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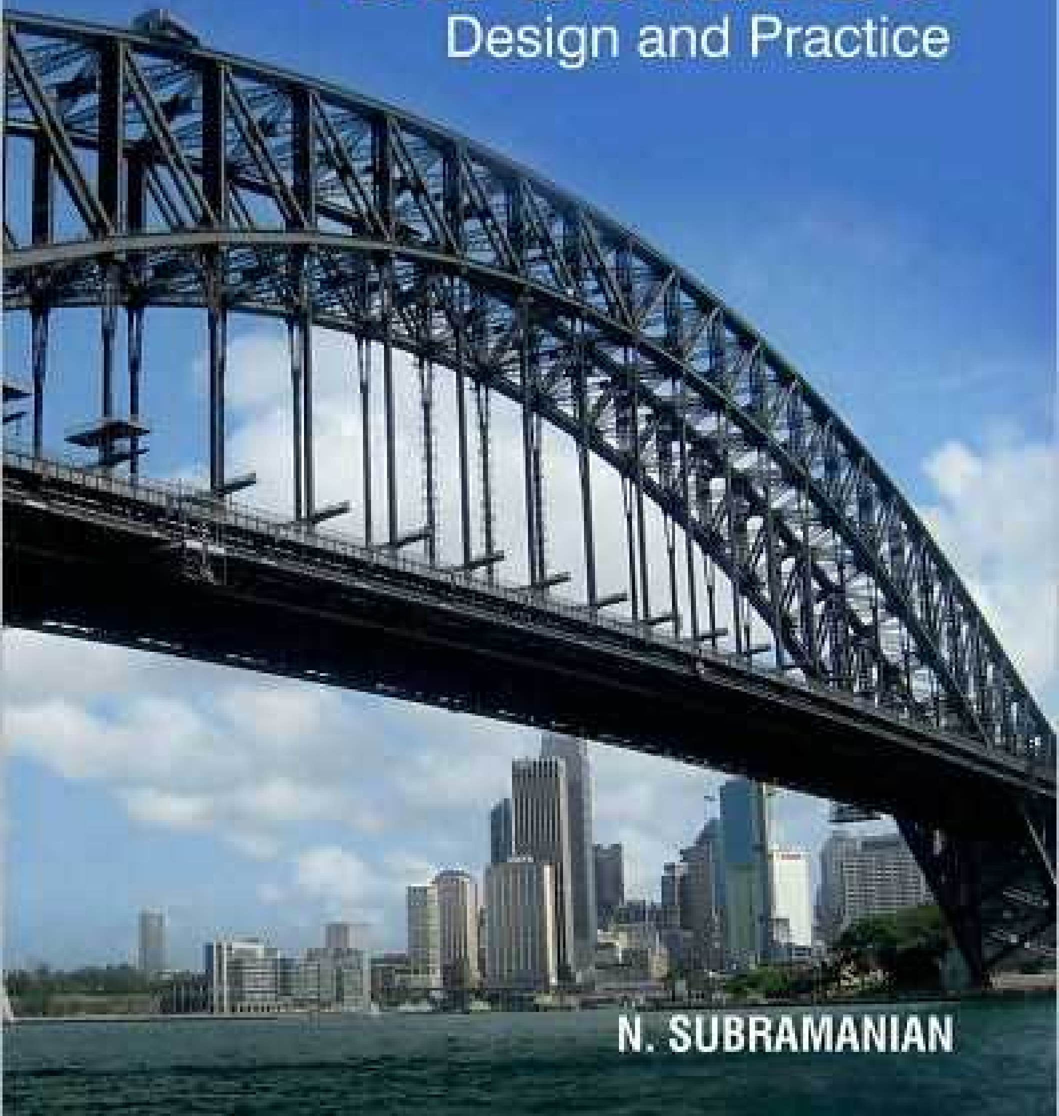
Steel Structures
Design and Practice



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Steel Structures

Design and Practice

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List of Symbols

A area of cross section; surface area of cladding

A_b area of bolt; gross area of horizontal boundary elements in SPSW

A_{br} required bearing area

A_c minor diameter area of the bolt; gross area of vertical boundary elements in SPSW

A_d area of diagonal member

A_e effective cross-sectional area; effective frontal area in wind

A_F moment amplification factor

A_f total flange area of the smaller connected column; floor area; required area of flange plates

A_g gross cross-sectional area

A_{gf} gross cross-sectional area of flange

A_{go} gross cross sectional area of outstanding leg

A_h design horizontal seismic coefficient

A_k design horizontal acceleration spectrum value of mode k

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

A_{ne} effective net area

A_{nf} net cross sectional area of each flange

A_{no} net sectional area of outstanding leg

A_o initial cross-sectional area of tensile test coupon

A_q cross-sectional area of the stiffener in contact with the flange

A_s tensile stress area

A_{sb} shank gross cross sectional area (nominal area) of a bolt

A_{st} area of stiffener

A_t total area of the compartment in fire

A_{tg} gross 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)

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)

A_v shear area

A_{vg} gross cross sectional area in shear along the line of transmitted force (block shear failure)

A_{vn} net cross sectional area in shear along the line of transmitted force (block shear failure)

A_w effective cross-sectional area of weld; window area; effective area of walls; area of web

a, b larger and smaller projection of the slab base beyond the rectangle considering the column, respectively

a_o peak acceleration
a_1 unsupported length of individual elements being laced between lacing points
B breadth of flange of I-section; length of side of cap or base plate of a column
B_s background factor
b_o outstand/width of the element
b_1 stiff bearing length; stiffener bearing length
b_e effective width of flange between pair of bolts
b_{fc} width of column flange
b_f breadth or width of the flange
b_p panel zone width between column flanges at beam column junction
b_s shear lag distance; stiffener width
b_{sh} average breadth of the structure between heights s and h
b_{0h} average breadth of the structure between heights 0 and h
b_t width of tension field
b_w width of outstanding leg
C centre-to-centre longitudinal distance of battens; coefficient related to thermal properties of wall, floor, etc.; spacing of transverse stiffener; moisture content of insulation
C_1 equivalent uniform moment factor
C_{dyn} dynamic response factor
C_e effective width of interior patch load
C'_f frictional drag coefficient
C_f force coefficient of the structure
C_{fs} cross-wind force spectrum coefficient
C_i specific heat of insulation material
C_{Lh} lateral horizontal load for cranes
C_m coefficient of thermal expansion; equivalent moment factor
C_p cost of purlin
C_{pe} external pressure coefficient
C_{pi} internal pressure coefficient
C_r cost of roof covering
C_s specific heat of steel
C_t cost of truss
c_m moment reduction factor for lateral torsional buckling strength calculation
D overall depth/diameter of the cross section
D_o outer diameter
d depth of web; nominal diameter; grain size of crystals; diagonal length; depth of snow; base dimension of the building
d_2 twice the clear distance from the compression flange angles, plates, or tongue plates to the neutral axis
d_a depth of angle
d_b beam depth; diameter of bolt
d_c column depth
d_g centre-to-centre of the outermost bolt of the end plate connection
d_h diameter of the hole
d_i thickness of insulation
d_o nominal diameter of the pipe column or the dimensions of the column in the depth direction of the base plate
d_p panel zone depth in the beam-column junction
E modulus of elasticity for steel; energy released by earthquake
$E(T)$ modulus of elasticity of steel at T°C
$E(20)$ modulus of elasticity of steel at 20°C
E_e equivalent elastic modulus of rope
EL_x earthquake load in x direction
EL_y earthquake load in y direction
E_p modulus of elasticity of the panel material
E_{sh} strain-hardening modulus
E_t tangent modulus of elasticity
E'_t reduced tangent modulus
e eccentricity, head diagonal of bolt
e_b edge distance of bolt
F net wind force on cladding
F' frictional drag force

F_{br} strength of lateral bracing
F_{cf} flange contribution factor
F_o minimum bolt pretension
F_{bsd} bearing strength of stiffener
F_q stiffener force
F_{qd} design buckling resistance of stiffener
F_w protection material density factor; ultimate web crippling load
F_{xd} design resistance of load carrying web stiffener
F_x external load; force or reaction on stiffener
F_z along-wind equivalent static load at height Z
f actual normal stress range for the detail category, uniaxial stress, frequency of vortex shedding
f_1, f_2, f_3 principal stresses acting in three mutually perpendicular directions
f_a stress amplitude
f_b actual bending stress; bending stress at service load
f_{bc} actual bending stress in compression
f_{bd} design bending compressive stress corresponding to lateral buckling
f_{bt} actual bending stress in tension at service load
f_c average axial compressive stress
f_{cc}, f_{cr} elastic buckling stress of a column or plate; Euler's buckling stress = $\pi^2E/(KL/r)^2$
f_{ck} characteristic compressive strength of concrete
$f_{cr,b}$ extreme fibre compressive elastic lateral buckling stress
f_{cd} design compressive stress
f_d stress range at constant amplitude
f_e equivalent stress
f_f fatigue strength corresponding to N_{sc} cycle of loading
f_{fd} design normal fatigue strength
f_{feq} equivalent constant amplitude stress range
f_{fmax} highest normal stress range
f_{fn} normal fatigue stress range for 5×10^6 cycles
f_k characteristic strength
f_l fatigue limit
f_L stress range at cut-off limit
f_m mean stress
f_{max} maximum stress
f_{min} minimum stress
f_o proof stress
f_0 first mode natural frequency of vibration of a structure in the along-wind direction
$f_{0.2}$ 0.2% proof stress
f_{pl} steel stress at proportional limit
f_q shear stress
f_R characteristic value of fatigue strength at loading cycle N_R
f_{st} shear stress due to torsion
f_{sw} warping shear stress
f_t tension field strength
f_u characteristic ultimate tensile stress
f_{ub} characteristic ultimate tensile stress of the bolt
f_{up} characteristic ultimate tensile stress of the connected plate
f_{uw} ultimate tensile stress of the weld
f_v yield strength in the panel utilizing tension field action
f_{wd} design strength of weld
f_y characteristic yield stress
$f_y(T)$ yield stress of steel at T°C
$f_y(20)$ yield stress of steel at 20°C
f_{yb} characteristic yield stress of bolt
f_{yf} characteristic yield stress of flange
f_{yn} nominal yield strength
f_{yst} design yield stress of stiffener
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
f_0 yield strength of very large isolated crystals

G shear modulus of rigidity for steel; thickness of grout
G_v gust factor
G^* design dead load
g gauge length between the centre of the holes perpendicular to the load direction; acceleration due to gravity; gap for clearance and tolerance
g_R peak factor for resonant response
H height of section; transverse load
H_p heated perimeter
H_v calorific value of the vth combustible material
H_w window height
h depth of the section; storey height
h_b total height from the base to the floor level concerned
h_c height of the column
h_e effective thickness
h_{ef} embedment length of anchor bolt
h_o distance between flange centroids of I-section
h_i thickness of protection material; height of floor i
h_L height of the lip
h_s storey height
h_y distance between shear centre of the two flanges of the cross section
I moment of the inertia of the member about an axis perpendicular to the plane of the frame; impact factor fraction; importance factor
I_b moment of inertia of brace
I_f second moment of area of the foundation pad; insulation factor
I_{fc} moment of inertia of the compression flange
I_{ft} moment of inertia of the tension flange
I_h turbulence intensity at height h
I_p polar moment of inertia
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 of stiffener
I_t St. Venant's torsional constant
I_w warping constant
I_y moment of inertia about the minor axis
I_z moment of inertia about the major axis
IF interference factor
K effective length factor
K_a area averaging factor
K_b effective stiffness of the beam and column; effective length factor for beams against lateral bending
K_c combination factor; stiffness of column
K_d wind directionality factor
K_h reduction factor to account for the bolt holes in HSFG connection
KL effective length of the member
KL/r appropriate effective slenderness ratio of the section
KL/r_y effective slenderness ratio of the section about the minor axis
KL/r_z effective slenderness ratio of the section about the major axis
$(\mathrm{KL}/r)_o$ actual maximum effective slenderness ratio of the laced or battened column
$(\mathrm{KL}/r)_e$ effective slenderness ratio of the laced column accounting for shear deformation
K_m mode share correction factor for cross-wind acceleration
K_y, K_z moment amplification factor about respective axes
K_w warping restraint factor
k regression coefficient; constant; mode shape power exponent for the fundamental mode of vibration
k_1 probability factor or risk coefficient
k_2 terrain, height and structure size factor
k_3 topography factor

k_4 importance factor for cyclonic region
k_b, k_{br} stiffness of bracing
k_s modulus of sub-grade reaction
k_{sec} web distortional stiffness
k_{sm} exposed surface area to mass ratio
k_t brace stiffness excluding web distortion, torsion parameter
k_{tb} stiffness of torsional bracing
k_v shear buckling coefficient
L actual length; unsupported length; centre-to-centre distance of the intersecting members; length of the end connection; cantilever length; land in weld
L_b laterally unbraced length or distance between braces
L_c length of end connection measured from the centre of the first bolt hole to the centre of the last bolt hole in the connection; distance between gantry girders
L_{cf} clear distance between flanges of vertical boundary elements
L_e effective horizontal crest length
L_g gauge length of tensile test coupon
L_h measure of the integral turbulence length scale at height h
L_m maximum distance from the restraint at plastic hinge to an adjacent restraint (limiting distance)
L_o length between points of zero moment (inflection) in the span
L_w effective length of weld; length of wall
l_a length of the angle
l_e distance between prying force and bolt centre line
l_g grip length of connection
l_j length of the joint
l_s length between points of lateral support
l_t elongation due to temperature; length of top angle
l_v distance from bolt centre line to the toe of the fillet weld or to half the root radius for a rolled section
M bending moment; magnitude of earthquake
M_{1sway} maximum first order end moment as a result of sway
M_{br} required flexural strength of torsional bracing
M_{cr} elastic critical moment corresponding to lateral torsional buckling
M_d design flexural or bending strength
M_{dv} design bending strength of the section under high shear
M_{dy} design bending strength as governed by overall buckling about minor axis
M_{dz} design bending strength as governed by overall buckling about major axis
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 strength under combined axial force (uni-axial moment acting alone)
$\mathrm{M_{ndy}}$, $\mathrm{M_{ndz}}$ design strength under combined axial force and the respective uniaxial moment acting alone
M_o cross-wind base overturning moment; first order elastic moment
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_{pf} plastic design strength of flanges only

M_{pr} reduced plastic moment capacity of the section due to axial force or shear
M_q applied moment on the stiffener due to eccentric load
M_{tf} moment resistance of tension flange
M_u second order elastic moment; factored moment; required ultimate flexural strength of a section
M_y factored applied moment about the minor axis of the cross section; yield moment capacity about minor axis
M_{yq} yield moment capacity of the stiffener about an axis parallel to web
M_z factored applied moment about the major axis of the cross section
m mass; slope of the fatigue strength curve
m_v mass of vth combustible material
m^1 non-dimensional moment parameter = M_u/M_{bp}
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 parallel planes of battens; mean probable design life of structure in years; reduced frequency; number of cycles to failure; factored applied axial force; number of bolts in the bolt group/ critical section; number of stress cycles; number of storeys
n_1, n_2 dispersion length
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 a bolted connection
n_s number of shear planes without threads intercepting the shear plane in a bolted connection
n' number of rows of bolts
P factored applied axial force; point load
P_{bf} design strength of column web to resist the force transmitted by beam flange
P_{cc} elastic buckling strength under axial compression
P_{crip} crippling strength of web of I-section
P_d design axial compressive strength
P_{dy}, P_{dz} design compression strength as governed by flexural buckling about the respective axis
P_{dw} design strength of fillet weld
P_e, P_{cr} elastic Euler buckling load; ^{2}EI/L^2
P_f probability of failure
P_k modal participation factor for mode k
P_{min} minimum required strength for each flange splice
P_N probability that an event will be exceeded at least once in N years
P_n nominal axial strength
P_{ny} axial strength of the member bent about its weak axis
P_u maximum load in the column
p pitch length between centres of holes parallel to the direction of the load; pitch of thread in bolt, pressure
p_d design wind pressure
p_s, p_1, p_2 staggered pitch length along the direction of the load between lines of the bolt holes (Fig. 5.21)
p_z wind pressure at height Z
Q prying force; nominal imposed load; static moment of the cross section = $A\bar{y}$
Q^* design imposed load
Q_a accidental load (action)
Q_c characteristic load (action)
Q_d design load (action)
Q_f fire load
Q_i load effect i; design lateral force at floor i

Q_m mean value of load
Q_p permanent loads (action)
Q_v variable loads (action)
q_f fire load/unit floor area
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; stress ratio; response reduction factor; resultant force; root opening of weld; local radius of curvature of beam; return period
R_d design strength of the member at room temperature
R_i net shear in bolt group at bolt i
R_k connection stiffness
R_m mean value of resistance
R_n nominal strength of resistance
R_{nw} design strength of fillet weld per unit length
R_{res} resultant force in the weld
R_r response reduction factor
R_{tf} resultant longitudinal shear in flange
R_u ultimate strength of the member at room temperature; ultimate strength of joint panel
r appropriate radius of gyration
r_a root radius of angle
r_b root radius of beam flange
r_1 minimum radius of gyration of the individual element being laced together
r_f ratio of the design action on the member under fire to the design capacity
r_{vv} radius of gyration about the minor axis (v-v)
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; strouhal number; size reduction factor; spacing of truss
S_a spectral acceleration
S_d spectral displacement
S_p spring stiffness
S_v spectral velocity
S_1, S_2 stability functions
s design snow load; size of weld
s_a actual stiffener spacing
s_c anchorage length of tension field along the compression flange
$s_{\text{ii}}, s_{\text{jj}}$ stability functions
$s_{\text{ii}}^*, s_{\text{jj}}^*$ stability function for semi-rigid frames
s_o ground snow load
s_t anchorage length of tension field along the tension flange (distance between adjacent plastic hinges)
T Temperature in °C; factored tension in bolt; natural period of vibration; applied torque
T_a approximate fundamental natural period of vibration
T_b applied tension in bolt
T_d design strength under axial tension
T_{db} design strength of bolt under axial tension; block shear strength of plate/angle
T_{dg} yielding strength of gross section under axial tension
T_{dn} rupture strength of net section under axial tension; design tension capacity
T_{dw} design strength of weld in tension
T_e externally applied tension
T_{eq} equivalent fire rating time
T_f factored tension force of friction type bolt; furnace temperature
$T_{\text{f,max}}$ maximum temperature reached in natural fire
T_l limiting temperature of the steel
T_{nb} nominal strength of bolt under axial tension
T_{nd} design tension capacity
T_{ndf} design tensile strength of friction type bolt
T_{nf} nominal tensile strength of friction type bolt

T_o ambient (room) temperature
T_s steel temperature at time t
T_t ambient gas temperature at time t
T_u ultimate net section strength
t thickness of element/angle, time in minutes
t_a thickness of top angle
t_b thickness of base plate
t_{cw} thickness of column web
t_e effective throat thickness of weld
t_f thickness of flange; required fire rating time
t_{fc} thickness of compression flange
t_{fail} time to failure of the element in case of fire
t_{fb} thickness of beam flange
t_p thickness of plate/end plate
t_{pkg} thickness of packing
t_q thickness of stiffeners
t_s thickness of web stiffener; duration of fire
t_v time delay in minutes
t_w thickness of web
U shear lag factor
V factored applied shear force; mean wind speed
V_B total design seismic base shear; basic wind speed
V_b shear in batten plate
V_{bf} factored frictional shear force in HSFG connection
V_{cr} critical shear strength corresponding to web buckling (without tension field action)
V_d design shear strength; design mean wind velocity
V_{db} shear capacity of outstanding leg of cleat
V_{dw} design strength of weld in shear
V_g gradient wind speed; gust speed
V_h design wind speed at height h
V_L longitudinal shear force
V_R vector resultant shear in weld
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 or shear strength of web
V_n nominal shear strength or resistance
V_{npb} nominal bearing strength of bolt
V_{nsb} nominal shear capacity of a bolt
V_{nsf} nominal shear capacity of a bolt as governed by slip or friction type connection
V_{sb} factored shear force in the bolt
V_{sd} design shear capacity
V_{sdf} design shear strength of friction type bolt
V_{sf} factored design shear force of friction bolts
V_T applied transverse shear
V_{tf} shear resistance in tension field
$\bar{V}$ average or mean velocity
V_{yw} yield strength of web plate of I-section
V_z mean or design wind speed at height z above the ground
V' Instantaneous velocity fluctuation above the mean velocity
W appropriate load; width; seismic weight; ventilation factor
W_e equivalent cross-wind static force per unit-height
w uniform pressure from below on the slab base due to axial compression under factored load; intensity of uniformly distributed load
w_{tf} width of tension field
X distance from a point to any other point
x_t torsional index
$\bar{x}$ distance from centre of gravity in x direction
Y_s yield stress
$\bar{y}$ distance from centre of gravity in y direction
y_g distance between point of application of the load and shear centre of the cross section

y_s coordinate of the shear centre with respect to centroid

Z section modulus; height above ground; zone factor

Z_e elastic section modulus

Z_g depth of boundary layer

Z_p plastic section modulus

Z_{pr} plastic modulus of the shear area about the major axis; reduced plastic modulus

α coefficient of linear expansion; imperfection factor; power law coefficient; included angle in groove weld

$_{LT}$ imperfection factor

α_t coefficient of thermal expansion

β reliability index; the ratio of structural damping to critical damping of a structure

β_{lj} reduction factor for overloading of end bolt

β_{lg} reduction factor for the effect of large grip length

β_{pkg} reduction factor for the effect of packing plates

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 stress reduction factor due to buckling under compression

χ_m stress reduction factor χ at f_{ym}

χ_{LT} strength reduction factor for lateral torsional buckling of a beam

δ, Δ storey deflection or drift; deflection

δ_b moment amplification factor for braced member

δ_L horizontal deflection of the bottom of storey due to combined gravity and notional load

$_m$ moment amplification factor

δ_p load amplification factor

δ_s moment amplification factor for sway frame

δ_U horizontal deflection of the top of storey to combined gravity and notional load

ε yield stress ratio; $(250/f_y)^{1/2}$; strain corresponding to stress f, resultant emissivity of surface

ε_p plastic strain

$_{sh}$ strain hardening strain

ε_u, ε_{br} ultimate strain

ε_y yield strain

. shape factor

ϕ strength or resistance reduction factor; cumulative distribution function; solidity ratio; inclination of the tension field stress in web; configuration factor; angle of twist

$\phi_{i,\,k}$ mode shape coefficient at floor i in mode k

ϕ_s sway index

γ unit weight of steel

γ_f, γ_{fk}, γ_{if} partial safety factor for load

γ_{fft} partial safety factor for fatigue load

γ_m partial safety factor for material

γ_{m0} 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 HSFG bolts

γ_{mft} partial safety factor for fatigue strength

γ_{mi} partial safety factor depending upon the type of failure as prescribed in IS: 800

γ_{mw} partial safety factor for strength of weld

$\bar{\lambda}$ non-dimensional slenderness ratio $= \left(KL/r/\sqrt{\pi^2 E/f_y} \right.$ $\left. = \sqrt{f_y/f_{cc}} = \sqrt{P_y/P_{cc}} \right)$

λ_{cr} elastic buckling load factor

λ_e equivalent slenderness ratio

λ_i effective thermal conductivity of insulation

λ_{LT} non-dimensional slenderness ratio

λ_y, λ_z non-dimensional slenderness ratio about respective axis

λ_w non-dimensional web slenderness ratio for shear buckling

μ Poisson's ratio; shape coefficient or factor for snow load

μ_c correction factor; capacity reduction factor for fatigue

μ_f coefficient of friction (slip factor)

μ_r capacity reduction factor for non-redundant load path

θ^1 non-dimensional rotation parameter $= \theta_r/\theta_p$

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, upwind slope of ground

θ_p plastic rotation

θ_r rotation of semi-rigid joint

ρ_s density of steel

ρ_i dry density of insulation

ρ_i' effective density of insulation

s Stefon–Boltzmann constant

τ actual shear stress for the detail category

τ_b shear stress corresponding to buckling

τ_{cr} elastic critical shear buckling stress

τ_e equivalent shear stress

τ_f fatigue shear stress range for N_{sc} cycle

τ_{fd} design fatigue shear strength

τ_{fmax} highest shear stress range

τ_{fn} fatigue shear stress range at 5×10^6 cycles for the detail category

τ_L shear stress range at cut-off limit

τ_o grout-concrete bond strength

τ_{vf} shear stress in the weld due to vertical force

τ_{vf1} shear stress in the weld due to bending moment

τ_w shear stress in weld throat; shear stress due to shear force

τ_y shear yield stress

ξ_i reduction factor for geometric imperfection

ψ ratio of the moment at the ends of the laterally unsupported length of a beam

ω circular natural frequency

Note: The subscripts y and z denotes the y-y and z-z axes of the section, respectively. For symmetrical sections, y-y denotes the minor principle axis whilst z-z denotes the major principal axis.

Preface

Structural design emphasizes that the elements of a structure are to be proportioned and joined together in such a way that they will be able to withstand all the loads (load effects) that are likely to act on it during its service life, without excessive deformation or collapse. Structural design is often considered as an art as well as a science. It must balance theoretical analysis with practical considerations, such as the degree of certainty of loads and forces, the actual behaviour of the structure as distinguished from the idealized analytical and design model, the actual behaviour of the material compared to the assumed elastic behaviour, and the actual properties of materials used compared to the assumed ones.

Steel is one of the major construction materials used all over the world. It has many advantages over other competing materials, such as high strength to weight ratio, high ductility (hence its suitability for earthquake-resistant structures), and uniformity. It is also a *green material* in the sense that it is fully recyclable. Presently, several grades and shapes of steel products exist.

Structural designers need to have a sound knowledge of structural steel behaviour, including the material behaviour of steel, and the structural behaviour of individual elements and of the complete structure. Unless structural engineers are abreast of the recent developments and understand the relationships between the structural behaviour and the design criteria implied by the rules of the design codes, they will be following the codal rules rigidly and blindly and may even apply them incorrectly in situations beyond their scope.

This text is based on the latest Indian Standard code of practice for general construction using hot-rolled steel sections (IS 800 : 2007) released in February 2008. This third revision of the code is based on the limit state method of design (the earlier versions of the code were based on the working or allowable stress method). The convention for member axis suggested in the code is adopted and SI units have been used throughout the book.

Readers are advised to refer to the latest code (IS 800 : 2007) published by the Bureau of Indian Standards, New Delhi. It is recommended that readers also refer to the latest version of the codes on design loads (IS 875 and IS 1893), dimension of sections (IS 808 or IS Handbook No. 1, IS 1161, IS 12778, IS 4923, and IS 811), specification of steel (IS 2062, IS 8500, IS 6639, and IS 3757), bolts (IS 1364 and IS 4000), and welding (IS 816).

About the Book

The objectives of writing this book are: (a) to explain the provisions of the latest version of IS 800:2007, which has been revised recently based on limit states design, (b) to provide ample examples so that the students understand the concepts clearly, (c) to give information on structural design failures and latest developments in structural steel design, and (d) to provide interested readers with the sources of further reading.

The book completely covers the requirements of undergraduate students of civil and structural engineering for a course on design of steel structures. Each chapter comprises numerous tables, figures, and solved examples to help students understand the concepts clearly. Review questions and exercises given at the end of each chapter will help students assimilate the ideas presented in the chapters and also to apply them to get a feel of the results obtained. Case studies of failures and some important aspects of structural design are sprinkled throughout the text, to enhance the usefulness of the book.

Contents and Coverage

Chapter 1 provides a brief discussion on the historical developments, steel making processes, and the metallurgy of steel.

Chapter 2 introduces the design considerations and the role of structural design in the complete design process as well the loads acting on structures. Many failures are attributed to the lack of determination of the loads acting on different structures. Hence, the various loads that can act on a structure are also briefly discussed, as per the latest Indian codes.

Chapter 3 deals with the design of tension members. Plastic and local buckling behaviour of steel sections are covered in Chapter 4, as they will be useful in understanding the design of axially loaded compression members and flexural members which are covered in Chapters 5 to 8.

The design of beam–columns, which are subjected to both axial loads and bending moments, is discussed briefly in Chapter 9. The two methods used to connect the elements of steel structures, namely, bolted and welded connections are discussed in Chapters 10 and 11.

With the information provided in Chapters 1 to 11, it is possible to design any type of structure consisting of tension members, compression members, flexural members, or beam–columns. To demonstrate this, the design of industrial buildings is dealt with in Chapter 12.

The design aids presented in the appendix (Appendix D) will be quite useful to designers and also to students to check the results.

Though care has been taken to present error-free material, some errors might have crept in inadvertently. I would highly appreciate if these errors and suggestions for improvement are brought to my notice.

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Dr N. Subramanian

Contents

CHAPTER 2

The Basis of Structural Design

2.1 Design Considerations

Structural design, though reasonably scientific, is also a creative process. The aim of a structural designer is to design a structure in such a way that it fulfils its intended purpose during its intended lifetime and be adequately safe (in terms of strength, stability, and structural integrity), and have adequate serviceability (in terms of stiffness, durability, etc.). In addition, the structure should be economically viable (in terms of cost of construction and maintenance), aesthetically pleasing, and environment friendly.

Safety is of paramount importance in any structure, and requires that the possibility of collapse of the structure (partial or total) is acceptably low not only under normal expected loads (service loads), but also under less frequent loads (such as due to earthquakes or extreme winds) and accidental loads (blasts, impacts, etc.). Collapse due to various possibilities such as exposure to a load exceeding the load-bearing capacity, overturning, sliding, buckling, fatigue fracture, etc. should be prevented.

Another aspect related to safety is structural integrity and stability—the structure as a whole should be stable under all conditions. (Even if a portion of it is affected or collapses, the remaining parts should be able to redistribute the loads.) In other words, *progressive failure* should be minimized.

Serviceability is related to the utility of the structure—the structure should perform satisfactorily under service loads, without discomfort to the user due to excessive deflection, cracking, vibration, etc. Other considerations of serviceability are durability, impermeability, acoustic and thermal insulation, etc. It may be noted that a design that adequately satisfies the safety requirement need not satisfy the serviceability requirement. For example, an I-beam at the roof level may have sufficient stiffness for applied loads but may result in excessive deflections, leading to cracking of the slab it is supporting, which will result in loss of permeability (leaking). Similarly, exposed steel is vulnerable to corrosion (thereby affecting durability).

Increasing the design margins of safety may enhance safety and serviceability, but increase the cost of the structure. For overall economy one should look into not only the initial cost but also the life-cycle cost and the long-term environmental effects. For example, using a very-high-strength steel to reduce weight often will not reduce cost because the increased unit price of high-strength steel will make the lighter design more costly. In bridges and buildings the type of corrosion and fire protection selected by the designer will greatly influence the economy of the structure.

While selecting the material and system for the structure the designer has to consider the long-term environmental effects. Such effects considered include maintenance, repair and retrofit, recycleability, environmental effects of the demolished structure, adoptability of fast track construction, demountability, and dismantling of the structure at a future date.

2.2 Steps Involved in the Design Process

The construction of any structure involves many steps. Although the structural designer is not responsible for each of these steps, he should be involved in most of them so that the resulting structure is safe, stable, serviceable, durable, and is economically viable and aesthetically pleasing, and does not have an adverse impact on the environment. The necessary steps may be listed as follows.

1. After receiving the plan and elevation of the building from the architect and the soil report from the geotechnical engineer, the structural engineer estimates the probable loads (dead, live, wind, snow, earthquake, etc) that are acting on the structure. Normally, the material of construction is chosen by the owner in consultation with the architect.
2. The structural engineer arrives at the structural system after comparing various possible systems. In a building, heating and air-conditioning requirements or other functional requirements may dictate the use of a structural system that is not the most efficient from a purely structural viewpoint, but which is the best bearing the total building in mind.
3. A suitable structural analysis, mostly with the aid of computers, is done to determine the internal forces acting on various elements of the structural system, based on the various loads and their combinations.
4. Considering the critical loading conditions, the sizes of various elements are determined following the codal provision.
5. The detailed structural drawings are then prepared once again following codal provisions and approved by the structural engineer.
6. The estimator arrives at the quantities involved and the initial cost of construction.
7. The contractor, based on the structural drawings, prepares the fabrication and erection drawings and a bill of quantity of materials (BOQ). The structural engineer again approves these drawings.

8. The contractor constructs the building based on the specifications given by the architect/project manager.
9. The structural engineer, with the help of quality control inspectors, inspects the work of the fabricator and erector to ensure that the structure has been fabricated/erected in accordance with his or her designs and specifications.
10. After the structure is constructed and handed over to the owner, the owner, by appointing suitable consultants and contractors, maintains the building till its intended age.

From these steps, it may be clear that accurate calculations alone may not produce safe, serviceable, and durable structures. Suitable materials, quality control, adequate detailing, good supervision, and maintenance are also equally important.

While executing the various steps, the structural engineer has to interact with the architect/project manager and also with others (electrical engineers, mechanical engineers, civil engineers, geotechnical engineers, surveyors, urban planners, estimators, etc.) and incorporate their requirements into the design (e.g., load due to mechanical and electrical systems). It has to be noted that steps 1 to 6, which are followed mainly in the design office, are not straightforward operations but are iterative (see Fig. 2.1). This book mainly covers only step 4—the design of structural elements to safely carry the expected loads and to ensure that the elements and the structure perform satisfactorily.

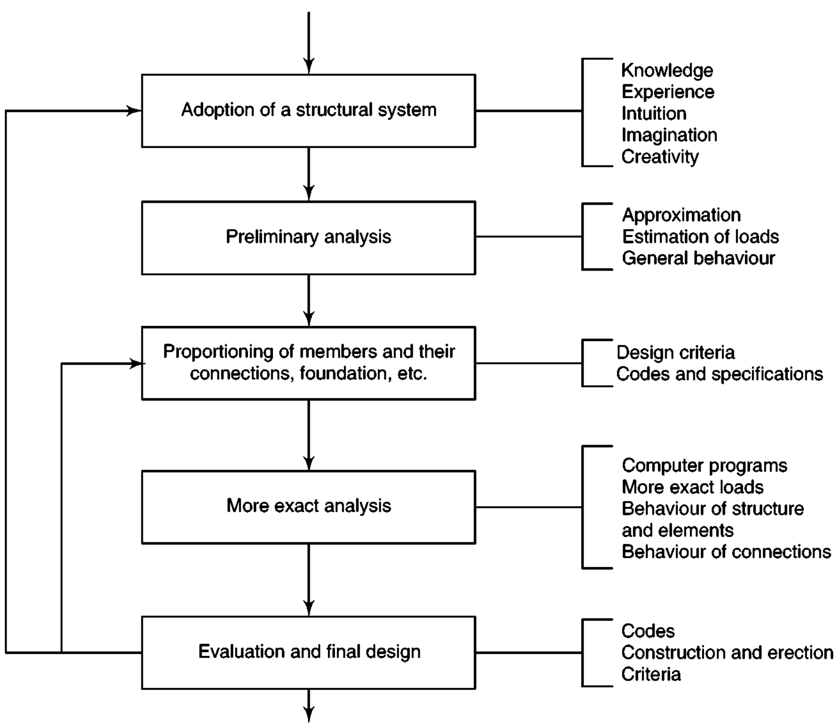


Fig. 2.1 Iterative structural design process

Compared to analysis (where all the parameters are known), design is a creative process. It involves the selection of the span, assessment of loads, and the choice of material, cross section, jointing method and systems, etc. Hence, there is no unique solution to a design problem. The designer has to make several decisions, which will affect the final construction and its cost. Therefore, the designer has to use his engineering judgment in order to reduce the cost and arrive at an efficient solution to the problem.

Today's structural engineer has several aids such as computer programs, handbooks, and charts, and hence should spend more time on thinking about design concepts and select the best structural system for the project at hand.

For most structures, the designer should specify a grade of structural steel that is readily available, keep the structural layout and structural details (e.g., connections) as simple as possible, use sections that are readily available, and use the maximum possible repetition of member sections and connection details. It is preferable for the designer to have a knowledge of fabricating shop capabilities (e.g., size of available zinc baths for galvanization) and erection techniques.

2.3 Structural Systems

The art of structural design is manifested in the selection of the most suitable structural system for a given structure. The arrangement of beams/girders/joints or trusses, and columns to support the vertical (gravity) loads and the selection of a suitable bracing system or a column and beam/truss arrangement to resist the horizontal (lateral) loads poses a great challenge to the structural engineer, since they will determine the economy and functional suitability of the building. The selection of a suitable system is made mainly based on previous data or experience. Steel structures may be classified into the following types (see Fig. 2.2).

(a) Single-storey, single, or multi-bay structures may have truss or stanchion frames, or rigid frames of solid or lattice members. Beams and open-web steel joists (light trusses) may also be supported at the ends by bearing walls of masonry construction. These types of structures are used for industrial buildings, commercial buildings, schools, and some residential buildings. Pitched roof portal frames consisting of rolled-steel sloped beams connected by welding to vertical columns have been used as industrial structures, arenas, auditoriums, and churches.

(b) Multi-storey, single, or multi-bay structures of braced or rigid frame construction (which are discussed in detail in the next section).

(c) Space structures, in the form of single-, double- or multi-layer grids, steel-frame folded plates, braced barrel walls, and domes, are required for very large column-free areas. Towers are also considered as space trusses. Often they require three-dimensional computer analysis. They also require special connectors to connect the various members at different angles (Subramanian 2006). Space frames are used to cover large spans such as those occurring in large arenas, auditoriums, swimming pools, theatres, airport hangars, tennis or baseball grounds, ballrooms, etc.

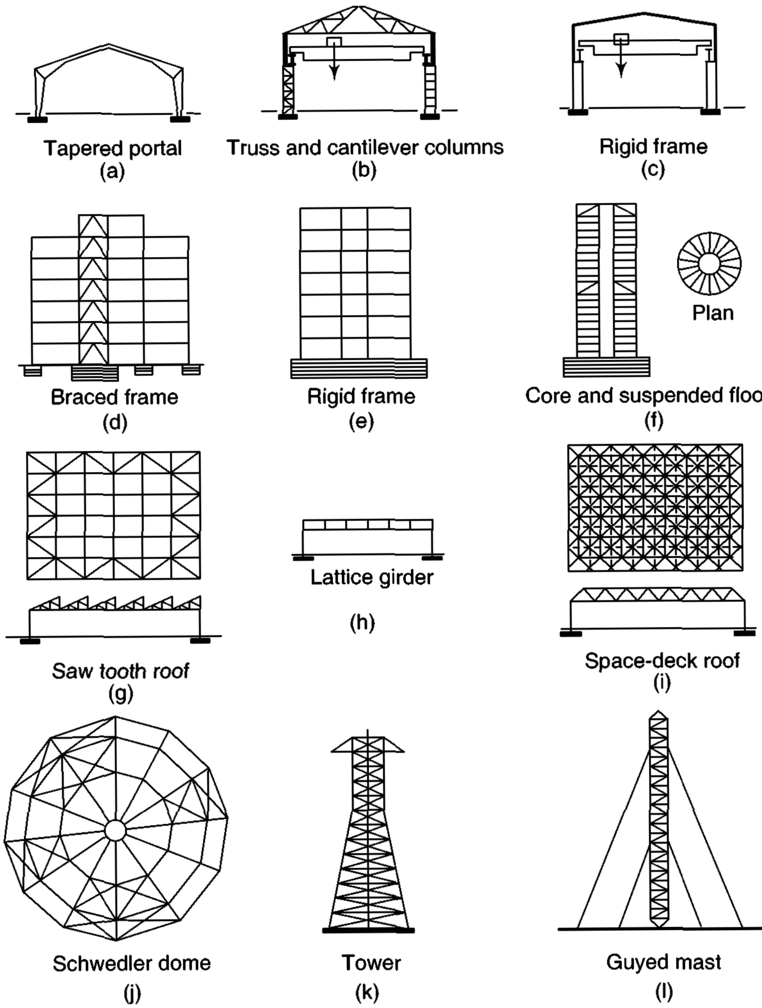


Fig. 2.2 Examples of steel-frame structures (MacGinley 1997)

(d) Tension structures, tensegritic and cable-supported (cable-suspended or cable-stayed) roof structures (Subramanian 2006).

(e) Stressed skin structure, where the cladding is also designed as a load-bearing member, thus stabilizing the structure; in such structures special shear connections are necessary, in order that the sheeting acts integrally with the main frames of the structure (Subramanian 2006).

(f) High-rise constructions: Tall buildings with more than 20 storeys are often considered in large cities where land costs are very high. In the design of such structures, the designer should pay attention to the system resisting the lateral loads (MacGinley 1999). Several interesting systems have been developed, and a few are discussed in the next section.

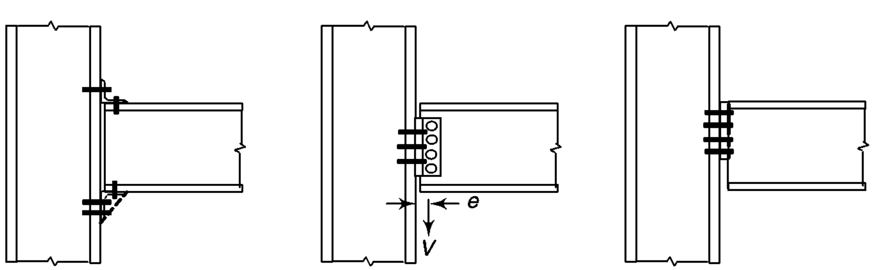
It has to be noted that combinations with concrete (in the form of shear walls or floor slabs) are structurally important in many buildings. If adequate interconnection between the concrete slab and the steel beam is provided in buildings and bridges (in the form of *shear connectors*), the resulting system, called *composite construction*, is both structurally and economically advantageous. Braced, rigid frame, truss roof, and space-deck construction are shown in Fig. 2.2 for comparison. Only framed structures are discussed in detail in the book. Analysis, design, and construction aspects of space frames, tension structures, and stressed skin systems are available in Subramanian (2006). Details of the design of composite constructions are available in Kulak and Grondin (2002), Salmon and Johnson (1996), and Johnson (1994).

For framed structures, the main elements are the beam, column, beam-column, tie, and lattice member. For long-span constructions, normal rolled sections may not have sufficient depth to act as beams. In such cases, deep welded plate girders, box girders, castellated girders, open-web joists or trusses may replace them. For very long spans, deep trusses or arches may be necessary.

2.3.1 Steel-Framed Buildings

Most steel structures belong to the category of braced and rigid frame construction. They consist essentially of regularly spaced columns joined by beams or girders. Secondary beams span between these main beams and provide support to the concrete floor or roof sheeting. Depending on the type of beam-column connections employed, such systems may be classified as *simple construction* or as *continuous construction*.

Simple construction In simple construction (see clause 4.2 and F.4 of the code), the ends of beams and girders are connected to transmit transverse shear only and are free to rotate under load in the plane of bending. Hence hinged ends are assumed for the beams. Connections are usually made by welding plates or angles to a beam or column in the fabricator's shop and bolted at site to the connecting beam or column (see Fig. 2.3).



Clip and seating angle
(a)

Web cleats (single or double)
(b)

Curtailed end plate (header plate)
(c)

Fig. 2.3 Simple beam to column connections

These simple constructions are statically determinate and hence the beams are designed as simply supported and the columns are designed for the axial loads (due to the reaction from the beams) and the moments produced by the eccentricity of the beam reactions as shown in Fig. 2.3(b). (A minimum distance of 100 mm from the face of the column is specified in the code clause 7.3.3.1.) In such frames lateral forces due to the wind or earthquake are generally resisted by bracings (usually made of angles), forming vertical or horizontal trusses as required.

The braced bays can be grouped around a central core, distributed around the perimeter of the building, or staggered through various elevations as shown in Fig. 2.2(d) and Fig. 2.4. The floors act as horizontal diaphragms to transmit load to the braced bays. Bracing must be provided in two directions and all connections are taken as pinned. The bracing should be arranged to be symmetrical with respect to the building plan, to avoid twisting. The unbraced portion of the building frame in effect 'leans' on the braced portion to keep from falling over. In multi-storey buildings, reinforced concrete shear walls may replace the vertical steel bracing trusses. This type of construction is used in frames up to about five storeys in height, where strength rather than stiffness governs the design. Manual analysis can be used for the whole structure.

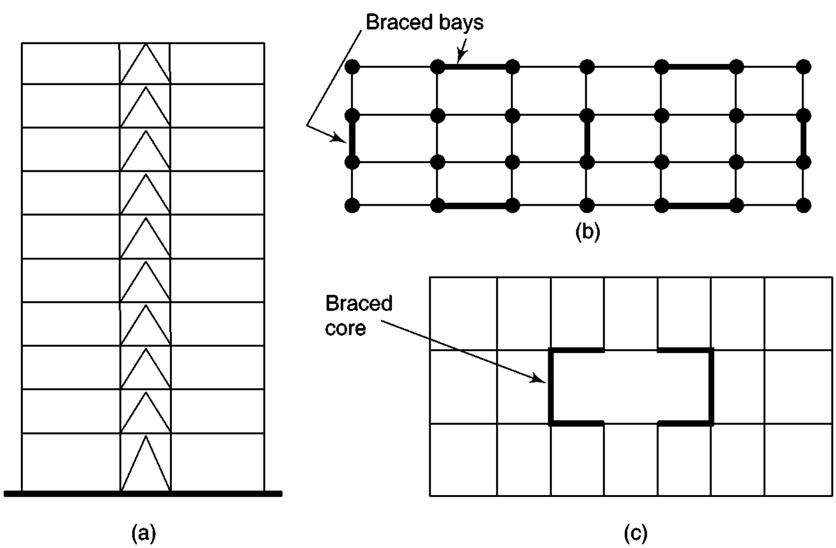


Fig. 2.4 All-steel braced structures: (a) vertical bracing, (b) bracing on perimeter/ interior walls, and (c) bracing around core

Continuous construction or rigid frame structures Continuous construction (also called rigid frame structures) assume sufficient rigidity in the beam-column connections, such that under the action of loads the original angles between intersecting members are unchanged (see clause 4.2 and F4.2 of the code). Connections are usually made in the fabricator's shop as well as the site, by welding and bolting. The connections shown in Fig. 2.5 can be adopted in rigid frame construction, which transfer both shear and moment from beam to column. Fully welded connections can also be considered as rigid beam-to-column connections. Such connections naturally involve additional fabrication and higher erection costs. However, the greater rigidity produced in the structure will result in reduced member sizes and the elimination of bracings. This form of construction is used for low-rise industrial buildings [Figs 2.2(a) and (c)] and for multi-storey buildings [Fig. 2.2(e)].

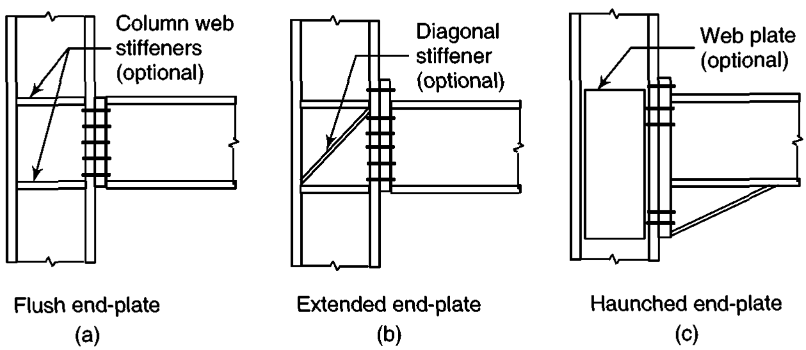


Fig. 2.5 Rigid beam-to-column connections

In rigid frame structures, bending in beams and columns resists horizontal load. The columns, particularly in the lower storeys, must resist heavy moments. So sections will be much larger than in braced frames.

The rigid frame structure deflects more than a braced structure. The deflection is made up of sway in each storey plus overall cantilever action. Due to excessive deflection, rigid frames are suitable only for low- or medium-rise buildings (up to about 15 floors).

Since rigid frames are *statically indeterminate*, they require several cycles of design. Computer programs are often used to analyse such rigid frames.

Frame with semi-rigid connections Semi-rigid connections fall between simple and rigid connections. As a matter of fact, any connection that is adopted in practice will be a semi-rigid connection. Hence, before analysing the frame, the moment-rotation characteristic of the adopted connection has to be established by a rational method or based on experiments (clause 4.2.1.2 of the code). In Appendix F, the code gives some recommendations for obtaining the moment-curvature relationship of single web-angle connections, double web-angle connections, top and seat angle connections (without double web-angle connections), and header plate connections. Computer programs are available for the analysis of frames with semi-rigid connections (Chen et al. 1995). Research is still being done on semi-rigid connections. In practice, most connections are either assumed as simple connections or rigid connections only.

Composite structures The composite steel-shear wall structure consists of a steel-framed building braced with vertical reinforced concrete shear walls, as shown in Fig. 2.6. The shear walls placed in two directions at right angles carry vertical and horizontal loads. The shear walls replace the braced bays in the all-steel building.

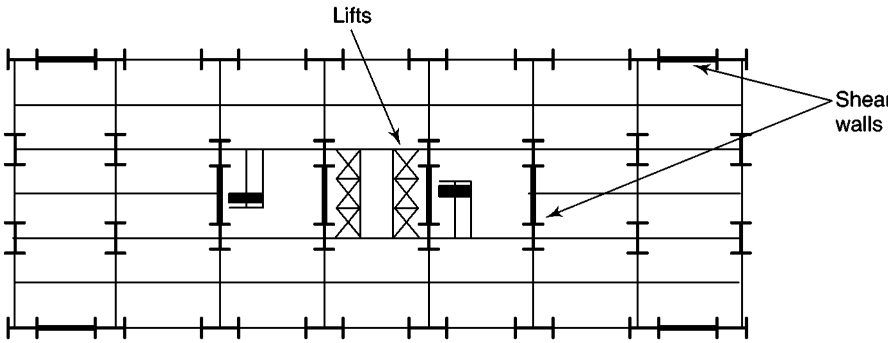


Fig. 2.6 Steel building with concrete shear walls

The shear walls can be located at the ends or sides or in appropriate locations within the building. They should be arranged so as to be symmetrical with respect to the plan, otherwise twisting will occur. They provide fire-proof walls at the lifts and staircase.

2.3.2 High-Rise Structural Systems

As pointed out previously, to build tall buildings economically, the designer must pay attention to the resistance of lateral forces. Several excellent systems have been invented in the past and are shown in Figs 2.7 and 2.8. They include outrigger and belt lattice girders systems, framed tube, braced tube, tube in tube, and bundled tube systems (Subramanian 1995). These systems are discussed briefly in this section.

