area of a rivet shall be taken as the cross sections area of the rivet hole.

The net sectional area of a bolt or screwed tension rod shall be taken as the tension area for the particular diameter of bolt as given in the table below :

Nominal Thread Dia (mm)	12	14	16	18	20	22	24	27	30	33
Nominal Stress Area (mm^2)	84	115	157	192	245	303	353	459	561	694

b) Calculation of stresses

Calculation of stresses in rivets and bolts shall be as per Clause **G2.5.2**

c) Diameter of Rivet and Bolt Holes

Refer to Clause **512.5.4(a)**

d) Edge Distances

Refer to Clause **512.5.4(b)**

e) Pitch of rivets or bolts

Refer to Clause **512.5.4(c)**

f) Long Rivets

The grip of rivets carrying calculated loads shall not exceed 8 times the diameter of the holes. Where the grip exceeds 6 times the diameter of the hole, the number of rivets required by normal calculation shall be increased by not less than half a per cent for each additional millimeter of length of grip above 6 times the hole diameter.

g) Rivets with Counter Sunk Head

Refer to Clause **512.5.4(d)**

h) Rivets or bolts Through packing

Number of rivets or bolts carrying calculated shear through a packing shall be increased above the number required by normal calculations by 2.5 percent for each 2.0 mm thickness of packing except that, for packing having a thickness of upto 6 mm, no increase need be made. For double shear connections packed on both sides, the number of additional rivets or bolts required shall be determined from the thickness of the thicker packing. The additional rivets or bolts shall be placed in an extension of the packing.

i) Staggered Pitch -

When rivets and bolts are staggered at equal intervals and the gauge does not exceed 75 mm, the distances between centres of rivets and bolts as specified earlier may be increased by 50 percent.