Outrigger and belt truss system In tall buildings, the lateral deflection can be excessive if the bracing is provided around the core only. This deflection can be reduced by bringing the outside columns also into action to resist the lateral loads by the provision of outrigger and belt lattice girders, as shown in Fig. 2.9. The tension and compression forces in the outer columns apply a couple to the core, which acts against the cantilever bending under wind loads. The belt truss surrounding the building brings all external columns into action (MacGinley 1997). A single outrigger and belt lattice girder system at the top or additional systems at different heights of a very tall building can be provided.

Tube structures The tube type of structure was developed by Dr Fazlur Khan of the USA for very tall buildings, say over 80 storeys in height. If the core type of structure were used, the deflection at the top would be excessive. The tube system is very efficient with respect to structured material used and results in a considerable saving in material when compared with conventional designs.

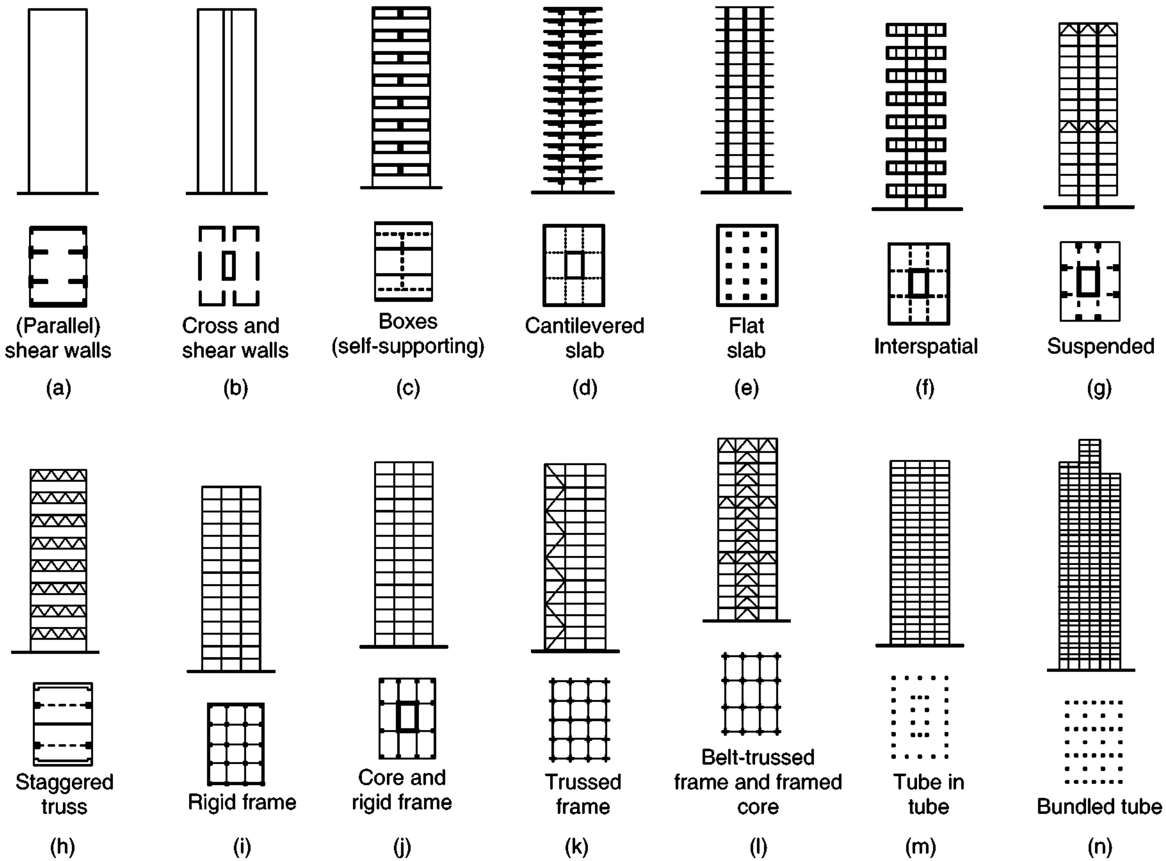


Fig. 2.7 High-rise structural systems

Twin Towers of World Trade Centre – Framed Tube Structure

The twin 110-story towers of the World Trade Center in New York City, USA, were designed by Architect Minoru Yamasaki and Structural Engineers Leslie E. Robertson Associates in the early 1960s using a *framed tube* structural system. The North Tower was completed in December 1970 and the South Tower was finished in July 1971.

Both had a 63 m^2 plan, 350,000 m^2 of office space, and the facades were sheathed in aluminum-alloy. The buildings were designed with narrow office windows 45-cm wide which also reflected Yamasaki's fear of heights as well as his desire to make building occupants feel secure.

The World Trade Center towers had high-strength, closely spaced load-bearing perimeter steel columns (acting like *Vierendeel trusses*) that supported all lateral loads such as wind loads, and shared the gravity load with the core columns [see Fig. 2.10(a)]. The 59 perimeter columns (per side) were constructed using prefabricated modular pieces, each consisting of three columns, three-storeys tall, connected by spandrel plates. Adjacent modules were bolted together with the splices occurring at mid-span of the columns and spandrels. The spandrel plates were located at each floor, transmitting shear stress between columns, allowing them to work together in resisting lateral loads. The joints between modules were staggered vertically, so the column splices between adjacent modules were not at the same floor.

The core of the towers housed the elevator and utility shafts, restrooms, three stairwells, and other support spaces. The core (a combined steel and concrete structure) of each tower was a rectangular area of 27 by 41 m and contained 47 steel columns running from the bedrock to the top of the tower. The large, column-free space between the perimeter and the core was bridged by prefabricated floor trusses. The floor trusses provided lateral stability to the exterior walls and distributed wind loads among the exterior walls. The trusses supported 100-mm thick lightweight concrete slabs that were laid on a fluted steel deck. The floors were connected to the perimeter spandrel plates with *viscoelastic dampers*, in order to reduce the sway of the buildings.

Outrigger truss systems were provided from the 107th floor to the top of the building. There were six such trusses along the long axis of the core and four along the short axis. This truss system allowed some load redistribution between the perimeter and the core columns and also supported the tall communication antenna on the top of each building. When completed in 1972, the North Tower became the tallest building in the world for two years, surpassing the Empire State Building after a 40-year reign.

On September 11, 2001, terrorists crashed a hijacked plane into the northern facade of the North Tower, impacting between the 93rd and 99th floors. Seventeen minutes later, a second team of terrorists crashed another hijacked plane into the South Tower, impacting between the 77th and 85th floors. After burning for 56 minutes, the South Tower collapsed due to the plane impact and as a result of buckling of steel columns due to the ensuing fire. The North Tower collapsed after burning for approximately 102 minutes. The attacks on the World Trade Center resulted in 2,750 deaths and a huge financial loss.

On the northwest corner of the WTC site, a new 551-m tall Freedom Tower, designed by David M. Childs of Skidmore, Owings & Merrill, is being built from April 2006.

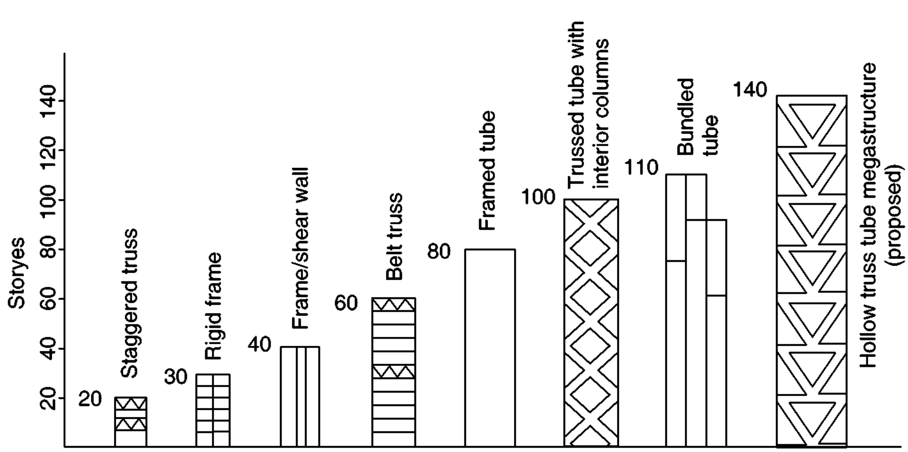


Fig. 2.8 Tall building system for steel structures

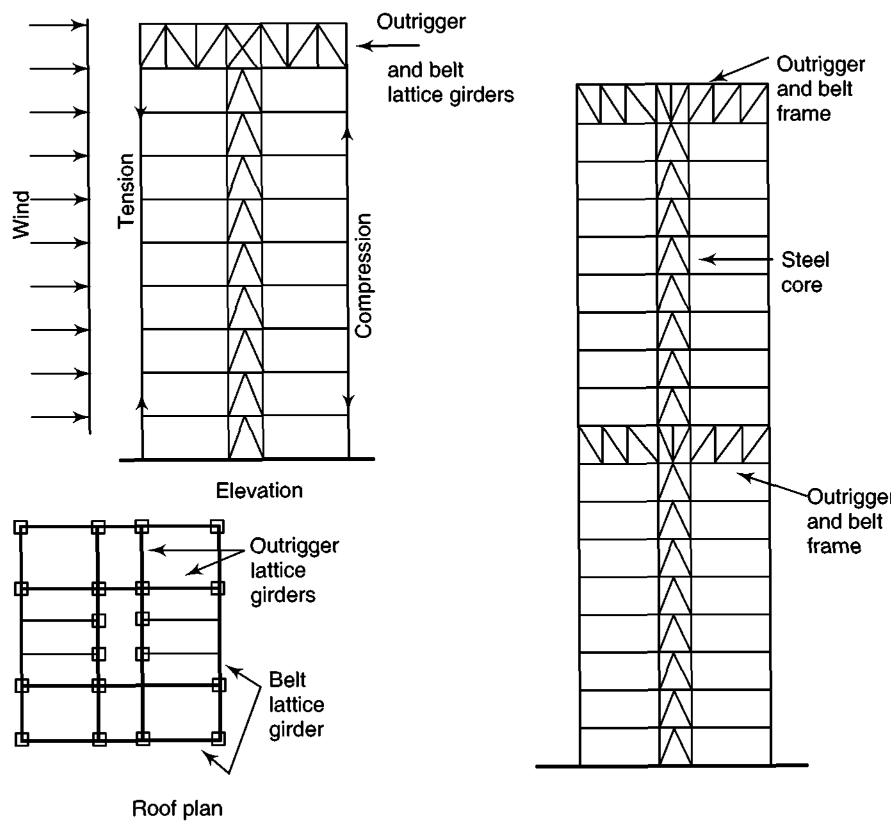


Fig. 2.9 Outrigger and belt lattice girder system (MacGinley 1997)

In this system the perimeter walls are so constructed that they form one large rigid or braced tube, which acts as a unit to resist horizontal load, as shown in Fig. 2.10(a) and (b). The small perforations form spaces for windows, and the normal curtain walling is eliminated.

In the single-tube structure, the perimeter walls carry the entire horizontal load and their share of the vertical load. Internal columns and/or an internal core, if provide, cary vertical loads only. In the tube-within-a tube system shown in Fig. 2.10(e), the internal tube can be designed to carry part of the horizontal load. Very tall stiff structures have been designed on the bundled tube system shown in Fig. 2.10(f) which consists of a number of tubes constructed together. This reduces the *shear lag* problem that is more serious if a single tube is used (MacGinley 1997). Shear lag is a phenomenon in which the stiffer (or more rigid) regions of the structure or structural component attract more stresses than the more flexural regions. Shear lag causes stresses to be unevenly distributed over the cross section of the structure or structural component, as shown in Fig. 2.10(c).

The framed tube can be relatively flexible and bracing the tube, as shown in Fig. 2.10(d), provides additional stiffness. This helps reduce shear lag in the flange tube faces as the diagonal members make all exterior columns act together as a rigid tube.

Dr F.Z. Khan carried the tube concept still further and constructed the Willis Tower (formerly known as Sears Tower), as a bundled tube. In this 108-storey, 442-m tall skyscraper in Chicago built in 1973, a number of relatively small-framed tubes or diagonally braced tubes are bundled together for great effeciency in resisiting lateral forces (Fig. 2.10(f)).

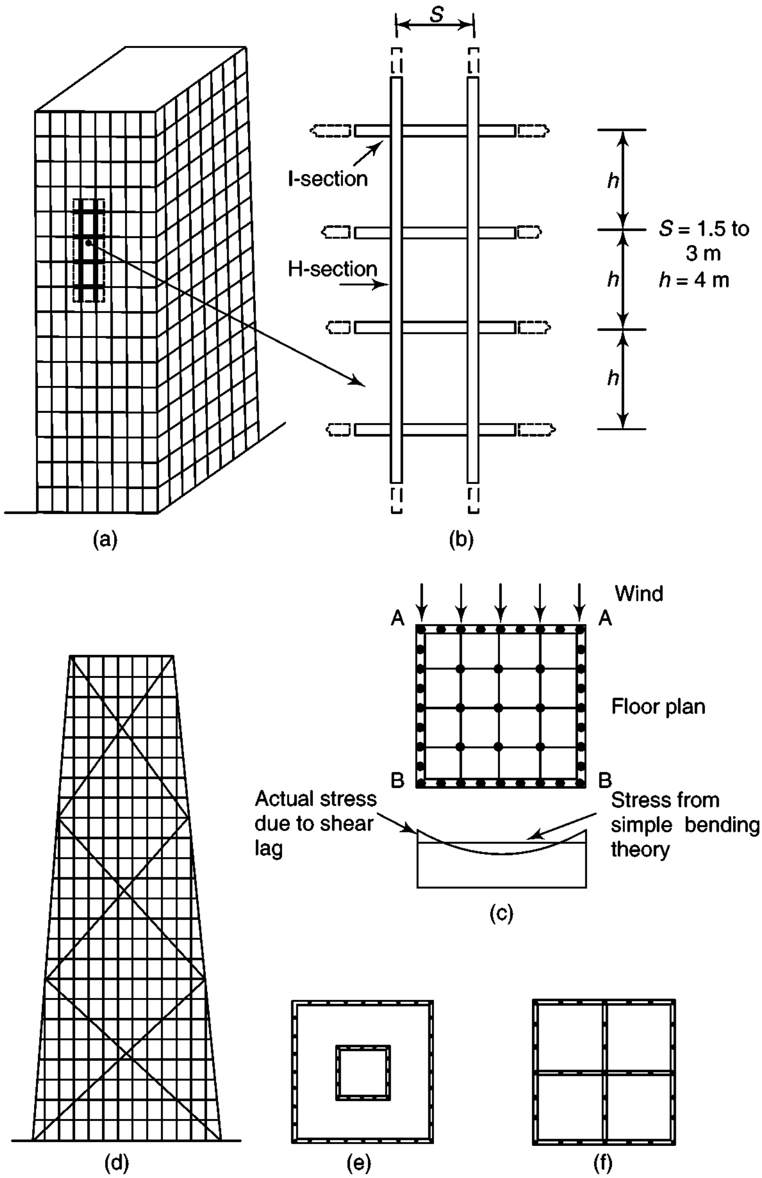


Fig 2.10 Tube structures-(a) framed tube, (b) prefabricated 'tree' unbit (c) stress distribution in walls AA, BB (d) braced tube, (e) tube in tube, and (f) bundled tube (Building may have more storeys than shown.)

John Hancock Center–Braced Tube Structure

The 100-storey, 344-m tall John Hancock Center of Chicago, Illinois, was a collaborative effort of Skidmore, Owings, and Merrill, and structural engineers Fazlur Khan and Bruce Graham. When completed in 1969, it was the tallest building in the world outside New York City.

The distinctive X-bracing at the exterior of the building, is a part of its 'trussed or braced tubular system'. The braced tube system, developed by engineer Fazlur Khan, helps the building stand upright during wind and earthquake loads as the X-bracing provides additional stiffness to the tube. The external bracings of the tube reduce the shear lag in the flange tube faces, as shown in Fig. 2.10(c), as the bracings make all the exterior columns to act together as a rigid tube. In addition to higher performance against lateral loads, it also increases the usable floor space. This original concept made the John Hancock Center an architectural icon.

2.4 Seismic Force Resisting Systems

Moment resisting frames, moment resistant frames with shear walls, braced frames with horizontal diaphragms or a combination of the above systems, may be provided to resist seismic forces (see Fig. 2.11). Out of these, moment resisting frames may be economical for buildings with only up to 5 to 10 storeys (the infill walls of non-reinforced masonry also provides some stiffness). Shear wall and braced systems (which are more rigid than moment resisting frames) are economical up to 15 storeys. When frames and shear walls are combined, the system is called a *dual system.* A moment resisting frame, when provided with specified details for increasing the ductility and energy absorbing capacity of its components, is called a *special moment resisting frame (SMRF)*, otherwise it is called an *ordinary moment resisting frame (OMRF)*.

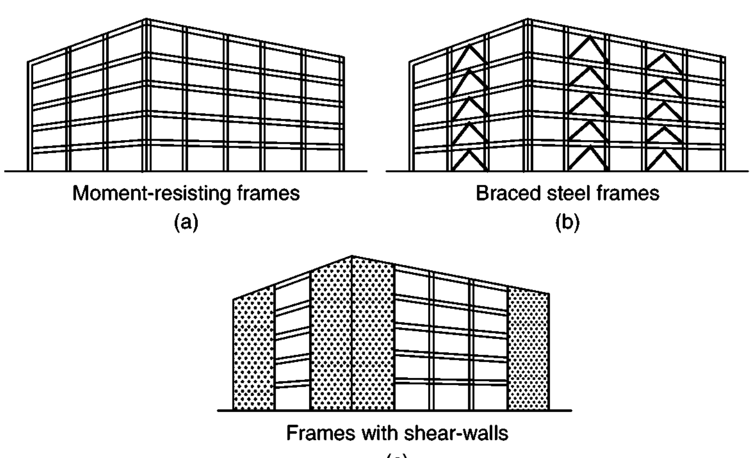


Fig. 2.11 Lateral-force-resisting system

The design engineer should not consider the structure as composed of a summation of different parts (such as beams, columns, trusses, walls, etc.) but as a completely integrated system, which has its own properties with respect to lateral force response. Thus, he or she should follow the flow of forces through the structure into the foundation and make sure that every connection along the path of stress is adequate to maintain the integrity of the system. It is also necessary to provide adequate redundancy in the structure. When a primary system yields or fails, the redundancy will allow the lateral forces to be redistributed to a secondary system to prevent progressive collapse (see Section 2.5). It is also important to note that the forces due to earthquakes are not static but dynamic, (cyclic and repetitive) and hence the deformations will be well beyond those determined from the elastic design.

2.4.1 Moment Resisting Frames

According to clause 12.10 of the code, ordinary moment resisting frames (OMRF) should be able to withstand inelastic deformation corresponding to a joint rotation of 0.02 radians (for special moment resisting frames it is 0.04 radians) without degradation in strength and stiffness below the full yield value (M_p). OMRFs should not be used in Seismic zones IV and V, and for buildings with an importance factor greater than unity in Seismic zone III. OMRFs and SMRFs with rigid moment connections should be designed to withstand a moment of at least 1.2 times the full plastic moment of the connected beam. For OMRFs, a semirigid moment connection is permitted. In such a case, the connection should be designed to withstand the lesser of the following: a moment of at least 0.5 times either the full plastic moment of the connected beam or the maximum moment that can be delivered by the system. In semi-rigid joints, the design moment should be achieved with a rotation of 0.01 radians.

The beam-to-column connection of SMRFs should be designed to withstand a shear resulting from the load combination of 1.2DL + 0.5LL plus the shear resulting from the application of 1.2 M_p in the same direction, at each end of the beam. A similar criterion is provided for the beam-to-column connection of OMRFs.

In a rigid, fully welded connection, continuity plates of thickness equal to or greater than the thickness of beam flanges are provided and welded to the column flanges and the web. In column connections along the strong axis, the panel zone is to be checked for shear buckling. Column web doubler plates or diagonal stiffeners may be used to strengthen the web against shear buckling. Beam and column sections should be either plastic or compact; at potential plastic hinge locations they should be plastic.

For providing strong-column and weak-beam design, the beams and columns should satisfy

$$\Sigma M_{pc} \geq 1.2 \, \Sigma M_{pb} \tag{2.1}$$

where ΣM_{pc} is the sum of the moment capacity in columns above and below the beam center line, and ΣM_{pb} is the sum of the moment capacity in the beams at the intersection of the beam and column center lines.

Note that engineers, during 1980s, tried to economize their designs by providing only a single bay of moment resistant framing on either side of buildings. The 1994 Northridge earthquake and the 1995 Kobe earthquake showed that such buildings are prone to brittle fracture at their welded-beam to column connections. Research conducted after this earthquake, resulted in several special provisions and some approved connections, which are discussed in the chapter on welded connections. Other guidelines may be found in Section 12 of IS 800.

2.4.2 Braced Frames

The members of braced frame act as a truss system and are subjected primarily to axial stress. Current research shows that significant inelastic deformation occurs in the beams and columns of braced frames in addition to the buckling of the brace. Depending on the diagonal force, length, required stiffness, and clearances, the diagonal members can be made of double angles, channels, tees, tubes or even wide flange shapes. Besides performance, the shape of the diagonal is often based on connection considerations. The braces are often placed around service cores and elevators, were frame diagonals may be enclosed within permanent walls. The braces can also be joined to form a closed or partially closed three dimensional cell so that torsional loads can be resisted effectively. A height to width ratio of 8 to 10 is considered to form a reasonably effective bracing system.

Braced frames may be grouped into *concentrically braced frames* (CBFs), and *eccentrically braced frames* (EBFs), depending on their ductility characteristics. In addition, concentrically braced frames are subdivided into two categories, namely, *ordinary concentrically braced frames* (OCBFs) and *special concentrically braced frames* (SCBFs).

Concentrically braced frames In CBFs, the axes of all members, i.e., columns, beams and braces intersect at a common point such that the member forces are axial. The Chevron bracing, cross bracing (X bracing), and diagonal bracing (single diagonal or K bracing) are classified as concentrically braced and are shown in Fig. 2.12(a)–(d).

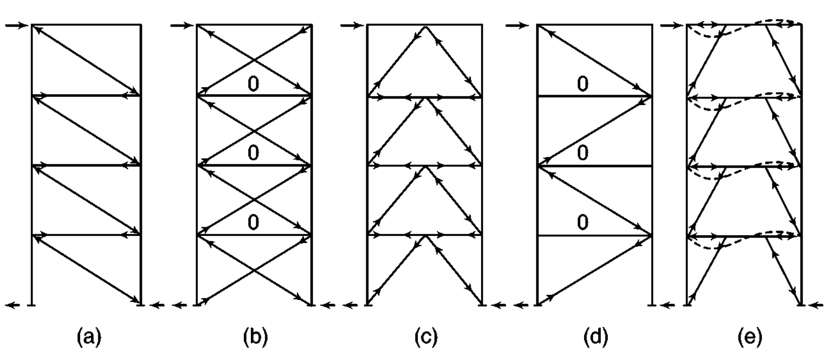


Fig. 2.12 Types of bracings and the load path (a) single diagonal bracing, (b) *X*-bracing, (c) chevron bracing, (d) single-diagonal, alternate direction bracing, and (e) knee bracing.

On the other hand EBFs utilize axial offsets to deliberately introduce flexure and shear into framing beams to increase ductility. For example, in the knee bracing shown in Fig. 2.12(e), the end parts of the beam are in compression and tension with the entire beam subject to double curvature bending. [Note that in all the frames shown in Fig. 2.12(a), a reversal in the direction of horizontal load will reverse all actions and deformations in each of the members]. EBFs are discussed in detail in the next section.

The inability to provide reversible inelastic deformation is the principle disadvantage of CBFs. After buckling, an axially loaded member loses strength and does not return to its original straight configuration. To reduce the possibility of this occurring during moderate earthquakes, more stringent design requirements are imposed on bracing members. Thus, ordinary concentrically braced frames are not allowed in Seismic zones IV and V and for buildings with an importance factor greater than unity (I > 1.0) in zone III; a K bracing is not permitted in earthquake zones by the code (the inelastic deformation and buckling of K bracing members may produce lateral deflection of the connected columns, causing collapse).

Ordinary concentrically braced frames should be designed to withstand inelastic deformation corresponding to a joint rotation of 0.02 radians without degradation in strength and stiffness, below the yield value. The slenderness of bracing members should not exceed 120 and the required compressive strength should not exceed 0.8 P_d, where P_d is the design strength in axial compression. Along any line of bracing, braces shall be provided such that for lateral loading in either direction, the tension braces will resist 30 to 70% of the load. This is to prevent an accumulation of inelastic deformation in one direction and to preclude the use of tension only diagonal bracing.

Special concentrically braced frames should be designed to withstand inelastic deformation corresponding to a joint rotation of 0.04 radians without degradation in strength and stiffness below the full yield value. They are allowed to be used in any zone and for any building. The slenderness ratio of the bracing members should not exceed 160 and the required compressive strength should not exceed the design strength in axial compression, P_d. Along the line of bracing, braces should be provided such that the lateral loading in either direction, the tension braces resist 30 to 70% of the load. The bracing and column sections used in special concentrically braced frames should be plastic sections.

The above provisions are for X braces only. For other types of bracings such as Chevron or V-type bracings and for eccentrically braced frames, the code does not give any guideline. More information about the design of such bracings may be found in Becker (1995), Becker and Ishler (1996), Bruneau, et al. (1997), Bozorgnia and Bertero (2004), Williams (2004), and Roeder and Lehman (2008).

The connections in a braced frame may be subjected to impact loading during an earthquake and in order to avoid brittle fracture, must be designed to withstand the minimum of the following:

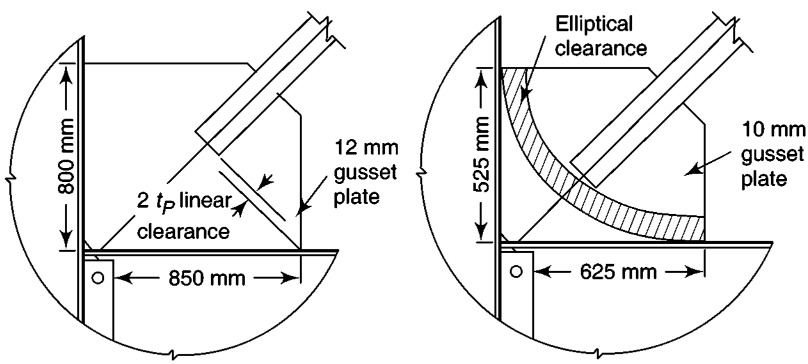
(a) A tensile force in the bracing equal to $1.1 f_y A_g$, and
(b) The force in the brace due to the following load combinations,

$$1.2\ DL + 0.5LL \pm 2.5\ EL,$$

$$0.9\ DL \pm 2.5EL$$

(c) The maximum forces that can be transferred to the brace by the system.

The connection should be checked to withstand a moment of 1.2 times the full plastic moment of the braced section about the buckling axis and for tension rupture, and block shear under the above loading. The gusset plates should be checked for buckling out of their plane, and sufficient length should be provided for plastic hinge formation. Recent research has shown that the current practice of providing a linear clearance of twice the thickness of gusset plates [see Fig. 2.13(a)], leads to thicker and larger size of gusset plates. This creates a rotationally stiff joint, which limits the rotation of the connection and leads to extensive frame yielding. Based on the recent research, Roeder and Lehman (2008) suggest to provide an elliptical clearance of eight times the thickness of the gusset plate [see Fig. 2.13(b)]. This will not only result in smaller, thinner, and compact gusset plates, but also greater ductility and inelastic deformation of the system. Welds joining the gusset plate to the beam and column should be sized using the plastic capacity of the gusset plate rather than the expected resistance of the brace.



(a) Linear 2t clearance (b) Elliptical 8t clearance

Fig. 2.13 Improved connection detail for CBFs (Roeder and Lehman 2008)

Eccentrically braced frames The bracing member in an EBF is connected to the beam so as to form a short link beam between the braces and the column or between two opposing braces (see Fig. 2.14). Thus, the eccentric bracing is a unique structural system that attempts to combine the strength and stiffness of a braced frame with the inelastic behaviour and energy dissipation characteristics of a moment frame. The link beam acts as a fuse to prevent buckling of the brace from large overloads that may occur during major earthquakes. After the elastic capacity of the system is exceeded, shear or flexural yielding of the link provides a ductile response in contrast to that obtained in an SMRF. In addition, EBFs may be designed to control frame deformations and minimize damage to architectural finishes during seismic loading (Williams 2004). Note that the connection between the column and beam are moment connected [see Fig. 2.14(f)] to achieve brace action. The web buckling is prevented by providing adequate stiffness in the link. Links longer than twice the depth of the beam tend to develop plastic hinges, while shorter links tend to yield in shear. Buildings using eccentric bracings are lighter than MRFs and, while retaining the elastic stiffness of CBFs, are more ductile. Thus, they provide an economical system in seismic zones. A premature failure of the link does not cause the structure to collapse, since the structure continues to retain its vertical load carrying capacity and stiffness. This facilitates easy repair of the system after a severe earthquake. The design and other details of eccentrically braced systems are provided by Williams (2004) and Bruneau, et al. (1997).

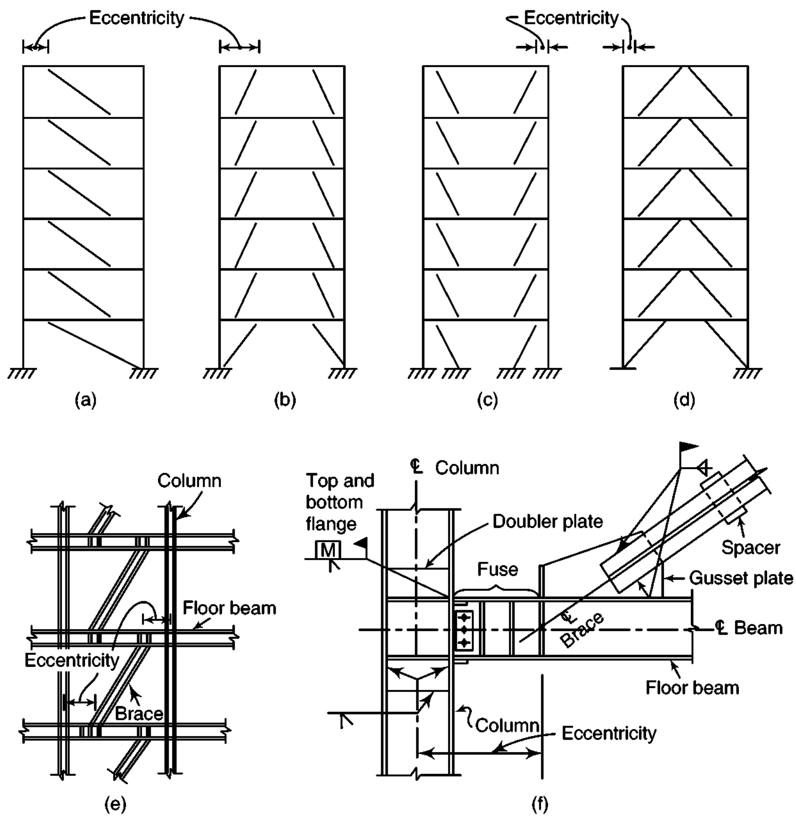


Fig. 2.14 Eccentric bracing system: (a–d) common types of bracing, (e) elevation, and (f) detail (Taranath 1998)

2.5 Structural Integrity

To reduce the risk of localized damage spreading to all parts, buildings should be effectively tied together at each principal floor level. It is important to effectively hold each column in position by means of horizontal ties in two directions (preferably at right angles), at each principal floor level supported by the column. Horizontal ties are also required at roof level, except where the steel work supports only cladding weighing 0.7 kN/m^2 or less and carrying only imposed roof loads and wind loads. At re-entrant corners the tie member nearest to the edge should be anchored into the steel framework, as shown in Fig. 2.15. All these horizontal tie members should be capable of resisting a minimum factored tensile load (should not be considered as additive to other loads) of 75 kN at floor level and 40 kN at roof level. A minimum tie strength of 0.5 $W_f S_t L_a$ for internal ties and one of 0.25 $W_f S_t L_a$ for external ties is also suggested in BS 5950, 2000 (where W_f is the total factored load/unit area, S_t is the tie spacing and L_a is the distance between columns in the direction of the ties). Note that these integrity considerations have a direct influence on connection design, since the tying action of beams requires the connection to possess adequate direct tension capacity. Experimental work done in the UK has established that end plates and web cleats of 8 mm thickness fastened to column flanges by top M20 grade 8.8 bolts will meet the requirement of ties resisting 75 kN factored tensile load (Nethercot 2001).

Each portion of a building between expansion joints should be treated as a separate building. By tying the structure together as shown in Fig. 2.15, alternative load paths, which enhance the safety, may be made available. To ensure sway resistance, no portion of the structure should be dependent on only one bracing system. All columns should be continuous vertically through the floors.

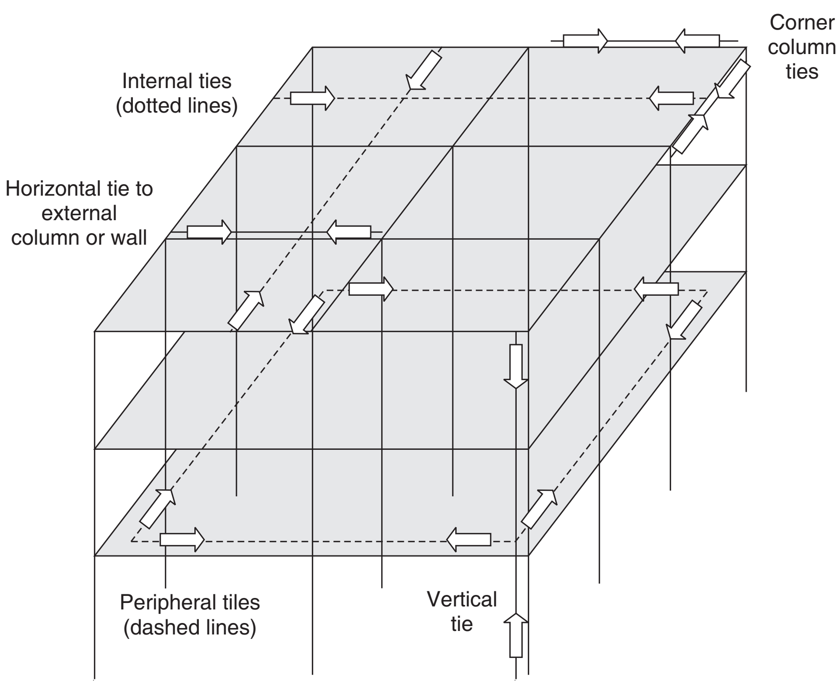


Fig. 2.15 Tying columns of building to achieve structural integrity

Precast concrete or other heavy floor or roof units must be properly anchored at both ends. At the edge of the structure, horizontal ties capable of resisting 1% of the maximum factored column loads should restrain columns. Key elements that would risk the collapse of greater area (greater than 15% of the floor area or 70 m^2, whichever is less) should be identified and designed for accidental loading (see clause 5.1.2 of IS 800 : 2007 for more on this).

Ronan Point Collapse

Progressive collapse provisions were introduced in the British code as early as 1970. This was a direct result of the Ronan Point collapse in 1968. This involved a 23-storey tower block in Newham, East London, which suffered a partial collapse when a gas explosion demolished a load-bearing wall, causing the collapse of one entire corner of the building. Four people were killed in the incident, and 17 were injured. (Ronan Point was repaired after the explosion; it was demolished in 1986 for a new low-rise housing development project.)

Due to the failure of Ronan Point apartment, many other similar large panel system buildings were demolished. The Building Research Establishment, UK, published a series of reports in the 1980s to advise councils and building owners on what they should do to check the structural stability of their blocks. As a result of terrorist attacks on embassies abroad, along with the Murrah Federal Building in Oklahoma City, abnormal load requirements were introduced in the US codes. Structural integrity requirements have been introduced in IS 800 only now.

2.6 Loading and Load Combinations

Before designing any structure or the different elements, such as beams, columns, etc., one has to determine the various natural and man-made loads acting on them. These loads on a structure may be due the following:

(a) Mass and gravitational effect ($m \times g$). The examples of these types of loads are dead loads, imposed loads, snow, ice, earth loads, hydraulic pressure, etc.

(b) Mass and its acceleration effect ($m \times a$). The examples of such loads are earthquake, wind, impact, blast, etc.

(c) Environmental effects. Examples include the loads due to temperature difference, settlement, shrinkage, etc. They are also termed as *indirect loads*.

In India, the basic data on dead, live and wind loads for buildings are given in IS 875, with more specialized information on matters such as load produced by cranes in industrial buildings provided by other codes (e.g., IS 807). The earthquake loads are specified in IS: 1893. For towers and other forms of structures, the necessary loading data are provided in the code of practice appropriate to that type of structure (e.g. IS 802, IS 9178, IS 6533). We will briefly discuss about a few important loads in this section.

2.6.1 Dead Loads

The load fixed in magnitude and in position is called a *dead load*. Determination of the dead load of the structure requires the estimation of the weight of the structure together with its associated *non-structural* components. Thus, we have to calculate and include the weight of bare steelwork (including items, such as bolts, nuts, and weld material) slabs, beams, walls, columns, partition walls, false ceiling, facades, cladding, water tanks, stairs, plaster finishes, and other services (cable ducts, water pipes, etc). After the design process, the initially assumed dead weight of the structure (based on experience), has to be compared with the actual dead load. If the difference between the two loads is significant, the assumed dead load should be revised and the structure redesigned. Dead weights of different materials are provided in the code IS 875 (part 1 – dead loads). The weights of some important building materials are given in Table 2.1.

Table 2.1 Weights of some building materials

Material	Unit Weight
Brick masonry in CM 1:4	20 kN/m^3
Plain concrete	24 kN/m^3
Reinforced cement concrete	25 kN/m^3
Stone masonry	20.4 to 26.5 kN/m^3
Cement Mortar	20.4 kN/m^3
Steel	78.5 kN/m^3
20-mm cement plaster	450 N/m^2
Roofing sheets	
(a) GI sheet 1.6mm thick	156 N/m^2
(b) Steel sheet 1mm thick	77.5 N/m^2
(c) AC sheet 6mm thick	160–170 N/m^2
5-mm glass	125 N/m^2
Floor finishes	600–1200 N/m^2

2.6.2 Imposed Loads

Imposed loads (previously referred to as *live loads*) are gravity loads other than dead loads and cover items, such as occupancy by people, movable equipment and furniture within the buildings, stored materials such as books, machinery, and snow. Hence, they are different for different types of buildings: domestic, office, warehouse, etc. Thus, they vary often in space and time.

Imposed loads are generally expressed as static loads for convenience, although there may be minor dynamic forces involved. The code gives uniformly distributed loads as well as concentrated loads for various occupational categories. The magnitude of a few imposed loads are as given below:

(a) Residential buildings : 2 kN/m^2
(b) Office buildings : 3–4 kN/m^2
(c) Storage facilities : 5–7.5 kN/m^2

Note that live load may change from room to room. For considering the load due to partition, increase the floor load by 33.3% per meter run of partition wall subject to a minimum of 1 kN/m^2; the total weight per meter run must be less than 4 kN/m. For complete guidance, the engineer should refer to IS 875 (Part 2).

When large areas are considered, the code allows for a reduction in live load; for single beam or girders, a reduction of 5% for each 50 m^2 floor area, subjected to a maximum of 25% is allowed. In multi-storey buildings, the probability that all the floors will be simultaneously loaded with the maximum live loads is remote, and hence reduction to column loads is therefore allowed. Thus, the live loads may be reduced in the design of columns, walls and foundations of multi-storey buildings, as given in Table 2.2. Note that such reduction is not permissible, if we consider earthquake loads.

Table 2.2 Reduction in live load applicable to columns

Floor measured from top	Percentage
1 (top or roof)	0
2	10
3	20
4	30
5–10	40
11 to ground floor	50

The imposed loads on roofs as per IS 875 (Part 2) are given in Table 2.3.

Table 2.3 Imposed loads on various types of roofs

Type of roof	Uniformly distributed imposed load measured on the plan area	Minimum imposed load measured on plan
Flat, sloping, or curved roof with slopes up to and including 10 degrees		3.75 kN uniformly distributed over any span of 1 m width of the roof slab and 9 kN uniformly distributed over the span of any beam or truss or wall.
(a) Access provided	1.5 kN/m^2	
(b) Access not provided (except ladder for maintenance)	0.75 kN/m^2	Half of case (a) above
Slooping roof with a slope greater than 10 degrees	For roof membrane sheets or purlins, 0.75 kN/m^2 less 0.02 kN/m^2 for every degree increase in the slope over 10 degrees	0.4 kN/m^2

Note: 1. The loads given above do not include the loads due to snow, rain, dust collection, etc. The roof should be designed for imposed loads given above or for snow/rain load, whichever is greater.

2. All roof covering (other than glass) should be capable of carrying an incidental load of 900 N concentrated over an area of 125 mm^2.

3. Trusses, beams, columns, and girders excluding purlins can be designed for 2/3 of the live load on the roof.

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IS 875 (Part 2) also gives *horizontal loads* acting on parapets, parapet walls, and balustrades. These loads should be assumed to act at handrail or coping level.

2.6.3 Crane and Impact Loads

In the design of crane runway girders (see Fig. 2.16) and their connections, the horizontal forces caused by moving crane trolleys must be considered, in addition to the vertical and impact loads. The intensity of the horizontal load (also called *lateral load*) is a function of the weight of the trolley, lifting load, and the acceleration of trolley. As per IS 875 – Part 2, the lateral load may be taken as,

(a) C_{Lh} = 10% of weight of trolley and lifted load in the case of electrically operated cranes (EOT) with a trolley having a rigid mast for the suspension of lifted weight, and

(b) C_{Lh} = 5% of weight of trolley and lifted load for all other EOT and hand-operated cranes.

The above force should be applied at the tip of the rail acting in either direction normal to the runway rails, and should be distributed amongst all the wheels on one side of the rail track.

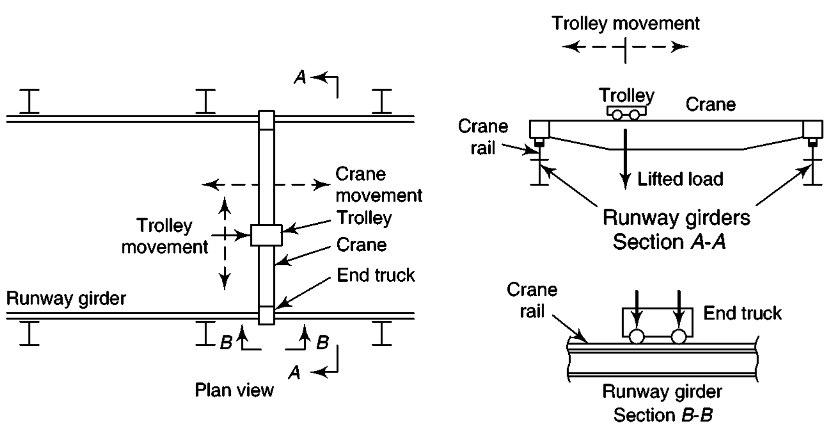


Fig. 2.16 Loads due to crane movement

In addition, due to acceleration and deceleration of the entire crane, a longitudinal tractive force is transmitted to the runway girder through the friction of the end track wheels with the crane rail. IS 875 (Part 2) specifies that 5% of the maximum static wheel load of the crane is to be applied as longitudinal force, at the top of the rail.

The impact due to vertical crane loads is converted empirically into equivalent static loads through an impact factor, which is normally a percentage of the crane load. Table 2.4 shows the impact factors as suggested by IS 875 code for cranes and lifts. Thus, if the impact is 25%, the live load is multiplied in the calculation of the forces by 1.25.

CHAPTER 3

Design of Tension Members

Introduction

Steel tension members are probably the most common and efficient members in the structural applications. They are those structural elements that are subjected to direct axial tensile loads, which tend to elongate the members. A member in pure tension can be stressed up to and beyond the yield limit and does not buckle locally or overall. Hence, their design is not affected by the classification of sections, for example, compact, semi-compact, etc. as described in Chapter 4. The design stress f_y as determined from Table 1.3, is therefore not reduced.

Tension members occur as components of trusses (bottom chord of roof trusses), bridges, transmission line and communication towers, and wind bracing systems in multi-storey buildings (see Fig. 3.1). Some truss web members and members in towers may carry tension under certain loading cases and may be subjected to compression for other loading cases. Steel cables used in suspension bridges and in cable supported roofs are also examples of tension members. Such cables are also used in guyed towers as well as power line poles where alignment changes occur.

Tension members carry loads most efficiently, since the entire cross section is subjected to uniform stress. The strength of these members is influenced by several factors such as the length of connection, size and spacing of fasteners, net area of cross section, type of fabrication, connection eccentricity, and shear lag at the end connection. To simplify the design procedure of tension members, considerable amount of research has been carried out (Salmon & Johnson 1996; Kulak & Wu 1997). This chapter discusses the effects of these parameters and the design of tension members as per IS 800.

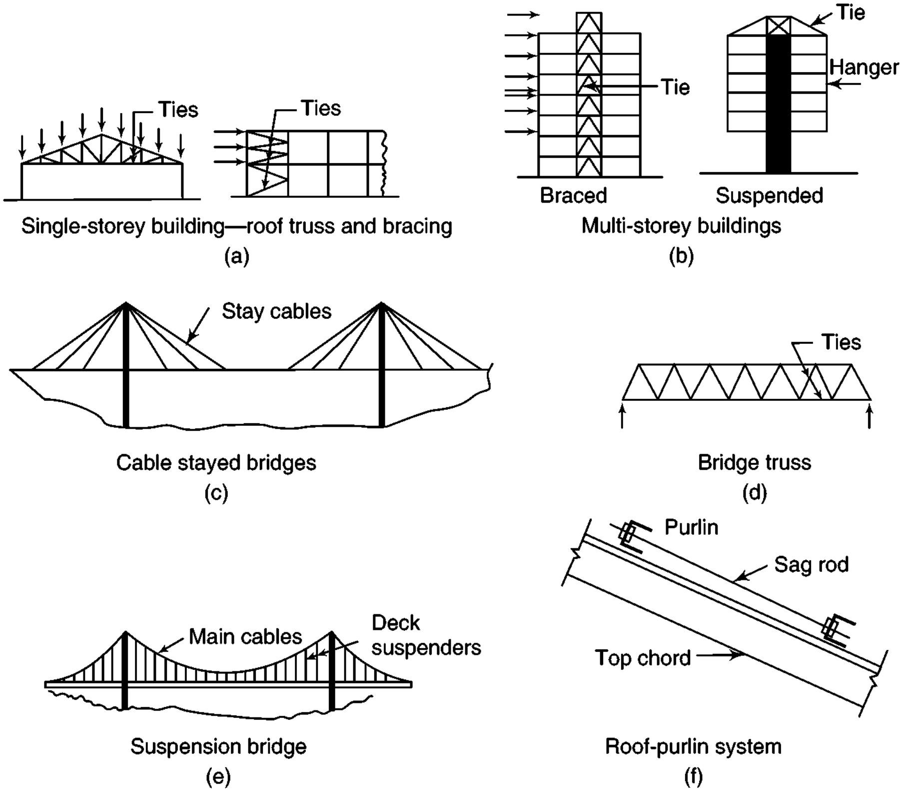


Fig. 3.1 Tension members in buildings and bridges

3.1 Types of Tension Members

Tension members may consist of single structural shape or they may be built using a number of structural shapes. The cross section of some typical tension members are shown in Fig. 3.2. When two elements such as two angles are used as a single member, they should be interconnected at reasonable intervals to enable them to act together as one member. These two separate elements are often placed parallel to each other with a gap of about 6–10 mm between them. They should also have *spacer plates* placed at regular intervals between them, which are connected to these individual elements by tack welds or bolts as shown in Fig. 3.3. Though in Fig. 3.3 only welding is shown, similar rules apply for a bolted spacer (also called as *stitch plates*). Single angle or double angles are either bolted to a single gusset plate at each end or may be welded directly to the webs or flanges of T- or I-chord members.

Structural T-sections may be used as chord members of lightly loaded trusses, instead of the back-to-back two angle sections. The stem of the T-sections may be used to connect the single or double angle web members, thus eliminating the use of any gusset plate, especially in welded connections. Tubular members are also used in roof trusses as tension members.

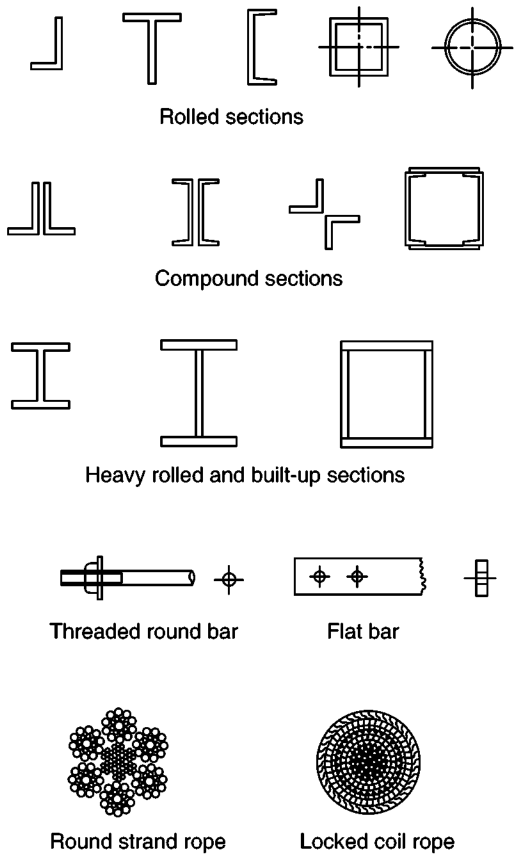


Fig. 3.2 Cross section of typical tension members

I-sections, channel sections, and built-up sections using angles, channels, etc. are used when greater rigidity is required and hence are often used in bridge structures.

Rods and bars are used as tension members in the bracing systems. As mentioned earlier, sag rods are used to support purlins, to support girt in industrial buildings, or as longitudinal ties. They are either welded to the gusset plates or threaded and bolted to the main members directly using nuts. When rods are used as wind bracings, they are pre-tensioned to reduce the effect of sway.

3.2 Slenderness Ratio

Although stiffness is not required for the strength of a tension member, a minimum stiffness is stipulated by limiting the maximum slenderness ratio of the tension member. The *slenderness ratio* of a tension member is defined as the ratio of its unsupported length (L) to its least radius of gyration. This limiting slenderness ratio is required in order to prevent undesirable lateral movement or excessive vibration. (As stated already, stability is of little concern in tension members.) The slenderness limits specified in the code for tension members are given in Table 3.1.

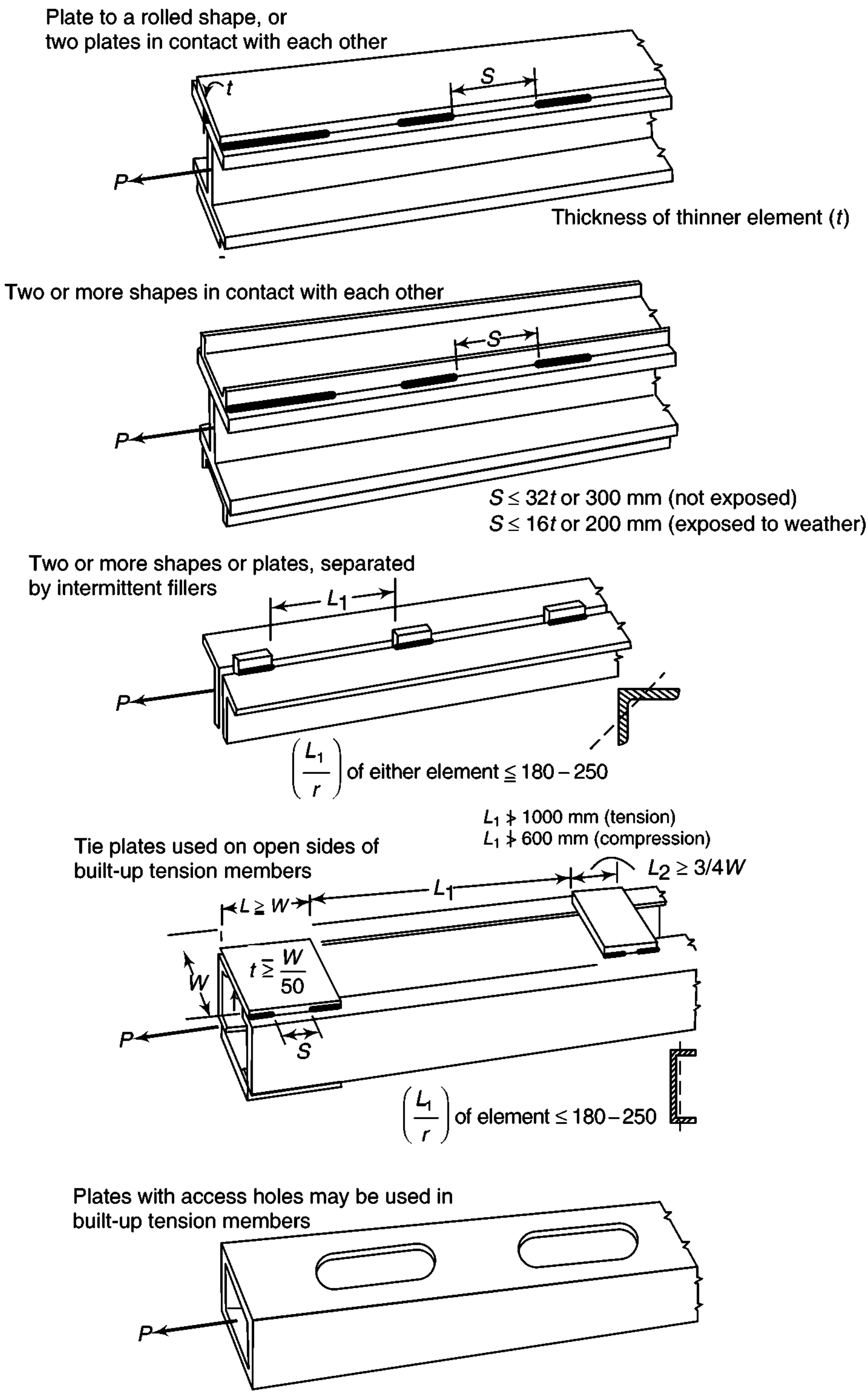


Fig. 3.3 Tack welding of built-up tension members

Table 3.1 Maximum values of effective slenderness ratios as per IS 800

Member	Maximum effective slenderness ratio (L/r)
A tension member in which a reversal of direct stress occurs due to loads other than wind or seismic forces	180
A member subjected to compressive forces resulting only from a combination of wind/earthquake actions, provided the deformation of such a member does not adversely affect the stresses in any part of the structure	250
A member normally acting as a tie in a roof truss or a bracing member, which is not considered effective when subject to reversal of stress resulting from the action of wind or earthquake forces	350
Members always in tension (other than pre-tensioned members)	400

3.3 Displacement of Tension Members

The increase in the length of a member due to axial tension under service loads is

$$\Delta = PL/(EA_g) \tag{3.1}$$

where is the axial elongation of the member (mm), P is the axial tensile force (un-factored) in the member (N), L is the length of the member (mm), and E is the modulus of elasticity of steel = 2.0×10^5 MPa. Note that displacement is a serviceability limit state criterion and hence is checked under service loads and not under factored loads.

3.4 Behaviour of Tension Members

The load–deformation behaviour of an axially loaded tension member is similar to the basic material stress–strain behaviour (see Fig. 1.4). When a member is subjected to tension, the area of cross section and the gauge length continuously change due to the Poisson effect and longitudinal strain, respectively (see Section 1.8.1 also). Stresses and strains may be calculated using the initial area of cross section and the initial gauge length, which is referred to as the engineering stress and engineering strain or using the current area of cross section and the current gauge length, which is referred to as the true stress and true strain.

The engineering stress–strain curve does not give a true indication of the deformation characteristics of a metal because it is based entirely on the original dimensions of the specimen, and these dimensions change continuously as the load increases. In fact, post-ultimate strain softening in engineering stress–strain curve caused by the necking of the cross section is completely absent in the true stress–strain curve. When the true stress based on the actual cross-sectional area of the specimen is used, it is found that the stress–strain curve increases continuously until fracture occurs. The true stress–strain curve is also known as *flow curve* since it represents the basic plastic flow characteristics of the material. Any point on the flow curve can be considered as the local stress for a metal strained in tension by the magnitude shown on the curve. However, since it is difficult to obtain the ordinates of true stress–strain curve, the engineering stress–strain curve is often utilized. As discussed in Section 1.8.1 and shown in Fig. 1.4(a), high-strength steel tension members do not exhibit a well-defined yield point and yield plateau. Hence the 0.2% offset load is usually taken as the yield point for such high-strength steel.

3.5 Modes of Failure

In the following sections, the different modes of failure of tension members are discussed.

3.5.1 Gross Section Yielding

Generally a tension member without bolt holes can resist loads up to the ultimate load without failure. But such a member will deform in the longitudinal direction considerably (nearly 10%–15% of its original length) before fracture. At such a large deformation a structure becomes unserviceable. Hence, code limits design strength in clause 6.2; substituing for γ_{m0}, which is the partial safety factor for failure in tension by yielding ($\gamma_{m0} = 1.10$), we get

$$T_{dg} = 0.909\, f_y A_g \tag{3.2}$$

where A_g is the gross area of cross section in mm^2, and f_y is the yield strength of the material (in MPa).

3.5.2 Net Section Rupture

A tension member is often connected to the main or other members by bolts or welds. When connected using bolts, tension members have holes and hence reduced cross section, being referred to as the *net area*. Holes in the members cause stress concentration at service loads, as shown in Fig. 3.4(a). From the theory of elasticity, we know that the tensile stress adjacent to a hole will be about two to three times the average stress on the net area, depending upon the ratio of the diameter of the hole to the width of the plate normal to the direction of stress. Stress concentration becomes very significant when repeated applications of load may lead to fatigue failure or when there is a possibility of a brittle fracture of a tension member under dynamic loads. Stress concentration may be minimized by providing suitable joint and member details.

When a tension member with a hole is loaded statically, the point adjacent to the hole reaches the yield stress f_y first. On further loading, the stress at that point remains constant at yield stress and each fibre away from the hole progressively reaches the yield stress f_y [see Fig. 3.4(b)]. Deformations continue with increasing load until finally rupture (tension failure) of the member occurs when the entire net cross section of the member reaches the ultimate stress f_u. The design strength due to net section rupture for plates is given in Section 6.3.1 of the code. Substituting the value for γ_{m1} which is the partial safety factor for failure due to rupture of cross section (= 1.25), we get,

$$T_{dn} = 0.72 f_u A_n \tag{3.3}$$

where A_n is the net effective area of the cross section in mm^2, and f_u is the ultimate strength of the material in MPa. Because of strain hardening, the actual strength of a ductile member may exceed that indicated by Eqn (3.2). However, since there is no reserve of any kind beyond the ultimate resistance an additional multiplier of 0.90 has been introduced in Eqn (3.3). Such a high margin of safety has been traditionally used in design when considering the fracture limit state than for the yielding limit state (Salmon & Johnson 1996). The 0.9 factor was included in the strength equation of Eqn (3.3), based on a statistical evaluation of a large number of test results for net section failure of plates.

Similarly, threaded rods subjected to tension could fail by rupture at the root of the threaded region. Thus, the design strength of the threaded rods in tension is given by Eqn (3.3) where A_n is the net root area at the threaded sections.

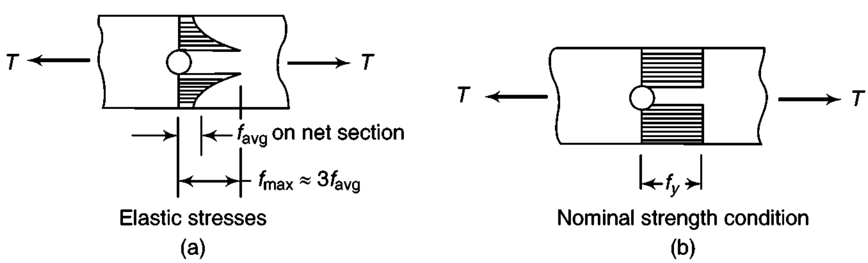


Fig. 3.4 Stress concentration due to holes

3.5.3 Block Shear Failure

Originally observed in bolted shear connections at coped beam ends, block shear is now recognized as a potential failure mode at the ends of axially loaded tension members also. In this failure mode, the failure of the member occurs along a path involving tension on one plane and shear on a perpendicular plane along the fasteners as shown in Fig. 3.5. Other examples of *block shear failures* including failures in welded connections are given in Fig. 3.6.

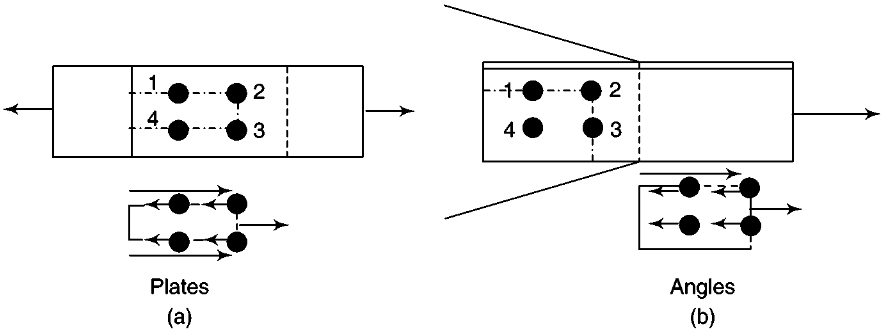


Fig. 3.5 Block shear failure in plates and angles

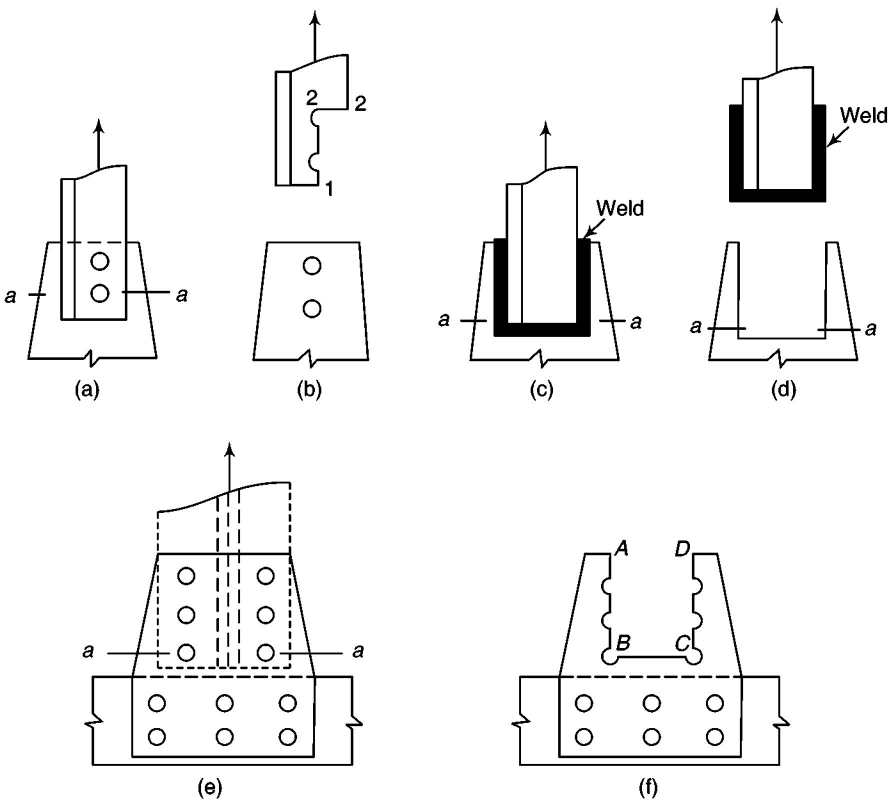


Fig. 3.6 Examples of block shear failure

It can be observed as shown in Fig. 3.6(a) that the gusset plate may fail in tension on the net area of section *a-a*, and in Fig. 3.6(c) it may fail on the gross area of section *a-a*. The angle member in Fig. 3.6(a) may also separate from the gusset plate by shear on net area 1-2 combined with tension on net area 2-2 as shown in Fig. 3.6(b). A similar fracture of the welded connection of Fig. 3.6(c) is shown in Fig. 3.6(d). The fracture of a gusset plate for a double angle member or of one of the gusset plates for an I-Section [Fig. 3.6(e)] is shown in Fig. 3.6(f). The gusset plate in Fig. 3.6(d) may also fail on the net section a-a. All these failures [Figs 3.6(b), (d), and (f)] are called block shear failures.

The block shear phenomenon becomes a possible mode of failure when the material bearing strength and bolt shear strength are higher. As indicated earlier, the appropriate model of the block shear failure is the rupturing of the net tension plane (BC) and yielding on the gross shear plane (AB and CD), as shown in Fig. 3.6(f), which results in rupturing of the shear plane as the connection lengths become shorter.

The block shear strength is given in section 6.4.1 of the code. Substituting the value of γ_{m0} (= 1.1) and γ_{m1} (1.25), we get the following:

(a) Plates: The block shear strength T_{db} of the connection is taken as the smaller of

$$T_{db1} = 0.525\, A_{vg} f_y + 0.72\, f_u A_{tn} \tag{3.4a}$$

$$T_{db2} = 0.416\, f_u A_{vn} + 0.909\, f_y A_{tg} \tag{3.4b}$$

where A_{vg} and A_{vn} are the minimum gross and net area in shear along a line of transmitted force, respectively [1-2 and 4-3 as shown in Fig. 3.5(a) and 1-2 as shown in Fig. 3.5(b); A_{tg} and A_{tn} are the minimum gross and net area in tension from the hole to the toe of the angle or next last row of bolt in plates, perpendicular to the line of force respectively [2-3 as shown in Figs 3.5(a) and (b)]; and f_u and f_y are the ultimate and yield stress of the material, respectively.

(b) Angles: Strength as governed by block shear failure in angle end connection is calculated using Eqn (3.4) and appropriate areas in shear and tension as shown in Fig. 3.5(b).

The lower values of the design tension capacities, as given by Eqns (3.2) to (3.4), govern the design strength of plates or members with hole and should be greater than the factored design tension. Note that no net areas are involved in the failures of welded connections [see Fig. 3.6(c)]. Therefore, in applying Eqn (3.4) to this case in the second term of Eqn (3.4a), use A_{tg} (instead of A_{tn}) and in the first term of Eqn (3.4b), use A_{vg} (instead of A_{vn}).

Recently, Driver et al. (2006) proposed a unified equation for block shear failure to predict the capacities of angles, tees, gusset plates, and coped beams.

Based on this work, the 2009 version of the Canadian code has adopted the following equation

$$T_{db} = 0.75[U_t A_{tn} f_u + 0.6\, A_{vg}(f_y + f_u)/2] \tag{3.4c}$$

where U_t is the efficiency factor and equals 1.0 for flange connected tees and for symmetric failure patterns and concentric loading; 0.6 for angles connected by one leg and stem connected tees; 0.9 for coped beams with one bolt line; and 0.3 for coped beams with two bolt lines. For f_y > 485 MPa, $(f_y + f_u)/2$ should be replaced by f_y. Other terms are defined already.

3.6 Factors Affecting the Strength of Tension Members

As discussed already, the yielding of the gross section of tension member causes excessive elongation and hence the load corresponding to the yielding of gross section is taken as one limit state. However, the net section through the bolt holes at the ends of the member may be subjected to tensile stresses well in excess of the yield stress to as high as ultimate stress without the member suffering excessive elongation. Hence, the rupture strength of the net section through the bolt holes at the ends is considered another limit state. Several factors affect the rupture strength of the net section of tension members. They are briefly described below.

3.6.1 Effect of Bolt Holes

In order to make connections, tension members are often bolted to adjacent members directly or by using gusset plates. These bolt holes reduce the area of cross section available to carry tension and hence affect the strength as discussed in the following section.

3.6.1.1 Methods of fabrication

There are generally two methods of making holes to receive bolts, namely punching and drilling. Due to punching, the material around the holes is deformed in shear beyond ultimate strength to punch out the hole.

Under cyclic loading the material around the punched holes present the greatest scope for crack initiation due to stress concentration, and hence punched hole is not allowed under fatigue environment.

Presently in many specifications, the punching effect upon the net section strength is accounted for by taking the hole diameter as 2 mm larger than the actual hole size when computing the net area (see clause 3.6.1 of IS 800).

3.6.1.2 Net area of cross section

The presence of a hole tends to reduce the strength of a tension member. When more than one bolt hole is present, the failure paths may occur along sections normal to the axis of the member, or they may include zigzag sections, if the fasteners are staggered (Fig. 3.7). Staggering holes improves the load carrying capacity of the member for a given row of bolts. When the bolts are arranged in a zigzag fashion with a pitch p and gauge g, the net effective area of the plate with a width B and thickness t is given by

$$A_n = [(B - nd_h + \Sigma(p^2/4g)]t \tag{3.5}$$

where n is the number of bolt holes in the critical section considered, the summation is over all the paths of the critical sections normal to the direction of the tensile force, and d_h is the diameter of the bolt hole. The above empirical relation was proposed by Cochrane in 1922 based on experimental evidence. All possible failure paths (straight as well as zigzag) are to be considered and the corresponding net areas are to be computed as per Eqn (3.5) to find the minimum net area of the plate.

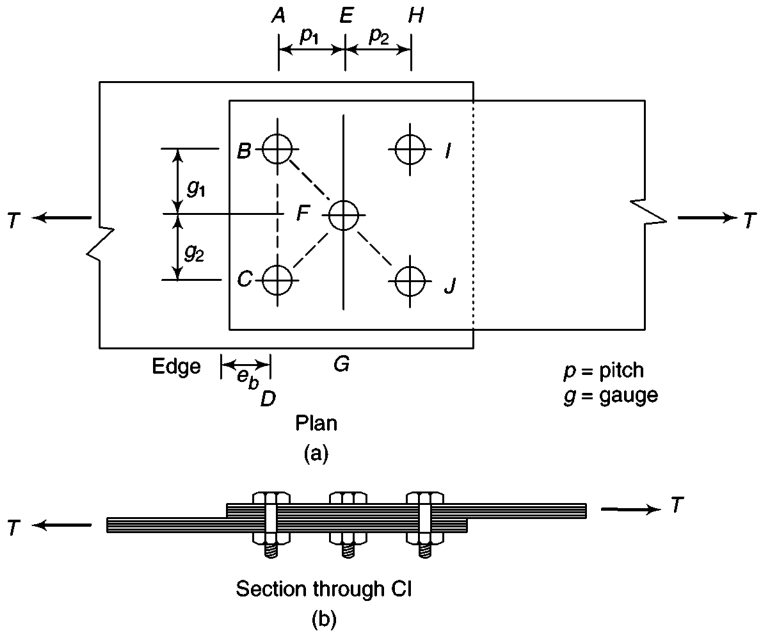


Fig. 3.7 Staggered bolt holes

3.6.1.3 Effect of bearing stress

When slip takes place between plates being joined by bolts, one or more fasteners come into bearing against the side of the hole. Consequently bearing stress is developed in the material adjacent to the hole and in the fastener. Initially this stress is concentrated at the point of contact. An increase in load causes local yielding and a larger area of contact resulting in a more uniform bearing stress distribution. The actual failure mode in bearing depends on the end distance, the bolt diameter, and the thickness of the connected material. Either the fastener splits out through the end of the plate because of the insufficient end distance or excessive deformations are developed in the material adjacent to the hole and the elongation of the hole takes place as shown in Fig. 3.8. Often a combination of these failure modes will occur.

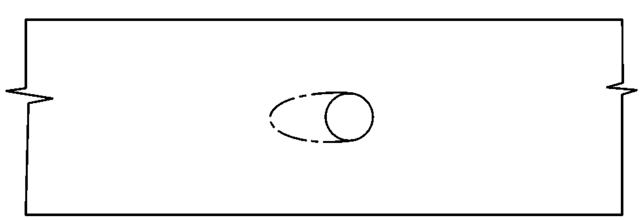


Fig. 3.8 Elongation of bolt hole due to local yielding under bearing stress

However, research (Munse & Chesson 1963) shows that as long as the bearing stress is less than 2.25 times the tensile stress, the effect of bearing stress can be neglected.

3.6.2 Effect of Shear Lag

The force is transferred to a tension member (angles, channels, or T-sections) by a gusset or the adjacent member connected to one of the legs either by bolting or welding. The force thus transferred to one leg by the end connection locally gets transferred as tensile stress over the entire cross section by shear. Hence, the tensile stress on the section from the first bolt hole up to the last bolt hole will not be uniform. The connected leg will have higher stresses at failure even of the order of ultimate stress while the outstanding leg stresses may be even below yield stress. However, at sections away from the end connection, the stress distribution becomes more uniform. (See Fig 3.9 and 3.11).

Let us consider an I-section as shown in Fig. 3.9(a), which is connected to the other members of a structure with two gusset plates attached to the flanges. This is the common connection found in bridge trusses as it is not practical to connect both webs and the flanges. It is obvious that the web is not fully effective in the region of the connection. As shown in Fig. 3.9(b), only a distance away from the connection, there will be uniform stress throughout the section. Because the internal transfer of forces from the flange region into the web region will be by shear (in the case of angles, transfer of force from one leg to the other will be by shear as indicated later in Fig. 3.11) and because one part 'lags' behind the other, the phenomenon is referred to as *shear lag*. The shear lag reduces the effectiveness of the component plates of a tension member that are not connected directly to a gusset plate.

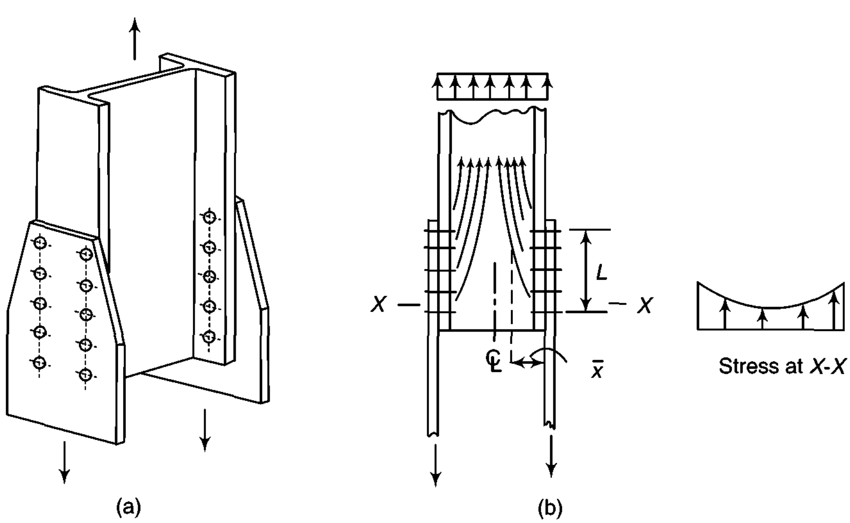


Fig. 3.9 Shear lag in tension member.

The shear lag effect reduces with increase in the connection length. In addition to its effect on shear lag, an increase in the connection length of a specimen also allows for a larger restoring moment at the eccentric connection. A longer connection length thus increases the net section efficiency.

The study conducted by Kulak & Wu (1997) revealed that the net section efficiency increases when the number of bolts in a line is increased up to four and after that there is no appreciable increase in the efficiency.

3.6.3 Geometry Factor

Tests on bolted joints show that the net section is more efficient if the ratio of the gauge length g to the diameter d is small (Kulak et al. 1987). The increase in the efficiency due to a smaller g/d ratio is due to the suppression of contraction at the net section.

To account for the effect of gauge or g/d ratio, Munse and Chesson (1963) proposed a geometry factor, K_3, given by Eqn (3.6), which is multiplied with the net section to account for this effect.

$$K_3 = 1.60 - 0.70(A_{ne}/A_g) \tag{3.6}$$

The value of K_3 generally varies in the range of 0.9 to 1.14.

3.6.4 Ductility Factor

Tension members with bolt holes made from ductile steels have proved to be as much as one-fifth to one-sixth times stronger than similar members made from less ductile steels having the same strengths (Kulak et al. 1987). To account for this effect, Munse and Chesson (1963) proposed a reduction factor

$$K_1 = 0.82 + 0.0032R_a \leq 1.0 \tag{3.7}$$

where R_a is the area reduction ratio before rupture. In case of commonly used structural steels exhibiting minimum prescribed ductility, the ductility factor k equals 1.0.

3.6.5 Spacing of Fasteners

The closer spacing of fasteners relative to their diameter may sometimes lead to block shear failure at the ends as discussed in Section 3.5.3, which has to be accounted for as a limit state.

3.7 Angles Under Tension

As mentioned earlier angles are used extensively as tension members in towers, trusses, and bracings. Angles, if axially loaded through centroid (as in the case of tower legs), could be designed as in the case of plates. However, in many cases, angles are connected to gusset plates (which in turn are connected to the other members of a structure) by welding or bolting only through one of the two legs (see Fig. 3.10). This kind of connection results in eccentric loading, causing non-uniform distribution of stress over the cross section (see Fig. 3.11). Further, since the load is applied by connecting only one leg of the member, there is a shear lag at the end connections.

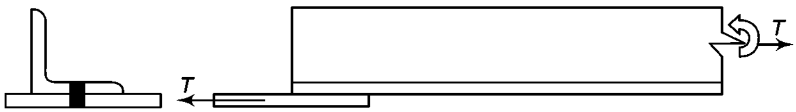


Fig. 3.10 Angles eccentrically loaded through gussets

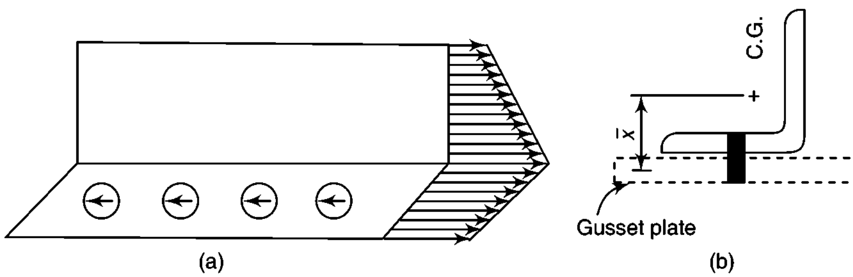


Fig. 3.11 Distribution of stresses in an angle

When the angles are connected to other angles through the centroid and when the holes are staggered on two legs of an angle (see Fig. 3.12), the gauge length g for use in Eqn (3.5) is obtained by developing the cross section into an equivalent flat plate [see Fig. 3.12(b)] by revolving about the centre lines of the component parts. The critical net section can then be established by the procedure described for plates. Thus referring to Fig. 3.12(b), the gauge distance g^* is obtained as

$$g^* = g_a - (t/2) + (g_b - t/2) = g_a + g_b - t \tag{3.8}$$

An illustration of the calculations involved is given in Example 3.3. Examples of net section calculations are also provided in Chapter 10.

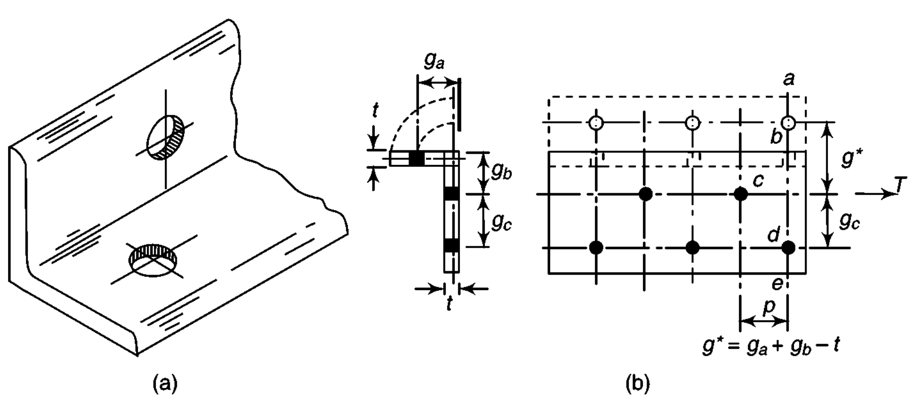


Fig. 3.12 Fasteners in more than one plane

3.7.1 Net Section Design

It was shown in Section 3.6 that the strength of a tension member with bolted or riveted connections can be predicted with good accuracy by taking into account the various factors affecting the strength of the net section. Hence, the following procedure has been suggested for the design of such members (Gaylord et al. 1992; Munse & Chesson 1963). To provide for necessary margin of safety against fracture, the capacity of the member should be determined by multiplying the effective net cross-sectional area with the specified minimum tensile strength f_u divided by the partial safety factor γ_{m1}. Thus, the effective net area is defined by (Gaylord et al. 1992).

$$A_{ne} = K_1 K_2 K_3 K_4 A_n \tag{3.8}$$

where A_n is the net area of the cross-section obtained by $p^2/4g$ rule [see Eqn (3.5)] and A_{ne} is the effective area of the cross-section modified by the other terms in the equation. These terms represent a ductility factor (k_1), a factor for the method of hole forming (k_2), a geometry factor reflecting hole spacing (k_3), and a shear lag factor (k_4).

Kulak & Wu (1997) conducted several tests on bolted angle tension members and studied the effects of various factors such as out-of-plane stiffness of gusset plates, angle thickness, connection by long leg and short leg, and connection length. They proposed the following net section strength formula based on these tests and the finite element analysis done by them (without the partial safety factor for material)

$$T_u = f_u A_{nc} + \beta f_y A_{go} \tag{3.9}$$

where A_{nc} is the net area of the connected leg at the critical cross section, A_{go} is the gross area of the outstanding leg and $\beta = 1.0$ for members with four or more fasteners per line in the connection, or $\beta = 0.5$ for members with two or three fasteners per line in the connection. Kulak & Wu compared the ultimate load predicted by Eqns (3.9) and found that it provides conservative results and falls in a narrower scatter band of results. They also proposed the following equation for single and double angles connected by only one of the legs and made of Fe 410 steel

$$T_u = U A_n f_u \tag{3.10}$$

where A_n is the net area of the critical cross section calculated using a hole diameter 2-mm greater than the nominal diameter of the bolt and using the $p^2/4g$ rule if staggered holes are present and $U = 0.80$ if the connection has four or more member of fasteners per line and 0.60 if there are two or three fasteners per line. They also concluded that Eqn (3.10) gives conservative results if it is applied to steel grades for which f_y/f_u is greater than 0.62. The above recommendations for angles have been adopted in ANSI/AISC code.

The research done by Kulak & Wu (1997) has also shown the following.

(a) The effect of the gusset plate thickness, and hence the out-of-plane stiffness of the end connection, on the ultimate tensile strength is not significant.
(b) The thickness of the angle has no significant influence on the member strength.

(c) When the length of the connection increases, the tensile strength increases up to four bolts and the effect of any further increase in the number of bolts, on the tensile strength of the member, is not significant.
(d) The net section efficiency is higher (7%–10%) when the long leg of the angle is connected, rather than the short leg.
(e) Because of local bending, each angle of a double angle member bends about the bolt line on each side of the gusset plate; thus the double angles seem to act individually rather than as a rigid unit.

3.7.2 Indian Code (IS 800 : 2007) Provisions for Angle Tension Members

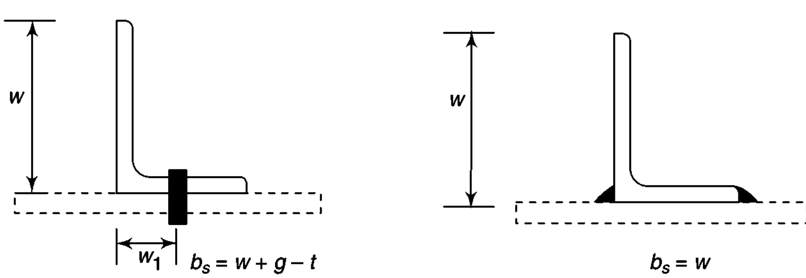
The net section strength of single and double angle tension members (either bolted or welded) and connected through one leg (including the shear lag effect) given in the code is based on the research done at IIT, Madras (Usha 2003; Usha & Kalyanaraman 2002). The design strength as governed by the tearing of the net section is given in clause 6.3.3 of the code. Substituting the values of $\gamma_{m1} = 1.25$ and $\gamma_{m0} = 1.1$, we get

$$T_{dn} = 0.72 f_u A_{nc} + \qquad A_{go} f_y \tag{3.11}$$

and

$$\beta = 1.4 - 0.076[(b_s/L_c)(w/t)(f_y/f_u)] \leq \; 0.88\, f_y/f_y \geq 0.7 \tag{3.12}$$

where f_u and f_y are the ultimate and yield stress of the material, w and t are the size and thickness of the outstanding leg, respectively, b_s is the shear distance from the edge of the outstanding leg to the nearest line of fasteners, measured along the centre line of the legs in the cross section (see Fig. 3.13), L_c is the length of the end connection measured from the center of the first bolt hole to the centre of the last bolt hole in the end connection or length of the weld along load direction and, A_{nc} is the net area of the connected leg at critical cross section, computed after deducting the diameter of hole (the diameter of the holes should be taken as 2-mm larger than the nominal size in the case of punched holes), and A_{go} is the gross area of the outstanding leg.



(a) Bolted connection (b) Welded connection

Fig. 3.13 Definition of b_s as per the code.

Alternatively, the IS code suggests the use of an equation, for preliminary design, similar to Eqn (3.10), with the partial factor of safety for material $\gamma_{m1} = 1.25$, we get

$$T_{dn} = \qquad A_n f_u \tag{3.13}$$

with $\alpha = 0.6$ for one or two bolts, 0.7 for three bolts, and 0.8 for four or more bolts in the end connections or equivalent weld length.

It is important to observe that in the case of welds, the determination of the value of α is difficult since the welds may be transverse, longitudinal, or combined. Designers have to use their judgement to arrive at an equivalent number of bolts.

3.8 Other Sections

The tearing strength T_{dn} of 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 (see Section 3.6.2). The code suggests that the design tensile strength of such sections, as governed by the tearing of the net section, may also be calculated by using Eqn (3.11) to (3.13), 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/weld line in the connected leg of the cross section. The net effective area of a single channel section connected through the web may be treated as for double angles connected by one leg each to the gusset. When a rolled or built-up channel or I-sections are connected through flanges, the web is found to be partially ineffective in resisting the tensile load. In such cases, the net area may be taken as the total area minus half the web area (Duggal 2000).

3.9 Tension Rods

A common and simple tension member is the threaded rod. Such rods are usually found as secondary members, where the required strength is small. Some examples of tension rods are as follows.

(a) Sag rods are used to help support purlins in industrial buildings. Various arrangements of sag rods are shown in Fig. 3.14. Sag rods reduce the bending moment about the minor axis (channels which are used as purlins are weak about the minor axis), resulting in economy. These rods are threaded at the ends and bolted to purlins. Since individual sag rods are placed between successive pairs of purlins, they may be designed individually, each to carry a tangential component from all the purlins below it. Thus, sag rods just below and suspended from the ridge purlin will be subjected to maximum force.
(b) Vertical ties to help support girts in the walls of industrial buildings [Fig. 3.15(a)].
(c) Hangers, such as tie rods supporting a balcony [Fig. 3.15(b)]. When providing such hangers, proper detailing should be adopted and communicated properly to the fabricator.
(d) Tie rods to resist the thrust of an arch.

As mentioned previously, tie rods are also often used with an initial tension as diagonal wind bracings in walls, roofs, and towers. The initial tension reduces the deflection and vibration and also increases the stiffness of these rods.

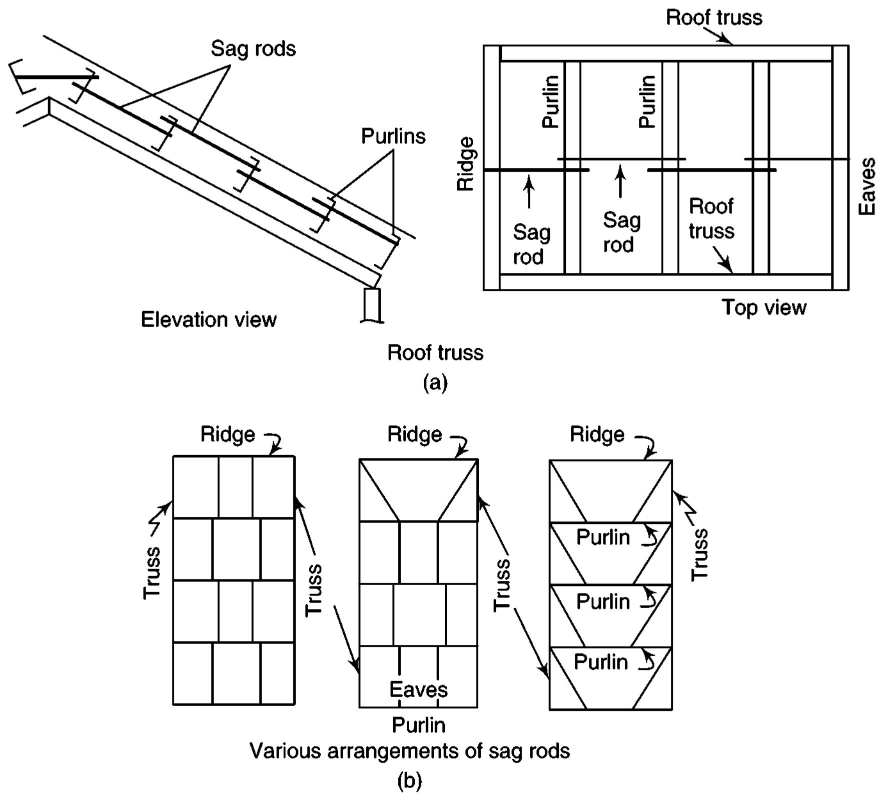


Fig. 3.14 Use of sag rods to support purlins

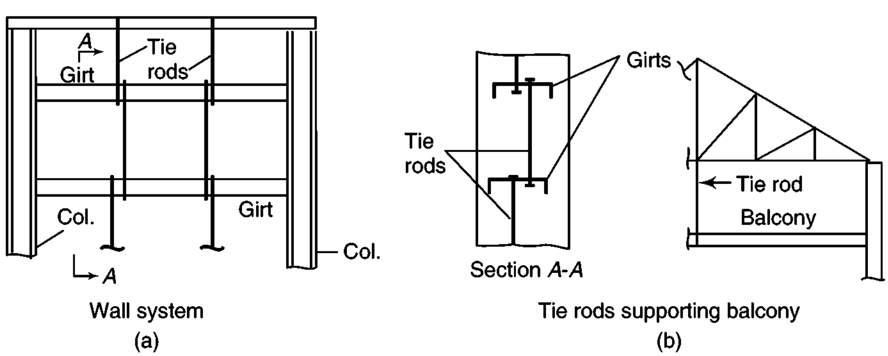


Fig. 3.15 Uses of tension rods

3.10 Design of a Tension Member

In the design of a tension member, based on the tensile force acting on the member, the designer has to arrive at the type and size of the member. The type of member is chosen based on the type of the structure and location of the member (e.g., double angles at the bottom chord or a rafter of roof trusses, angles or pipes for web members of roof trusses, etc.).

The design is iterative, involving a choice of a trial section and an analysis of its capacity. The various steps are as follows:

1. The net area required A_n to carry the design load T is obtained by the equation
$$A_n = T_u/(f_u/\gamma_{m1}) \tag{3.14}$$
2. From the required net area, the gross area may be computed by increasing the net area by about 25% to 40%. The required gross area may also be checked against that required from the yield strength of the gross section as follows
$$A_g = T_u/(f_y/\gamma_{m0}) \tag{3.15}$$
A suitable trail section may be chosen from the steel section tables (IS 808: 1989) to meet the required gross area.
3. The number of bolts or welding required for the connections is calculated. They are arranged in a suitable pattern and the net area of the chosen section is calculated. The design strength of the trial section is evaluated using Eqns (3.2) to (3.4) in the case of plates and threaded bars and additionally using Eqns (3.11) to (3.13) in the case of angles.
4. If the design strength is either small or too large compared to the design force, a new trial section is chosen and Step 3 is repeated until a satisfactory design is obtained.
5. The slenderness ratio of the member is checked as per Table 3.1.

3.11 Lug Angles

When a tension member is subjected to heavy load, the number of bolts or the length of weld required for making a connection with other members becomes large; resulting in uneconomical size of the gusset plates. In such situations, an additional short angle may be used to reduce the joint length and shear lag as shown in Fig. 3.16. Such an angle is called the lug angle. The location of the lug angle is of some importance; it is more effective at the beginning of the connection, as in Fig. 3.16, rather than at the end. The use of lug angles with angles or channels reduces the net area of the main members due to the additional bolt holes in projected members. This reduction in the net area of the member should not be excessive. In the connections of the lug angles to the member or the gusset plate more than two bolts are used. Since both legs of the angles or channels are connected to the lug angles, the net area of the members should be calculated simply as gross area minus the area of the holes.

Lug angles may be eliminated by providing unequal angle sections with the wider leg as the connected leg and using two rows of staggered bolts. In many cases, the cost of providing the lug angles (including their connection and the extra fabrication required to make the holes) may be found to be expensive than providing extra length and thickness of gusset plate. Hence, they are not used in practice.

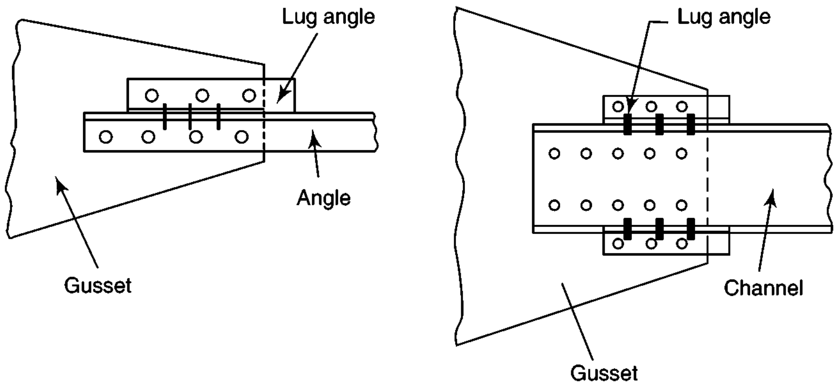


Fig. 3.16 Lug angles

In the case of angle members, the lug angles and their connection to the gusset or other supporting member should be capable of developing a strength of not less than 20% in excess of the force in the outstanding leg of the angle, and the connection of the lug angle to the angle member should be capable of developing 40% in excess of the force (clause 10.12.2).

In channel members, however, the lug angles and their connections to the gusset or other supporting member should be capable of developing a strength of not less than 10% in excess of the force in the flange of the channel and the attachment of the lug angle to the member should have a strength not less than 20% in excess of that force (clause 10.12.3).

3.12 Splices

When the available length is less than the required length of a tension member, splices are provided. The various types of splices that can be provided are shown in Figs 3.17(a) to (c). If the sections are not of the same thickness, packings are introduced, as shown in Fig. 3.17(d). Moreover, reduction in capacity of bolt has to be considered for long joints or if the packing thickness is greater than 6 mm (see Chapter 10 for these calculations).

In the design of a tension splice, the effect of eccentricity is neglected; as far as possible it should be avoided. Thus Fig. 3.17(e) shows an angle section spliced on one leg of the angle only by a plate. Such an arrangement causes eccentricity and introduces bending moments. To overcome this, both the legs of the angle should be spliced, as shown in Fig. 3.17(a). The splice as shown in Fig. 3.17(b) is used in the legs of transmission line or communication towers and aids transfer of tensile loads, without any eccentricity.

The splice cover plates or angles and its connections should be designed to develop the net tensile strength of the main member. The forces in the main member are transferred to the cover plate angle sections through the bolts/welding and carried through these covers across the joint and is transferred to the other portion of the section through the fasteners. For examples of tension splices, see Chapter 10.

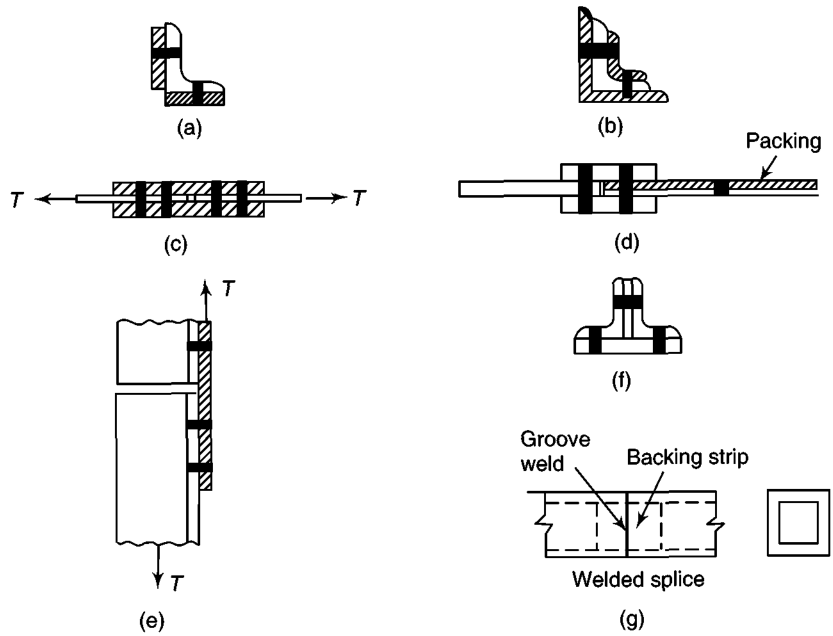


Fig. 3.17 Splices in tension members

Hyatt Regency Walkway Collapse

The 40-story Hyatt Regency Hotel in Kansa City, Missouri, USA, was opened on 1 July 1980. The lobby of the hotel featured a multistory atrium, which had suspended concrete walkways on the second, third, and fourth levels. These three separate pedestrian walkways connected the north and south buildings. The fourth- and second-floor walkways hung one above the other and the third-floor walkway hung offset to one side (see Fig. CS1). These walkways all connected to steel trusses that hung from the atrium ceiling.

The two walkways were suspended from a set of steel tension rods of 32 mm, with the second-floor walkway hanging directly underneath the fourth-floor walkway. The walkway platform was supported on three cross-beams suspended by steel rods retained by nuts. The cross-beams were box beams made from C-channels welded toe-to-toe. The original design called for three pairs of rods running from the second floor all the way to the ceiling.

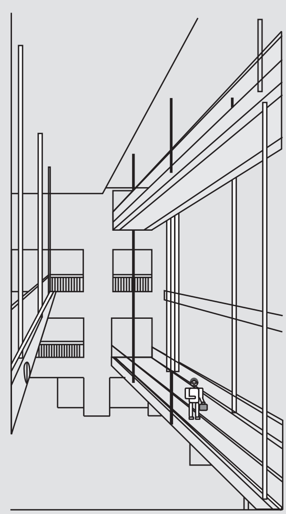


Fig. CS1

On 17 July 1981, when a party was going on, the fourth-floor walkway failed and fell on the lower walkway, both walkways crashing onto the floor three stories below, killing 114 people and injuring 185. The third-floor walkway was not involved in the collapse.

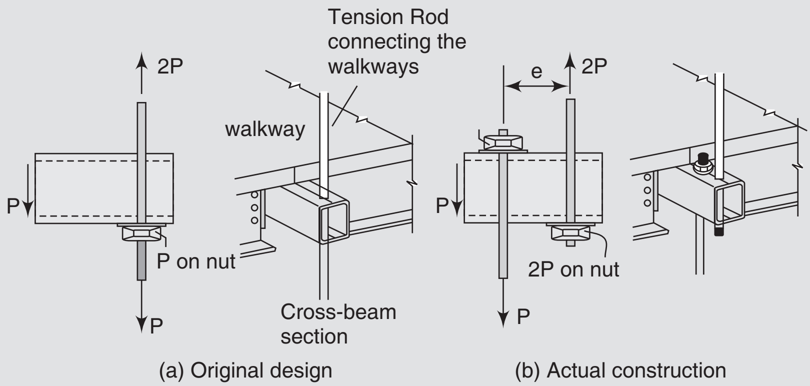


Fig. CS2 Difference between the design and construction of the walkway support system

The cause of the failure is that the contractor replaced the one vertical suspension rod specified by the original designer, by two shorter rods; one from the upper support to the first walkway, and another from the bottom beam of the first walkway down to the second walkway (see Figure CS2(b)). Now the nut and washer under the upper rod is subjected to double the design load (in addition the eccentricity created a local bending moment), which led to the failure. Photographs of the wreckage showed excessive deformations of the cross-section; the box beams split at the weld, and the nut supporting them slipped through. Lack of proper communication and overlooking the details were cited as the main problems for the faulty connection detail; the connection that failed was never shown on any drawings, and it was not designed.

References:

1. http://en.wikipedia.org/wiki/Hyatt_Regency_walkway_collapse
2. http://ethics.tamu.edu/ethics/hyatt/hyatt2.htm
3. http://failurebydesign.info/

3.13 Gussets

A gusset plate is a plate provided at the ends of tension members through which the forces are transferred to the main member. Gusset plates may be used to join more than one member at a joint. The lines of action of truss members meeting at a joint should coincide. If they do not coincide, secondary bending moments and stresses are created, which should be considered in the design.

The size and shape of the gusset plates are decided based on the direction of various members meeting at a joint. The plate outlines are fixed so as to meet the minimum edge distances specified for the bolts that are used to connect the various members at a particular joint. The shape of the gusset plate should be such that it should give an aesthetic appearance, in addition to meeting the edge distances of bolts, as mentioned earlier.

It is tedious to analyse the gusset plate for shear stresses, direct stresses, and bending stresses, and hence empirical methods have been used in the past to arrive at the thickness of the gusset plate (e.g., Whitmore method). More details of these methods are discussed in Chapter 10. The block shear model (see Section 3.5.3) could also be used to find the thickness of the gusset plate. It is a usual practice to provide the thickness of the gusset plate equal to or slightly higher than the thickness of members that are connected by the gusset plate. It is interesting to note that the failure of the I-35W bridge at Minnesapolis, USA in August 2007, was due to the inadequate thickness of the gusset plate.

Examples

Example 3.1 *What is the net area A_n for the tension member shown in Fig. 3.18, in case of (a) drilled holes, (b) punched holes?*

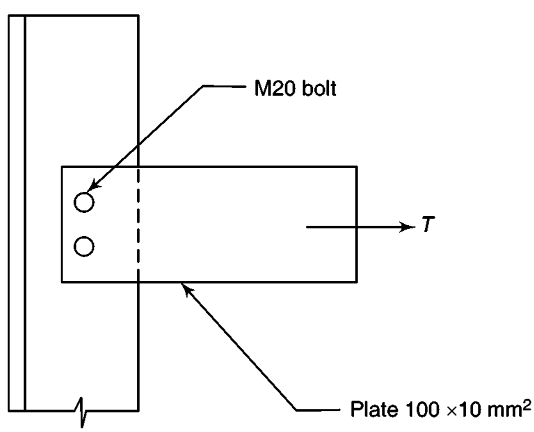


Fig. 3.18

Solution

$$A_g = 100 \times 10 = 1000 \text{ mm}^2$$

(a) Hole made by drilling

Hole for M20 bolt = 22 mm

$$A_n = A_g - n \text{ (hole} \times \text{thickness of plate)}$$
$$= 1000 - 2\,(22 \times 10) = 560 \text{ mm}^2$$

(b) Holes made by punching

$$\text{Hole} = 22 + 2 = 24 \text{ mm}$$

$$A_n = 1000 - 2(24 \times 10) = 520 \text{ mm}^2$$

Example 3.2 *Determine the minimum net area of the plates as shown in Figs. 3.19(a) and (b) with a plate of size of 210 × 8 mm and 16-mm bolts.*

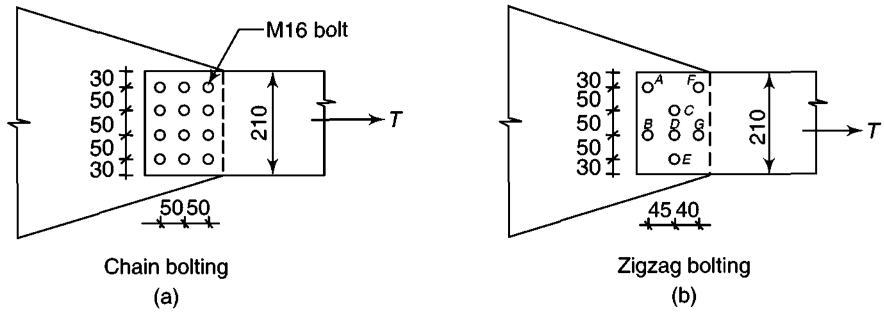


Fig. 3.19

Solution

(a) Chain bolting

For a 16-mm bolt, hole diameter = 18 mm

Net area = $(b - nd)t$

$= (210 - 4 \times 18) \times 8$

$= 1104 \text{ mm}^2$

(b) Zigzag bolting

Staggered length correction = $p_i^2/4g_i$

Path AB and FG (two holes):

Net area = $(210 - 2 \times 18) \times 8 = 1392 \text{ mm}^2$

Path CDE (three holes):

Net area = $(210 - 3 \times 18) \times 8 = 1248 \text{ mm}^2$

Path ACDE (four holes and one stagger):

Net area = $[210 - 4 \times 18 + 45^2/(4 \times 50)]8 = 1185 \text{ mm}^2$

Path FCDE (four holes and one stagger):

Net area = $[210 - 4 \times 18 + 40^2/(4 \times 50)]8 = 1168 \text{ mm}^2$

Path ACG or FCB (three holes and two staggers):

Net area = $[210 - 3 \times 18 + 45^2/(4 \times 50) + 40^2/(4 \times 50)]8 = 1393 \text{ mm}^2$

Path FCG (three holes and two staggers):

Net area = $210 - 3 \times 18 + 2 \times 40^2/(4 \times 50)]8 = 1376 \text{ mm}^2$

The minimum net area is for path FCDE = 1168 mm^2. Note that the minimum net area occurs at a path which has the maximum number of holes and minimum number of staggers.

Example 3.3 *Determine the net area A_n for the 200 × 150 × 10 angle with M20 bolt holes as shown in Fig. 3.20*

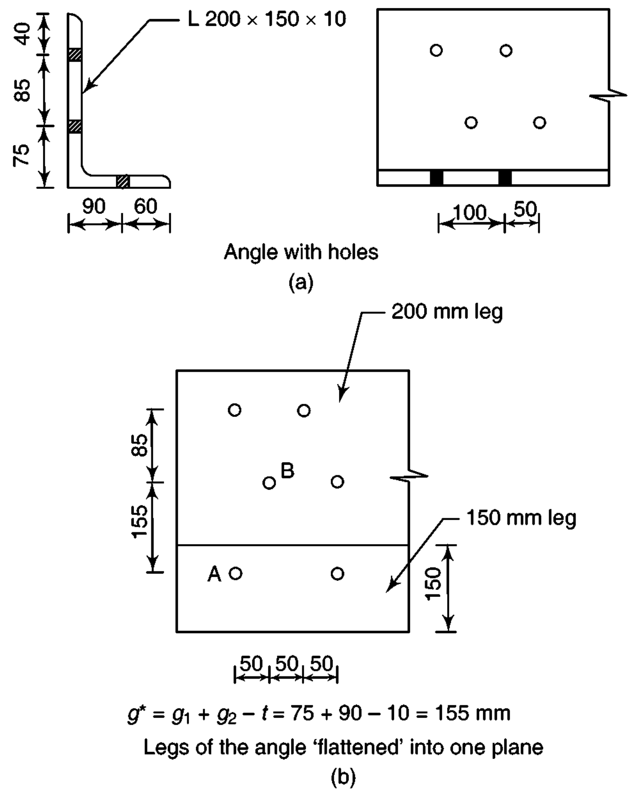


Fig. 3.20 Effective net area of angle

Solution

For an M20 bolt, $d_h = 22$ mm

For net area calculation, the angle may be visualized as being flattened into a plate as shown in Fig. 3.20(b).

$$g^* = g_1 + g_2 - t = 75 + 90 - 10 = 155 \text{ mm}$$

$$A_n = A_g - \Sigma d_h t + \Sigma(p^2/4g)t$$

Gross area of angle = 3430 mm^2

Path AC:

$$\text{Net area} = 3430 - 2 \quad 22 \quad 10 = 2990 \text{ mm}^2$$

Path ABC:

$$\text{Net area} = 3430 - 3 \quad 22 \quad 10 + [50^2/(4 \quad 85) + 50^2/(4 \quad 155)] \quad 10$$

$$= 2883.85 \text{ mm}^2$$

Since the smallest net area is 2883.85 mm^2 for path ABC, therefore, that value governs.

Example 3.4 *Determine the design tensile strength of plate (200 × 8 mm) connected to 10-mm thick gusset using 20 mm bolts as shown in Fig. 3.21, if the yield and the ultimate stress of the steel used are 250 MPa and 410 MPa, respectively.*

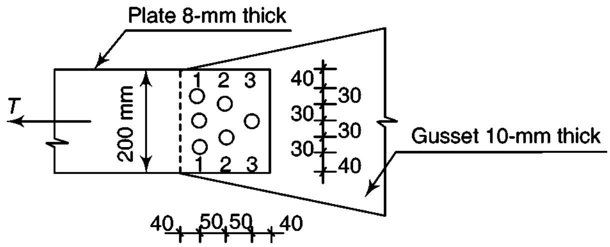


Fig. 3.21

Solution

$f_y = 250$ MPa

$f_u = 410$ MPa

Calculation of net area

A_n (Section 11) = $(200 - 3 \times 22) \times 8 = 1072$ mm^2

A_n (Section 1221) = $[(200 - 4 \quad 22) + (2 \quad 50^2)/(4 \quad 30)] \quad 8 = 1229.3$ mm^2

A_n (Section 12321) = $[(200 - 5 \times 22) + (4 \times 50^2)/(4 \times 30)] \times 8 = 1386.6$ mm^2

Strength of member in tension is given by

(i) Yielding of gross-section

$$T_{dg} = [f_y \times A_g/\gamma_{m0}]$$
$$= [250 \times (200 \times 8)/1.10] \times 10^{-3} = 363.64 \text{ kN}$$

(ii) Rupture of net section

$$T_{dn} = (0.9 \times f_u \times A_n/\gamma_{ml})$$
$$= (0.9 \times 410 \times 1072/1.25) \times 10^{-3} = 316.45 \text{ kN}$$

Therefore, the design tensile strength of the plate = 316.45 kN

Check for minimum edge distance

Provided edge and end distance = 40 mm > 1.5 × 20 = 30 mm

Hence, the edge distance is as required.

Example 3.5 *A single unequal angle 100 × 75 × 6 is connected to a 10-mm thick gusset plate at the ends with six 16-mm-diameter bolts to transfer tension as shown in Fig. 3.22. Determine the design tensile strength of the angle assuming that the yield and the ultimate stress of steel used are 250 MPa and 410 MPa:*

(i) if the gusset is connected to the 100-mm leg

(ii) if the gusset is connected to the 75-mm leg

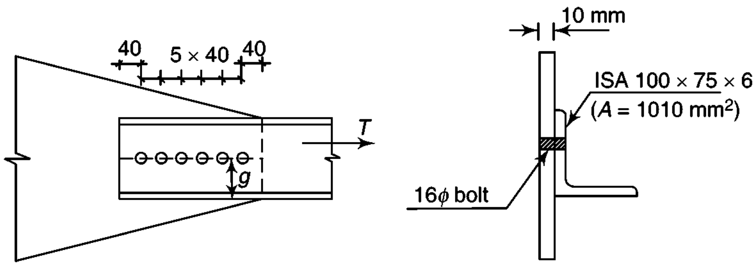
Solution

(i) *Gusset is connected to the 100-mm leg of the angle*

$A_{nc} = (100 - 6/2 - 18) \times 6 = 474$ mm^2

$A_{go} = (75 - 6/2) \times 6 = 432$ mm^2

$A_g = 1010$ mm^2



g = 40 mm if 75 mm leg is connected
= 60 mm if 100 mm leg is connected

Fig. 3.22

(a) Strength governed by yielding of gross section

$$T_{dg} = A_g f_y / \gamma_{mo} = (1010 \times 250/1.10) \times 10^{-3} = 229.55 \text{ kN}$$

(b) Strength governed by rupture of critical section

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

$$\beta = 1.4 - 0.076(w/t)(f_y/f_u)(b_s/L_c)$$

$$= 1.4 - 0.076[(75 - 3)/6](250/410)[(72 + 60)/(5 \times 40)]$$

$$= 1.4 - 0.367 = 1.033 > 0.7 \text{ and } < 1.44[(410/250)(1.1/1.25)]$$

$$T_{dn} = [0.9 \times 410 \times 474/1.25 + 1.033 \times 432 \times 250/1.10] \times 10^{-3}$$

$$= 139.92 + 101.42 = 241.34 \text{ kN}$$

Alternatively,

$$T_{dn} = \alpha A_n f_u / \gamma_{m1} = 0.8 \times [(474 + 432) \times 410/1.25] \times 10^{-3} = 237.73 \text{ kN}$$

Hence, take

$$T_{dn} = 241.34 \text{ kN}$$

(c) Strength governed by block shear

$$A_{vg} = 6 \times (5 \times 40 + 40) = 1440 \text{ mm}^2$$

$$A_{vn} = 6 \quad [(5 \quad 40 + 40) - 5.5 \quad 18] = 846 \text{ mm}^2$$

$$A_{tg} = 6 \quad 40 = 240 \text{ mm}^2$$

$$A_{tn} = 6 \quad (40 - 0.5 \quad 18) = 186 \text{ mm}^2$$

$$T_{db1} = A_{vg} f_y / (\sqrt{3} \quad _{m0}) + 0.9 f_u A_{tn} / \quad _{m1}$$

$$= [1440 \times 250/(\sqrt{3} \times 1.1) + 0.9 \times 410 \times 186/1.25] \times 10^{-3} = 243.85 \text{ kN}$$

$$T_{db2} = 0.9 f_u A_{vn} / (\sqrt{3} \gamma_{m1}) + f_y A_{tg} / \gamma_{m0}$$

$$= [0.9 \quad 410 \quad 846/(\sqrt{3} \quad 1.25) + 250 \quad 240/1.10] \quad 10^{-3} = 198.73 \text{ kN}$$

Hence,

$$T_{db} = 198.73 \text{ kN}$$

Thus, the design tensile strength of the angle = 198.73 kN (least of 198.73, 229.55, and 241.34).

The efficiency of the tension member $= 198.73 \times 1000 \times 100/(1010 \times 250/1.10)$
$= 86.57\%$

(ii) *Gusset is connected to the 75-mm leg of the angle*

$$A_{nc} = (75 - 6/2 - 18) \times 6 = 324 \text{ mm}^2$$

$$A_{go} = (100 - 6/2) \times 6 = 582 \text{ mm}^2$$

$$A_g = 1010 \text{ mm}^2$$

(a) Strength as governed by yielding of gross-section

$$T_{dg} = A_g f_y/\gamma_{m0} = 229.55 \text{ kN}$$

(b) Strength governed by tearing of net section

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

$$\beta = 1.4 - 0.076(w/t)(f_y/f_u)(b_s/L_c)$$

$$= 1.4 - 0.076[(100 - 3)/6](250/410)[(97 + 40)/(5 \times 40)]$$

$$= 0.8868 > 0.7$$

$$T_{dn} = [0.9 \times 410 \times 324/1.25 + 0.8868 \times 582 \times 250/1.1] \times 10^{-3}$$

$$= 212.94 \text{ kN}$$

Alternatively,

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

$$= [0.8 \times (324 + 582) \times 410/1.25] \times 10^{-3}$$

$$= 237.73 \text{ kN}$$

Hence, take $T_{dn} = 212.94$ kN

(c) Strength governed by block shear

$$A_{vg} = 6 \times (5 \times 40 + 40) = 1440 \text{ mm}^2$$

$$A_{vn} = 6 \times (5 \times 40 + 40 - 5.5 \times 18) = 846 \text{ mm}^2$$

$$A_{tg} = 6 \times 35 = 210 \text{ mm}^2$$

$$A_{tn} = 6 \times (35 - 0.5 \times 18) = 156 \text{ mm}^2$$

$$T_{db1} = A_{vg}\, f_y/(\sqrt{3}\,\gamma_{m0}) + 0.9\, f_u A_{tn}/\gamma_{m1}$$

$$= [1440 \times 250/(\sqrt{3} \times 1.1) + 0.9 \times 410 \times 156/1.25] \times 10^{-3}$$

$$= 235 \text{ kN}$$

$$T_{db2} = 0.9\, f_u A_{vn}/(\sqrt{3}\,\gamma_{m1}) + f_y A_{tg}/\gamma_{m0}$$

$$= [0.9 \times 410 \times 846/(\sqrt{3} \times 1.25) + 250 \times 210/1.10] \times 10^{-3}$$

$$= 191.91 \text{ kN}$$

Hence,

$$T_{db} = 191.91 \text{ kN}$$

Thus, the design tensile strength of the angle = 191.91 kN (least of 229.55, 212.94 and, 191.91)

The efficiency of the tension member = 191.91 × 1000 × 100/(1010 × 250/1.10)
= 83.6%

Hence, in this case, by connecting the short leg, the efficiency is reduced by about 3%. Note that as the outstanding leg increases, gross net area increases and hence block shear may govern.

Example 3.6 *Determine the tensile strength of a roof truss diagonal 100 × 75 × 6 mm (f_y = 250 MPa) connected to the gusset plate by 4-mm welds as shown in Fig. 3.23.*

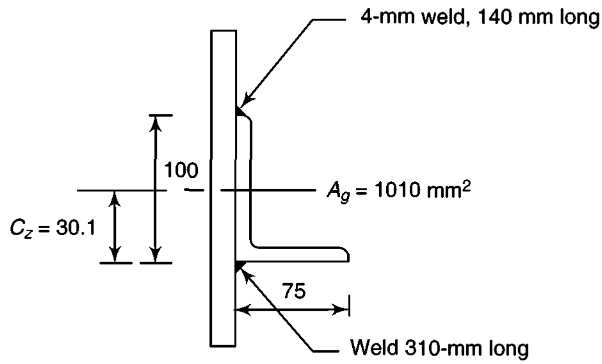


Fig. 3.23

Solution

Area of the connected leg = (100 – 6/2) × 6 = 582 mm²

Area of the outstanding leg = (75 – 6/2) × 6 = 432 mm²

$$A_g = 1010 \text{ mm}^2$$

(a) Strength governed by yielding of cross section

$$T_{dg} = A_g f_y/\gamma_{mo} = (1010 \times 250/1.10) \times 10^{-3} = 229.55 \text{ kN}$$

(b) Strength governed by rupture of critical section

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

Assuming average length of weld L_w = 225 mm

$$\beta = 1.4 - 0.076(w/t)(f_y/f_u)(b_s/L_w)$$
$$= 1.4 - 0.076[(75 - 3)/6](250/410)(75/225)$$
$$= 1.215$$

Hence,

$$T_{dn} = [0.9 \times 410 \times 582/1.25 + 1.215 \times 432 \times 250/1.10] \times 10^{-3}$$
$$= 291.1 \text{ kN}$$

Hence, T_{dn} = 291.1 kN

(c) Strength governed by block shear

Since the member is welded to the gusset plate, no net areas are involved and hence A_{vn} and A_{tn} in the equation for T_{db} (Section 6.3.1 of the code) should be taken as the corresponding gross areas (Gaylord et al. 1992). Assuming average length of the weld on each side as 225 mm and the gusset plate thickness as 8 mm,

$$T_{db1} = [8 \times (225 \times 2) \times 250/(\sqrt{3} \times 1.1) + 0.9 \times 410 \times 8 \times 100/1.25] \times 10^{-3}$$
$$= 708.53 \text{ kN}$$

$$T_{db2} = [0.9 \times 410 \times 8 \times 225 \times 2/(\sqrt{3} \times 1.25) + 250 \times 8 \times 100/1.1] \times 10^{-3}$$
$$= 798.38 \text{ kN}$$

Hence,

$$T_{db} = 708.53 \text{ kN}$$

Thus, tensile strength = 229.55 kN (least of 229.55, 291.1, and 708.53)

The efficiency of the tension member = 229.55 × 1000 × 10/(1010 × 250/1.10)
= 100%

It is clear that since there is no reduction in the area in the welded connection, the efficiency of the tension member is not reduced.

Note that in the calculation, we have assumed the average length of weld as 225 mm on each side. However, the welding should be proportioned based on the position of the neutral axis.

Thus, for the tensile capacity = 229.55 kN, with capacity of 4-mm weld = 0.530 kN/mm

Length of the weld at the upper side of the angle

$$= (229.55 \times 30.1/100)/0.530 = 130 \text{ mm, say } 140 \text{ mm}$$

Length of the weld at the bottom side of the angle

$$= [229.55 \times (100 - 30.1)/100]/0.530 = 302 \text{ mm, say } 310 \text{ mm}$$

Example 3.7 *Select a suitable angle section to carry a factored tensile force of 290 kN assuming a single row of M24 bolts and assuming design strength as $f_y = 250$ N/mm^2*

Solution

Approximate required area = $1.1 \times 290 \times 10^3/250 = 1276$ mm^2

Choose 90 × 90 × 8 angle with $A = 1380$ mm^2

Strength governed by yielding = $[1380 \times 250/1.1] \times 10^{-3} = 313.64$ kN

A_{nc} = area of connected leg = $(90 - 4 - 22) \times 8 = 512$ mm^2

$A_{go} = (90 - 4) \times 8 = 688$ mm^2

Required number of M24 bolts (Appendix D) = 313.64/65.3 = 4.8

Provide five bolts at a pitch of 60 mm

Strength governed by rupture of critical section

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

$$\beta = 1.4 - 0.076(w/t)(f_y/f_u)\ (b_s/L_c)$$
$$= 1.4 - 0.076(90/8)\ (250/410)\ (82 + 50)/(4 \times 60)$$
$$= 1.113 < 1.44 \text{ and } > 0.7$$
$$T_{dn} = [0.9 \times 410 \times 512/1.25 + 1.113 \times 688 \times 250/1.10] \times 10^{-3}$$
$$= 325.18 \text{ kN}$$

Strength governed by block shear

Assuming an edge distance of 40 mm,

$$A_{vg} = 8 \times (4 \times 60 + 40) = 2240 \text{ mm}^2$$
$$A_{vn} = 8 \times (4 \times 60 + 40 - 4.5 \times 26) = 1304 \text{ mm}^2$$
$$A_{tg} = 8 \times 40 = 320 \text{ mm}^2$$
$$A_{tn} = 8 \times (40 - 0.5 \times 26) = 216 \text{ mm}^2$$
$$T_{db1} = A_{vg} f_y /(\sqrt{3}\gamma_{m0}) + 0.9 f_u A_{tn}/\gamma_{m1}$$
$$= [2240 \times 250/(\sqrt{3} \times 1.1) + 0.9 \times 410 \times 216/1.25] \times 10^{-3} = 357.68 \text{ kN}$$
$$T_{db2} = 0.9 f_u A_{vn}/(\sqrt{3}\ \gamma_{m1}) + f_y A_{tg}/\gamma_{m0}$$
$$= [0.9 \times 410 \times 1304/(\sqrt{3} \times 1.25) + 250 \times 320/1.10] \times 10^{-3} = 294.97 \text{ kN}$$

Tension capacity of the angle = 294.97 kN > 290 kN

Hence the angle is safe.

Exampe 3.8 *A tie member in a bracing system consists of two angles 75 × 75 × 6 bolted to a 10-mm gusset, one on each side using a single row of bolts [See Fig. 3.24(a)] and tack bolted. Determine the tensile capacity of the member and the number of bolts required to develop full capacity of the member. What will be the capacity if the angles are connected on the same side of the gusset plate and tack bolted [Fig. 3.24(b)]? What is the effect on tensile strength if the members are not tack bolted?*

Solution

(a) Two angles connected to opposite side of the gusset as in Fig. 3.24(a)

(i) Design strength due to yielding of gross section $T_{dg} = f_y(A_g/\gamma_{m0})$

$$A_g = 866 \text{ mm}^2 \text{ (for a single angle)}$$
$$T_{dg} = 250 \times 2 \times 866/1.10 \times 10^{-3}$$
$$T_{dg} = 393.64 \text{ kN}$$

(ii) The design strength governed by tearing at net section

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

Assume a single line of four numbers of 20-mm-diameter bolts ($\alpha = 0.8$)

$$A_n = [(75 - 6/2 - 22)6 + (75 - 6/2)6]2$$
$$A_n = (300 + 432)2 = 1464 \text{ mm}^2$$
$$T_{dn} = (0.8 \times 1454 \times 410/1.25) = 384.15 \text{ kN}$$

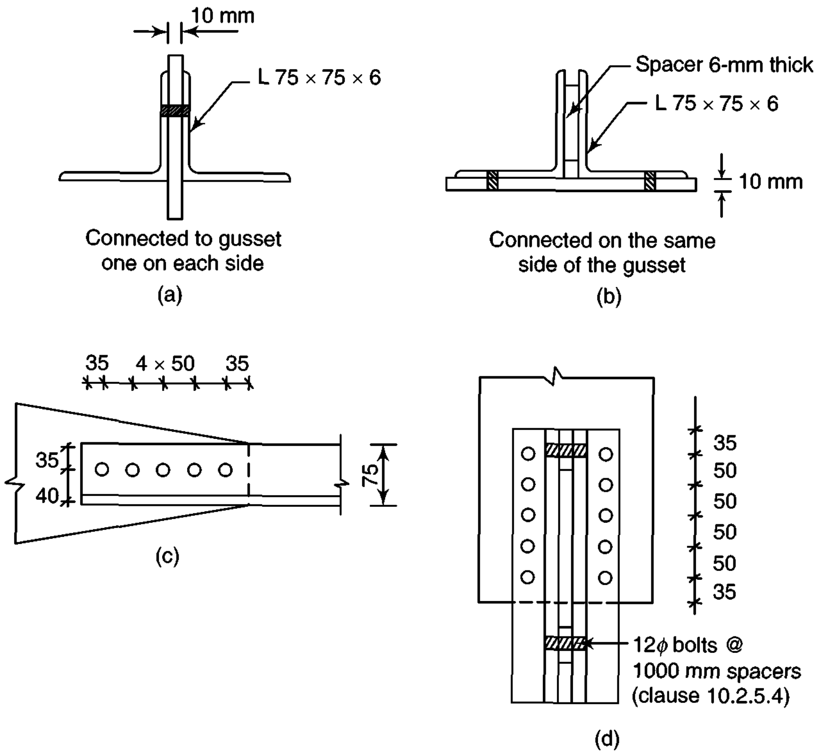


Fig. 3.24

Therefore,

Tensile capacity = 384.15 kN

Design of bolts

Choose edge distance = 35 mm

Capacity of bolt in double shear (Appendix D)

$$= 2 \quad 45.3 = 90.6 \text{ kN}$$

Bearing capacity of the bolt does not govern.

Hence,

Strength of a single bolt = 90.6 kN

Provide five bolts. Then,

Total strength of the bolts = 5 × 90.6 = 453 kN > 384.15 kN

Hence the connection is safe.

Minimum spacing = $2.5t = 2.5 \times 20 = 50$ mm

Hence, provide a spacing of 50 mm.

The arrangements of bolts are shown in Fig. 3.24(c).

Check for block shear strength: (clause 6.4)

Block shear strength T_{db} of connection will be taken as

$$T_{db1} = [A_{vg} f_y / \sqrt{3}\gamma_{m0}) + (0.9 A_{tn} f_u / \gamma_{m1})]$$

or

$$T_{db2} = 0.9 f_u A_{vn}/\sqrt{3}\gamma_{m1} + (f_y A_{tg}/\gamma_{m0})$$

whichever is smaller.

$$A_{vg} = (4 \times 50 + 35)6 = 1410 \text{ mm}^2$$

$$A_{vn} = (4 \times 50 + 35 - 4.5 \times 22)6 = 816 \text{ mm}^2$$

$$A_{tn} = (35.0 - 22/2)6 = 144 \text{ mm}^2$$

$$A_{tg} = (35 \times 6) = 210 \text{ mm}^2$$

$$T_{db1} = \{[(1410 \times 250)/(\sqrt{3} \times 1.10)] + [(0.9 \times 144 \times 410)/1.25]\} \times 10^{-3}$$
$$= 227.5 \text{ kN}$$

$$T_{db2} = \{[(0.9 \times 410 \times 816)/(\sqrt{3} \times 1.25)] + [(250 \times 210)/1.10]\} \times 10^{-3}$$
$$= 186.8 \text{ kN}$$

For double angle,
block shear strength = 2 × 186.8 = 373.6 kN
Therefore,

Tensile capacity = 373.6 kN (least of 393.64 kN, 384.14 kN, and 373.6 kN)

(b) Two angles connected to the same side of the gusset plate [Fig. 3.24(b)]

(i) Design strength due to yielding of the gross section = 393.64 kN
(ii) Design strength governed by tearing at the net section = 384.14 kN
Assuming ten bolts of 20 mm diameter, five bolts in each connected leg
Capacity of an M20 bolt in single shear = 45.3 kN
Total strength of bolts = 10 × 45.3 = 453 kN > 393.64 kN
Hence the connection is safe.

The arrangement of bolts is shown in Fig. 3.24(d). Since it is similar to the arrangement in Fig. 3.24(c), the block shear strength will be the same, i.e., 373.6 kN.

Hence, the tensile capacity = 373.6 kN

The tensile capacity of both the arrangements (angles connected on the same side and connected to the opposite side of gusset) are same, as per the code though the load application is eccentric in this case. Moreover, the number of bolts are ten whereas in case (a) we used only five bolts since the bolts were in double shear.

(c) If the angles are not tack bolted, they behave as single angles connected to gusset plate.

In this case also the tensile capacity will be the same and we have to use ten M20 bolts. This fact is confirmed by the test and FEM results of Usha (2003).

Example 3.9 *Design a single angle to carry 350 kN. Assume that the length of the member is 3 m and f_y = 250 KPa.*

Solution

Required area = 1.1   350   1000/250 = 1540 mm^2

Let us choose an unequal angle of size 150 × 75 × 8 mm with a weight of 13.7 kg/m and area = 1750 mm^2, r_{vv} = 16.2 mm.

(i) Design strength due to yielding of cross-section

$T_{dg} = f_y A_g / \gamma_{m0} = 250 \times 1750/1.10 = 397.7$ kN > 350 kN

(ii) Design strength governed by tearing of net section

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

Assuming nine M20 bolts, with strength = 9   45.3 = 407.7kN

$A_n = (150 - 4 - 2 \times 22)\ 8 + (75 - 4)8 = 1384\ \text{mm}^2$

$T_{dn} = (0.8 \times 1384 \times 410/1.25) \times 10^{-3} = 363.1$ kN > 350 kN

(iii) Assuming a staggered bolting and block shear failure as shown in Fig. 3.25

$A_{vg} = (3 \times 50 + 25 + 35) \times 8 = 1680\ \text{mm}^2$

$A_{vn} = (4 \times 50 + 25 + 35 - 3.5 \times 22)\ 8 = 1064\ \text{mm}^2$

$A_{tn} = (65 + 30 + 25^2/(4 \times 65) - 1.5 \times 22)\ 8 = 515\ \text{mm}^2$

$A_{tg} = (65 + 30 + 25^2/(4 \times 65)) \times 8 = 779\ \text{mm}^2$

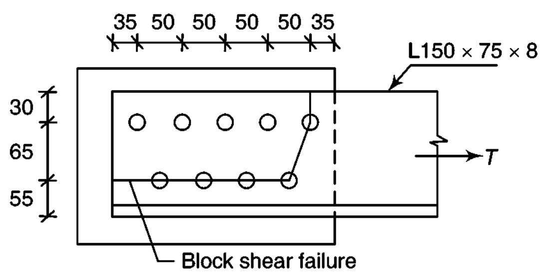


Fig. 3.25

Block shear capacity

$T_{db1} = A_{vg} f_y/(\sqrt{3}\gamma_{m0}) + 0.9A_{tn} f_u/\gamma_{m1}$

$= [1680 \times 250/\sqrt{3} \times 1.1) + 0.9 \times 515 \times 410/1.25] \times 10^{-3} = 372.47$ kN

$T_{db2} = 0.9A_{vn} f_u/\sqrt{3}\gamma_{m1} + A_{tg} f_y/\gamma_{m0}$

$= [0.9 \times 1064 \times 410/(\sqrt{3} \times 1.25) + (779 \times 250/1.1) \times 10^{-3} = 398.46$ kN

Hence $T_{db} = 363.1$ kN > 350 kN

Check for stiffness (Table 3 of code)

$L/r = 3000/16.2 = 185 < 250$

Hence the section is safe.

It is seen from Examples 3.8 and 3.9 that unequal angle section with its long leg connected has a higher load carrying capacity than two equal angles of the same weight connected on the same side or opposite side of gusset plate. Hence wherever possible, unequal angles (with its long leg connected) should be used. But unfortunately unequal angles are not freely available in the market.

Example 3.10 *Design sag rods for consecutive purlins near the supported end of a roof truss system as shown in Fig. 3.26. The purlins are supported at one-third points by sag rods. Also design the ridge rod between ridge purlins. Assume c/c spacing of truss = 6 m, spacing of purlin = 1.4 m, self weight of roofing = 200 N/*

m^2, intensity of wind pressure = 1500 N/m^2, slope of the roof truss = 25°, and no access is provided to the roof.

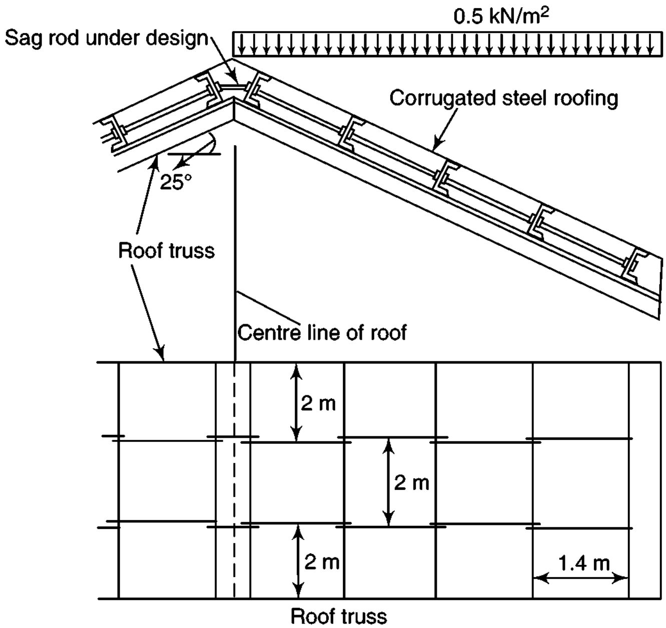


Fig. 3.26 Roof and sag rods for Example 3.10

Solution

Dead load from roofing = 200 × 1.4 = 280 N/m

Self weight of purlin = 100 N/m (assumed)

Live load = (0.75 – 0.02   15)   1000 = 450 N/m^2 > 0.40 kN/m^2

Live load on purlin = 450 × 1.4 = 630 N/m

Toad gravity load = 630 + 280 + 100 = 1010 N/m

Wind load = 1500 × 1.4 = 2100 N/m (Normal to roof)

Component of gravity load parallel to roof

= 1010 × sin 25° = 426.8 N/m

As the sag rods are placed at third points on the purlin

Pull on sag rod = 426.8 × 6/3 = 853.6 N

Factored load = 1.5 × 853.6 = 1280.4 N

Required net area = $T_{dn} \times \gamma_{m1}/(0.9 f_u)$

= 1280.4 × 1.25/(0.9 × 410) = 4.34 mm^2

Provide a 16-mm sag rod with a threaded area of 157 mm^2 between purlins (Note that the provided rod should not be less than 16-mm diameter).

Tie rod between ridge purlins

Pull in the tie rod = 4 × 853.6 × sec 25° = 3767 N

Factored load = 3767 × 1.5 = 5650 N

Required net area = $T_{dn}\gamma_{m1}/(0.9\ f_u)$

$= 5650 \times 1.25/\ (0.9 \times 410) = 19.14\ \text{mm}^2$

Hence, provide 16-mm diameter tie rods between ridge purlins.

Example 3.11 *A diagonal member of a roof carries an axial tension of 450 kN. Design the section and its connection with a gusset plate and lug angle. Use f_y = 250 MPa and f_u = 410 MPa.*

Solution

Factored tensile load = 450 kN

Required net area of section = $T_u\gamma_{m1}/(0.9f_u)$

$= 450 \times 1000 \times 1.25/(0.9 \times 410)$

$= 1524\ \text{mm}^2$

Choose ISA 150 × 75 × 10 with A = 2160 mm², r_{vv} = 16.1 mm

Providing 20-mm-diameter bolts; strength of a bolt in single shear = 45.3 kN

(Strength in bearing will not govern.)

Required number of bolts = 450/45.3 ≈ 10

Using a pitch of 2.5 × 20 = 50 mm and an edge distance of 30 mm

Length of gusset plate = 9 × 50 + 2 × 30 = 510 mm

Area of connected leg A_{nc} = [150 – 22 – (10/2)] × 10 = 1230 mm²

Area of outstanding leg A_{go} = [75 – (10/2)] × 10 = 700 mm²

$A_n = 1230 + 700 = 1930\ \text{mm}^2 > 1524\ \text{mm}^2$

Tearing strength of the net section

$T_{dn} = \alpha A_n f_u/\gamma_{m1} = 0.8 \times 1930 \times 410/1.25$

$= 506.4\ \text{kN} > 450\ \text{kN}$

Hence safe.

Without lug angle, the length of the gusset plate is 510 mm. If the bolts are staggered and arranged in two rows, the length of the gusset plate may be reduced. We will now provide a lug angle (see Fig. 3.27).

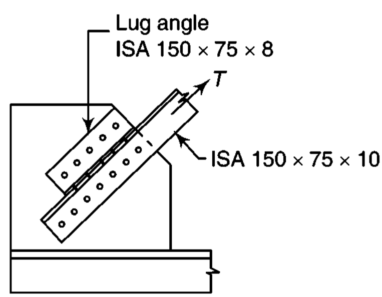


Fig. 3.27

Design of lug angle

Total factored tensile load = 450 kN

Gross area of the connected leg = [150 – (10/2)] × 10 = 1450 mm²

Gross area of outstanding leg = [75 – (10/2)] × 10 = 700 mm²

In an unequal angle, the load gets distributed in the ratio of the gross area of connected and outstanding legs.

Load shared by outstanding leg of main angle

= 450 × 700/(1450 + 700) = 146.5 kN

Load on lug angle = 1.2 × 146.5 = 175.8 (clause 10.12.2)

Required net area for lug angle $= 175.8 \times 10^3 \times 1.25/(0.9 \times 410)$

$= 596 \text{ mm}^2$

Use ISA 150 × 75 × 8 angle with $A = 1750 \text{ mm}^2$

Assuming that the section is weakened by one row of 20-mm-diameter bolt

Net area $= 1750 - 22 \times 8 = 1574 \text{ mm}^2 > 596 \text{ mm}^2$

The lug angle is also kept with its 75-mm long leg as outstanding leg

Number of bolts to connect 150-mm leg of lug angle with gusset plate

$= 175.8/45.3 \approx 4$

Provide five bolts of 20 mm diameter to connect lug angle with gusset plate.

Check

Load on connected leg = 450 × 1450/(1450 + 700) = 303.5 kN

Required number of bolts = 303.5/45.3 ≈ 7

Hence provide seven 20-mm-diameter bolts to connect the diagonal tension member with the gusset.

Required number of bolts to connect outstanding legs of the two angles (clause 10.12.2)

$= 1.4 \times 146.5/45.3 \approx 5$

Hence, provide five bolts of 20 mm diameter.

Required length of gusset plate = 6 × 50 + 2 × 30 = 360 mm (compared with 510 mm without lug angle).

Summary

Steel tension members are the most common and efficient members in structural applications. These members are connected to other members of a structure using gusset plates and by bolts or welds. A variety of cross sections can be used for tension members. The behaviour of tension members has been discussed. A brief discussion about the various factors such as the net area of cross section, method of fabrication, effect of bearing stress, and effect of shear lag have been provided. The various modes of failures of tension members have been identified. It has been found that the rupture of the net section at the end connections where the tensile stresses are the largest or the block shear failure at the end connections or the yield strength of gross section may govern the failure of tension members. The yielding of the gross section may be the governing failure mode of tension members connected by welding at the ends, whereas the other two failure modes will govern when the members are connected at the ends by bolts. Shorter connections are governed by block shear and longer connections by net sections failure. The effect of connecting the end gusset plate to only one element of the cross section has been empirically accounted for by reducing the effectiveness of the outstanding legs, while calculating the net effective area. The methods for accounting for these factors in the design of tension members are discussed with emphasis to the formulae given in the Indian code. The iterative method of design of tension members is presented. The concepts presented are explained with the use of several examples.

Exercises

1. What is the net area A_n for the tension member shown in Fig. 3.28, when (a) the holes are made by drilling, (b) holes are made by punching. Assume M20 bolts.

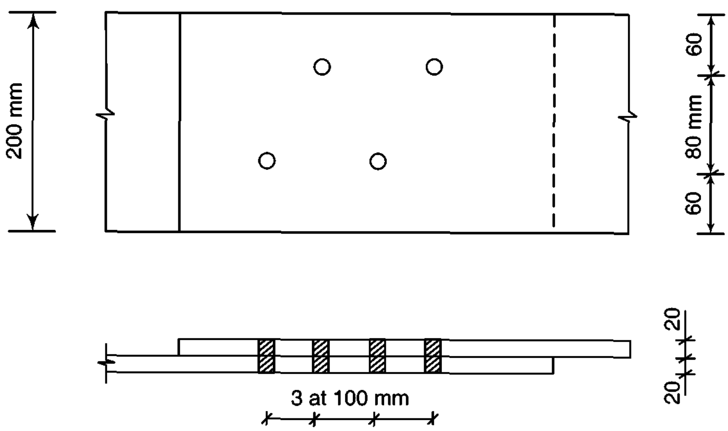


Fig. 3.28

2. Determine the net area A_n for the $150 \times 75 \times 8$ angle with M20 bolt holes as shown in Fig. 3.29.

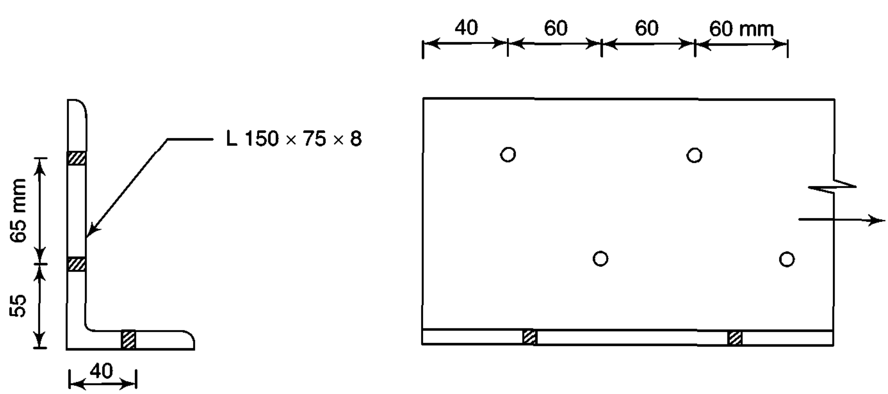


Fig. 3.29

3. Determine the design tensile strength of a plate (160×8 mm) connected to a 10-mm thick gusset using 16 mm-diameter bolts as shown in Fig. 3.30, if the yield and the ultimate stress of the steel used are 250 MPa and 410 MPa, respectively.

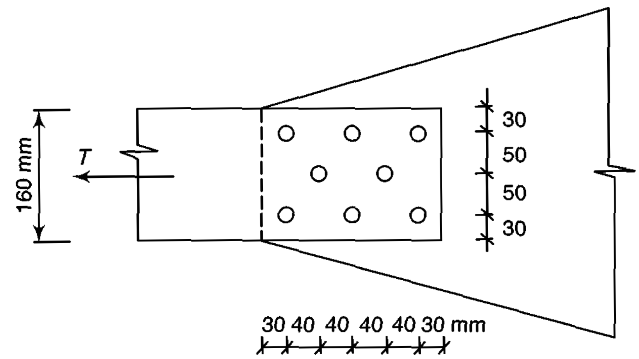


Fig. 3.30

4. What tensile load can an ISA 75 × 75 × 6 carry with the connections shown in Fig. 3.31? Assume that the connection is stronger than the members connected. Assume M30 bolts with an edge distance of 50 mm and a pitch of 75 mm, for bolted connections. The size of the fillet weld is 4 mm and the length on each side is 225 mm. Assume that the yield and the ultimate stress of steel used are 250 MPa and 410 MPa, respectively.

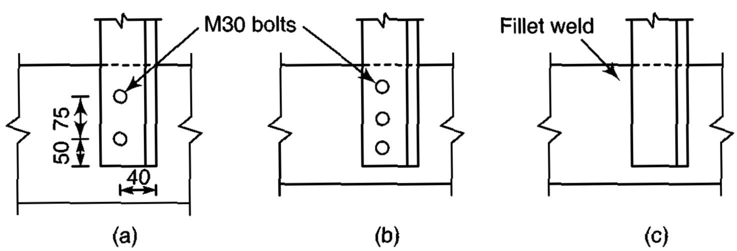


Fig. 3.31

5. Determine the tensile strength of a roof truss diagonal of 150 75 10 mm (f_y = 250 MPa) connected to the gusset plate by 6 mm welds as shown in Fig. 3.32.

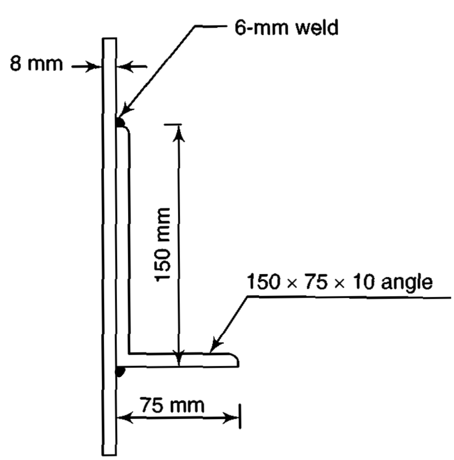


Fig 3.32

6. Select a suitable angle section to carry a factored tensile force of 150 kN, assuming (a) single row of M16 bolts, (b) welded end connection. Assume design strength as $f_y = 250$ N/mm^2.
7. Determine the tension capacity of $125 \times 75 \times 6$ mm angle in Fe410 steel, assuming
   (a) Connection through longer leg by two rows of three M20 bolts,
   (b) Connection through shorter leg by a single row of six M24 bolts.
8. A tie member in a bracing system consists of two angles of $100 \times 100 \times 6$ bolted to a 12-mm gusset, one on each side, using single row of bolts (See Fig.3.33) and tack bolted. Determine the tensile capacity of the member and the number of bolts required to develop full capacity of the member. What will be the capacity if the angles are connected on the same side of the gusset plate and tack bolted?

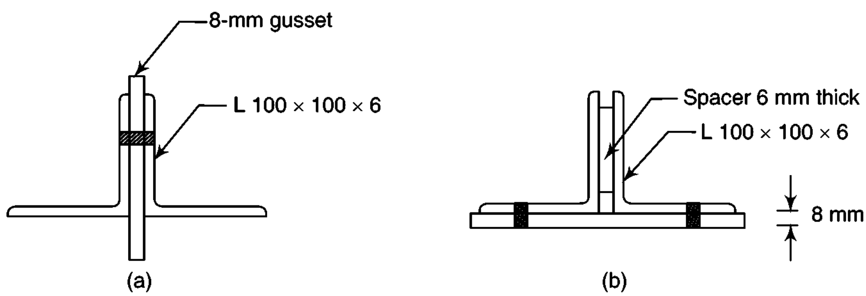


Fig. 3.33

9. Design a single angle to carry a tensile load of 500 kN. Assume that the length of the member is 3 m.
10. Design the tension member of the bottom chord of a bridge structure shown in Fig. 3.34. Assume $f_u = 410$ MPa and $f_y = 250$ MPa.

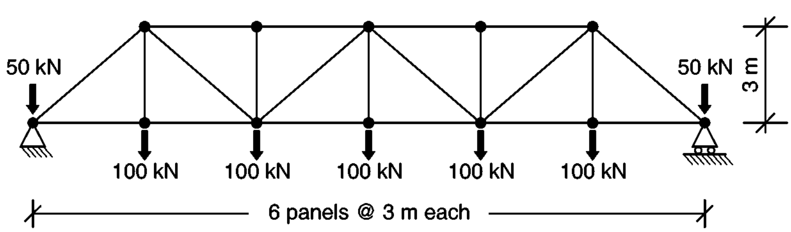


Fig. 3.34

11. Design sag rods for consecutive purlins near the supported end of a roof truss system as shown in Fig. 3.26. The purlins are supported at one-third points by sag rods. Also design the ridge rod between ridge purlins. The data given are as follows:
    C/C Spacing of truss = 5 m
    Spacing of purlin = 1.4m
    Self weight of roofing = 160 N/m^2
    Intensity of wind pressure = 1000 N/m^2
    Slope of the roof truss = 30°

    No access is provided to the roof.

12. A diagonal member of a roof carries an axial tension of 300 kN. Design the section and its connection with a gusset plate and lug angle. Use $f_y = 250$ MPa and $f_u = 410$ MPa.

Review Questions

1. List the type of cross section that can be used as tension members and their use in typical structures.
2. Why are rods, which are used as tension members, required to be pre-tensioned?
3. What is the use of spacer plates or stitch plates? At what spacing are they connected to the members?
4. What is meant by slenderness ratio?
5. The maximum slenderness ratio permissible in steel ties is
   (a) 250 (b) 350 (c) 450 (d) 400
   (e) indirectly controlled by deflection
6. The maximum slenderness ratio permissible in steel ties which may be subjected to compression under wind load condition is
   (a) 250 (b) 350 (c) 400 (d) 180 (e) no limit
7. Write down the expression for the axial elongation of the member subjected to a tensile force.
8. List the different modes of failures of a tension member.
9. What is the design stress of a tension member based on
   (a) gross-section yielding (b) net section rupture
10. Write short notes on block shear failure in plates and angles.
11. Why are drilled holes preferred over punched holes?
12. What are the methods by which the effect of punched holes can be considered in the calculation?
13. Write down the expression to calculate the net area of cross-section of a plate of width B, thickness t, and having staggered holes of pitch p and gauge g.
14. What is meant by shear lag?
15. How can the effects of shear lag be considered in the design calculation?
16. Do the geometry and ductility factors affect the design strength of tension member considerably?
17. How is the net area calculated when the angles are connected through both the legs with staggered bolts?
18. Write down the expression given in IS 800 for the net section design of angle tension members.
19. List the use of tension rods in building structures.
20. List the various steps in the design of a tension member.
21. What is a lug angle? Why is it not used in practice?
22. Write short notes on splices to tension members.
23. How are the sizes of gussets determined?

CHAPTER 4

Plastic and Local Buckling Behaviour

Introduction

The two aspects of structural behaviour under stress which are of particular importance and have considerable influence on the design of steel members are the following.

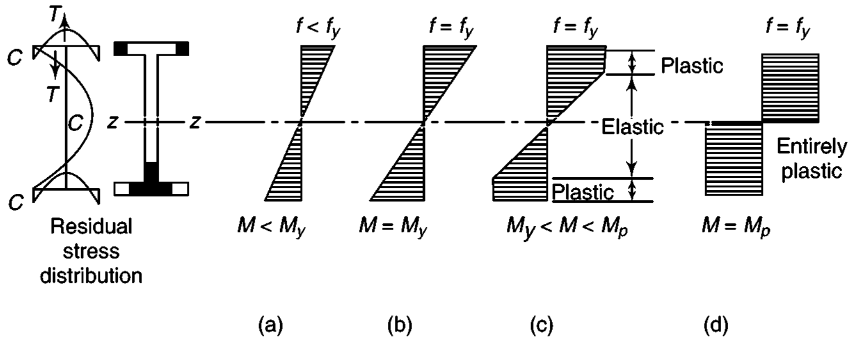
1. Behaviour of steel in the plastic region of the stress–strain curve [see Fig. 1.4(a)].
2. The tendency of unsupported compression members to ‘buckle’ and become unstable. Buckling may be defined as a structural behaviour in which a deformation develops in a direction or plane perpendicular to that of the load which produced it; this deformation changes rapidly with variations in the magnitude of applied load. Buckling occurs mainly in members that are subjected to compression (Dowling et al. 1988). Its effect is to decrease the load carrying capacity of a structure (i.e., reduce the strength) and also to increase the deformation (i.e., reduce the stiffness).

These two behavioural aspects will be briefly considered in this chapter to provide the necessary theoretical background material for the later design-oriented chapters on columns and beams.

4.1 Plastic Theory

4.1.1 Basis of Plastic Theory

To explain the concepts of plastic analysis, consider an I-beam subjected to a steadily increasing bending moment M as shown in Fig. 4.1. In the service load range the section is elastic as shown in Fig. 4.1(a) and the elastic condition exists until the stress at the extreme fibre reaches the yield stress f_y [Fig. 4.1(b)]. Once the strain ε reaches ε_y (see Fig. 4.2), increasing the strain does not induce any increase in stress. This elastic–plastic stress–strain behaviour is the accepted idealization for structural steel having yield stresses up to about f_y = 450 MPa.



Note: Shaded portion indicates portion that have reached f_y due to residual stress of stage *b*.

Fig. 4.1 Stress distribution at different stages of loading

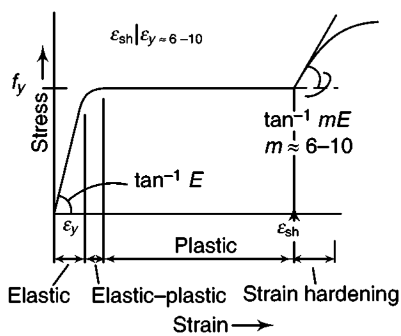


Fig. 4.2 Idealized tensile stress–strain diagram for steel

When the yield stress reaches the extreme fibre as shown in Fig. 4.1(b), the nominal moment strength M_n of the beam is referred to as the *yield moment* M_y and is given by

$$M_n = M_y = Z_e f_y \tag{4.1}$$

where Z_e is the elastic section modulus.

A further increase in the bending moment causes the yield to spread inwards from the lower and upper surfaces of the beam as shown in Fig. 4.1(c). The spread of yielding in an I-section with residual stresses is also shown in the left hand side of Fig. 4.1. This stage of partial plasticity occurs because of the yielding of the outer fibres without increase of stresses, as shown by the horizontal line of the idealized stress–strain diagram shown in Fig. 4.2. Upon increasing the bending moment further, the whole section yields as shown in Fig. 4.1(d). When this condition is reached, every fibre has a strain equal to or greater than $\varepsilon_y = f_y/E_s$. The nominal moment strength M_n at this stage is referred to as the *plastic moment* M_p and is given by

$$M_p = f_y \int_A y \, dA = f_y Z_p \tag{4.2}$$

where $Z_p = \int y \, dA$ is the *plastic section modulus.*

Any further increase in the bending moment results only in rotation, since no greater resisting moment than the fully plastic moment can be developed until strain hardening occurs. The corresponding moment–curvature relationship of the beam at various stages of loading is shown in Fig. 4.3(b); curvature may be defined as the reciprocal of radius of curvature [see Fig. 4.3(a)]. The portion DE of the curve in Fig. 4.3(b), a rising curve, is due to strain hardening. In the simple plastic theory it is conservatively assumed that the maximum moment M_p is reached at a point where the curvature can increase indefinitely (point C on the curve shown in Fig. 4.3), i.e., neglecting the benefits of strain hardening. The maximum moment M_p is called the *plastic moment of resistance*; the portion of the member where M_p occurs, termed a *plastic hinge*, can sustain large local increases of curvature [a plastic hinge is shown in Fig. 4.3(c)].

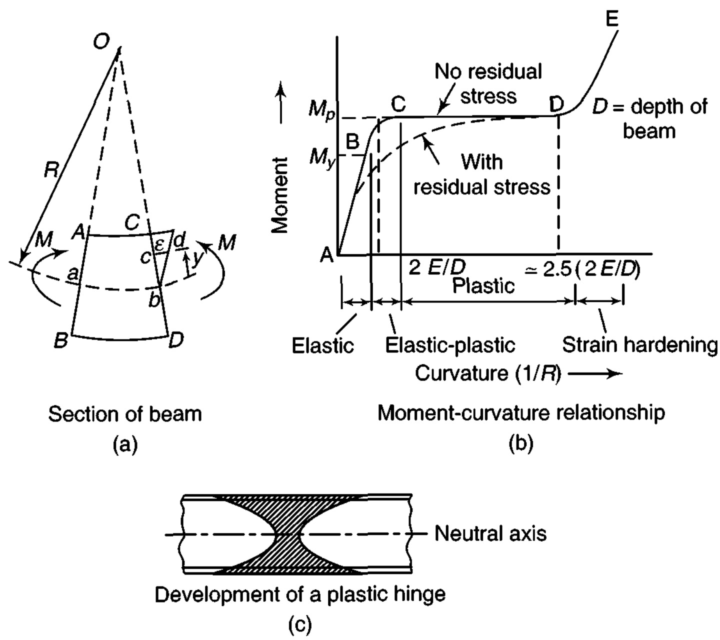


Fig. 4.3 Moment-curvature relationship at various stages of loading

Thus the plastic hinge may be defined as a *yielded zone*, which can cause an infinite rotation to take place at a constant plastic moment M_p of the section. As shown above, plastic hinges form in a member at the maximum bending moment locations. However, at the intersections of two members, where the bending moment is the same, a hinge forms in the weaker member. Generally, hinges are located at restrained ends, intersections of members, and point loads.

Generally the number of plastic hinges are $n = r + 1$, where r is the number of redundancies or the indeterminacy. (However, there are exceptions, e.g., the partial collapse of a beam in a structure.)

In the fully plastic stage, because the stress is uniformly equal to the yield stress, equilibrium is achieved when the neutral axis divides the section into two equal areas. Thus, considering the general cross section shown in Fig. 4.4 and equating the compressive and tensile forces, we get

$$f_y A_1 = f_y A_2$$

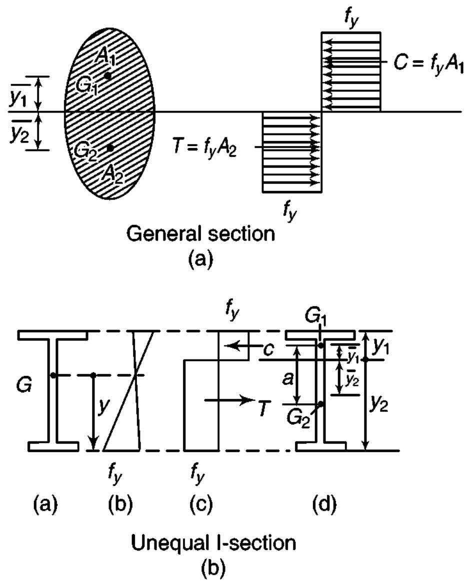


Fig. 4.4 Force equilibrium of a section

Since $A_1 = A_2$ and $A = A_1 + A_2$,

$$A_1 = A_2 = A/2$$

Plastic moment of resistance

$$M_p = f_y A_1 \bar{y}_1 + f_y A_2 \bar{y}_2$$

$$= f_y A/2 \ (\bar{y}_1 + \bar{y}_2)$$

Thus, $M_p = f_y Z_p$

where $Z_p = (A/2) \ (\bar{y}_1 + \bar{y}_2)$ is the plastic modulus of the section. (4.3)

Thus, the *plastic modulus* of the section may be defined as the combined statical moment of the cross-sectional area above and below the equal-area axis. It is also referred to as the *resisting modulus of the completely plasticized section.*

4.1.2 Shape Factor

The ratio M_p/M_y is a property of the cross-sectional shape and is independent of the material properties. This ratio is known as the shape factor v and is given by

$$v = M_p/M_y = Z_p/Z_e \tag{4.4}$$

For wide flange I-sections in flexure about the strong axis (z-z), the shape factor ranges from 1.09 to about 1.18, with the average value being about 1.14. One may conservatively take the plastic moment strength M_p of I-sections bent about their strong axis to be at least 15% greater than the strength M_y when the extreme fibre alone reaches the yield stress f_y. On the other hand, the shape factor for I-sections bent about their minor axis is about the same as for a rectangle, i.e., about 1.5. Note that when the material at the centre of the section is increased, the value of v increases. Examples 4.1 to 4.5 given at the end of this chapter explain the methods of determining the shape factors and plastic section moduli of different sections.

For the theoretically ideal section in bending, i.e., two flange plates connected by a web of insignificant thickness, the value of v will be 1. A value of shape factor nearly equal to one shows that the section is efficient in resisting bending.

4.2 Plastic-collapse Load

The load at which a sufficient number of plastic hinges are formed in a structure such that a collapse mechanism is created is called the *plastic-collapse load* or *plastic limit load*. The feature of plastic design which distinguishes it from elastic design is that it takes into account the favourable redistribution of bending moment which takes place in indeterminate structures after the first hinge forms at the point of maximum bending moment. This redistribution may be considerable and the final load at which the collapse mechanism forms may be significantly higher than the load at which the first hinge forms. Thus, for example, in a fixed beam having a concentrated load at the one-third point, the first hinge forms at one of the supports; as the load is increased further, the moment at this hinge remains constant at M_p, while the moments at the other support and the load point increase until a second hinge is formed. When the load is increased further, the moments at these two hinges remain constant at M_p, until a third and final hinge is formed to make the beam a mechanism. In this beam, the final ultimate load will be 33% higher than the first hinge load.

A number of examples of plastic-collapse mechanisms in cantilevers and single- and multiple-span beams are shown in Fig. 4.5.

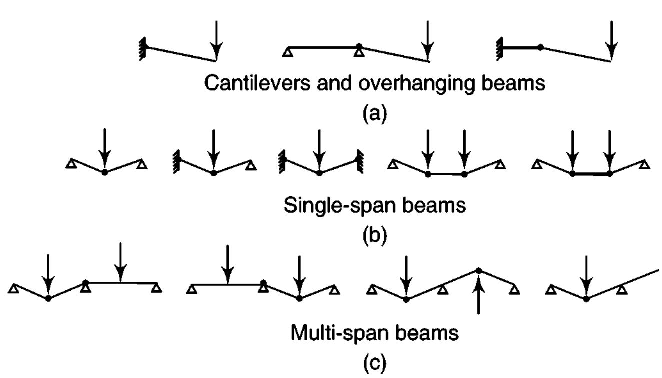


Fig. 4.5 Plastic-collapse mechanisms in beams

As noted already, plastic hinges normally occur at supports, points of concentrated load, and points where cross sections change. The location of a plastic hinge in a beam with a uniformly distributed load is, however, not well defined.

4.3 Conditions of Plastic Analysis

The basic conditions that are to be satisfied for any structure in elastic and plastic analysis are shown in Table 4.1.

Table 4.1 Conditions to be satisfied in elastic and plastic analysis

Elastic analysis	Plastic analysis
Equilibrium The summation of the forces and moments acting on any free body must be equal to zero, i.e., $\Sigma F_x = 0$, $\Sigma F_y = 0$, $\Sigma M_{xy} = 0$	*Equilibrium* The bending moment distribution defined by the assumed plastic hinges must be in static equilibrium with the applied loads and reactions.
Compatibility Strain in one section is assumed to be equal to that in the adjoining section.	*Mechanism* There must be sufficient number of plastic and frictionless hinges for the beam/structure to form a mechanism. The ultimate or collapse load is reached when a mechanism is formed.
Moment–curvature relations $M/I = f/y = E/R$ (Navier's theorem) where f = stress at any layer, E = Young's modulus of elasticity, R = local radius of curvature of beam, M = maximum bending moment, I = moment of inertia of the cross section of beam, and y = the distance of the layer under stress from the neutral axis.	*Plasticity* The bending moments in any section of a structure must be less than the plastic moment of the section $-M_p \le M \le M_p$

If all the three conditions are satisfied, the lowest plastic limit load (a unique value) is obtained. If only the equilibrium and mechanism conditions are satisfied (this forms the basis for the mechanism method of plastic analysis), an upper bound solution for the true ultimate load is obtained. A lower bound solution for the true ultimate load is obtained when the equilibrium and plasticity conditions only are satisfied (statical method of plastic analysis). See Section 4.5 for the mechanism and statical methods of analysis.

4.3.1 Principle of Virtual Work

The principle of virtual work may be stated as follows: If a system of forces in equilibrium is subjected to a virtual displacement, then the work done by the external forces equals the work done by the internal forces, i.e.,

$$W_E = W_I \tag{4.5}$$

where W_E and W_I are the work done by the external and internal forces, respectively. This is simply a method to express the equilibrium condition. While applying this method to determine the moment at collapse, an arbitrary displacement is assumed at a plastic hinge location (the arbitrary displacement must be one for which only the internal moments at the plastic hinges contribute to the internal work) and the work done by the external and internal forces is equated. This is accomplished by allowing rotations of the structure only at points of simple support and at points where plastic moments are expected to occur in producing the mechanism.

4.4 Theorems of Plastic Collapse

The plastic analysis of structures is governed by three theorems, which are as follows. The *static* or *lower bound theorem* states that a load computed on the basis of an assumed equilibrium moment diagram, in which the moments are nowhere greater than the plastic moment, is less than, or at the best equal to, the correct collapse load. Hence the static method represents the lower limit to the true ultimate load and has a maximum factor of safety. The static theorem was first suggested by Kist and its proof was given by Gvozder, Greenberg, and Horne (Horne 1979).

The *kinematic* or *upper bound theorem* states that a load computed on the basis of an assumed mechanism will always be greater than, or at the best equal to, the correct collapse load. Hence the kinematic method represents an upper limit to the true ultimate load and has a smaller factor of safety compared to the static method. A proof of this theorem was provided by Gvozder, Greenberg, and Prager (Horne 1979).

The upper and lower bound theorems can be combined to produce the *uniqueness theorem*, which states that the load that satisfies both the theorems at the same time is the correct collapse load.

4.5 Methods of Plastic Analysis

Using the principle of virtual work and the upper and lower bound theorems, a structure can be analysed for its ultimate load by any of the following methods: (1) static method, (2) kinematic method.

In the mechanism or kinematic method, a mechanism is assumed and virtual work equations are used to determine the collapse load. The number of independent mechanisms (n) is related to the number of possible plastic hinge locations (h) and the number of degrees of redundancy (r) of the frame by the equation

$$n = h - r \tag{4.6}$$

In the statical or equilibrium method, an equilibrium moment diagram is obtained such that the moment at any section is less than or equal to the plastic moment capacity. Even though the equilibrium method gives a lower bound solution, the virtual work method is often used due to its simplicity of application in comparison with the equilibrium method.

If the upper and lower bound solutions obtained by the mechanism and statical methods coincide or are sufficiently close, then the assumed plastic hinge locations are correct. If, however, these bounds are not precise enough, then the location of the assumed hinge should be modified (an indication of this will be provided by the bending moments determined in the statical analysis) and the analysis repeated. The use of the mechanism and equilibrium methods of plastic analysis in single-span beams is explained in Examples 4.6–4.10.

A load factor of 1.7 is normally used in plastic design.

4.6 Plastic Design of Portal Frames

Plastic design is used extensively for the design of single-storey portal frame structures. The design of portals is a little complicated because there can be more than one mechanism of failure, and the mechanism giving the least collapse load has to be chosen. Single-bay portal frames with fixed bases have three redundancies and require four hinges to produce a mechanism.

Furthermore, as the number of redundancies increases, so does the number of possible modes of collapse. The possible mechanisms are classified into two groups: elementary and combined mechanisms. Whereas the former are independent of each other, the latter are linear combinations of elementary mechanisms. The elementary and combined mechanisms of a single-storey portal frame are shown in Fig. 4.6. The *beam mechanism* results when three plastic hinges form within any one of the elements of the frame [see Fig. 4.6(b)]. A set of columns in a storey may develop plastic hinges at the top and bottom of each column so as to generate

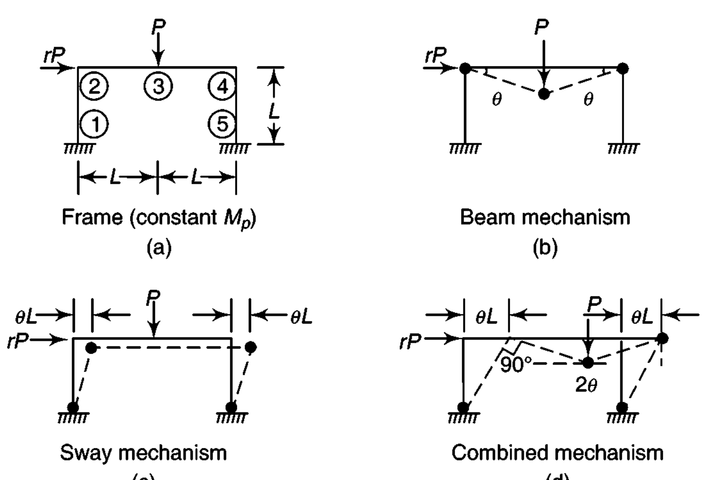


Fig. 4.6 Collapse mechanisms of a single-bay, fixed-base portal frame

a simple sway mechanism of the structure [see Fig. 4.6(c)]. The sway and beam mechanisms may be combined to form a combined mechanism as shown in Fig. 4.6(d). Thus, a structure may collapse partially or as a whole. As mentioned earlier, if the order of indeterminacy is r, then the minimum number of plastic hinges required for total collapse is $r + 1$, whereas the number of hinges required for partial collapse may be less.

Examples 4.11 and 4.12 illustrate the methods used to determine the collapse load of simple portal frames.

4.7 Special Considerations

There are a number of special considerations while attempting a plastic design. These considerations are as follows:

- Local buckling (concerned with the width-to-thickness ratios for the flange and web of a section)
- Flange stability
- Column action (effect of axial force and shear on plastic moment capacity)
- Deflection
- Connection.

Local buckling is discussed in Section 4.8. For the effect of axial force on plastic moment capacity and other considerations/limitations, refer to Davies and Brown (1996), and Horne and Morris (1981).

4.8 Local Buckling of Plates

The concept of stability as it applies to structural systems may be understood best by considering the conditions of equilibrium. If a structural system that is in equilibrium is disturbed by a force, it has two basic alternatives when the disturbing force is removed.

1. It will return to its original position, in which case we refer to the system as being *stable*.
2. It will continue to deform and as a consequence be incapable of supporting the load it supported before the disturbance occurred, in which case the system is called *unstable*.

Instability is thus characterized as a change in geometry, which results in the loss of the ability to support load. Stability, specifically the loss of ability to support load, is an extremely important consideration in the development of the limit states design criterion. Thus *buckling* may be defined as a structural behaviour in which a mode of deformation develops in a direction or plane perpendicular to that of the loading which produces it; such a deformation changes rapidly with increase in the magnitude of the applied loading. It occurs mainly in members or elements that are subjected to compressive forces.

While using plastic design, it is assumed that plastic deformation can take place without the geometry of the structure changing to such an extent that the conditions of equilibrium are significantly modified. Such changes in geometry can arise at three levels, namely (Horne & Morris 1981),

1. deformation within the cross section of a member (resulting from *local buckling* in the plate elements constituting the web or the flange),
2. displacements within the length of the member relative to straight lines drawn between the corresponding points of the end sections (due to the bending and/or twisting of the member), and
3. overall change of geometry of the structure, causing the joints to displace relative to each other (e.g., the sway deformation in multi-storey frames).

These three levels of deformation are thus associated, respectively, with local, member, and frame instability. The problem of member stability is discussed in Chapter 5. We will discuss local buckling and its consequences in this section.

4.8.1 Elastic Buckling of Plates

Local stability of the compressed elements of a section without transverse stiffeners can be studied by reference to the elastic stability of an infinite plate having width b and thickness t, as shown in Fig. 4.7. Assume that the plate is loaded by compressive forces acting on the simply supported sides having width b. The critical stress of this plate is given by (Timoshenko & Gere, 1961)

$$f_{cr} = (k\,\pi^2\,E)/[12\,(1 - \mu^2)(b/t)^2] \tag{4.7}$$

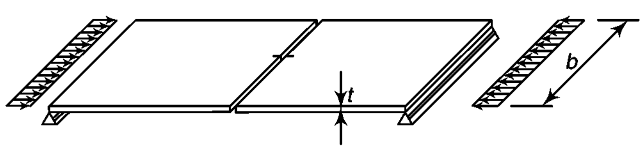
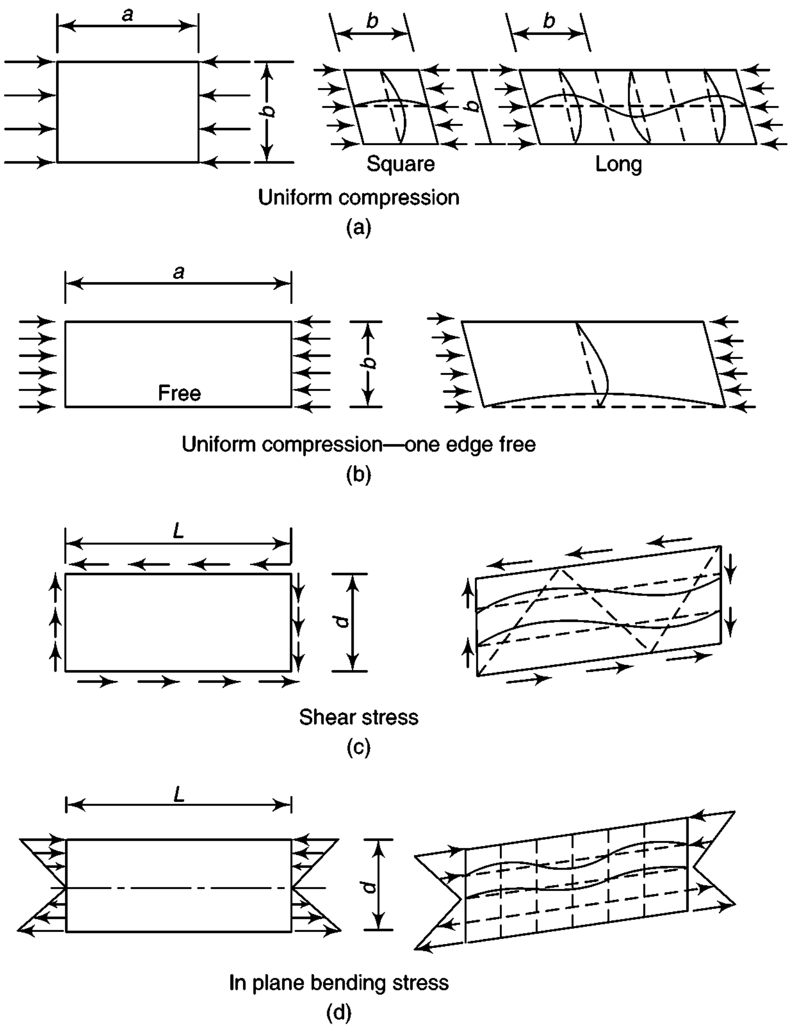


Fig. 4.7 Infinite plate subjected to compressive forces

where μ is Poisson's ratio of the material, b/t is the width-to-thickness ratio of the plate, k is the buckling coefficient, and E is Young's modulus of rigidity of the material.

The value of the coefficient k depends on the constraints along the non-loaded edges of the plate. The way in which the plates buckle and also the value of their critical buckling stress depend on the edge conditions, dimensions, and loading. The buckled plate patterns of four different plates with different loading and boundary conditions are shown in Fig. 4.8. Some values of k for the four plates are given in Table 4.2. It has to be noted that the edge conditions affect not only the critical buckling stress but also the post buckling behaviour.



All edges simply supported except as noted in (b)

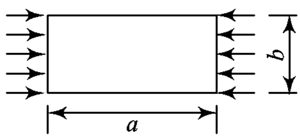
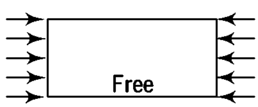
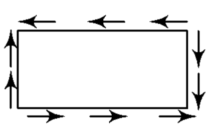
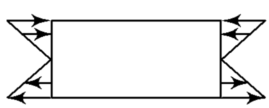
Fig. 4.8 Elastic buckling of plates with different loading and boundary conditions

On the basis of the boundary conditions, plate elements in structural members can be divided into two categories: unstiffened elements and stiffened elements. *Unstiffened elements* are supported along one edge parallel to the axial stress (e.g., legs of single angles, flanges of beams, and stems of T-sections). *Stiffened elements* are supported along both the edges parallel to the axial stress (e.g., flanges of square and rectangular hollow sections, perforated cover plates, and webs of I-sections and channel sections (see Fig. 4.9).

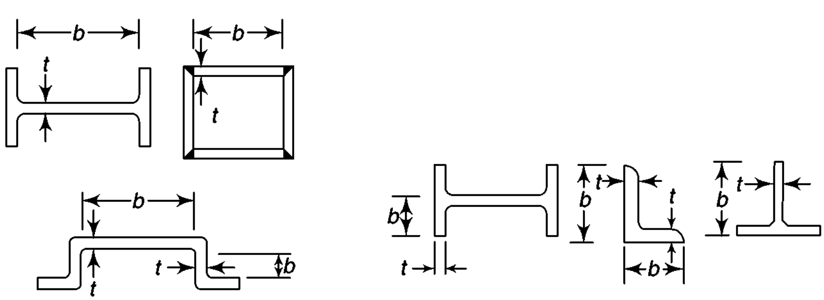
4.8.2 Plates with Different Boundary Conditions

The edges of flat plates may be fixed or elastically restrained, instead of being simply supported or free. The elastic buckling loads of flat plates with various support conditions have been determined, and many values of buckling coefficient

Table 4.2 Buckling coefficients and limiting values of width/thickness ratios. All edges simply supported except as noted. *a* = plate length/stiffener spacing on a plate girder.

Plate and load	$\frac{\text{Length}}{\text{Width}} = \frac{a}{b}$	Buckling coefficient, k	Limiting value of b/t for f_{cr} = yield stress
	1.0 5.0	4.0 4.0	53.8 53.8
	1.0 ∞	1.425 0.425	32.1 17.5
	1.0 ∞	9.35 5.35	108.2 81.9
	1.0 ∞	25.6 minimum 23.9	136.1 131.4

a = plate length/stiffener spacing on a plate girder
All edges simply supported except as noted



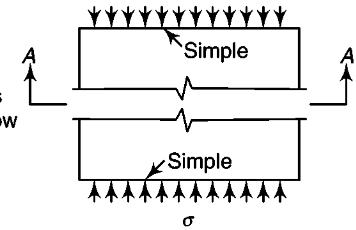
Elements supported along two edges (stiffened elements)
(a)

Elements supported along one edge only (unstiffened elements)
(b)

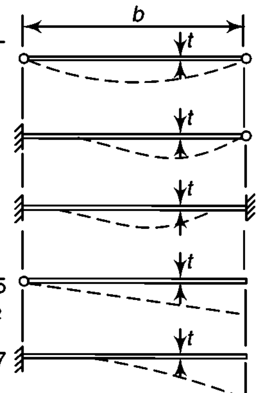
Fig. 4.9 Stiffened and unstiffened compression elements

k to be used with Eqn (4.7) are given in Timoshenko and Gere (1961), Bleich (1952), and Allen and Bulson (1980). Some of these coefficients are shown in Fig. 4.10.

Segment of long plate having thickness t, width b, and various edge conditions as tabulated below



Case	Description of edge support	k
1	Both edges simply supported	4.00
2	One edge simply supported, the other fixed	5.42
3	Both edges fixed	6.97
4	One edge simply supported, the other free	0.425 $0.425 + (b/a)^2$
5	One edge fixed, the other free	1.277



Section A-A

Fig. 4.10 Coefficients k for Eqn (4.7)

Many structural steel compression members are assemblies of flat plate elements which are rigidly connected together along their common boundaries. The local buckling of such an assembly (e.g., welded box girder, plate girder) can be analysed approximately by assuming that the plate elements are simply supported along their common boundaries and free along any unconnected boundary. Thus, the buckling stress of each plate element can be determined using Eqn (4.10) with $k = 4.0$ or 0.425 as appropriate. This approximation is conservative, as the rigidity of the joints between the plate elements causes all connected plate elements to buckle simultaneously. Solutions considering the simultaneous buckling of different elements have been obtained for channels, I-sections, and rectangular tubes by Allen and Bulson (1980), which show that the approximate solutions as discussed above underestimate the true value of k.

It has to be noted that the b/t or d/t limits given in Table 4.2 are applicable for perfectly flat plates and do not account for residual stresses and various imperfections, which lower the proportional limit. Therefore smaller values of (b/t) or (d/t) have to be used in design methods to avoid local buckling of plate elements in sections. However, it is not easy to determine the values to be used in design, since there can be considerable variations in out-of-flatness, residual stress, etc. Hence design codes suggest the limiting values based on judgement and experience. Table 4.3 gives the limiting values of b/t as adopted in IS 800 (See also Table 2 of code).

Table 4.3 Cross-sectional limits necessary to prevent local buckling in members

Type of element	Method of manufacture	Ratio	Limiting proportions for sections		
			Plastic	Compact	Semi-compact
Outstand element of compression flange	Welded	b/t_f	8.4	9.4	13.6
	Rolled	b/t_f	9.4	10.5	15.7
Internal element of compression flange	Welded or rolled	b/t_f	29.3	33.5	42
Web of an I, H or box section	Welded or rolled				
	Neutral axis at mid depth	d/t_w	84	105	126
	Generally if r_1 is negative	d/t_w	$84/(1 + r_1)$ but ≤ 42	$105/(1 + r_1)$	$126/(1 + 2r_2)$ but ≤ 42
	If r_1 is positive	d/t_w		$105/(1 + 1.5\, r_1)$ but ≤ 42	
Web of channel	—	d/t_w	42	42	42
Angle (both criteria should be satisfied)	—	b/t and d/t	9.4	10.5	15.7
Stem of a T-section (rolled or cut from a rolled I- or H-section)	—	D/t_f	8.4	9.4	18.9

Notes

1. The above values should be multiplied by $\varepsilon = (250/f_y)^{0.5}$ if f_y is not equal to 250 MPa.
2. The stress ratios r_1 and r_2 are defined as average axial compressive stress/design compressive stress of web alone and average axial compressive stress/design compressive stress of overall section, respectively. r_1 and r_2 are negative if axial stress is tensile.

4.9 Cross Section Classification

Determining the resistance (strength) of structural steel components requires the designer to consider first the cross-sectional behaviour and second the overall member behaviour—whether in the elastic or inelastic material range; cross-sectional resistance and rotation capacity are limited by the effects of local buckling.

In the code (IS 800), cross sections are placed into four behavioural classes depending upon the material yield strength, the width-to-thickness ratios of the individual components (e.g., webs and flanges) within the cross section, and the loading arrangement. The four classes of sections are defined as follows.

(a) *Plastic or class 1* Cross sections which can develop plastic hinges and have the rotation capacity required for the failure of the structure by the formation of a plastic mechanism (only these sections are used in plastic analysis and design).
(b) *Compact or class 2* Cross sections which can develop their plastic moment resistance, but have inadequate plastic hinge rotation capacity because of local buckling.
(c) *Semi-compact or class 3* Cross sections in which the elastically calculated stress in the extreme compression fibre of the steel member, assuming an elastic distribution of stresses, can reach the yield strength, but local buckling is liable to prevent the development of the plastic moment resistance.
(d) *Slender or class 4* Cross sections in which local buckling will occur even before the attainment of yield stress in one or more parts of the cross section. In such cases, the effective sections for design are calculated by deducting the width of the compression plate element in excess of the semi-compact section limit.

The moment–rotation characteristics of these four classes of cross sections are shown in Fig. 4.11. As seen from this figure, class 1 (plastic) cross sections are fully effective under pure compression, and capable of reaching and maintaining their full plastic moment in bending and hence used in plastic design. These sections will exhibit sufficient ductility ($\theta_2 > 6\ \theta_1$), where θ_1 is the rotation at the onset of plasticity and θ_2 is the lower limit of rotation for treatment as a plastic section).

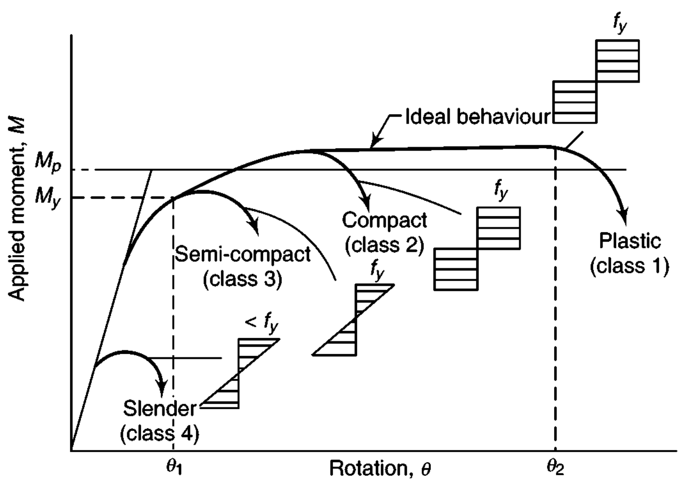


Fig. 4.11 Moment–rotation behaviour of the four classes of cross sections as defined by IS 800

Class 2 (compact) cross sections have lower deformation capacity, but are also fully effective in pure compression and are capable of reaching their full plastic moment in bending (they have ductility in the range $\theta_1 < \theta_2 < 6\theta_1$). Class 3 (semi-compact) cross sections are fully effective in pure compression but local buckling prevents the attainment of the full plastic moment in bending; bending moment resistance in these cross sections is limited to the (elastic) yield moment only. For class 4 (slender) cross sections, the local or lateral buckling of the member occurs in the elastic range. An effective cross section is therefore defined based on the width-to-thickness ratios of the individual plate elements and this is used to determine the resistance of the cross section. The majority of the hot-rolled cross sections belong to class 1, 2, or 3, and hence their resistances may be based on the gross cross-section properties obtained from section tables (IS 808).

For cold-formed cross sections, which predominantly have open cross sections (e.g., channel) and light gauge materials (with thickness less than 6 mm), the design will seldom be based on the gross cross-section properties. The class 4 cross sections of Table 4.3 contain slender elements that are susceptible to local buckling in the elastic range of material behaviour. For the design of slender compression elements considering the strength beyond the elastic local buckling of elements (such as those of cold-formed sections), the reader should refer to IS 801. The effective cross sections are to be calculated in the case of slender welded plate girders (see Example 4.13).

The design moment capacity M_d of each of the four classes of sections defined above may be calculated as follows (of course a partial factor of safety for materials as per IS 800 should also be applied to the design moment capacity):

Plastic:	$M_d = Z_p f_y$	(4.8a)
Compact:	$M_d = Z_p f_y$	(4.8b)
Semi-compact:	$M_d = Z_e f_y$	(4.8c)
Slender:	$M_d < Z_e f_y$	(4.8d)

where Z_p and Z_e are the plastic and elastic section moduli, respectively.

Each compressed (or partially compressed) element is assessed individually against the limiting width-to-thickness ratios for the class 1, 2, and 3 elements defined in Table 4.3. If an element fails to meet the class 3 limits, it should be considered as class 4. The different elements of a cross section can belong to different classes. In such cases, the section is classified based on the least favourable classification. A member with one or more slender elements has its effective area reduced to

$$A_{\text{eff}} = \sum (b_{\text{eff}} t) \tag{4.9}$$

in which the effective width of any slender plate element is determined from

$$B_{\text{eff}} = \lambda_{L3} t\varepsilon \le b \tag{4.10}$$

where λ_{L3} is the limiting width-to-thickness ratio of the semi-compact section. The limiting width-to-thickness ratios are modified by a factor ε, which takes into account the yield strength of the material (for a circular hollow section, the width-to-thickness ratios are modified by ε^2). The factor ε is defined by

$$\varepsilon = (250/f_y)^{0.5} \tag{4.11}$$

where f_y is the nominal yield strength of the steel. Note that increasing the nominal yield strength results in stricter classification limits.

The terms internal element, outstanding elements, etc., as used in Table 4.3, are described as follows.

(a) *Internal elements* These are elements attached along both longitudinal edges to other elements or to longitudinal stiffeners connected at suitable intervals to transverse stiffeners (e.g., a web of I-sections and a flange and a web of box sections).

(b) *Outstanding elements or outstands* These are elements attached along only one of the longitudinal edges to an adjacent element, the other edge being free to displace out of the plane (e.g., the flange overhang of an I-section, stem of a T-section, and leg of an angle section).

(c) *Tapered elements* These elements may be treated as flat elements having the average thickness defined in IS 808.

Compound elements in built-up section

In the case of a compound element consisting of two or more elements bolted or welded together as shown in Fig. 4.12(i), the following width-to-thickness ratios should be considered:

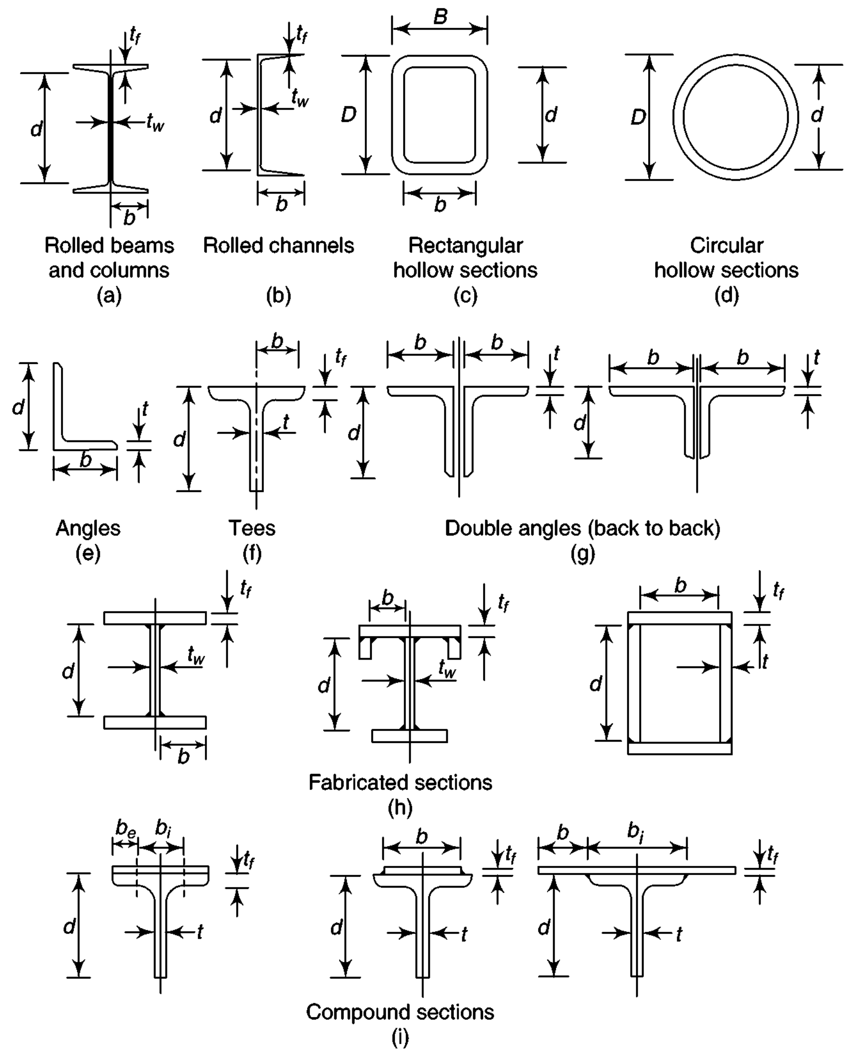


Fig. 4.12 Definition of b, d, t_f, and t_w as used in Table 4.3

(a) Outstand width of compound element 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 compared to its own thickness
(c) Any outstand of the added plates beyond the line of welds or fasteners connecting it to original section compared to its own thickness.

Note that stricter limits are imposed for welded elements, in recognition of the weakening effect of the more severe residual stresses present.

Hollow section members

A circular hollow section member has no buckling effects when its diameter-to-thickness ratio D/t satisfies

$$D/(t\varepsilon^2) \leq 88 \tag{4.12}$$

A circular hollow section member which does not satisfy Eqn (4.12) is considered a slender member. Its effective area is reduced to

$$A_{\text{eff}} = A_g\{88/[D/(t\varepsilon^2)]\}^{0.5} \tag{4.13}$$

The limiting width-to-thickness ratios of hollow rectangular sections are not specified in IS 800. However, for these members, the values shown in Table 4.4, may be used which is based on BS: 5950.

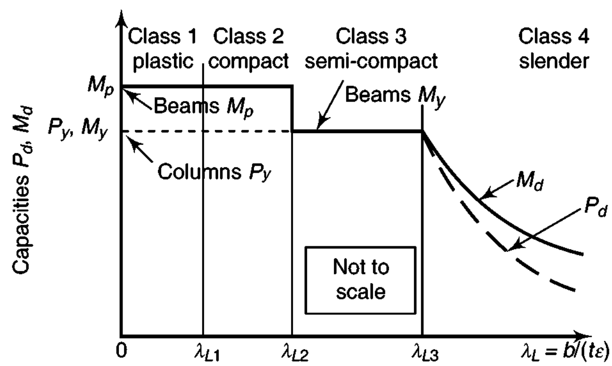
Table 4.4 Limiting width-to-thickness ratios for a circular hollow section (CHS) and a rectangular hollow section (RHS) for f_y = 250 MPa

Compression element		Ratio	Limiting value		
			Plastic	Compact	Semi-compact
Circular hollow section	Compression due to bending	D/t	$42\varepsilon^2$	$52\varepsilon^2$	$88\varepsilon^2$
Hot-rolled (HR) RHS	Flange: compression due to bending	b/t	29.3ε	33.5ε	42ε
	Web: neutral axis at mid-depth	d/t	67.1ε	84ε	125.9ε
	Generally	d/t	$64\varepsilon/(1 + 0.6\, r_1)$ but $\leq 40\varepsilon$	$84\varepsilon/(1 + r_1)$ but $\leq 40\varepsilon$	$125.9\varepsilon/(1 + 2r_2)$ but $\leq 40\varepsilon$
Cold formed (CF) RHS	Flange: compression due to bending	b/t	27.3ε	29.3ε	36.7ε
	Web: neutral axis at mid-depth	d/t	58.7ε	73.4ε	110.1ε
	Generally	d/t	$58.7\varepsilon/(1 + 0.6r_1)$ but $\leq 35\varepsilon$	$73.4\varepsilon/(1 + r_1)$ but $\leq 35\varepsilon$	$110.1\varepsilon/(1 + 2r_2)$ but $\leq 35\varepsilon$

Notes

1. For a RHS, the dimensions b and d should be as follows:
   For HR RHS: $b = B - 3t$; $d = D - 3t$
   For CF RHS: $b = B - 5t$; $d = D - 5t$
   where B, D, and t are the breadth, depth, and thickness. For a RHS subject to bending, D must always be equal to the web dimension.
2. $\varepsilon = (250/f_y)^{0.5}$

The relationship between the design moment capacity M_d and the compression flange slenderness b/t indicating the λ limits is shown in Fig. 4.13. In this figure, the value of M_d for the semi-compact section is conservatively taken as M_y. Note that in the above classifications, it is assumed that the web slenderness d/t is such that its buckling before yielding is prevented. When a beam is subjected to bending, the entire web may not be in uniform compression, and if the neutral axis lies at mid-depth, half of the web will actually experience tension. In such cases, the slenderness limits for webs are somewhat relaxed.



		λ_{L1}	λ_{L2}	λ_{L3}
Flange outstands	Hot-rolled	9.4	10.5	15.7
	Welded	8.4	9.4	13.6
Simply supported flanges	Hot-rolled	29.3	33.5	42
	Welded	29.3	33.5	42
	HF RHS	29.3	33.5	42
	CF RHS	27.3	29.3	36.7

Fig. 4.13 Moment capacities of fully braced beams and columns

As mentioned earlier, only plastic sections should be used in indeterminate frames forming plastic-collapse mechanisms. In elastic design, semi-compact sections can be used with the understanding that the maximum stress reached will be M_y. Slender sections also have stiffness problems and are not preferable for hot-rolled structural steelwork. However, they are used extensively in cold-formed members. Plate girders are usually designed based on the tension field approach (which takes into account the post-buckling behaviour of plates) to achieve economy.

Since the classification into plastic, compact, etc. is based on bending, it cannot be used for a compression member. In compression members, a criterion is used to determine whether the member is slender or not (see Chapter 5). However, in practice, compact or plastic sections are used for compression members, since they have more stiffness than semi-compact or slender members.

4.10 Behaviour and Ultimate Strength of Plates

Unlike columns, plates continue to support loads even after buckling. The post buckling strength has been considered in most of the codes, by using an effective width concept. This difference in behaviour of plates is explained in this section by comparing the bahaviour of plates with that of columns.

4.10.1 Behaviour of Plates

Though the behaviour of columns is discussed in the next chapter, it may be interesting to compare the behaviour of a column and a plate. In the case of an ideal column, as the axial load is increased, the lateral displacement remains zero until the attainment of the critical buckling load called the *Euler load* (see Chapter 5 for more information on column buckling). If the axial load versus lateral displacement is plotted, we will get a line along the load axis up to $P = P_{cr}$ (Fig. 4.14). This is called the 'fundamental path'. When the axial load reaches the Euler buckling load, the lateral displacement increases indefinitely at constant load. This is the 'secondary path', which bifurcates from the fundamental path at the buckling load. The secondary path for columns represents *neutral equilibrium*. A smooth transition from the stable path to the neutral equilibrium path will occur for practical columns, having initial imperfections [as shown by the dashed curves in Fig. 4.14(a)].

The fundamental path for a perfectly flat plate is similar to that for an ideal column. At the critical buckling load, this path bifurcates into a secondary path as shown in Fig. 4.14(b). However in this case, the secondary path shows that the plate can carry loads higher than the elastic critical load. Unlike columns, the secondary path for a plate is stable. Therefore, the elastic buckling of a plate need not be considered a collapse. However, plates having one edge free and other edges (outstands) simply supported have very little post-buckling strength.

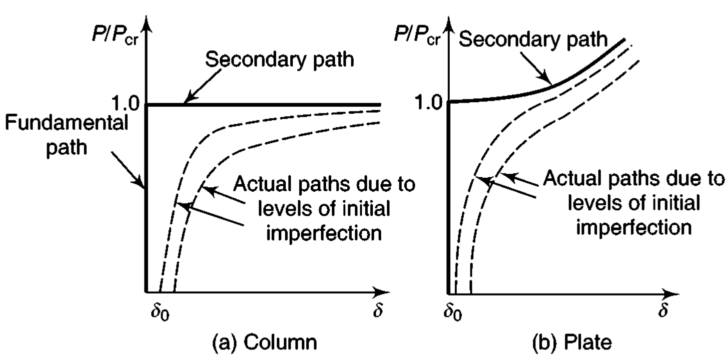


Fig. 4.14 Load versus out-of-plane displacement curves

The actual failure load of columns and plates is reached when the yielding spreads from the supported edges, triggering collapse; thereafter, the unloading occurs.

Local buckling causes loss of stiffness and redistribution of the stresses. Uniform edge compression in the longitudinal direction results in non-uniform stress distribution after buckling as shown in Fig. 4.15, and the buckled plate derives most of its stiffness from the longitudinal edge supports.

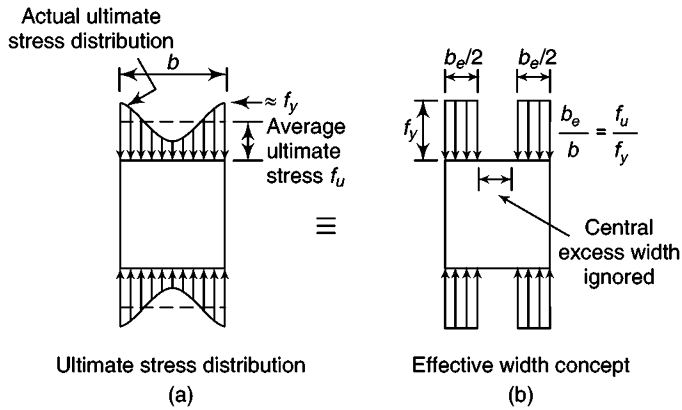


Fig. 4.15 Effective width concept for plates with simply supported edges

Examples

Example 4.1 *Determine the shape factor for a rectangular beam of width b and depth d.*

Solution

For a rectangular section (Fig. 4.16), the elastic modulus

$$Z_e = (bh^3/12)/(d/2) = bd^2/6$$

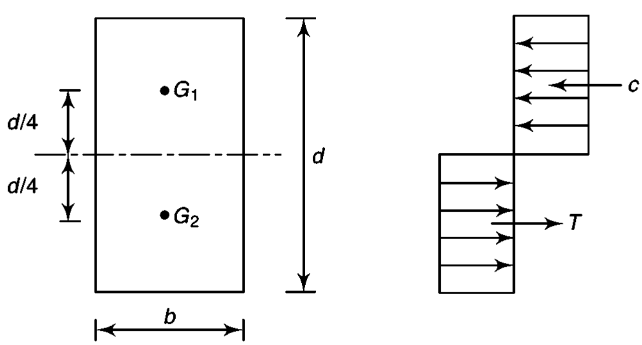


Fig. 4.16

the plastic modulus

$$Z_p = A/2\,(\bar{y}_1 + \bar{y}_2)$$
$$= bd/2(d/4 + d/4) = bd^2/4$$

Hence the shape factor

$$v = Z_p/Z_e = (bd^2/4)/(bd^2/6) = 1.5$$

Example 4.2 *Determine the plastic moment capacity and shape factor of the I-section shown in Fig. 4.17. This section is ISMB 400 with the root radius omitted. Assume f_y = 250 MPa.*

Solution

Plastic section modulus about major axes

To determine the plastic section modulus about the z-z axis, divide the section into areas A_1 and A_2 as shown in Fig. 4.17, where

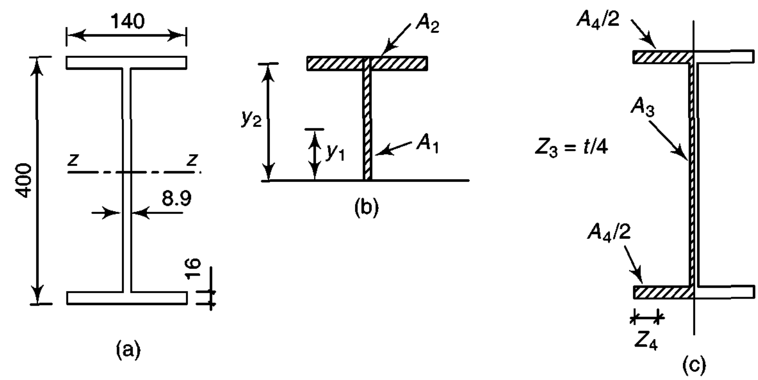


Fig. 4.17

$$A_1 = (D/2)t = (400/2) \times 8.9 = 1780 \text{ mm}^2$$

$$A_2 = (B - t)T = (140 - 8.9) \times 16 = 2097.6 \text{ mm}^2$$

and

$$\bar{y}_1 = D/4 = 400/4 = 100 \text{ mm}$$

$$\bar{y}_2 = (D/2 - T/2) = (400/2 - 16/2) = 192 \text{ mm}$$

The plastic section modulus

$$Z_p = 2(A_1 \bar{y}_1 + A_2 \bar{y}_2)$$
$$= 2(1780 \times 100 + 2097.6 \times 192)$$
$$= 1.1615 \times 10^6 \text{ mm}^3$$

The value given in Annex H of the code is 1176.18×10^3 mm^3, which is slightly greater (1.27%) because of the additional material at the root radius.

$$I_{zz} = 2(140 \times 16^3/12) + 2 \times 140 \times 16 \times (200 - 8)^2 + (400 - 16 \times 2)^3 \times 8.9/12$$
$$= 202.20 \times 10^6 \text{mm}^4$$

$$Z_e = 202.20 \times 10^6/200 = 1011.04 \times 10^3 \text{ mm}^3 \text{(compared to}$$
$$1020.00 \times 10^3 \text{mm}^3 \text{ in Annex H of the code)}$$

Hence the shape factor

$$Z_p/Z_e = 1.1615 \times 10^6/(1011.04 \times 10^3)$$
$$= 1.1488 \text{ (compared to 1.1498 given in Annex H)}$$

Note that this value together with the stress diagrams shown in Fig. 4.1 explain the use of the limit for moment capacity of $M_d \leq 1.2\, Z_e f_y/\gamma_{m0}$, which prevents plasticity at service load (clause 8.2.1.2)

Plastic section modulus about minor axis

Similarly, to determine the plastic section modulus about the y-y axis, divide the section into areas A_3 and A_4 as shown in Fig. 4.17(c), where

$$A_3 = (D - 2T)t/2 = (400 - 2 \times 16)8.9/2 = 1637.6 \text{ mm}^2$$

$$A_4 = 2\,(B/2)T = 2 \times 140/2 \times 16 = 2240 \text{ mm}^2$$

and

$$\bar{z}_3 = t/4 = 8.9/4 = 2.225 \text{ mm}$$

$$\bar{z}_4 = B/4 = 140/4 = 35 \text{ mm}$$

The plastic section modulus

$$Z_{py} = 2(A_3\bar{z}_3 + A_4\bar{z}_4)$$

$$= 2(1637.6 \times 2.225 + 2240 \times 35) = 0.1641 \times 10^6 \text{ mm}^3$$

$$I_{yy} = 2 \times 16 \times 140^3/12 + (400 - 32) \times 8.9^3/12 = 7338.95 \times 10^3 \text{ mm}^4$$

$Z_{ey} = 7338.95 \times 10^3/70 = 104.842 \times 10^3 \text{ mm}^3$ (compared to $88.9 \times 10^3 \text{mm}^3$ as given in IS 808-1989)

Hence the shape factor about the y-y axis

$$v = Z_{py}/Z_{ey} = 164.1 \times 10^3/104.842 \times 10^3$$

$$= 1.565$$

and the plastic moment capacity

$$= 164.1 \times 10^3 \times 250 \times 10^{-6} = 41.025 \text{ kNm}$$

Example 4.3 *Determine the shape factor for a triangular section of base b and height h as shown in Fig. 4.18.*

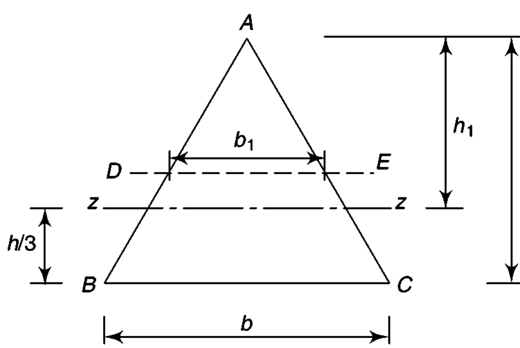


Fig. 4.18

Solution

Moment of inertia

$$I_{zz} = bh^3/36$$

Elastic section modulus

$$Z_e = (bh^3/36)/(2h/3)$$

$$= bh^2/24$$

Let DE be the equal-area axis (see Fig. 4.18). Then,

$$0.5\, b_1\, h_1 = 0.5(bh/2) \text{ or } b_1\, h_1 = bh/2 \qquad (1)$$

From similar triangles ADE and ABC

$$h_1/b_1 = h/b \qquad (2)$$

From Eqns (1) and (2), we get

$$h_1 = h/\sqrt{2} \text{ and } b_1 = b/\sqrt{2}$$

$$A = bh/2$$

$$\bar{y}_1 = h_1/3 = h/(3\sqrt{2})$$

$$\bar{y}_2 = [(h - h_1)/3]\, [(b_1 + 2b)/(b_1 + b)]$$

$$= [(h - h/\sqrt{2})/3]\, [(2b + b/\sqrt{2})/(b + b/\sqrt{2})]$$

$$= [h(\sqrt{2} - 1)/(3\sqrt{2})]\, (2\sqrt{2} + 1)/(\sqrt{2} + 1)$$

$$= 0.1548\, h$$

$$Z_p = A/2(\bar{y}_1 + \bar{y}_1) = bh/4\, [h/(3\sqrt{2}) + 0.1548h]$$

$$= 0.0976\, bh^2$$

The shape factor

$$v = Z_p/Z_e = 0.0976\, bh^2/(bh^2/24) = 2.343$$

The shape factor about the y-y axis will be different and the calculation of the same is left as an exercise to the reader.

Example 4.4 *Find out the collapse load of a simply supported beam subjected to a concentrated load at mid-span as shown in Fig. 4.19.*

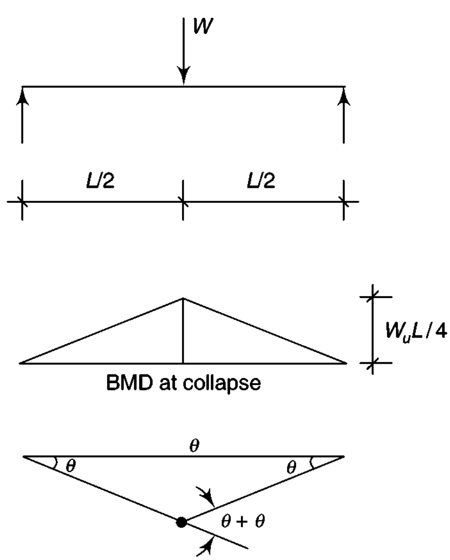


Fig. 4.19

Solution

Kinematic method

Since it is a determinate structure, one hinge is sufficient to make the beam a mechanism. Let Δ be the vertical virtual deflection at the load point. The slope of the beam at the supports

$$\theta = \Delta/(L/2) \quad \text{or} \quad \Delta = \theta L/2$$

Work done by the plastic hinge = $2M_p\theta$; work done by the load = $W_u\theta L/2$. At collapse, work by hinge = work by load

$$\therefore \qquad 2M_p\theta = W_u\theta L/2$$

$$\therefore \qquad W_u = 4M_p/L$$

Static method

$$M_p = W_uL/4 \text{ or } W_u = 4M_p/L$$

Example 4.5 *Determine the collapse load of a fixed beam with a concentrated load at mid-span as shown in Fig. 4.20.*

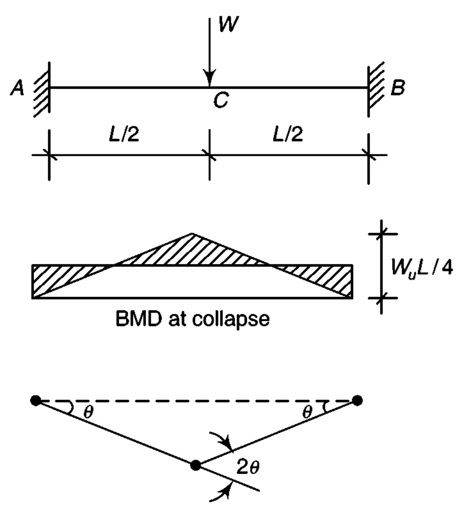


Fig. 4.20

Solution

Note that three hinges are to be formed at A, B, and C for the beam to be converted into a mechanism.

Kinematic method

Work done by the plastic hinge = $M_p\theta + M_p(2\theta) + M_p\theta = 4M_p\theta$; work done by the displacement of the load = W_u $(\theta L/2)$. Equating these expressions, we get

$$4M_p\theta = W_u\ (\theta L/2)$$

or

$$W_u = 8\ M_p/L$$

Static method

$$W_u L/4 = 2M_p \quad \text{or} \quad W_u = 8\ M_p/L$$

Example 4.6 *Determine the collapse load of the fixed beam as shown in Fig. 4.21.*

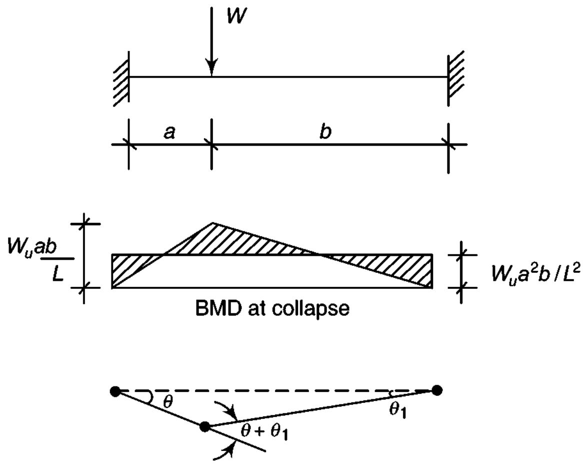


Fig. 4.21

Solution

Kinematic method

Work done by the plastic hinge

$$= M_p\theta + M_p\theta_1 + M_p(\theta + \theta_1)$$

$$= M_p\theta + M_p(\mathrm{a/b})\theta + M_p[\theta + (a/b)\theta]$$

$$= 2M_p\theta\,(1 + a/b)$$

$$=2M_p\theta\,(a + b)/b = 2\ M_pL\ \theta/b$$

Work done by the displacement of the load = $W_u a\theta$. Equating the two expressions, we get

$$W_u a\theta = 2M_pL\theta/b \quad \text{or} \quad W_u = 2M_pL/ab$$

Static method

$$W_u ab/L = 2M_p \quad \text{or} \quad W_u = 2\ M_pL/ab$$

Example 4.7 *Determine the collapse load of the fixed beam shown in Fig. 4.22 using the kinematic method.*

Solution

Kinematic method

Work done by the plastic hinges = $M_p\theta + M_p\theta + M_p(2\theta) = 4M_p\theta$; work done by the displacement of the load = $W_u(1/2)(L/2\theta) = W_uL\theta/4$. Equating the two, we get

$$4M_p\theta = W_uL\theta/4 \quad \text{or} \quad W_u = 16\ M_p/L$$

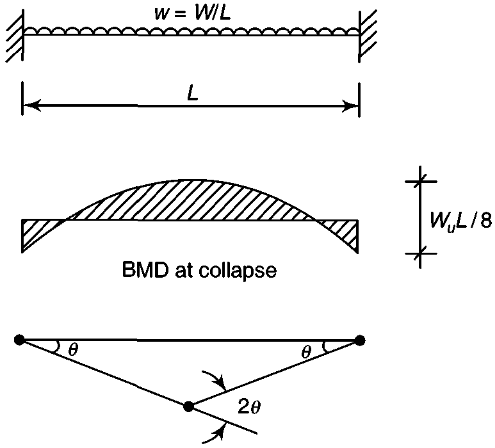


Fig. 4.22

Example 4.8 *Determine the collapse load of the propped cantilever beam shown in Fig. 4.23 using the kinematic method.*

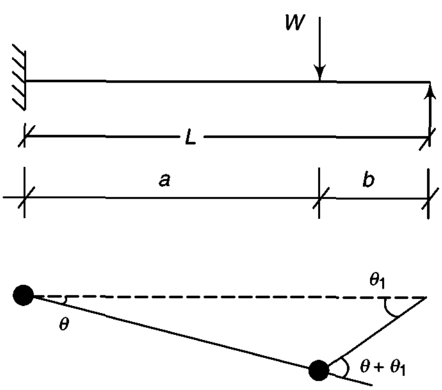


Fig. 4.23

Solution

Since the order of determinacy of this beam is 1, two hinges are required to generate a mechanism. Work done by the plastic hinges

$$= M_p\theta + M_p\,(\theta + \theta_1)$$
$$= M_p\theta + M_p\,[\theta + (\text{a/b})\theta]$$
$$= M_p\theta(1 + 1 + a/b)$$
$$= M_p\theta[b + (a + b)]/b$$
$$= M_p\theta(L + b)/b$$

Work done by the displacement of two loads = $W_u a\theta$. Equating the two, we get

$$W_u a\theta = (L + b)M_p\theta/b$$
$$W_u = M_p(L + b)/ab$$

Example 4.9 *Determine the collapse load of a fixed-end beam subjected to a load of W at one-third span as shown in Fig. 4.24.*

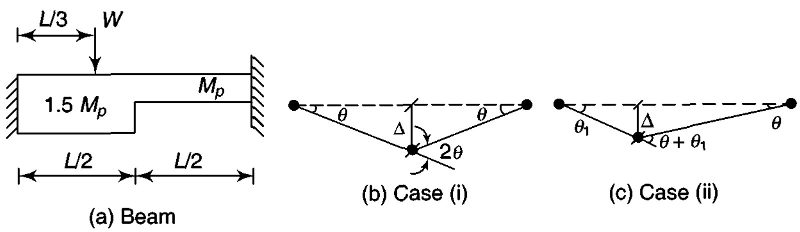


Fig. 4.24

Solution

To convert the beam into a mechanism, three plastic hinges are required. A plastic hinge can form at the point where the cross section changes. Hence we have two possible mechanisms.

Case 1 Two plastic hinges at the supports and one plastic hinge at the point where the cross section changes [see Fig. 4.24(b)]:

External work done = load × deflection

$= W_u(L/3)\theta$

Internal work done = moment × rotation

$= 1.5\ M_p\theta + M_p(\theta + \theta) + M_p\theta$

$= 4.5M_p\theta$

By the principle of virtual work,

$$4.5M_p\theta = W_u(L/3)\theta$$

or

$$W_u = 13.5M_p/L$$

Case 2 Two plastic hinges at the supports and one just below the concentrated load:

$$\Delta = (L/3)\ \theta_1 = (2/3)L\theta \quad \text{or} \quad \theta_1 = 2\theta$$

External work done $= W_u(L/3)\theta_1$

$= W_u(L/3)2\theta = (2/3)W_uL\theta$

Internal work done $= 1.5M_p\theta_1 + 1.5\ M_p(\theta + \theta_1) + M_p\theta$

$= 1.5M_p(2\theta) + 1.5\ M_p(\theta + 2\theta) + M_p\theta$

$= 8.5M_p\theta$

By the principle of virtual work,

$$(2/3)W_uL\theta = 8.5M_p\theta$$

Hence

$$W_u = 12.75M_p/L$$

The collapse load will be the smaller of the above two values. Hence the collapse load of the beam is $12.75M_p/L$.

Example 4.10 *Find the collapse load of the beam of uniform cross section shown in Fig. 4.25.*

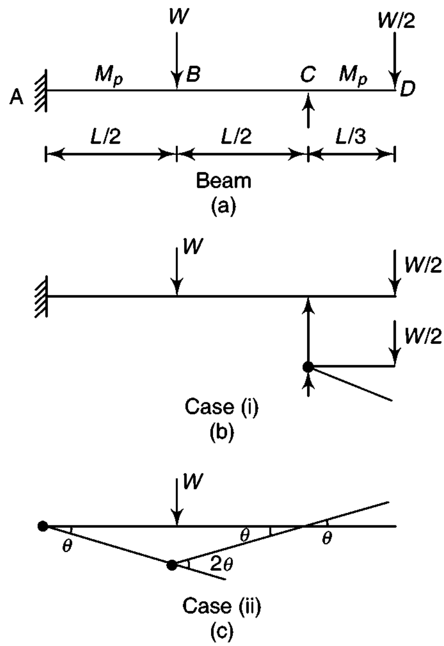


Fig. 4.25

Solution

Span CD

Since the end D is free, a plastic hinge will form at C.

$$\text{External work done} = \text{load} \times \text{deflection}$$
$$= (W/2) \times (L/3)\theta$$
$$= (WL/6)\theta$$

$$\text{Internal work done} = M_p\theta$$

By the principle of virtual work,

$$(WL/6)\theta = M_p\theta$$

or $\quad W_u = 6\,M_p/L$

Span AC

Since the end C is propped, hinges will form at A and B only. The end D will be lifted up when a mechanism is formed and negative work is done by the load acting at D. Thus,

$$\text{External work done} = W(L/2)\theta + (-W/2)(L/3)\theta$$
$$= WL\theta[(1/2) - (1/6)] = (WL/3)\,\theta$$

$$\text{Internal work done} = M_p\theta + M_p(\theta + \theta) = 3M_p\theta$$

By the principle of virtual work,

$$(WL/3)\,\theta = 3M_p\theta$$

or

$$W_u = 9M_p/L$$

Hence the collapse load for the beam is $6M_p/L$.

Example 4.11 *Determine the collapse load for the portal frame shown in Fig. 4.26, assuming that the beams and columns have the same cross section.*

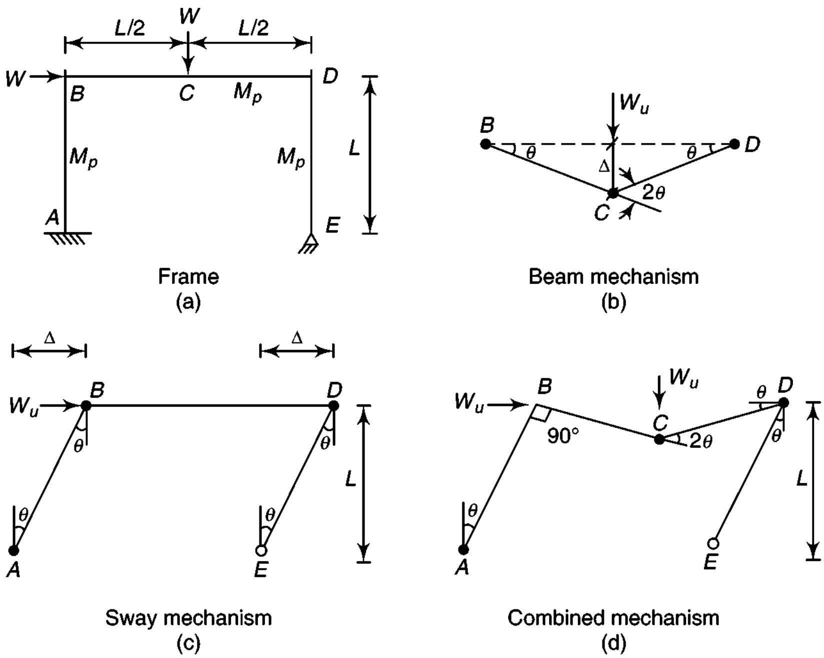


Fig. 4.26

Solution

In the case of portal frames, the first step is to find the number of independent mechanisms and then all the possible combined mechanisms. The possible locations for the plastic hinges are A, B, C, and D. Hence the number of possible plastic hinges, $N = 4$; the degree of redundancy, $r = 5 - 3 = 2$; and the number of possible independent mechanisms, $n = N - r = 4 - 2 = 2$. The two independent mechanisms are the (a) beam mechanism and (b) sway mechanism. These two independent mechanisms may be combined to form other combined mechanisms.

Beam mechanism

Assuming that plastic hinges form at B, C, and D to give a beam mechanism,

External work done $= W_u(L/2)\theta$

Internal work done $= M_p\theta + M_p(\theta + \theta) + M_p\theta = 4M_p\theta$

By the principle of virtual work,

$$W_u(L/2)\theta = 4M_p\theta$$

Hence

$$W_u = 8M_p/L$$

Sway mechanism

With a plastic hinge forming at A, B, and D (E is a mechanical hinge with zero moment and hence no plastic hinge will develop at E), the sway mechanism shown in Fig. 4.26(c) will result.

External work done = $W_u L\theta$

Internal work done = $M_p\theta + M_p\theta + M_p\theta = 3\ M_p\theta$

By the principle of virtual work,

$$W_u L\theta = 3\ M_p\theta$$

Hence

$$W_u = 3\ M_p/L$$

Combined mechanism

A combined mechanism with plastic hinges at A, C, and D is shown in Fig. 4.26(d).

External work done = load × deflection

$$= W_u L\theta + W_u(L/2)\theta = 1.5\ W_u L\theta$$

Internal work done = moment × rotation

$$= M_p\theta + M_p\,(2\theta) + M_p\,(2\theta) = 5\ M_p\theta$$

By the principle of virtual work,

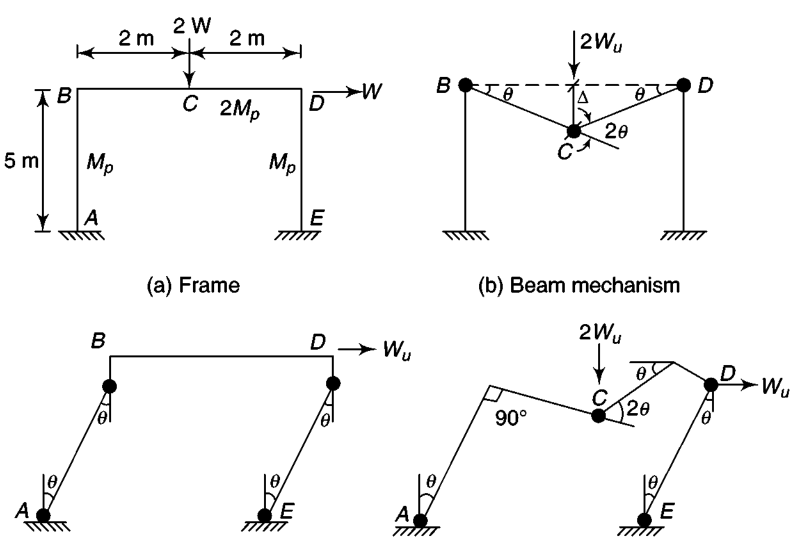
$$1.5 W_u L\theta = 5 M_p\theta$$

or

$$W_u = (10/3) M_p/L$$

As the collapse load has been calculated by the kinematic method in this example, it is the least of the three ultimate loads corresponding to the various mechanisms. Hence $W_u = 3M_p/L$.

Example 4.12 *Compute the collapse load for the portal frame shown in Fig. 4.27 and design the members if factored W_u = 72 kN and f_y of steel is 250 MPa.*



(a) Frame

(b) Beam mechanism

(c) Sway mechanism

(d) Combined mechanism

Fig. 4.27

Solution

The possible locations of the plastic hinges are A, B, C, D, and E.

The number of possible plastic hinges, $N = 5$; degree of redundancy, $r = 6 - 3 = 3$; number of possible independent mechanisms, $n = N - r = 5 - 3 = 2$. As stated in the previous example, the beam and sway mechanisms can be combined to form combined mechanisms.

Beam mechanism This mechanism is generated with three plastic hinges at B, C, and D and is associated with partial collapse [see Fig. 4.27(b)].

$$\text{External work done} = \text{load} \times \text{deflection}$$
$$= 2W_u \times 2\theta = 4W_u\theta$$
$$\text{Internal work done} = \text{moment} \times \text{rotation}$$
$$= M_p\theta + 2M_p(\theta + \theta) + M_p\theta = 6M_p\theta$$

By the principle of virtual work, $4W_u\theta = 6M_p\theta$. Hence $W_u = 1.5M_p$.

Sway mechanism The sway mechanism is shown in Fig. 4.27(c), with plastic hinges in the columns at A, B, D, and E.

$$\text{External work done} = W_uL\theta = W_u5\theta$$
$$\text{Internal work done} = M_p\theta + M_p\theta + M_p\theta + M_p\theta = 4M_p\theta$$

By the principle of virtual work, $5W_u\theta = 4M_p\theta$. Hence $W_u = (4/5)M_p$.

Combined mechanism

The combined mechanism is shown in Fig. 4.27(d) with hinges forming at A, C, D, and E.

$$\text{External work done} = 2W_u \times 2\theta + W_u5\theta = 9W_u\theta$$
$$\text{Internal work done} = M_p\theta + 2M_p(2\theta) + M_p\theta + M_p\theta + M_p\theta = 8M_p\theta$$

By the principle of virtual work, $9W_u\theta = 8M_p\theta$. Hence $W_u = (8/9)M_p$.

Therefore, the collapse load for the frame is $(4/5)M_p$ [least of $1.5M_p$, $(4/5)M_p$, and $(8/9)M_p$].

Design

The section must be so designed that it resists the maximum value of M_p. From the preceding calculations,

$$M_p = (9/8)W_u = (9/8) \times 72 = 80 \text{ kN m}$$

Required Z_p for column $= 80 \times 10^6/250 = 320 \times 10^3$ mm^3

Required Z_p for beam $= 2 \times 320 \times 10^3 = 640 \times 10^3$ mm^3

Hence, ISMB 225 (with $Z_p = 348.27$ mm^3) is required for the columns and ISMB 300 (with $Z_p = 651.74$ mm^3) is required for the beam of the portal frame.

Example 4.13 *Check whether the moment capacity of a welded plate girder comprising two 650- × 25-mm flange plates and one 1500- × 15-mm web plate will be affected by flange local buckling assuming (a) Fe 410 steel with a design strength of f_y = 250 MPa (b) Fe 540 steel with a design strength of f_y = 410 MPa (see Fig. 4.28).*

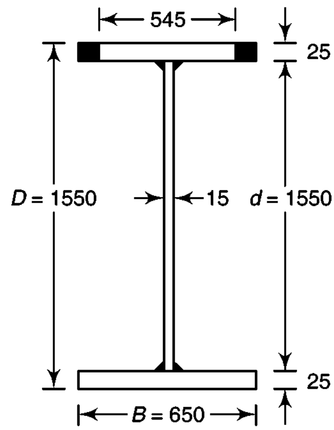


Fig. 4.28

Solution

(a) For $f_y = 250$ MPa, from Table 4.3, (Table 2 of IS 800), maximum outstand b/t for the flange to be compact = 8.4.

Actual b/t, using Fig. 4.28 = (325 – 15/2)/25 = 12.7 > 8.4

Maximum b/t for the flange to be semi-compact = 13.6

Hence the section is semi-compact and $M_d = Z_e f_y$

$$I_z = BD^3/12 - (B - t_w)d^3/12$$

$$= 650 \times 1550^3/12 - (650 - 15) \times 1500^3/12 = 23116.15 \times 10^6 \text{ mm}^4$$

$$Z_{ez} = 23116.15 \times 10^6/(1500/2 + 25) = 29827.3 \times 10^3 \text{ mm}^3$$

$$M_d = 250 \times 29827.3 \times 10^3/10^6 = 7456.8 \text{ kN m}$$

$$Z_p = 2Bt_f(D - t_f)/2 + t_w d^2/4$$

$$= 2 \times 650 \times 25\ (1500 - 25)/2 + 15 \times 1500^2/4$$

$$= 32406.25 \times 10^3 \text{ mm}^3$$

$$M_p = 32406.25 \times 10^3 \times 250/10^6 = 8101.5 \text{ kN m}$$

Hence reduction in capacity from that corresponding to compact behaviour

$$= (8101.5 - 7456.8)/8101.5 \times 100 = 7.96\%$$

(b) For $f_y = 410$ MPa, maximum b/t for the flange to be semi-compact

$$= 13.6 \times (250/410)^{0.5} = 10.6$$

Therefore the section is slender. Limit for the effective flange width for semi-compact behaviour

$$= 10.6 \times 25 = 265 \text{ mm}$$

Hence effective top width of the flange = 265 × 2 + 15 = 545 mm (see Fig. 4.28)

Location of neutral axis

Taking moments about the top edge, the distance of the neutral axis from the top edge,

$$\bar{y} = [545 \times 25 \times 25/2 + 1500 \times 15 \times (1500/2 + 25) + 650 \times 25 \times (1525 + 25/2)]/(545 \times 25 + 1500 \times 15 + 650 \times 25)$$

$$= 813.2 \text{ mm}$$

$$I_z = (545 \times 25)\,(813.2 - 12.5)^2 + (15 \times 1500^3/12) + (15 \times 1500) \times (813.2 - 775)^2 + 650 \times 25\,(736.8 - 12.5)^2$$

$$= 2.1511 \times 10^{10} \text{ mm}^4$$

$$Z_z \text{ (top flange)} = 2.1511 \times 10^{10}/813.2 = 26453 \times 10^3 \text{ mm}^3$$

$$M_d = 410 \times 26453 \times 10^3/10^6 = 10{,}845.8 \text{ kN m}$$

Capacity, if the whole section were as effective as the semi-compact section,

$$M_{d1} = 410 \times 29827.3 \times 10^3/10^6 = 12{,}229.2 \text{ kN m}$$

Hence reduction in capacity from that corresponding to semi-compact section behaviour

$$= (12{,}229.2 - 10{,}845.8/12{,}229.2) \times 100 = 11.3\%$$

Capacity, if the whole section is as effective as the plastic section,

$$M_p = 32406.25 \times 10^3 \times 410/10^6 = 13{,}286.5 \text{ kN m}$$

Hence reduction in capacity due to slenderness

$$= (13{,}286.5 - 10{,}845.8/13286.5) \times 100 = 18.4\%$$

Thus, 18.4% of the capacity of the cross section could not be utilized, due to the slenderness of the cross section.

Summary

The basic concepts of plastic analysis have been introduced in this chapter. The section modulus, either elastic or plastic depending on the design philosophy adopted, is the most appropriate property to be considered while selecting a beam section. Methods for calculating the plastic modulus and shape factor (ratio of M_P/M_y) of any cross section have been explained with a number of examples. The plastic hinge concept, conditions to be satisfied for elastic/plastic analysis, and the methods of plastic analysis (mechanism and statical method) have been discussed. The theorems of plastic collapse and alternative patterns of hinge formation triggering plastic collapse have been discussed. The plastic designs of simply supported beams, fixed beams propped cantilevers, continuous beams, and portal frames have been explained with the use of worked examples. A number of special considerations, such as local buckling, flange stability, deflection, connection, etc., should be satisfied while attempting a plastic design.

Most structural sections are assemblages of plates of slender proportions, which are prone to local buckling. Local buckling has the effect of reducing the overall load carrying capacity of columns and beams due to the reduction in the stiffness and strength of the locally buckled elements. Therefore it is desirable to avoid local buckling before the yielding of the member. The local buckling action can be analysed approximately by considering each plate of a cross section in isolation. The critical stress formulae of plates subjected to different loading and boundary conditions have been given is this chapter. From these formulae it is found that the buckling capacity of a plate is inversely proportional to the square of its width-to-thickness ratio (b/t or d/t). One way of avoiding local buckling is to ensure that the width-to-thickness ratio of each component plate does not exceed a certain value. In IS 800, a comprehensive system has been introduced for classifying the cross section of members that are subjected to compression due to moment and/or axial loading. The classification depends primarily on the geometry of the cross section and has the following four classes: plastic (class 1), compact (class 2), semi-compact (class 3), and slender (class 4). These four classes have been described for various cross sections in this chapter.

Plastic and compact sections are preferable if limit state design is used, and only plastic sections can be used in mechanism-forming indeterminate frames (plastic design). Slender sections are to be avoided even in elastic design. Most hot-rolled steel sections have individual elements of sufficient thickness (may be classified as plastic, compact, or semi-compact) and hence local buckling is avoided before the yielding of the member. However, fabricated (welded) sections and thin-walled cold-formed steel members usually experience local buckling of plate elements before the yield stress is reached. For such sections, only the effective area and moment of inertia should be used for calculating the capacity.

Substantial reserve strength exists in plates beyond the point of elastic buckling. The utilization of this reserve capacity may also be a design objective. Post-buckling reserve strength is normally taken into account in the design of cold-formed sections, approximately by using the empirical effective width concept.

Further details on plastic analysis, plastic design of frames (especially pitched roof portals and multi-storey frames), and buckling of plate elements may be found in the references given at the end of the book.

Exercises

1. Find the shape factor for the following sections:
   (a) square of side a with its diagonal parallel to z-z axis as shown in Fig. 4.29(a) [Ans: $v = 2$]
   (b) Hollow square section of external side D, internal side d and wall thickness t as shown in Fig. 4.29(b) [Ans: $v = 1.12$]
   (c) Circular section of radius r as shown in Fig. 4.29(c) [Ans: $v = 1.7$]
   (d) Hollow tube section of external diameter D and internal diameter d

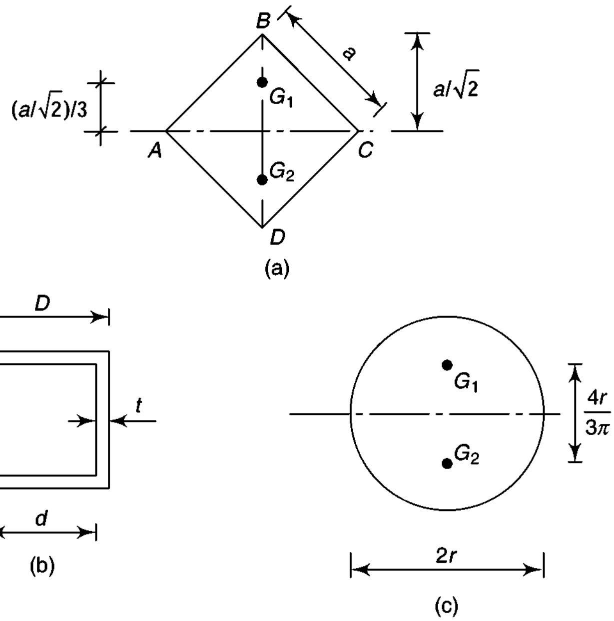


Fig. 4.29

2. Determine the plastic moment capacity of the sections shown in Figs 4.30(a) to (c), assuming $f_y = 250$ MPa

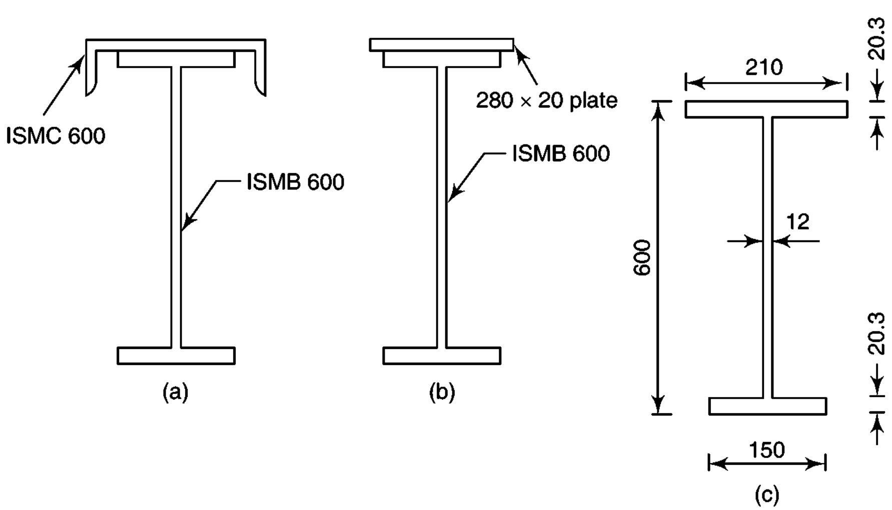
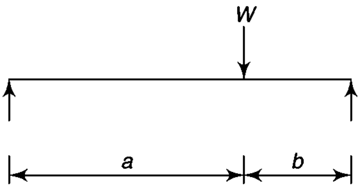


Fig. 4.30

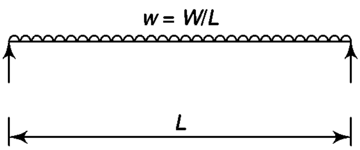
3. Find out the collapse load of the following beams (assume that the beam is of uniform cross section, unless otherwise shown).

(a)



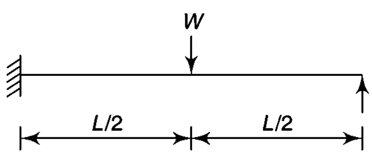
Ans: $W_u = M_p L/ab$

(b)



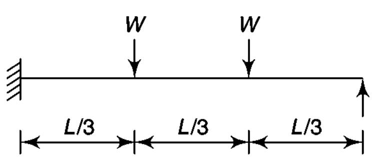
Ans: $W_u = 8M_p/L$

(c)



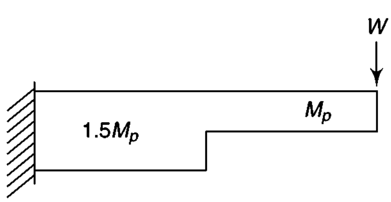
Ans: $W_u = 6M_p/L$

(d)



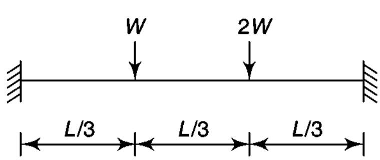
Ans: $W_u = 4M_p/L$

(e)



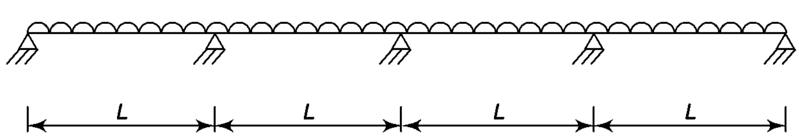
Ans: $W_u = 0.75M_p/L$

(f)



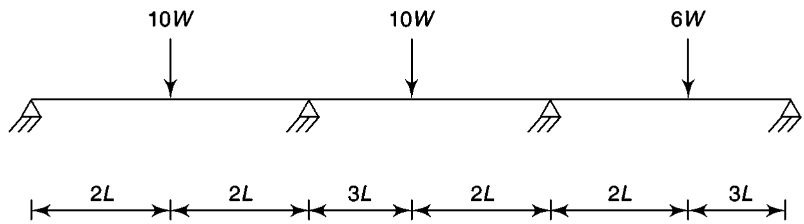
Ans: $W_u = 3.6M_p/L$

(g)



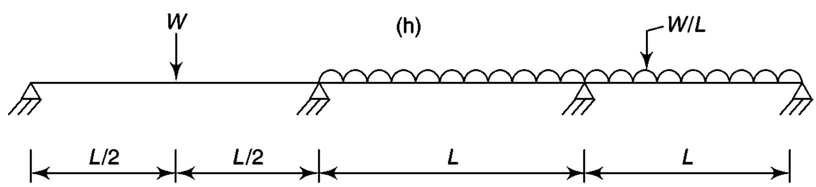
Ans: $W_u = 12M_p/L^2$

(h)



Ans: $W_u = 0.15M_p/L$

(i)



Ans: $W_u = 8M_p/L$

4. Find out the collapse load for a portal frame of uniform cross section as shown in Figs 4.31(a) and (b).

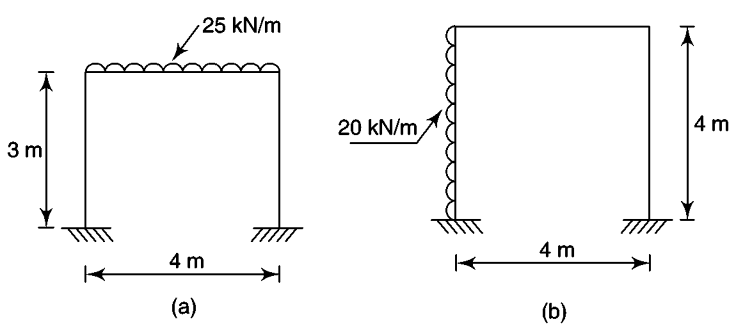


Fig. 4.31

5. Design a suitable section for a two-span continuous beam, each having a span of 10 m and supporting a dead load of 15 kN/m and a live load of 25 kN/m by (i) elastic design procedure and (ii) plastic design procedure.
6. Check whether the moment capacity of a welded plate girder consisting of two 600- 20-mm flange plates and one 1000- 15-mm web plate will be affected by flange buckling, assuming (a) Fe 410 grade steel (f_y = 250 MPa), (b) Fe 540 grade steel (f_y = 410 MPa).

Review Questions

1. Distinguish between the elastic modulus and plastic modulus of a section.
2. Define shape factor.
3. Will the shape factor of rectangular and circular cross sections be higher than that of I-sections? If yes, state the reason.
4. What is a plastic hinge? In what way is it different from an ordinary hinge?
5. What are the points at which a plastic hinge is likely to form?

6. Describe the collapse of a fixed-ended beam, as assumed in plastic design.
7. What are the basic conditions to be satisfied in an elastic analysis and a plastic analysis?
8. Illustrate the difference between the yield moment and plastic moment of resistance by taking an example of a simply supported rectangular beam subjected to a central concentrated load.
9. What is the difference in collapse mechanisms of single- and multi-span beams?
10. State the principle of virtual work.
11. State the following theorems of plastic collapse: (a) static theorem (b) kinematic theorem and (c) uniqueness theorem.
12. What are the two methods of plastic analysis by which the collapse load can be determined?
13. What is the load factor usually adopted in plastic design?
14. What are the two groups of mechanisms considered in the plastic design of portal frames?
15. Define the phenomenon of buckling.
16. What are the three levels of changes in geometry that are associated with local, member, and frame instability?
17. Write the general equation for buckling of plates in the elastic region.
18. State the four classifications of sections as per IS 800.
19. Is it possible to use slender section for beams?
20. If different elements of a cross section are of different classes, how will the whole section be classified?
21. How can the reduced effectiveness of slender sections be taken into account?
22. Why are only plastic sections to be used in indeterminate frames?
23. What is the basic difference between a plastic and a compact section?
24. What is the basic difference between a semi-compact and a compact section?
25. Give the b/t or d/t ratio for the following cases:
    (a) Outstand element (flange) of rolled beam of plastic section
    (b) Outstand element (flange) of rolled beam of semi-compact section
    (c) Web of an I-beam of plastic cross section
    (d) Web of an I-beam of semi-compact section
26. Describe the difference in behaviour of columns and plates.

CHAPTER 5

Design of Compression Members

Introduction

A structural member which is subjected to compressive forces along its axis is called a *compression member*. Thus, compression members are subjected to loads that tend to decrease their lengths. Except in pin-jointed trusses, such members (in any plane or space structure), under external loads, experience bending moments and shear forces. If the net end moments are zero, the compression member is required to resist load acting concentric to the original longitudinal axis of the member and is termed *axially loaded column,* or simply *column.* If the net end moments are not zero, the member will be subjected to an axial load and bending moments along its length. Such members are called *beam-columns* and are treated in Chapter 9.

Let us consider an example of an axially loaded column shown in Fig. 5.1. For this column, the axial load P, to be resisted by it, is the sum of the beam shears $V_1 + V_2$. For the column shown in Fig. 5.1, the net end moment is assumed to be zero; this is true if the end moments and shears developed by the two beams are equal. Such situations arise in many interior columns of buildings having equal column spacing. Where the beam is not connected rigidly to the column, the beams will not develop significant end moments and in such situations also the column has to resist only the difference in end shears. In several interior columns, the net moment will be small and the member is designed as an axially loaded column.

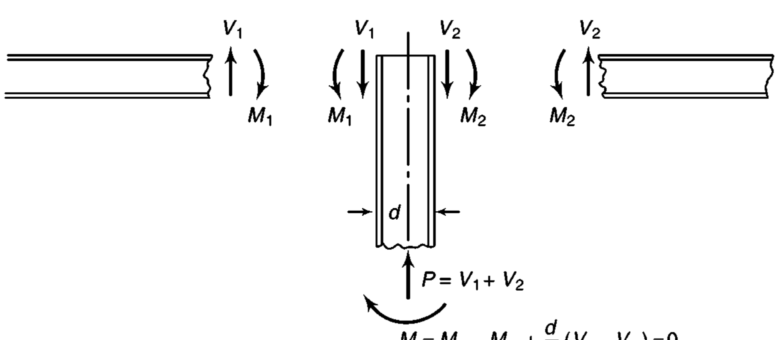


Fig. 5.1 Axially loaded column

Multi-storey steel building under construction in Maryland, USA

Close-up view of the column of the above building

Different terms are used for a compression member depending upon its position in a structure. The vertical compression members in a building supporting floors or girders are normally called as columns (referred sometimes as *stanchions* in UK). They are subjected to heavy loads. Sometimes vertical compression members are called *posts*. The compression members used in roof trusses and bracings are called *struts*. They may be vertical or inclined and normally have small lengths. The top chord members of a roof truss are called the *principal rafter*. The principal compression member in a crane is called the *boom*. Short compression members at the junction of columns and roof trusses or beams are called *knee braces*. Some of these compression members are shown in Fig. 5.2.

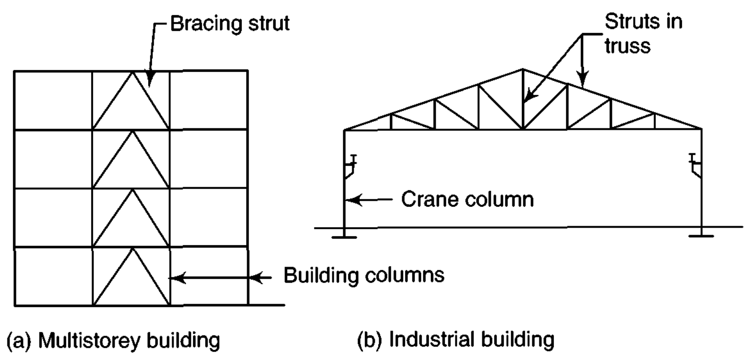


Fig. 5.2 Types of compression members

It is well known from basic mechanics of materials that only very short columns can be loaded up to their yield stress. For long columns (see Section 5.3.1 also), *buckling* (deformation in the direction normal to the load axis) occurs prior to developing the full material strength of the member. Hence, a sound knowledge of stability theory is necessary for designing compression members in structural steel.

Since compression members have to resist buckling, they tend to be stocky and 'square' and circular tubes are found to be the ideal sections, since their radius of gyration is same in the two axes. This situation is in contrast to the slender and more compact tension members and deep beam sections. Unlike the member subjected to tension, a compression member is designed on the assumption that its gross cross-sectional area will be effective in resisting the applied loads. Bolts may be used to connect columns to adjacent members. As the load is applied, the member will contract. It is assumed that the action of bolts is such that they will replace the material removed for holes. Thus, the bolt holes are often ignored in the design. Since compression members comprise of thin plates, they also experience local buckling, as discussed in Chapter 4.

The strength of a column depends on the following parameters:

- Material of the column
- Cross-sectional configuration
- Length of the column
- Support conditions at the ends (called restraint conditions)
- Residual stresses
- Imperfections

The imperfections include the following:

- The material not being isotropic and homogenous
- Geometric variations of columns
- Eccentricity of load

It is difficult to assess the residual stress acting on each column cross section and also to assess the degree of support condition offered by a variety of connection details adopted in practice. Due to the large number of variables, which influence the strength of columns and beam-columns in the elastic and inelastic ranges, several researchers throughout the world have done extensive experimental and theoretical investigations on the behaviour of columns due to these variables.

In this chapter, the effect of these parameters on the strength and stability of columns are briefly discussed and the design, as per the Indian code of practice IS 800, is given. The method of design of open web columns (latticed, battened) and base plates are also provided.

5.1 Possible Failure Modes

The possible failure modes of an axially loaded column are discussed as follows:

1. *Local buckling* Failure occurs by buckling of one or more individual plate elements. This failure mode may be prevented by selecting suitable width-to-thickness ratios of component plates. Alternatively, when slender plates are used, the design strength may be reduced (see Section 5.2).
2. *Squashing* When the length is relatively small (stocky column) and its component plate elements are prevented from local buckling, then the column will be able to attain its full strength or 'squash load' (yield stress × area of cross section).
3. *Overall flexural buckling* This mode of failure normally controls the design of most compression members. In this mode, failure of the member occurs by excessive deflection in the plane of the weaker principal axis. An increase in the length of the column, results in the column resisting progressively less loads.
4. *Torsional and flexural–torsional buckling* Torsional buckling failure occurs by twisting about the shear centre in the longitudinal axis. A combination of flexure and twisting, called *flexural–torsional buckling* is also possible. Torsional buckling is a possible mode of failure for point symmetric sections. Flexural torsional buckling must be checked for open sections that are singly symmetric and for sections that have no symmetry. Note that open sections that are doubly symmetric or point symmetric are not subjected to flexural-torsional buckling, since their shear centre and centroid coincide. Closed sections are also immune to flexural–torsional buckling.

In addition to the above failure modes, in compound members (two or more shapes joined together to form a lattice cross section), failure of a component member may occur, if the joints between members are sparsely placed. Codes and specifications usually have rules to prevent such failures (see Section 5.11).

5.2 Classification of Cross Section

If individual plate elements which make up the cross section of a compression member, (for example, the web and two flanges in the case of an I-section), are thin, local buckling may occur. For columns, it is frequently possible to eliminate this problem by limiting the proportions of component plates as given in Table 5.1, such that local buckling will not influence the strength of the cross sections.

These limits are applicable to semi-compact sections. When more slender plates, having width-to-thickness ratios higher than the limits given in Table 5.1 are to be used, the strength of the section should be suitably reduced. However, it is a good practice to use plastic or compact sections for compression members, since they provide more stiffness than semi-compact or slender sections.

Table 5.1 Limiting width-to-thickness ratios for axial compression elements

Element	Ratio	Upper limit for semi-compact section
Outstand element of compression flange (I, H, or C)		
Rolled section	b/t_f	15.7ε
Welded section	b/t_f	13.6ε
Internal element of compression flange (box)	b/t_f	42ε
Web of an I-H or box-section	d/t_w	$\leq 42\varepsilon$
Single angle or double angle with the components separated (all three criteria should be satisfied)	b/t d/t $(b+d)/t$	15.7ε 15.7ε 25ε
Circular hollow section	D/t	$88\varepsilon^2$
Hot rolled RHS—Flange	b/t	42ε
Web	d/t	$\leq 40\varepsilon$
Cold formed RHS—Flange	b/t	36.7ε
Web	d/t	$\leq 35\varepsilon$

$\varepsilon = (250/f_y)^{0.5}$

For HR RHS: $b = B - 3t$; $d = D - 3t$

For CF RHS: $b = B - 5t$; $d = D - 5t$

5.3 Behaviour of Compression Members

Before discussing the behaviour of compression members, it may be useful to know about the classification of compression members based on their length.

5.3.1 Long, Short, and Intermediate Compression Members

Compression members are sometimes classified as being long, short, or intermediate. A brief discussion about this classification is as follows:

Short compression members For very short compression members the failure stress will equal the yield stress and no buckling will occur. Note that for a compression member to fall into this classification, it has to be so short (for an initially straight column $L \leq 88.85r$, for $f_y = 250$ MPa) that it will not have any practical application.

Long compression members For these compression members, the Euler formula [see Section 5.4 and Eqn (5.3)] predicts the strength of long compression members very well, where the axial buckling stress remains below proportional limit. Such compression members will buckle elastically.

Intermediate length compression members For intermediate length compression members, some fibres would have yielded and some fibres will still be elastic. These compression members will fail both by yielding and buckling and their behaviour is said to be 'inelastic'. For the Euler formula Eqn (5.3) to be applicable to these compression members, it should be modified according to the reduced modulus concept or the tangent modulus concept (as is done in AISC code formula) to account for the presence of residual stresses. Now let us discuss the behaviour of short and slender columns.

5.3.2 Short Compression Members

Consider an axially compressed member of short length which is initially straight and made of material having the ideal rigid–plastic stress–strain relationship as shown in Fig. 5.3. At low values of external load P, there will be no visible deformation—neither lateral nor axial. Since P is applied at the centroid of the section, apart from possible localized effects at the ends of the member, all parts of the member will experience the same value of compressive stress $f_c = P/A$. Large deformation is possible only when f_c reaches the yield stress f_y. At this stage the member deforms axially. The value of the axial force at which this happens is termed as the 'squash load' and is given by

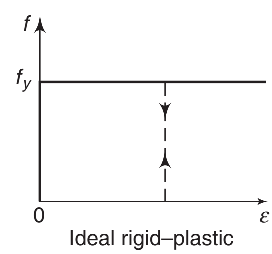


Fig. 5.3 Idealized material behaviour

$$P_y = f_y A \tag{5.1}$$

Typical laboratory compression test on the short length of the rolled section is often referred to as the *stub column test*. Since rolled steel sections have residual stresses, there will be non-uniform yielding of the cross section of the stub column. Thus, those fibres which contain residual compression have their effective yield point reduced; while those containing residual tension have theirs increased. However, in both cases, the squash load of the stub column can be achieved provided there is no local buckling. (Nethercot 2001).

5.3.3 Slender Compression Members

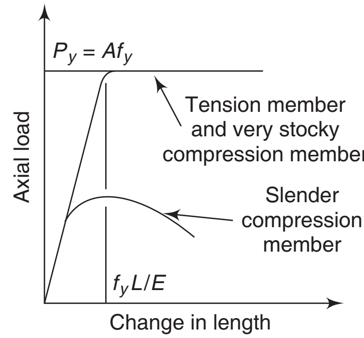


Fig. 5.4 Strengths of axially loaded members

As mentioned earlier, the strength of the compression member decreases as its length increases, in contrast to the axially loaded tension member whose strength is independent of its length. Thus, the compressive strength of a very slender member may be much less than its tensile strength, as shown in Fig. 5.4 (also see Fig. 5.6). This decrease in strength is due to the following parameters, which are often grouped under the heading of imperfections: the initial lack of straightness, accidental eccentricities of loading, residual stresses, and variation of material properties over the cross section. In order to understand the effect of these parameters, we should study the behaviour of ideal, straight pin-ended columns, which is discussed in the next section.

5.4 Elastic Buckling of Slender Compression Members

Though the first qualitative remarks on column strength and stability were given by Erone of Alexandria (75 B.C.) and similar descriptions of buckled columns by Leonardo Da Vinci (1452–1519), the theory of column buckling was first originated by Euler during 1744–1759 (Timoshenko 1953; Euler 1759). Euler considered an ideal column with the following attributes.

- Material is isotropic and homogenous and is assumed to be perfectly elastic.
- The column is initially straight and the load acts along the centroidal axis (i.e., no eccentricity of loads).
- Column has no imperfections.
- Column ends are hinged.

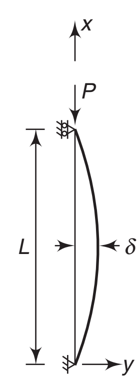
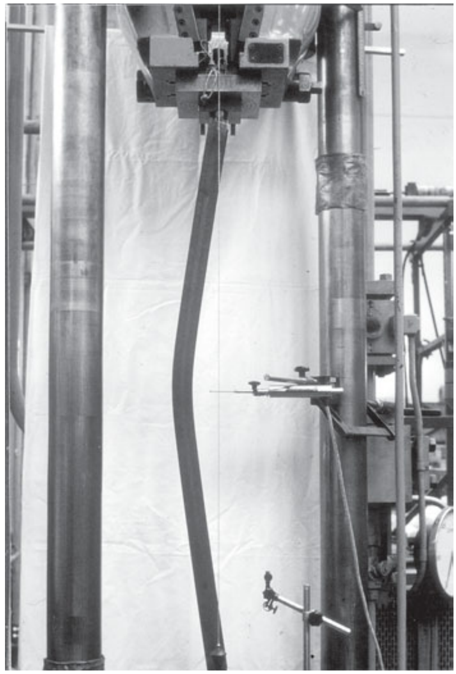


Fig. 5.5 Behaviour of a perfectly straight elastic pin-ended column

Such a column is also known as an *Euler column* and is shown in Fig. 5.5. A concentric load P is applied to the upper end of the member, which remains straight until buckling occurs, when it is slightly bent as shown in Fig. 5.5.

The Euler critical load for a column with both ends hinged may be derived as

$$P_{cr} = \pi^2 EI/L^2 \tag{5.2}$$



Buckling of a pin-ended column in a laboratory test
(Courtesy: Late Dr. W. Kurth of Germany)

Or in terms of average critical stress, using $I = A_g r^2$

$$f_{cr} = P_{cr}/A_g = \pi^2 E/\lambda^2 \tag{5.3}$$

where λ is the slenderness ratio defined by

$$\lambda = L/r \tag{5.4}$$

Note that the buckling phenomenon is associated with the stiffness of the member. A member with low stiffness will buckle early than one with high stiffness. Increasing member lengths causes reduction in stiffness. The stiffness of the member is strongly influenced by the amount and distribution of the material in the cross section of the column; the value of r reflects the way in which the material is distributed. Also note that any member will tend to buckle about the weak axis (associated with lesser ability to resist buckling).

Critical loads and critical stresses can be similarly found for struts having other end-restraint conditions. For example, if both ends are prevented from rotation as well as from lateral movement (fixed ends), the critical load may be $4\pi^2 EI/L^2$. In such cases, the slenderness ratio is defined as KL/r, where KL is the *effective length* of the member. This concept is discussed in Section 5.8.

Inspection of Eqn (5.3) indicates that the critical stress is inversely proportional to the slenderness ratio of the column and very large values of f_{cr} can be obtained by using $L/r \rightarrow 0$. However, as per the differential equation for bending, stress should be proportional to strain. Thus, the upper limit of validity is the proportional limit, which is often taken as $f_{cr} \rightarrow f_y$, though short columns may be loaded even into the strain-hardening range. Thus, one may write,

$$f_y = \pi^2 E / \lambda_p^2 \tag{5.5}$$

$$\text{Hence, } \lambda_p = \pi\sqrt{(E/f_y)}\text{ ; for } f_y = 250 \text{ MPa}, \lambda_p = 88.85 \tag{5.6}$$

Thus, the changeover from yielding to buckling failure occurs at point C (see Fig. 5.6), defined by the slenderness ratio λ_p. Figure 5.6(a) shows the strength curve for an axially loaded initially straight pin-ended column indicating the plastic yield (stocky columns) and elastic buckling (long column) behaviour. Figure 5.6(a) is often presented in a non-dimensional form as shown in Fig. 5.6(b).

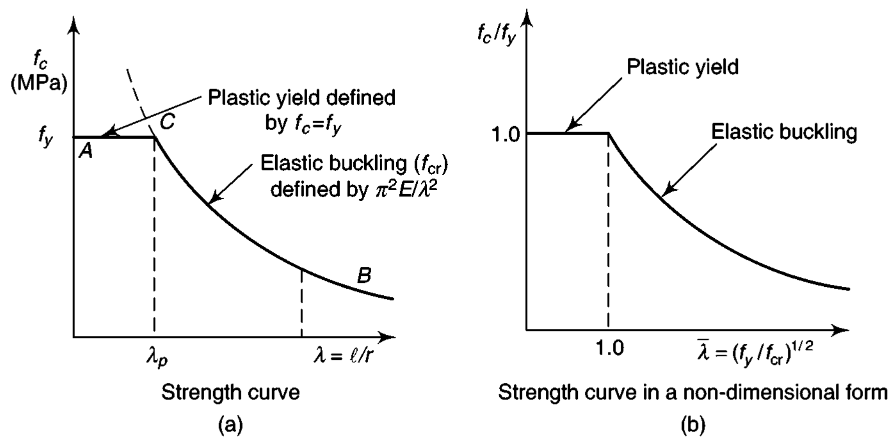


Fig. 5.6 Strength curve for an axially loaded initially straight pin-ended column

5.5 Behaviour of Real Compression Members

Euler's approach was generally ignored for design because test results did not agree with it (see Fig. 5.7). Test results included effects of initial crookedness of the member, accidental eccentricity of load, end restraint, local or lateral buckling, and residual stress. The effects of these parameters are discussed in Galambos (1938), Salmon and Johnson (1996), and Subramanian (2008). Note that columns are usually an integral part of a structure and as such cannot behave independently. Due to the imperfections, the load deflection curve of a real column will be much different than that of an ideal column.

5.6 Development of Multiple Column Curves

In real columns, all the effects mentioned in Section 5.5 occur simultaneously, i.e., out-of-straightness, eccentricity of loading, residual stresses, also depending on the material, lack of clearly defined yield point, and strain hardening. Note that

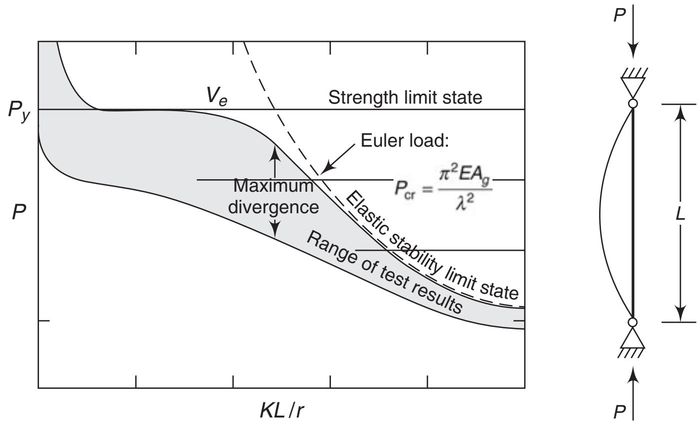


Fig. 5.7 Typical experimental results showing column strength vs slenderness ratio

only strain hardening tends to raise the column strength (that too only for short stocky columns), while the other effects lower the column strength for all or part of the slenderness ratio range.

Thus, the determination of the maximum strength of columns is a complicated process, often involving numerical integration, especially when initial imperfections and material non-linearities or residual stresses must be considered (see Allen & Bulson 1980—for the details of numerical integration techniques). These procedures are not suitable for design office use since they require lengthy calculations. Hence, simplified design formulae were developed and used in various design codes. The establishment of an acceptable single formula for the design stress had been a subject of controversy, since, as we have discussed, there are several parameters that influence the strength.

In the past, four basic methods were used to establish column design formulae, curves or charts (Galambos 1998).

1. Empirical formulae such as Merchant-Rankine formula
2. Formulae based on the yield limit state, e.g. Perry-Robertson formula
3. Formula based on tangent modulus theory, and
4. Formula based on maximum strength

In order to develop a rational basis for predicting the strength of real columns (which have initial imperfections, end restraints, eccentricity of loading, and residual stresses), the committee 8 on instability (headed by Prof. Beer from Austria) of the European Convention for Constructional Steel work (founded in the year 1955 and shortly known as ECCS) launched an extremely large series of experiments on axially loaded columns. Seven European countries were involved and about thousand buckling tests were performed (Sfintesco 1970).

Based on these results, the ECCS committee concluded that in order to represent the real strength of columns, five different curves have to be used, depending on different cross sections of columns. These curves are called the European multiple column curves.

Based on an extensive numerical study, Bjorhovde (1972) generated a set of 112 column curves for members for which measured residual stress distribution were available, assuming that the initial crookedness was of a sinusoidal shape having maximum amplitude of $1/1000^{th}$ of the column length and that the end restraint is zero. These curves were reduced to a set of three column curves and adopted by SSRC. These curves are known as SSRC column strength curves.

Several European countries have adopted the ECCS curves. Eurocode 3 has adopted a modified form of ECCS curves. The Canadian code has adopted the SSRC column curves 1 and 2 and uses the following equation (Galambos 1998).

$$(f_u/f_y) = 0.9A(1 + \bar{\lambda}^{2n})^{-(1/n)} \tag{5.7}$$

with $n = 2.24$ (welded members) and 1.34 (hot rolled members) for curves 1 and 2 respectively.

5.6.1 Multiple Column Curves in the IS Code

The Indian Code (IS 800) has adopted the multiple column curves, as shown in Fig. 5.8, which are similar to the curves given in the British Code BS 5950 (Part 1)-2000, which is based on the Perry–Robertson approach. The design compressive strength is given by

$$P_d = A_e f_{cd} \tag{5.8}$$

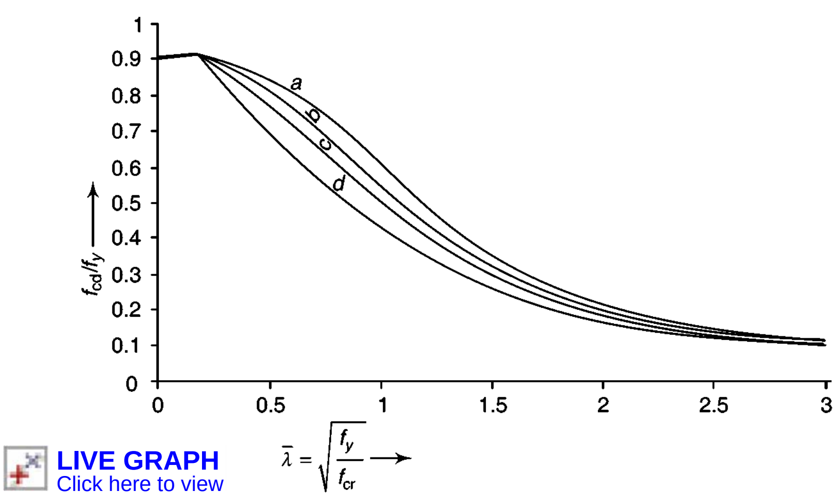


Fig. 5.8 Column buckling curves as per IS 800-2007

where A_e is the effective sectional area.

The design stress in compression f_{cd} is given by (substituting for γ_{m0})

$$f_{cd} = [(0.909\, f_y)/\{\phi + (\phi^2 - \bar{\lambda}^2)^{0.5}\}] = 0.909\, \chi f_y \le 0.909\, f_y \tag{5.9a}$$

where $\phi = 0.5[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2]$ (5.9b)

and $\bar{\lambda}$ is the non-dimensional effective slenderness ratio = $\sqrt{(f_y/f_{cr})}$, f_{cr} is the Euler buckling stress = $\pi^2 E/(KL/r)^2$, KL/r is the effective slenderness ratio, α is the imperfection factor (see Table 5.2), and χ is the stress reduction factor.

Table 5.2 Imperfection factor α

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

The design compressive stress f_{cd} for different buckling curves and effective slenderness ratio are given in Table 5.3 for f_y = 250 MPa. For other values of the yield stress, refer IS 800-2007. The classification of different sections under different buckling curves *a*, *b*, *c*, or *d* is given in Table 5.4.

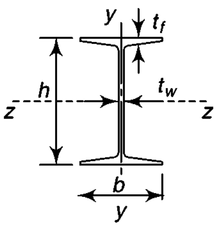
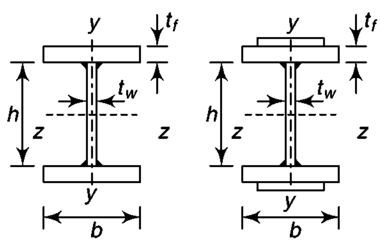
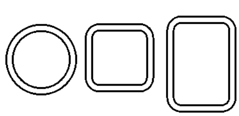
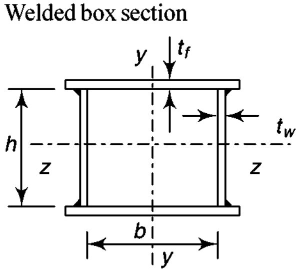
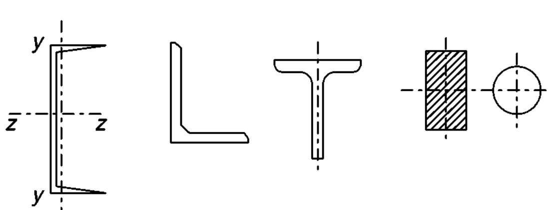
Table 5.3 Design compressive stress f_{cd} (MPa) for f_y = 250 MPa

$KL/r\downarrow$	Curve *a*	Curve *b*	Curve *c*	Curve *d*
10	227	227	227	227
20	226	225	224	223
30	220	216	211	204
40	213	206	198	185
50	205	194	183	167
60	195	181	168	150
70	182	166	152	133
80	167	150	136	118
90	149	134	121	105
100	132	118	107	92.6
110	115	104	94.6	82.1
120	101	91.7	83.7	73.0
130	88.3	81.0	74.3	65.2
140	77.8	71.8	66.2	58.4
150	68.9	64.0	59.2	52.6
160	61.4	57.3	53.3	47.5
170	55.0	51.5	48.1	43.1
180	49.5	46.5	43.6	39.3
190	44.7	42.2	39.7	35.9
200	40.7	38.5	36.3	33.0
210	37.1	35.2	33.3	30.4
220	34.0	32.3	30.6	28.0
230	31.2	29.8	28.3	26.0
240	28.8	27.5	26.2	24.1
250	26.6	25.5	24.3	22.5

5.7 Sections used for Compression Members

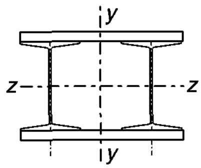
Though numerous sections may be selected to safely resist a compressive load in a given structure, from a practical viewpoint, the possible solutions are limited mainly due to the considerations of availability of sections, connection problems, and the type of structure in which they will be used. Figure 5.9 shows the possible configurations that may be used as compression members. From this figure it can be observed that the sections used as compression members are similar to tension

Table 5.4 Selection of buckling curve depending on cross section

Cross section	Limits		Buckling about axis	Buckling curve
Rolled I-sections 	$h/b > 1.2$:	$t_f \leq 40$ mm	z-z	*a*
			y-y	*b*
		40 mm $< t_f \leq 100$ mm	z-z	*b*
			y-y	*c*
	$h/b \leq 1.2$:	$t_f \leq 100$ mm	z-z	*b*
			y-y	*c*
		$t_f > 100$ mm	z-z	*d*
			y-y	*d*
Welded I-section 		$t_f \leq 40$ mm	z-z	*b*
			y-y	*c*
		$t_f > 40$ mm	z-z	*c*
			y-y	*d*
Hollow section 	Hot rolled	Any		*a*
	Cold formed	Any		*b*
Welded box section 	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*

(Contd)

(Contd)

Built-up member 	Any	*c*

members with some exceptions. These exceptions are caused by the fact that the strength of compression members vary inversely to the slenderness ratio [see Section 5.4 and Eqn (5.3)]. Thus, rods, bars, and plates are too slender to be used as compression members.

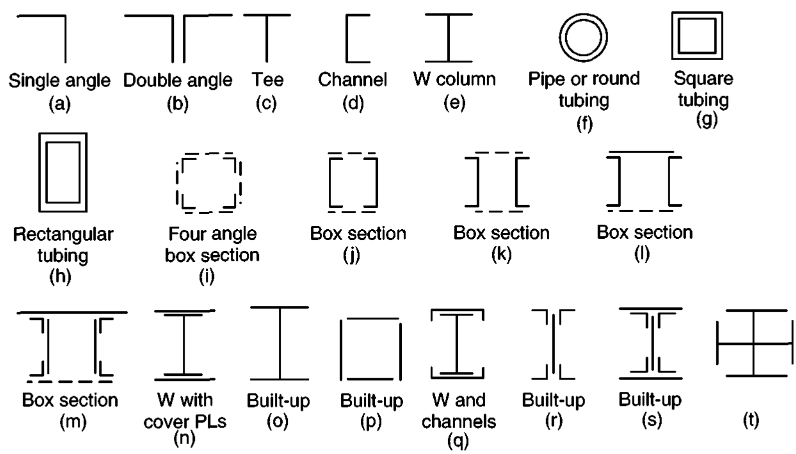


Fig. 5.9 Shapes of compression members

The choice of the sections in a particular situation depends on the following:

- Functional aspects of the structure
- Functional aspects of the member
- Easy connectivity to other members
- High radius of gyration

Table 5.5 gives some recommendations for the above aspects.

Table 5.5 Recommended structural shapes

Structure/member	Type of cross section
Small span roof trusses (up to 20 m span)	
(a) Rafters and bottom ties	Double angles (connected back-to-back), Ts, CHS, SHS, RHS
(b) Web members	Single angles, CHS, SHS, RHS
Medium and long span roofs	
(a) Rafters	4 angles and plate {Fig. 5.9(r)}, two channels with plates, box-section

(Contd)

(Contd)

(b) Web members	4 angles, 2 channels
Small towers (up to 20 m high)	
(a) Leg members	CHS, 2 angles (star formation)
(b) Bracings	CHS or angles
Towers about 150 m high	
(a) Bottom 40 m legs	4 angles (star formation)
(b) Bracings and horizontal members	2 angles (connected back-to-back)
Members of space frames	CHS, SHS
Columns of multi-storey buildings	I-sections, built-up I-sections, box sections (for heavy loads)
Columns of industrial buildings	I-sections, built-up channels or I-sections

5.8 Effective Length of Compression Members

The effect of end restraints on column strength is usually incorporated in the design by the concept of effective length. (It must be emphasized that the concept of effective lenght is only a simple and convenient short cut and the designer is not obliged to use it, if he/she knows a better, yet sufficiently simple solution.) Hence, in this section the effective length concept is first explained for idealized boundary conditions and then extended to cases where intermediate restraints are present. The Julian and Lawrence alignment charts are used in American and Canadian codes. Note that Wood's Curves (used in British and Indian codes), which are used to determine the effective lenght of columns of multi-storey buildings are also explained. The Indian code recommendations for the effective length factors of truss members are also discussed.

5.8.1 Effective Length for Idealized Boundary Conditions

Till now, the discussion in this chapter has been centered around pin-ended columns. Under ideal conditions, the boundary conditions of a column may be idealized in one of the following ways:

- Both ends pinned (as already discussed in Section 5.4)
- Both ends fixed
- One end fixed and the other end pinned
- One end fixed and the other end free

For all these conditions, the differential equations can be set up and the appropriate boundary conditions applied to get the following critical loads (Timoshenko & Gere 1961; Allen & Bulson 1980).

1. For column with both ends fixed:

$$P_{cr} = 4\pi^2 EI/L^2 = 4\pi^2 E/(L/r)^2 \tag{5.10}$$

2. For column with one end fixed and the other end pinned:

$$P_{cr} = 2\pi^2 E/(L/r)^2 \tag{5.11}$$

3. For columns with one end fixed and the other end free:

$$P_{cr} = \pi^2 E/[4(L/r)^2] \tag{5.12}$$

Using the length of the pin-ended column, as the basis for comparison, the critical load in the three cases mentioned earlier can be obtained by employing the concept of *effective length KL* where *K* is called the *effective length ratio* or effective length coefficient. (It is the ratio of the effective length to the unsupported length of the columns.) The value of *K* depends on the degree of rotational and translational restraints at the column ends. The *unsupported length L* is taken as the distance between lateral connections, or actual length in the case of a cantilever. In a conventional framed construction, *L* is taken as the clear distance between the floor and the shallower beam framing into the columns in each direction at the next higher floor level.

In other words, the *effective length* of a column in a given plane may be defined as the distance between the points of inflection (zero moment) in the buckled configuration of the column in that plane (see Fig. 5.10). Note that when there is relative translation at the ends of the column, the points of inflection may not lie within the member. In such a case, they may be located by extending the deflection curve beyond the column end(s) and by applying conditions of symmetry, as shown in Fig. 5.11. Thus, the effective length can also be defined as the length of an equivalent pin-ended column, having the same load-carrying capacity as the member under consideration. The smaller the effective length of a particular column, the smaller is its danger of lateral buckling and the greater its load carrying capacity. It must be recognized that it is very difficult to get perfectly fixed or perfectly hinged end conditions in practice.

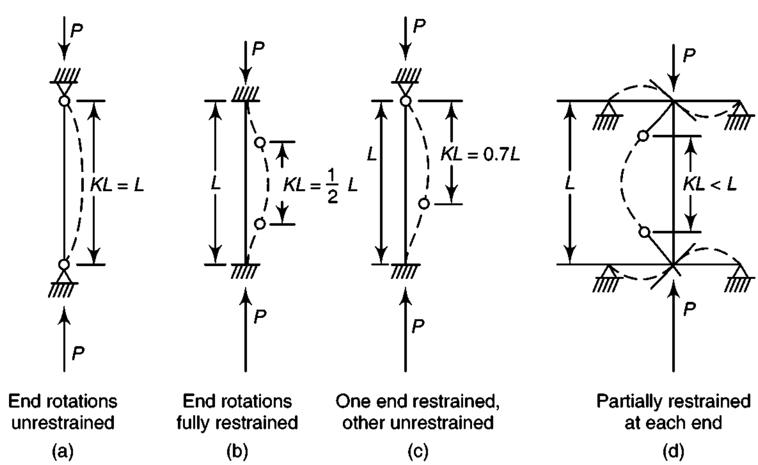


Fig. 5.10 Effective length *KL* when there is no joint translation at the ends

Using *KL*, Eqns (5.10), (5.11), and (5.12) can be written as

$$P_{cr} = \pi^2 EI/(KL/r)^2 \tag{5.13}$$

where $K = 1.0$ for columns with both ends pinned,
$K = 0.5$ for columns with both ends fixed,
$K = 0.707$ for columns with one end fixed and the other end pinned,
$K = 2.0$ for columns with one end fixed and the other end free,
$K \leq 1.0$ for columns partially restrained at each end, and
$K \geq 2.0$ for columns with one end unstrained and the other end rotation partially restrained.

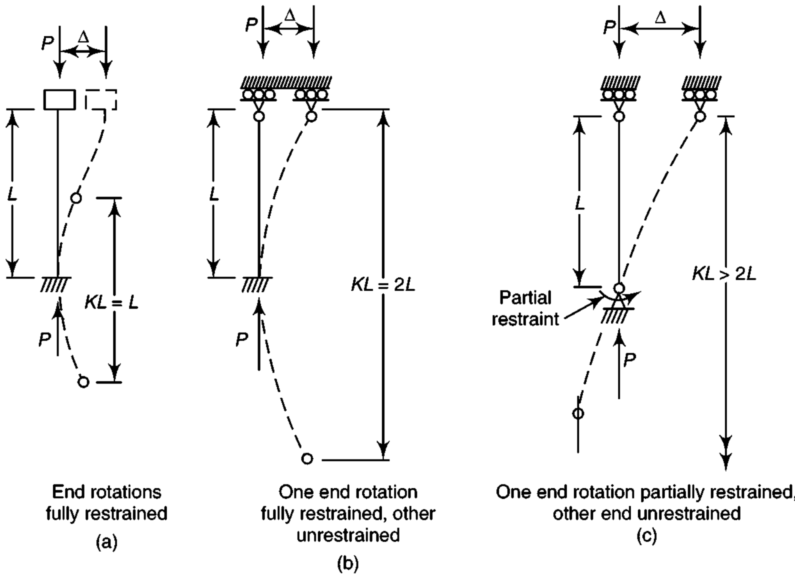


Fig. 5.11 Effective length KL when there is joint translation at the ends

Thus, the effective length is important in design calculations because the buckling load is inversely proportional to the square of the effective length.

Approximate values for effective lengths, which can be used in design, are given in IS 800 and are shown in Fig. 5.12 along with the theoretical K values {as given by Eqn (5.13)}. Note that the values given in the code are slightly more than the theoretical values given in (a), (b), and (c) of Fig. 5.12. It is because, fully rigid end restraints are difficult to achieve in practice; partial end restraints are much

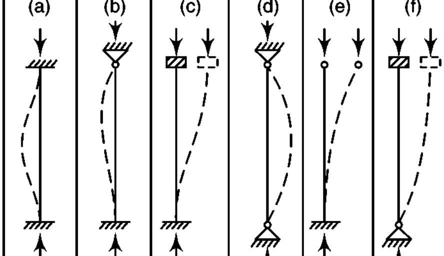
	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended K value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.0	2.0
End condition code		Rotation fixed, translation fixed Rotation free, translation fixed Rotation fixed, translation free Rotation free, translation free				

Fig. 5.12 Effective-length factors K for centrally loaded columns with various end conditions

more common in practice. For example, at the base, shown fixed for conditions (a), (b), (c), and (e) in Fig. 5.12, full fixity can be assumed only when the column is anchored securely to a footing, for which rotation is negligible. (Individual footings placed on compressible soils, will rotate due to any slight moment in the column.) Similarly, restraint conditions, (a), (c), and (f) at the top can be achieved only when the top of the column is framed integrally to a girder, which may be many times stiffer than the column. Condition (c), as shown in Fig. 5.12 is applicable to columns supporting heavy loads at the top (e.g., columns supporting storage tanks).

5.8.2 Intermediate Restraints and Effective Lengths in Different Planes

In the previous sections, it was assumed that the compression member was supported only at its ends as shown in Fig. 5.12. If the member has an additional lateral support (bracing) which prevents it from deflecting at its centre, as shown in Fig. 5.13, the elastic buckling load is increased by a factor of four (i.e., the value of P_{cr} will be $4\pi^2EI/L^2$).

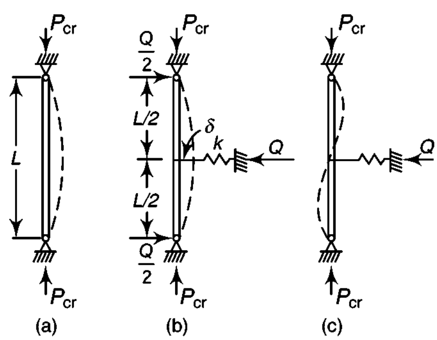


Fig. 5.13 Compression member with an elastic intermediate restraint

This restraint need not be completely rigid, but may be elastic, provided its stiffness exceeds a certain minimum value.

Many specifications and codes suggest a rule-of-thumb of using a bracing having a strength of 2 to 2.5% of the compressive strength of the compression elements being braced. This seems to be a conservative alternative to a rigorous analytical study (Lay & Galambos 1966). Such a clause is also available in the code (IS 800 clause 7.6.6.1).

When such restraints are provided, the buckling behaviour will be different about the two column axes. For example, consider a pin-ended column which is braced about the minor axis against lateral movement (but not rotationally restrained) at a spacing of $L/4$. Now the minor axis buckling mode will be such that $K = 1/4$. If there is no bracing in the major axis, the effective length for buckling about major axis will be L. Hence, the slenderness ratio about the major and minor axis will be L/r_z and $L/4r_y$ respectively. For ISHB sections $r_z < 4r_y$ and hence the major axis slenderness ratio will be greater, giving lower value of critical load and failure will occur by major axis buckling. If this is not the case, checks have to be carried out about both the axes.

In many situations, it may not be possible to use bracing in more than one plane. For example, columns are often used with walls, where these walls can serve as lateral bracing in one plane; in other planes for functional reasons it may not be possible to provide bracings. In such situations, non-symmetric members could be employed advantageously with their strong axis oriented in the out-of-braced-plane buckling mode and their weak axis in the in-plane mode. A great level of efficiency may be obtained by keeping the buckling load about one axis exactly equal to the buckling load about the other axis. The number of braces required to achieve this may be found out by the relationship $r_z/r_y = L_z/L_y$. (This relationship may also be useful in the design of built-up columns—see Example 5.14.)

5.8.3 Columns in Multi-storey or Framed Buildings

As mentioned already, isolated columns are rare and they normally form a part of any framework. Moreover, their end conditions are influenced by the members to which they are connected. The more accurate determination of K for such a compression member as part of any framework requires the application of methods of indeterminate structural analysis, modified to take into account the effects of axial load and inelastic behaviour on the rigidity of the members. These procedures are not directly applicable to routine design and hence simple models are often used to determine the effective length factor for framed members. In such simplified approaches, a distinction is always made about sway (unbraced) and non-sway (braced) frames because of their distinct buckling modes (see Fig. 5.14). In non-sway (braced) frames, the columns buckle in single curvature and hence their effective length factor will always be less than unity; whereas the columns in sway frames buckle in double curvature and hence their effective length factor will always be greater than unity.

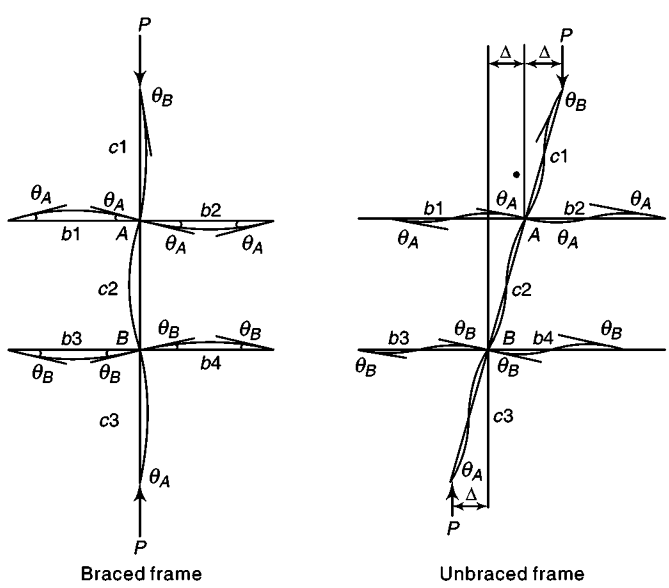


Fig. 5.14 Sub-assemblage model for braced and unbraced frames

5.8.3.1 Braced Frames

A braced frame is considered to be a frame in which lateral stability is provided by diagonal bracing, shear walls, in-fill walls, or by any other equivalent means. It is not necessary that every bay of a multi-storey building has to be braced, since the unbraced bays are restrained by those that are braced. The code (IS 800) gives criteria for considering the frame as a non-sway frame in clause 4.1.2. The ACI code (ACI 318-2008) suggests the calculation of the stability index Q, which is given by

$$Q = [(\Sigma P_u)/h_s](\Delta_u/H_u) \tag{5.14}$$

where ΣP_u is the sum of axial loads on all columns in the storey, h_s is the height of the storey, Δ_u is the elastic first-order lateral deflection of the storey, and H_u is the total lateral force acting on the storey. If the value of Q is less than 0.05, the column may be assumed as braced.

In the absence of bracing elements, the *lateral flexibility* measure of the storey Δ_u/H_u (storey drift per unit storey shear) may be taken for a typical intermediate storey as (Taranath 1988)

$$(\Delta_u/H_u) = h_s^2/[(12E\ \Sigma(I_c/h_s)] + h_s^2/(12E\Sigma I_b/L_b) \tag{5.15}$$

where $\Sigma I_c/h_s$ is the sum of ratios of the second moment of area to the height of all columns in the storey in the plane under consideration, $\Sigma I_b/L_b$ is the sum of ratios of the second moment of the area to the span of all floor beams in the storey in the plane under consideration, and E is Young's modulus of elasticity of steel.

The application of this concept is demonstrated in Example 5.6. Note that Eqn (5.15) does not consider the effect of infills, bracings, or shear walls. However, it gives an idea of the lateral flexibility of the storey under consideration.

Various investigators have provided charts to permit easy determination of effective lengths for commonly encountered situations. Two simplified approaches are often recommended in the codes of practices. The American and Canadian codes follow the alignment charts originally developed by Julian and Lawrence in 1959 (Kavanagh 1962). The British and Indian codes follow the charts originally developed by Wood (1974). The Australian code also gives a chart similar to that of the Wood's chart.

5.8.3.2 Effective length as per IS 800

The code (IS 800), gives the following equations for the effective length factor K, based on Wood's curves—see Annex D of the code for these curves.

For non-sway frames (braced frames):

$$K = \frac{[1 + 0.145(\beta_1 + \beta_2) - 0.265\beta_1\beta_2]}{[2 - 0.364(\beta_1 + \beta_2) - 0.247\beta_1\beta_2]} \tag{5.16a}$$

For sway frames (moment-resisting frames):

$$K = \left\{ \frac{[1 - 0.2(\beta_1 + \beta_2) - 0.12\beta_1\beta_2]}{[1 - 0.8(\beta_1 + \beta_2) + 0.6\beta_1\beta_2]} \right\}^{0.5} \tag{5.16b}$$

with $\beta_i = \Sigma K_c / (\Sigma K_c + K_b)$ (5.16c)

where K_c and K_b are the effective flexural stiffness of the columns or beams meeting at the joint at the ends of the columns and rigidly connected at the joints.

$$K_c \text{ or } K_b = C(I/L)$$

where I is the moment of inertia about an axis perpendicular to the plan of the frame, L is the length of the member, taken as centre-to-centre distance of the intersecting member, and C is the connection factor as shown in Table 5.6.

Table 5.6 Connection factor C

Fixity conditions at far end	Connection factor C	
	Braced frame	Unbraced frame
Pinned	$1.5(1 - \bar{n})$	$0.5(1 - \bar{n})$
Rigidly connected column	$1.0(1 - \bar{n})$	$1.0(1 - 0.2\bar{n})$
Fixed	$2.0(1 - 0.4\bar{n})$	$0.67(1 - 0.4\bar{n})$

Note $\bar{n} = P/P_{cr}$, where P is the applied load and P_{cr} is the elastic buckling load $= \pi^2 EI/(KL)^2$

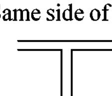
Note that for calculating C we need the effective length and hence the determination of effective length is an interactive process. Initially, we can assume $K = 1$ for calculating the value of C.

A review of the IS code provisions for effective length of columns in frames is provided by Dafedar et al. (2001).

5.8.4 Compression Members in Trusses

The general recommendations for effective length of compression members in trusses are given in Table 5.7 (see also clauses 7.2.4 and 7.5 of the code).

Table 5.7 Effective length for angle struts as per IS 800

Type	End connection	Effective length
Discontinuous single angle	(a) One bolt or rivet	Distance between the centre of end fastening (as per IS 800-1984)
	(b) Two or more bolts or Rivets or equivalent Welding	0.85 of distance between node points (as per IS 800-1984)
Discontinuous double angle, stitched together by bolts or welding at regular intervals	(a) Same side of gusset 	
	(i) One bolt or rivet	Distance between centre of end fastenings
	(ii) Two bolts or rivets or equivalent welding	0.7–0.85 of distance between nodes

(Contd)

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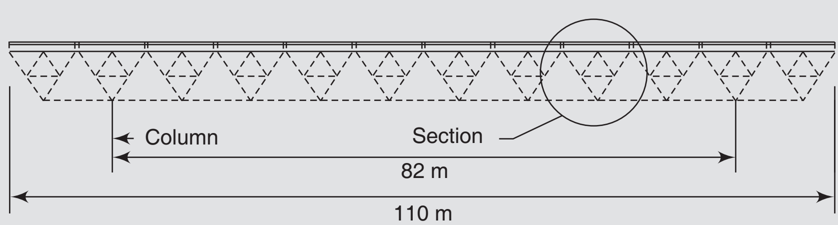
	(b) Both sides of gusset	
	(i) One bolt or rivet	Distance between centre of end fastenings
	(ii) Two bolts or rivets or equivalent welding	0.7-0.85 of distance between nodes
	In a plane perpendicular to that of end gusset —for both (a) and (b)	Distance between centre of end fastening
Continuous angles (e.g., top and bottom chords of trusses, tower legs)	(a) Continuous	0.7–1.0 of distance between nodes
	(b) In a plane perpendicular to the plane of truss	Distance between centres of nodes

The effective length factor of single angle discontinuous struts connected by single or more bolts/rivets or equivalent welding may be determined as discussed in Section 5.9.1.

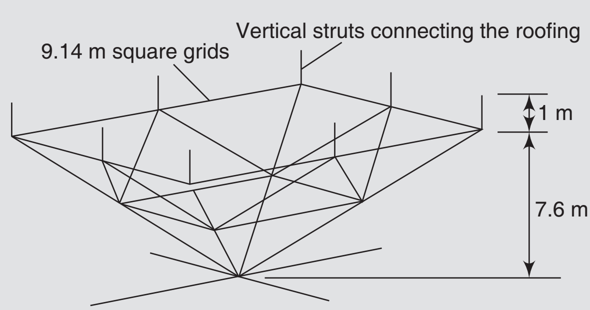
For the design of tapered columns, column cross sections with one axis of symmetry (for example, channel columns), and columns with no axis of symmetry, specialized literature should be consulted.

Hartford Civil Centre Roof Collapse

The Hartford Civil Centre coliseum, Connecticut, USA, was completed in 1973. The space frame roof structure was 7.6 m high and covered 110 m by 91 m, with clear spans of 64 to 82 m.



(a) Elevation



(b) Section

(c) View of the collapsed roof
Space frame roof of Hartford Civil Centre Coliseum roof

Fig. CS 1

The space frame consisted of warren trusses with triangular bracing between top and bottom chords (see Fig. CS 1). The two main layers were arranged in 9.14 m by 9.14 m grids composed of horizontal steel bars. The 9.14 m diagonal bars connected the nodes of the upper and lower layers, and in turn, were braced by horizontal bars at a middle layer (see Fig. CS 1). Struts 1 m long attached to the space truss were used to support the 75 mm wood fibre composition roofing.

On January 17, 1978, at 4:15 a.m., the roof crashed down 25.2 m to the floor, due to a large snow storm. Luckily it was empty by the time of the collapse, and no one was hurt. Though there were several causes for the collapse, the main cause was the relatively minor changes in the connections between steel components, i.e., the fabrication deviating from design. A few centimeters shift of the fabricated connection, cut down the axial force capacity to less than tenth of the design value! Some angle sections found at the wreckage were found to have failed in block shear. Epstein and Thacker, in 1991, used finite element analysis and found that block shear was the mode of failure for these angles. This study also established the difference in behaviour of coped beams (where the load is applied to the connection in the plane of the web, which also is the block shear plane) and angles (where the load is applied eccentric to the failure plane).

In addition, the Hartford Civil Centre coliseum roof design was extremely susceptible to the torsional buckling of compression members which, as a mode of failure, was not considered by the computer analysis used by the designers. Had the designers chosen tubular or even I sections, instead of the cruciform section adopted in the roof members, the failure might have been averted (the four steel angles forming the cruciform cross-section has much smaller radius of gyration than tubes or I-sections, and hence not efficient in resisting compressive loads). The failure also showed that computer software should be used only as a software tool, and not as a substitute for sound engineering experience and judgement.

References:

1. Smith, E. and Epstein. H. (1980). Hartford Coliseum Roof Collapse: Structural Collapse and Lessons Learned. Civil Engineering, ASCE, April, pp. 59–62.
2. Epstein, H.I. and Thacker, B. (1991) Effect of bolt stagger for block shear tension failures in angles, Computers and Structures, V.39,N.5, pp. 571–76.
3. Epstein, H.I. and Aleksiewicz , L.J. (2008) Block shear equations revisted…. Again, Engineering Journal, AISC, V.45, No.1, First Quarter, pp.5–12.

5.9 Single Angle Struts

Angles are perhaps the most basic and widely used of all rolled structural steel shapes. There is a wide range of sizes available and end conditions are relatively simple. Single angles are commonly used in many applications such as web members in steel joists and trusses, members of latticed transmission towers and communication structures, elements of built-up columns, and bracing members. In roof trusses, the single angle web members are often connected by one leg (thus introducing eccentricity with respect to the centroid of the cross sections) on one side of the chords and sometimes alternatively on opposite sides of T-sections as shown in Fig. 5.15. In towers, the bracing members meeting at a joint are connected to the opposite sides of the leg member, in order to reduce eccentricity as well as to reduce (or eliminate) gusset plates. The ease with which connections can be made contributes to the popularity of their use.

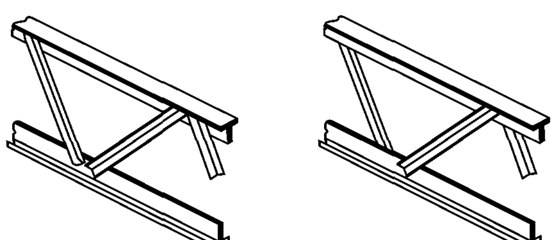


Fig. 5.15 Web members in trusses: (a) same sides and (b) opposite sides

Due to the asymmetry of the angle cross section, the determination of the compression capacity under eccentric loading along with end restraints is complex. Many researchers have studied the load carrying capacity of single angle members (e.g., Madugula & Kennedy 1985; Woolcock & Kitipornchai 1986; Kitipornchai & Lee 1986; Bathon et al. 1993; Adluri & Madugula, 1996). A review of experimental and analytical research on angle members is provided by Galambos (1998).

Angle members loaded through the centroid by a compressive axial force will buckle in flexural buckling about the minor principal axis of the cross section or in a torsional-flexural mode. When the width-to-thickness ratio of the legs of the angle are larger, the greater will be the possibility of torsional or flexural-torsional buckling being the controlling limit states. Note that the end restraint can significantly increase the ultimate strength of single angles of higher slenderness ratios, whereas it may weaken the ultimate strength in the lower slenderness ratio ranges. One of the most difficult tasks for designers is to judge or determine the end restraint and eccentricity condition for their specific application. Failure to consider the end restraints may lead to uneconomical designs whereas ignoring the end eccentricity may result in an unsafe design.

An empirical approach is adopted in ASCE Standard 10-1992, ASCE Manual 52-1988, ECCS recommendations-1985, and BS 5950-2000, to include the effects of end restraints and joint eccentricities by modifying the member's effective slenderness ratio. For example, in BS-5950, for single-angle struts connected to a gusset or directly to another member at each end by two or more fasteners in line along the angle or by equivalent welded connection, the slenderness ratio L/r should not be less than

$$0.85L/r_v \quad \text{or} \quad 0.7L/r_a + 30 \tag{5.17}$$

where r_v is the radius of gyration about the minor principal axis and r_a is the radius of gyration about the axis parallel to the plane of the gusset or the supporting member. Consideration of the rotational restraint permits the use of an effective length factor 0.85 while the end eccentricity effect is taken into account in the out-of-plane buckling. If a single fastener is used at each ends, the L/r should be taken as not less than

$$L/r_v \quad \text{or} \quad 0.7L/r_a + 30 \tag{5.18}$$

Thus, the rotational restraint effect is ignored for this connection.

The complete treatment of single angles is outside the scope of this book and the interested reader may refer to Galambos (1998). A separate publication is available from AISC for the design of single angle members subjected to compression bending and beam-column action (AISC-1993), though these provisions have been integrated in the current version of the code (AISC-360, 2005). Based on an inelastic analysis of steel angle columns with residual stresses Adluri and Madugula (1996) developed an average column curve for steel angles (similar to column c of Fig. 5.8).

5.9.1 Indian Code Provisions

The Indian code (IS 800) suggests that when the single angle is loaded concentrically in compression, the design strength may be evaluated using Fig. 5.8, choosing class c curve. When the single angle is loaded eccentrically through one of its legs, the flexural torsional buckling strength may be evaluated using an equivalent slenderness ratio λ_e given by

$$\lambda_e = \sqrt{[k_1 + k_2\lambda_{vv}^2 + k_3\lambda_\phi^2]} \tag{5.19a}$$

where k_1, k_2, k_3 are constants depending on end conditions as shown in Table 5.8.

$$\lambda_{vv} = (L/r_{vv})/\{88.86\varepsilon\} \tag{5.19b}$$

and

$$\lambda_{\phi} = (b_1 + b_2)/\{88.86\varepsilon \times 2t\} \tag{5.19c}$$

where L is the unsupported length of the member, r_{vv} is the radius of gyration about the minor axis, b_1 and b_2 are the widths of the two legs of the angle, t is the thickness of the leg, and ε is the yield stress ratio $(250/f_y)^{0.5}$.

Table 5.8 Constants k_1, k_2, and k_3

No. of bolts at the end of member	Gusset/connecting member fixity*	k_1	k_2	k_3
≥ 2	fixed	0.20	0.35	20
1	hinged	1.25	0.50	60

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

Equations (5.19a) to (5.19c) are based on the research conducted at the Indian Institute of Technology, Madras.

5.10 Design of Compresssion Members

The strength of a compression member is based on its gross area A_g (for slender cross section, A_{eff} should be used). The strength is always a function of the effective slenderness ratio KL/r, and for short columns the yield stress f_y of the steel. Since the radius of gyration r depends on the section selected, the design of compression members is an iterative process, unless column load tables are available (see Appendix D). The usual design procedure involves the following steps.

1. The axial force in the member is determined by a rational frame analysis, or by statics for statically determinate structures. The factored load P_u is determined by summing up the specified loads multiplied by the appropriate partial load factors γ_f.
2. Select a trial section. Note that the width/thickness limitations as given in Table 4.3 to prevent local buckling must be satisfied (most of the rolled sections satisfy the width-to-thickness ratios specified in Table 4.3). If it is not satisfied and a slender section is chosen, the reduced effective area A_{eff} should be used in the calculation. The trial section may be chosen by making initial guesses for A_{eff}/A, and f_{cd} (say 0.4–0.6f_y) and calculating the target area A.
   The following member sizes may be used as a trial section:
   (a) *Single angle size* 1/30 of the length of compression member
   (b) *Double angle size* 1/35 of the length of compression member
   (c) *Circular hollow section* diameter = 1/40 of length
   The slenderness ratios as given in Table 5.9 will help the designer to choose the trail sections.

Table 5.9 Slenderness ratios to be assumed while selecting the trial sections

Type of member	Slenderness ratio (L/r)
Single angles	100–150
CHS, SHS, RHS	90–110
Single channels	90–150
Double angles	80–120
Double channels	40–80
Single I-section	80–150
Double I-sections	30–60

3. Compute KL/r for the section selected. The computed value of KL/r should be within the maximum limiting value given in Table 5.10. Using Fig. 5.8 and Tables 5.3 and 5.4 compute f_{cd} and the design strength $P_d = Af_{cd}$.

Table 5.10 Maximum slenderness ratio of compression members

Type of Member	KL/r
Carrying loads resulting from dead loads and superimposed loads	180
Carrying loads resulting from wind and seismic loads only, provided the deformation of such a member does not adversely affect the stress in any part of the structure	250
Normally acting as a tie in a roof truss or a bracing system but subject to possible reversal of stress resulting from the action of wind or seismic forces	350
Lacing bars in columns	145
Elements (components) in built-up sections	50

4. Compare P_d with P_u. When the strength provided does not exceed the strength required by more than a few per cent, the design would be acceptable; otherwise repeat steps 2 through 4.

5.10.1 Limiting Slenderness Ratio

The maximum slenderness ratio of compression members under axial load is limited by the code as shown in Table 5.10.

The limit of 180 may be taken as normally applicable to primary members in compression, such as columns, compound column sections, etc. The limit of 250 may be applied to secondary members, which themselves do not support any structure, but form a part of one. The limit of 350 may be applied to bracing systems, which lend rigidity to a structure, but by themselves carry only nominal loads.

It may be of interest to note that the latest version of AISC code (AISC: 360-2005) does not specify any upper limit on the slenderness ratio, though the commentary to this code recommends an upper limit of 200. The upper limits provided are based on professional judgement and practical considerations such as economics, ease of handling, and care required to minimize any inadvertent damage during fabrication, transport, and erection.

5.11 Built-up Compression Members

For large loads and for efficient use of material, *built-up columns* (also called as *combined columns* or *open-web columns*) are often used. They are generally made up of two or more individual sections such as angles, channels, or I-sections and properly connected along their length by lacing or battening so that they act together as a single unit. Such laced combined compression members are often used in bridge trusses. According to the type of connection between the chords, built-up members may be classified as follows:

- Laced members {Fig. 5.16(a)}
- Struts with batten plates {Fig. 5.16(b)}
- Battened struts {Fig. 5.16(c)}
- Members with perforated cover plates {Fig. 5.16(f)}

In general, such struts can be considered either as simple or built-up struts depending on the plane of bending. A strut having a cross section as shown in Fig. 5.16(d), for example, must be considered as a simple strut if it bends about the z-axis and considered as a built-up strut if it bends about the y-axis (Ballio & Mazzolani 1983). Struts having cross sections of the type shown in Fig. 5.16(e) behave as built-up struts both in z and y directions.

The effects of shear in built-up columns sets apart the design of these members from that of other columns. The importance of designing the elements connecting the main longitudinal members for shear was tragically demonstrated by the failure of the first Quebec Bridge in Canada during construction in 1907 (Galambos 1998). It has been found that about three fourths of the early failure of laced columns resulted from local rather than general column failure. Moreover, the critical load for a built-up column is less than that of a comparable solid column because the effect of shear on deflections is much greater for the former (Galambos 1998). The shear in column may be due to the following:

1. Lateral loads from wind, earthquake, gravity, or other causes
2. The slope of column with respect to the line of thrust due both to un-intentional initial curvature and the increased curvature during buckling
3. The end eccentricity of the load due to either end connections or fabrication imperfections

The slope effect is most important for slender columns and the eccentricity effect for short columns. Lin et al. (1970) suggested a shear flexibility factor μ, using which the equivalent slenderness λ_e can be computed and using curve c of Fig. 5.8 the design stress can be computed (for more details see Lin et al. 1970; Galambos 1998).

For columns with batten plates, the strut may be designed as a single integral member with a slenderness given by (Bleich 1952)

$$\lambda_e = \sqrt{[\lambda_m^2 + \lambda_c^2(\pi^2/12)]} \tag{5.20}$$

With the limitations $\lambda_e \not> 50$ and $\lambda_e \not< 1.4\,\lambda_c$ where $\lambda_m = L/r_{\min}$ = strut slenderness, $\lambda_c = L_o/r_o$ = local chord slenderness between one batten plate and the next, and L_o is the centre-to-centre distance of batten plate.

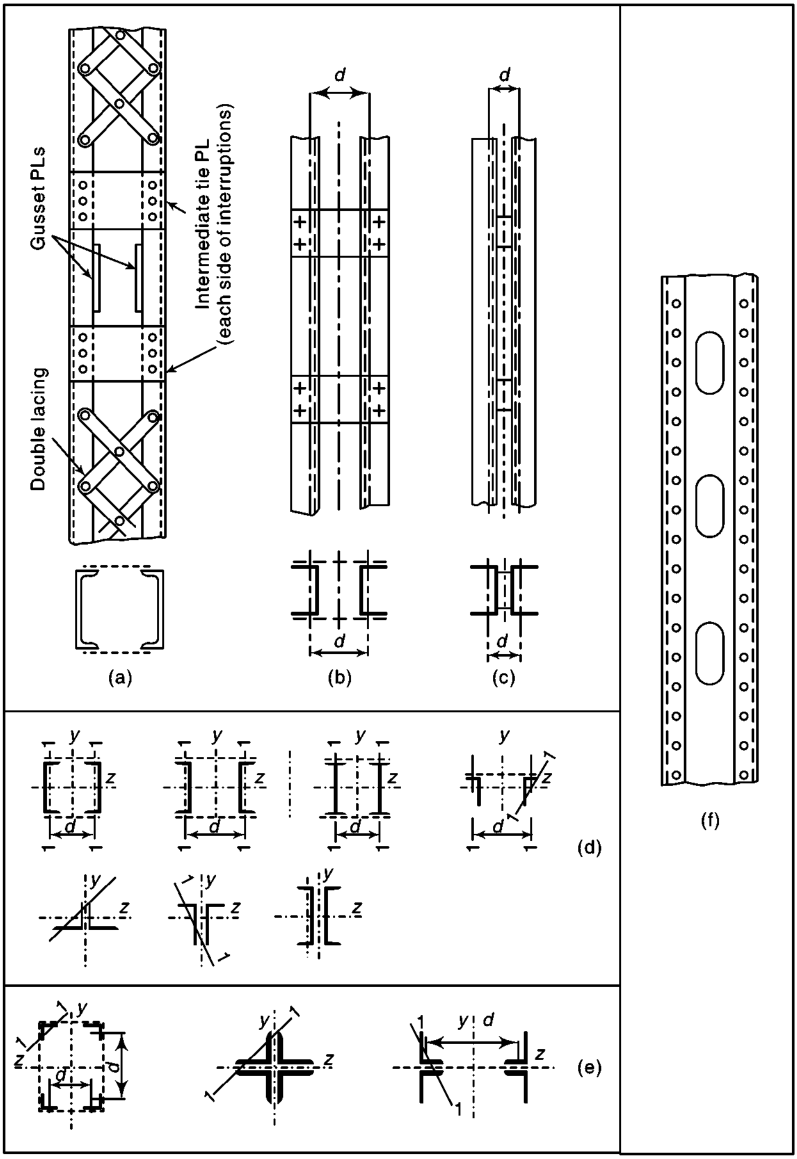


Fig. 5.16 Built-up columns

The most commonly adopted lacing systems are shown in Fig. 5.17. The simplest form of lacing consists of single bars connecting the two components [Fig. 5.17(b)]. The double lacing [Fig. 5.17(d)] is sometimes considered preferable, although a well-designed single lacing is equally effective. In single or double lacing systems, cross members perpendicular to the longitudinal axis of the strut should not be used [see Figs 5.17(c) and (e)]. The 'accordion' like action of the lacing system without cross members permits the lateral-expansion of the column. The introduction of cross members prevents the lateral expansion and thus forces the lacing bar to

Built-up column in a portal frame, Mumbai (Note the stubs in the column, which carry a small crane).

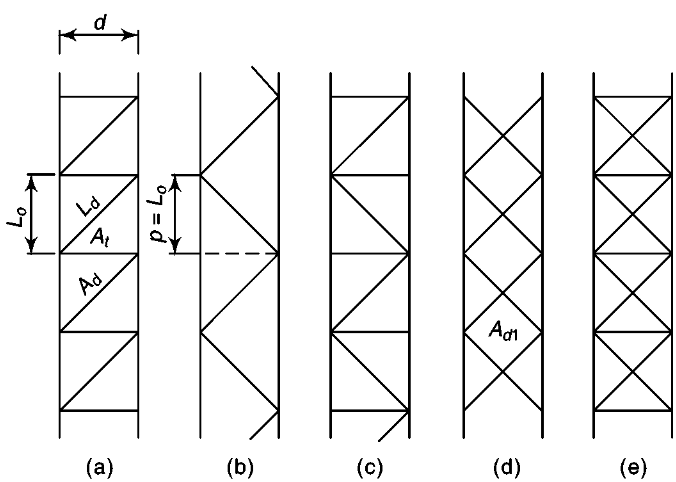


Fig. 5.17 Commonly adopted lacing systems

share the axial load on the strut. Note that the lacing bars and batten plates are not designed as load carrying elements. Their function is primarily to hold the main component members of the built-up column in their relative position and equalize the stress distribution in them. At the ends and at intermediate points where it is necessary to interrupt the lacing (for example, to admit gusset plates), the open sides are connected with *tie plates* (also called batten plates or stay plates). Tie plates are also provided at the top and bottom of the column (see Fig. 5.18).

It should to be noted that the battened columns have the least resistance to shear compared to columns with lacings and perforated plates, and may experience an appreciable reduction in strength. Hence, they are not generally used in the United States. Columns with perforated plates require no special considerations

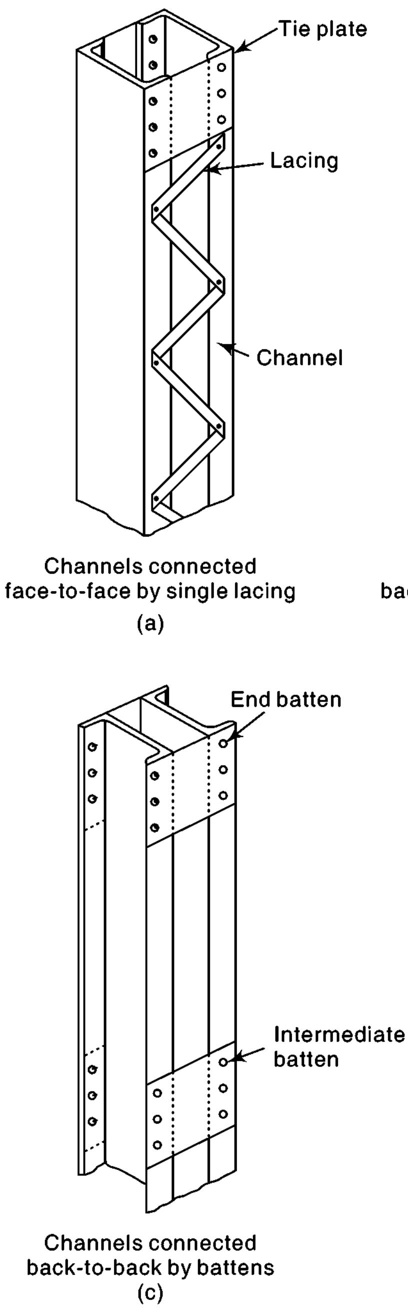


Fig. 5.18 Isometric view of built-up columns

for shear effects. These cover plates contain perforations spaced axially, which afford access for welding or painting. After the advent of automatic cutting machines, the production of such perforated plates have become simpler. Hence, they are used extensively in USA. They result in reduction of fabrication and maintenance cost and offer superior stiffness and straightness. At present, they are not used in India and interested readers may refer to Salmon and Johnson (1996) for the design of built-up columns with perforated plates.

The effective slenderness of laced struts of the types shown in Figs 5.17(c), (d), and (e) (with two chords connected by lacings) may be obtained by using the following equation (Ballio & Mazzolani 1983)

$$\lambda_{eq} = \sqrt{[\lambda^2 + \pi^2(A/A_d)L_d^3/(L_o d^2)]} \tag{5.21}$$

where λ is the strut slenderness = L/r_{min}, A is the overall cross-sectional area = $2A_1$, A_1 is the area of the individual chord, A_d is the cross-sectional area of the diagonal lacing {= $2A_{d1}$ for Figs 5.17(a) and (d)}, L_d is the length of the diagonal lacing, d is the distance between the centroid of the chords, and L_o is the chord length between the two successive joints.

Equation (5.21) is also applicable to the laced strut type shown in Fig. 5.17(b), provided L_o is replaced by p.

5.11.1 Rules Specified in the Indian Code

In most of the codes, latticed members are designed and proportioned according to detailed empirical rules, most of which are related to local buckling requirements. In the Indian code (IS 800 : 2007), the following rules are given.

5.11.1.1 Lacings

(a) The radius of gyration of the combined column about the axis perpendicular to the plane of lacing should be greater than the radius of gyration about the axis parallel to the plane of lacing [see Fig. 5.19(a)].

(b) Lacing system should be uniform throughout the length of the column.

(c) Single [Fig. 5.19(b)] and double laced [Fig. 5.19(c)] systems should not be provided on the opposite sides of the same member. Similarly lacings and battens should not be provided on opposite sides of the same member.

(d) Single laced system on opposite sides of the main component shall be in the same direction viewed from either side so that one is the shadow of the other. Thus, the lacing as shown in Fig. 5.19(b), for face *cd*, is not recommended.

(e) The lacing shall be designed to resist a total transverse shear V_t at any point in the member, equal to 2.5% of the axial force in the member; and this shear shall be divided among the lacing systems in parallel planes.

(f) The lacings in addition should be designed to resist any shear due to bending moment or lateral load on the member.

(g) The slenderness ratio of lacing shall not exceed 145.

(h) The effective length shall be taken as the length between inner end bolts/rivets of the bar for single lacings and 0.7 times the length for double lacings effectively connected at intersections. For welded bars, the effective length is taken as 0.7 times the distance between the inner ends of the welds connecting the single bars to the members.

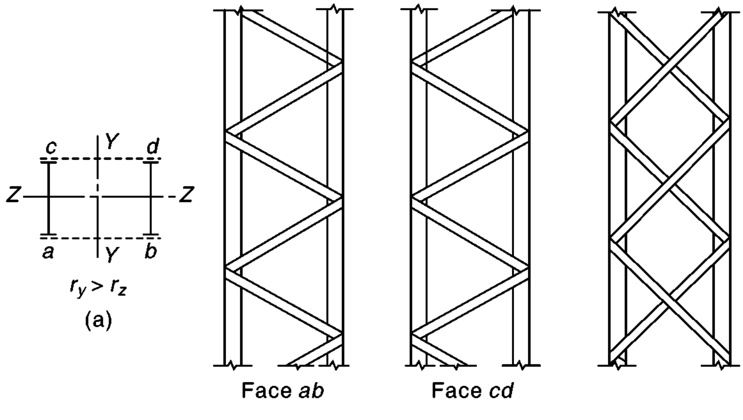


Fig. 5.19 Lacing systems—(a) In plan (b) Single lacing (c) Double lacing

(i) The minimum width of the lacing bar shall not be less than approximately three times the diameter of the connecting bolt/rivet; the thickness shall not be less than $1/40^{th}$ of the effective length for single lacing and $1/60^{th}$ for double lacing.

(j) The spacing of lacing bars shall be such that the maximum slenderness ratio of the components of the main member between two consecutive lacing connections is not greater than 50 or 0.7 times the most unfavourable slenderness ratio of the combined column.

(k) When welded lacing bars overlap the main members, the amount of lap should be not less than four times the thickness of the bar and the welding is to be provided along each side of the bar for the full length of lap. Where lacing bars are fitted between main members, they should be connected by fillet welds on each side or by full penetration butt weld.

(l) Where lacing bars are not lapped to form the connection to the components of members, they should be so connected that there is no appreciable interruption in the triangulated system.

(m) Plates shall be provided at the ends of laced compression members and shall be designed as battens.

(n) Flats, angles (normally adopted in practice), channels, or tubes may be used as lacings.

(o) Lacing bars, whether in double or single shear shall be inclined at an angle of 40° to 70° to the axis of the built-up member.

(p) The effective slenderness ratio $(KL/r)_e$ of the laced column shall be taken as 1.05 times $(KL/r)_o$, where $(KL/r)_o$ is the maximum actual slenderness ratio of the column, to account for shear deformation effects.

(q) The required sections of lacing bars as compression/tension members may be determined using the appropriate design stresses f_{cd} as given in Section 5.5.1 and T_d in Sections 3.5 and 3.7.2.

5.11.1.2 Battens

The rules for the design of battens shall be the same as for lacings except for the following conditions (see Fig. 5.20).

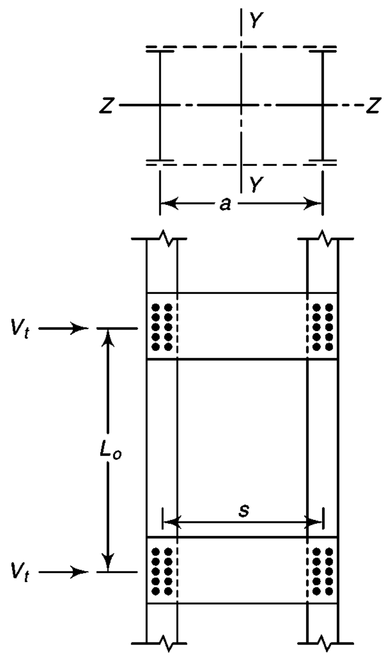


Fig. 5.20 Battening system

(a) The number of battens shall be such that the member is divided into not less than three bays.

(b) Battens shall be designed to resist simultaneously

$$\text{Longitudinal shear } V_b = V_t L_o / ns \tag{5.22}$$

and

$$\text{Moment } M = V_t L_o / 2n \tag{5.23}$$

where V_t is the transverse shear force, L_o is the distance between centre-to-centre of battens, longitudinally, n is the number of parallel planes of battens, and s is the minimum transverse distance between the centroids of the bolt/rivet group/welding connecting the batten to the main member.

(c) When plates are used for battens, the effective depth between the end bolts/rivets or welds shall not be less than twice the width of one member in the plane of battens; nor less than three quarters of the perpendicular distance between centroids of the main members for intermediate battens; and not less than the perpendicular distance between the centroids of main members for end battens.

(d) The thickness of batten plates shall not be less than $1/50^{th}$ of the distance between the innermost connecting transverse bolts/rivets or welds.

(e) The requirement of size and thickness does not apply when other rolled sections are used for battens with their legs or flanges perpendicular to the main member.

(f) When connected to main members by welds, the length of the weld connecting each end of the batten shall not be less than half the depth of the plate; at least one third of its length should be placed at each end of the edge; in addition the weld shall be returned along the other two edges for a length not less than the minimum lap (i.e., not less than four times the thickness of the plate). The length of the weld and depth of batten should be measured along the longitudinal axis of the main member.

(g) The effective slenderness ratio of battened column shall be taken as 1.10 times $(KL/r)_o$, where $(KL/r)_o$ is the maximum actual slenderness ratio of the column, to account for shear deformation effects.

(h) Battened compression members, not complying with the preceding rules or those subjected to eccentricity of loading, applied moments, and lateral forces in the plane of the battens, shall be designed according to exact theory of elastic stability (see Bleich 1952; Timoshenko & Gere 1961) or empirically but verified by test results.

It should be noted that in Western countries such as USA and UK, due to the prohibitive labour and fabrication costs and the availability of larger rolled steel sections, built-up columns are seldom used nowadays.

5.12 Compression Members Composed of Two Components Back-to-back

Compression members may also be composed of two angles, channels, or Ts back-to-back in contact or separated by a small distance and connected together by bolting, rivetting, or welding [Fig. 5.16(c)]. In such a case, the code (IS 800 : 2007) gives the following specifications (see clauses 7.8 and 10.2.5 of the code).

(a) The slenderness ratio of each member between the connections should not be greater than 40 or 0.6 times the minimum slenderness ratio of the strut as a whole.

(b) The ends of the strut should be connected with a minimum of two bolts/rivets or equivalent weld length (weld length must not be less than the maximum width of the member) and there should be a minimum of two additional connections in between, spaced equidistant along the length of the member. Where there is small spacing between the two sections, washers (in case of bolts) and packing (in case of welding) should be provided to make the connections. Where the legs of angles or Ts are more than 125-mm wide, or where the web of channels is 150-mm wide, a minimum of two bolts should be used in each connection. Spacing of tack bolts or welds should be less than 600 mm. If bolts are used, they should be spaced longitudinally at less than 4.5 times the bolt diameter and the connection should extend to at least 1.5 times the width of the member.

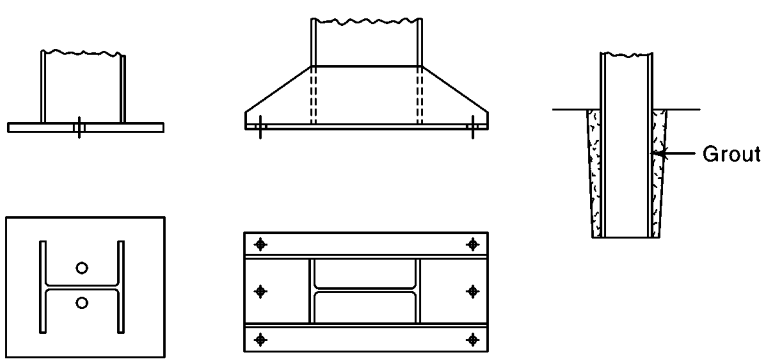
(c) The bolts/rivets should be 16 mm or more in diameter for a member ≤ 10 mm thick and 20 mm in diameter for a member ≤ 16 mm thick, and 22 mm in diameter for members greater than 16 mm thick.
(d) Such members connected by bolts/welding should not be subjected to transverse loading in a plane perpendicular to the rivetted, bolted, or welded surfaces.
(e) When placed back-to-back, the spacing of bolts/rivets should not exceed $12t$ or 200 mm and the longitudinal spacing between intermittent welds should not be more than $16t$, where t is the thickness of the thinner section.

5.13 Column Bases and Caps

For transmitting the load from columns to its foundations, *base plates* are used. Base plates assist in reducing the intensity of loading and distributing it over the foundations. The area of base plate is so chosen that the intensity of load distributed is less than the bearing capacity of concrete on which it rests.

The safety of a column and thus of a structure depends mainly upon the stability of the foundations and consequently on the bases, in the case of steel columns. Hence, column base plates should be designed with great care. The design of a base plate is generally assumed to be on the condition that the distribution of load under the base is uniform and the outstanding portions of the base plate are treated as cantilevers.

The main types of bases used are shown in Fig. 5.21. These are as follows:
(a) Slab base
(b) gussetted base; and
(c) pocket base



Slab base for axially loaded columns (a)   Gussetted base (b)   Pocket base (c)

Fig. 5.21 Column bases

With respect to slab and gussetted bases, depending on the values of axial load and moment, there may be compression over the whole base or compression over part of the base and tension in the holding-down bolts. Horizontal loads are restricted by shear in the weld between column and base plates, friction, and bond between the base and the concrete. Though the AISC code does not allow the anchor rods to transfer substantial shear, ACI-318, Appendix D gives the limit states to be checked in the anchorage, including the steel strength of the anchor in shear, as well as the various concrete limit states. If the base plate has a grout pad of any substantial thickness and the anchor rod does not bear against the base plate (the base plate holes will be larger than the anchor rod and hence in many cases the base connection will not bear against the side of the hole), then bending will be introduced in the rod in addition to shear. The bending capacity of the anchor rods is limited and hence the AISC code does not allow shear transfer through anchor rods (AISC design guide 1-2005). Hence AISC suggests the use of a shear key or lug or embedded plate with welded side plates to transfer a large horizontal shear force from the column base to the foundation (see Fig. 5.22). Note that the horizontal loads will be substantial for earthquake loading or wind loading. We will consider only the design of base plates with concentric loading here. Base plates subjected to bending moments are covered in Chapter 9.

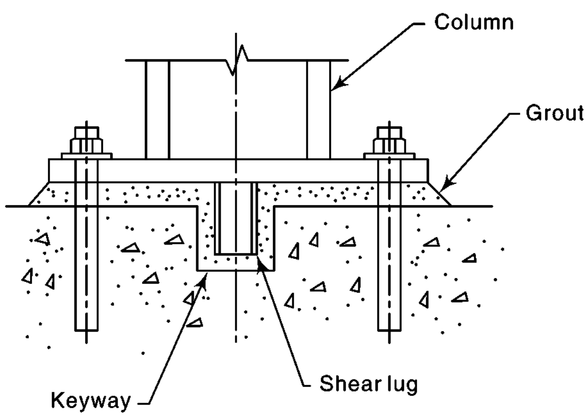


Fig. 5.22 Use of shear lug to transfer heavy shear force

Lightly loaded columns are provided with thick slab bases. The slab base is free from pockets where corrosion may start. Base plates with especially large loads require more than a simple plate. This may result in a double layer of plates, a grillage system, or the use of stiffeners to reduce the plate thickness. The *column caps* serve similar purpose except that they act as a link between load coming on the columns and the column itself (see Fig. 5.23).

The design of slab bases with concentric load is covered in Section 7.4.3 of IS 800 : 2007. This states that where the rectangular plate is loaded by *I*-, *H*-, channel, box, or rectangular hollow sections, the minimum thickness of base plate t_s should be

$$t_s = [2.75w(a^2 - 0.3b^2)/f_y]^{0.5} > t_f \tag{5.24}$$

where w is the pressure on the underside of the slab base due to the factored compressive load on the column (assumed as uniformly distributed over the area

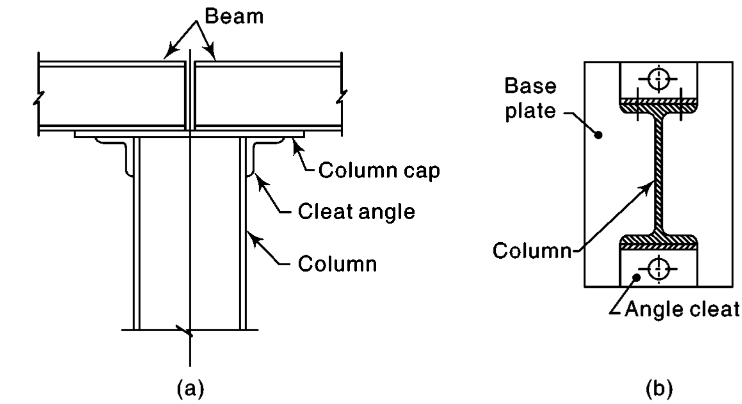


Fig. 5.23 (a) Column cap (elevation), (b) column base (plan)

of the slab base), a and b are the larger and smaller projections of the slab base beyond the rectangle, circumscribing the column, respectively, f_y is the yield strength of the base plate, and t_f is the flange thickness of the compression member.

Equation (5.24) takes into account plate bending in two directions. The moment in the direction of the greater projection is reduced by the co-existence moment at right angles. Poisson's ratio of 0.3 is used to allow for this effect.

Consider an element at A and the two cantilever strips 1-mm wide shown in Fig. 5.24. The bending moments at A are

$$M_x = wa^2/2$$
$$M_y = wb^2/2$$

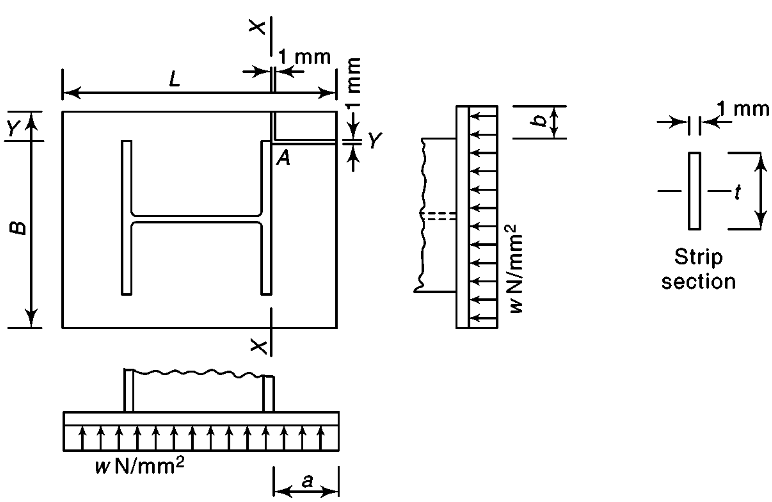


Fig. 5.24 Moment in column base plate

The projection a is greater than b and hence the net moment, with $\mu = 0.3$, is

$$M_x = wa^2/2 - 0.3wb^2/2$$
$$= w/2(a^2 - 0.3b^2) \tag{5.25a}$$

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The moment capacity of the plate is given by

$$M_p = 1.2f_y Z_e$$

The elastic modulus for the cantilever strip is $t^2/6$. Thus,

$$M_p = 1.2f_y t^2/6 \tag{5.25b}$$

Equating Eqns (5.25a) and (5.25b), solving for t, and applying the partial factor of safety for material we get Eqn (5.24), given in the code.

5.13.1 Weld: Column to Slab Base

The code states in clause 7.4.3.4 that where the cap or slab base is fillet welded directly to the column, the contact surfaces should be machined to give a perfect bearing and the welding shall be sufficient to transmit the forces. When full strength butt welds are provided, machining of contact surfaces is not required. Also, where the end of the column is connected directly to the base plate, by means of full penetration butt welds, the connection is deemed to transmit to the base all the forces and moments to which the column is subjected.

5.13.2 Design of Base Plate

The design of base plate consists of finding out its size and thickness. The following are the design steps to be followed:

1. Assuming the grade of concrete, calculate the bearing strength of concrete, which is given by $0.45f_{ck}$ as per clause 34.4 of IS 456-2000
2. Required area of slab base may be computed by

   $$\text{Required } A = P_u/\text{bearing strength of concrete} \tag{5.26}$$

   where P_u is the factored concentric load on the column
3. From the above, choose a size for the base plate $L \times B$, so that $L \times B$ is greater than the required area. Though a few designers prefer to have a square base plate, it is advisable to keep the projections of the base plate beyond column edges a and b as equal (see Fig. 5.24). Hence, the size can be worked out as

   $$(D + 2b)\,(b_f + 2a) = A \tag{5.27}$$

   where D is the depth of the column section (in mm), b_f is the breadth of flange of column (in mm), and a and b are the larger and smaller projections of the base plate beyond the column (in mm).
4. Calculate the intensity of pressure w acting below the base plate using

   $$w = P_u/A_1$$

   where A_1 is the provided area of the base plate ($L \times B$)
5. Calculate the minimum thickness of slab base as per Eqn (5.24). If it is less than t_f, thickness of flange of column, provide the thickness of slab base = thickness of flange of column
6. Provide nominal four 20-mm holding-down bolts
7. Check the weld length connecting the base plate with the column (this check is required only for fillet welds)

CHAPTER 6

Design of Beams

Introduction

Beams are structural members that support loads which are applied transverse to their longitudinal axes. They are assumed to be placed horizontally and subjected to vertical loads. Beams have a far more complex load-carrying action than other structural elements such as trusses and cables.

The load transfer by a beam is primarily by bending and shear. Any structural member could be considered as a beam if the loads cause bending of the member. If a substantial amount of axial load is also present, the member is referred to as a beam-column (beam-columns are covered in Chapter 9). Though some amount of axial effects will be present in any structural member, in several practical situations, the axial effect can be neglected and the member can be treated as a beam.

If we consider a normal building frame, the beams that span between adjacent columns are called *main* or *primary beams/girders*. The beams that transfer the floor loading to the main beams are called *secondary beams/joists*. Beams may also be classified as follows:

1. *Floor beams* A major beam of a floor system usually supporting joists in buildings; a transverse beam in bridge floors
2. *Girder* In buildings, girders are the same as floor beams; also a major beam in any structure. Floor beams are often referred to as girders
3. *Girt* A horizontal member fastened to and spanning between peripheral columns of an industrial building; used to support wall cladding such as corrugated metal sheeting
4. *Joist* A beam supporting floor construction but not a major beam
5. *Lintels* Beam members used to carry wall loads over wall openings for doors, windows, etc.
6. *Purlin* A roof beam, usually supported by roof trusses
7. *Rafter* A roof beam, usually supporting purlins
8. *Spandrels* Exterior beams at the floor level of buildings, which carry part of the floor load and the exterior wall

9. *Stringers* Members used in bridges parallel to the traffic to carry the deck slab; they will be connected by transverse floor beams

Beams may be termed as *simple beams* when the end conditions do not carry any end moments from any continuity developed by the connection. A beam is called a *continuous beam*, when it extends continuously across more than two supports. A *fixed beam* has its ends rigidly connected to other members, so that the moments can be carried across the connection. In steel frames, the term 'fixed end' is somewhat a misnomer, since the ends of rigid connections are not fixed but allow some joint rotation.

Beam loadings will consist of both dead and live loads. A part of the beam dead load is its own weight. When long, heavy members are used, the self weight may be a significant part of the total load. Hence, the beam section should be checked for adequacy for the applied loads, including the beam weight. For the usual structural frame of a steel building, bending effects are predominant in beams with negligible torsional effects.

In order to resist the bending effects in the most optimum way, much of the material in a beam has to be placed as far away as possible from the neutral axis. The web area of the beam has to be adequate for resisting the shear. Usually, maximum bending moment and maximum shear occur at different cross sections. However, in continuous beams, they may occur at the same cross section near the interior supports. Here, the interaction effects are normally neglected. Commonly used cross-sectional shapes are the rolled I-sections and the built-up I-sections or box sections, though channel sections are also often used as purlins (Fig. 6.1). Doubly symmetric shapes such as the standard rolled I-shapes are the most efficient.

Generally a beam may be subjected to simple, unsymmetrical, or biaxial bending. Simple bending takes place if the loading plane coincides with one of the principal planes of a doubly symmetric section such as an I-section or, in the case of a singly symmetric open section (C-section), the loading passes through the *shear centre* and is parallel to the principal plane. Unsymmetrical bending occurs if the plane of loading does not pass through the shear centre. In this case, bending is coupled with torsion. If simple bending takes place about both the principal planes without torsion, the beam is said to undergo biaxial bending. A typical example for biaxial bending in a beam is the roof purlin.

Several complications may arise in a beam design due to a variety of reasons, such as complex stress conditions, tendency of the member to buckle, and conflicting design requirements. Complex stress states may arise when loads are inclined to the principal axes, when unsymmetrical sections are used or where large values of shear and bending moment occur at a section. Buckling may take place in many ways: (a) lateral buckling of the whole beam between supports, (b) local buckling of flanges, and (c) longitudinal buckling of the web and buckling in the depth direction under concentrated loads or near the supports. As far as design is concerned, the depth of the beam is a very important parameter. While a large depth is desirable for moment resistance, the resulting thin web may not be able to resist lateral or web buckling.

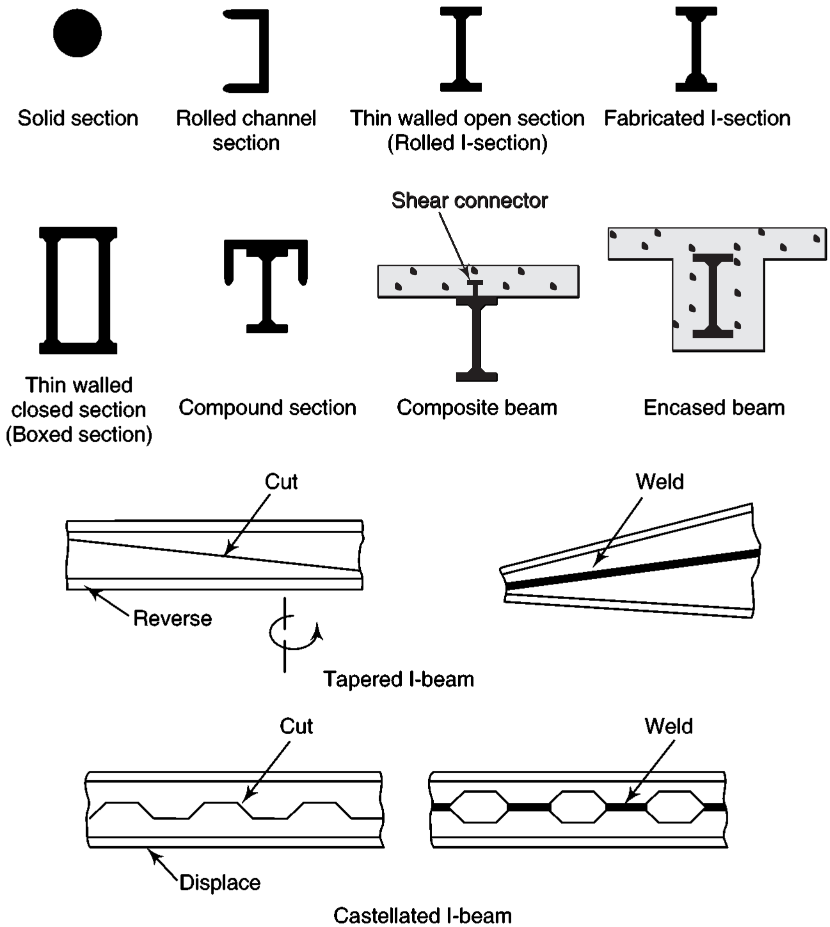


Fig. 6.1 Types of steel beams

In the design of a beam, two aspects are of primary consideration: (a) strength requirement, that is, the beam has adequate strength to resist the applied bending moments and accompanying shear forces and (b) stability consideration, that is, the member is safe against buckling. Another requirement is adequate bending stiffness, which is often satisfied by members meeting the requirement of strength. Beams develop higher stresses compared to axial members for the same loads and bending deflections are considerably high. Bending effects rather than shear govern the proportions of a rolled beam section, except in cases where shear-to-moment ratio is very large. Occasionally, bending deflections may form the primary design consideration. Shear deflections are usually very small and hence neglected except in the case of deep beams with very short spans.

6.1 Beam Types

There are various forms of beam cross sections used in practice, as shown in Fig. 6.1. The selection of a section would depend upon the use for which it is intended and on the overall economy. The beam chosen should possess required strength and it should not deflect beyond a limit. Members fabricated out of rolled plates are called *plate girders*, which are dealt in detail in the next chapter. Compound sections may also be fabricated out of rolled sections for special purposes. Similarly, in *hybrid sections*, the flanges of higher yield stress compared to the web can also be formed. Certain specific features, such as the passage of services below the floor of a building, may necessitate the use of sections with openings in the web. Other economical types of beams that can be fabricated out of rolled sections are the *castellated* and *tapered beams*. Composite steel beams and concrete-encased steel beams, derive added strength, economy, and in the latter case fire protection also. Table 6.1 gives the important types of steel beams used in practice with their optimum span range.

Table 6.1 Beam types

Type of beam	Optimum span range (m)	Application
Angles	3–6	For lightly loaded beams such as roof purlin and sheeting rail
Rolled I-sections	1–30	Most frequently used as a beam
Castellated beams	6–60	Long spans and light loads
Plate girders	10–100	Long spans with heavy loads such as bridge girders
Box girders	15–200	Long spans and heavy loads such as bridge girders

6.2 Section Classification

The bending strength of a section depends on how well the section performs in bending. When a stocky beam (thick-walled beam) (see Fig. 4.1) is subjected to bending, extreme fibres in the maximum moment region reach the yield stress [see Fig. 4.1(b)]. Upon further loading, more and more fibres reach the yield stress and the stress distribution is as shown in Fig. 4.1(c). The bending moment that causes the whole cross section to reach yield stress [Fig. 4.1(d)], is termed as the plastic moment of the cross section M_p. The cross section is incapable of resisting any additional moment, but may maintain the plastic moment, acting as a plastic hinge, for some more amount of rotation. If the section is thin-walled (slender), it may fail by local buckling, even before reaching the yield stress. The ratio of ultimate rotation to yield rotation is called the rotation capacity. Four classes of section have been identified: (a) plastic (class 1), (b) compact (class 2), (c) semi compact (class 3), and (d) slender (class 4) based on their yield and plastic moments along with their rotation capacities. Refer to Section 4.9 and Fig. 4.11 for more details of this classification.

Local buckling can be prevented by limiting the width–thickness ratio. In the case of rolled sections, higher thickness of the plate is adopted to prevent local buckling. For built-up and cold-formed sections, longitudinal stiffeners are provided to reduce the width into smaller sizes. It is evident that only plastic sections can be used in indeterminate frames which form collapse mechanisms. Compact sections can be used in simply supported beams which fail after reaching M_p at one section. Semi-compact sections can be used for elastic design, where the section fails after reaching M_y at the extreme fibres. Slender sections are not preferred in hot-rolled structural steelwork; but they are extensively used in cold-formed members.

6.3 Lateral Stability of Beams

A beam loaded predominantly in flexure would attain its full moment capacity if the local and lateral instabilities of the beam are prevented. To ensure the first condition, the cross sections of flanges and web of the beam chosen must be *plastic* or *compact*. If significant ductility is required, sections must invariably be plastic. If the laterally unrestrained length of the compression flange of the beam is relatively long, then a phenomenon, known as *lateral buckling* or *lateral torsional buckling* of the beam may take place. The beam would fail well before it attains its full moment capacity. This phenomenon (see Section 4.1.1 for the behaviour of beams, which are laterally restrained) has a close similarity to the Euler buckling of columns, where collapse is triggered before the column attains its squash load (full compressive yield load).

In steel structures, especially in buildings, beams are restrained laterally by the floor decks, which are placed on top of them. During the construction stage, before the floor decks are in place, if the beams are not adequately supported laterally, they may be susceptible to lateral buckling. Therefore, during construction stage, they may need special attention with regard to their lateral stability. If adequate lateral restraints are not provided to beams in the plane of their compression flanges, the beams would buckle laterally resulting in a reduction of their maximum moment capacity. Lateral buckling can be prevented, if adequate restraints are provided to the beam in the plane of the compression flange; such beams are called *laterally restrained beams.*

Beams may also fail by local buckling or local failure (shear yielding of web, local crushing of web or buckling of thin flanges). These may be prevented by providing stiffeners/additional flange plates. It is interesting to explain the phenomenon of lateral buckling. Consider a simply supported, but laterally unsupported (except at the ends of the beam) beam subjected to incremental transverse loads at its mid-section as shown in Fig. 6.2(a).

In the case of axially loaded columns, the deflection takes place sideways and the column is said to buckle in a pure flexural mode. A beam, under transverse loads, has a part of its cross section in compression and the other in tension. The part under compression becomes unstable while the portion under tension tend to stabilize the beam and keep it straight. Thus, beams when loaded exactly in the plane of the web, at a particular load, will fail suddenly by deflecting sideways and then twisting about its longitudinal axis. This form of instability is more complex (compared to column instability which is two-dimensional) since the lateral buckling

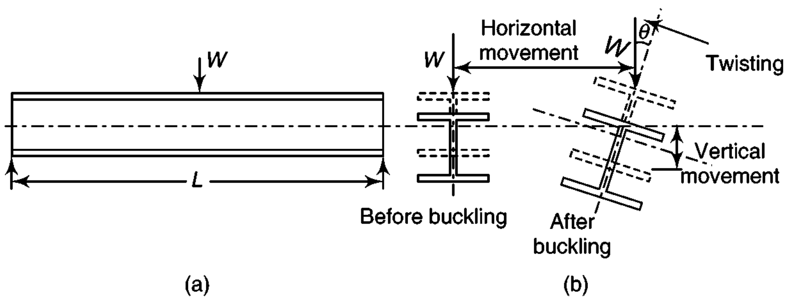


Fig. 6.2 (a) Long span beam (b) Laterally deflected shape of the beam

problem is three-dimensional in nature. It involves coupled lateral deflection and twist. Thus, when the beam deflects laterally, the applied moment exerts a torque about the deflected longitudinal axis, which causes the beam to twist. The bending moment at which a beam fails by lateral buckling when subjected to a uniform end moment is called its *elastic critical moment* (M_{cr}). In the case of lateral buckling of beams, the elastic buckling load provides a close upper limit to the load carrying capacity of the beam. Lateral instability occurs only if the following two conditions are satisfied.

- The section possesses different stiffness in the two principal planes.
- The applied loading induces bending in the stiffer plane (about the major axis).

Similar to columns, the lateral buckling of unrestrained beams is also a function of its slenderness. Several factors influence the buckling capacity of beams. Some of them are cross-sectional shape of the beam, type of support, type of loading and position of the applied load.

6.3.1 Lateral Torsional Buckling of Symmetric Sections

As explained earlier, when a beam fails by lateral torsional buckling, it buckles about its weak axis, even though it is loaded in the strong plane as shown in Figs. 6.3(a) and (b)].

Let us consider the lateral torsional buckling of an ideal I-section with the following assumptions.

1. The beam has no initial imperfections.
2. Its behaviour is elastic.
3. It is loaded by equal and opposite end moments in the plane of the web.
4. The beam does not have residual stresses.
5. Its ends are simply supported vertically and laterally.

Note, in practice, the above ideal conditions are seldom met.

The differential equation for the angle of twist for the beam shown in Fig. 6.3 is

$$EI_w \frac{d^4\phi}{dx^4} - GI_t \frac{d^2\phi}{dx^2} - \frac{M^2}{EI_y}\phi = 0$$

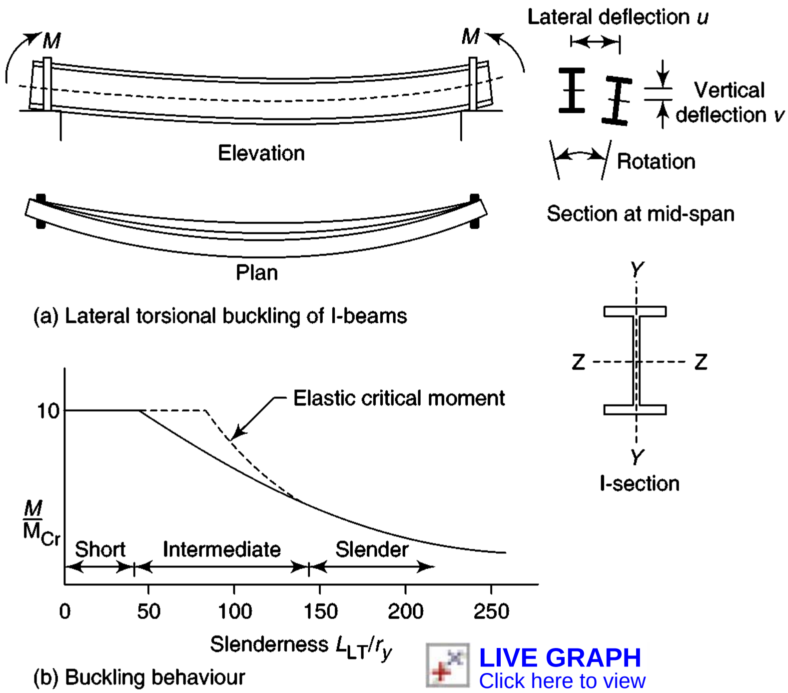


Fig. 6.3 Lateral torsional buckling of I-section beams

The solution to this differential equation is given by (Bleich 1952; Timoshenko & Gere 1961; Salmon & Johnson 1996)

$$M_{cr} = (\ \ /L)\sqrt{EI_yGI_t \quad \left(\frac{E}{L}\right)^2 I_wI_y} \tag{6.1a}$$

where EI_y is the minor axis flexural rigidity, GI_t is the torsional rigidity, and EI_w is the warping rigidity.

Equation (6.1a) may be rewritten as

$$M_{cr} = \frac{\pi}{L}(EI_yGI_t)^{0.5}\left[1 \quad \frac{\pi^2 EI_w}{L^2GI_t}\right]^{0.5} \tag{6.1b}$$

The cross-sectional shape is a particularly important parameter in assessing the lateral buckling capacity of a beam. Conversely, the problem of lateral instability can be minimized or even eliminated by a judicious choice of the section as shown in Table 6.2. Although the five cross sections shown are having the same cross-sectional area, the values of their flexural and torsional properties relative to those of the square cross section show considerable variation. It is observed from Table 6.2 that the flat and deep I-sections, which have largest in-plane bending stiffness, have less buckling strength compared to the box section (Kirby & Nethercot 1979).

Table 6.2 Types of cross-section and their relative values of section properties (Kirby & Nethercot 1979)

Section type	Square	Flat	Shallow beam (typical)	Deep beam (typical)	RHS (typical)
Section properties					
A	1	1	1	1	1
I_z	1	25	12.45	45.59	16.94
I_y	1	0.04	3.20	3.20	8.10
I_t	1	0.04	0.034	0.033	4.731

For a beam with central load acting at the level of centroidal axis, the critical buckling load is given by

$$M_{cr} = \frac{4.24}{L}\sqrt{(EI_y GI_t)}\sqrt{[1+\pi^2 EI_w/(L^2 GI_t)]} \tag{6.2}$$

The ratio of the two constants $4.24/\pi = 1.35$ is often termed as 'equivalent uniform moment factor' C_1. Its value is a direct measure of the severity of a particular pattern of moments relative to the basic case given in Eqn (6.1). Taking into account C_1, Eqn (6.1) may be rewritten as

$$M_{cr} = C_1(EI_y GI_t)^{1/2}\left[\frac{\pi}{L}\left(1+\frac{\pi^2}{B^2}\right)^{1/2}\right] \tag{6.3}$$

where $B^2 = L^2\, G\, I_t/EI_w$ or

$$M_{cr} = C_1(EI_y\, GI_t)^{1/2}\,\gamma \tag{6.4}$$

where $\gamma = \pi/\ell(1+\pi^2/B^2)^{1/2}$

Equation (6.4) is a product of three terms: the first term C_1 varies with the loading and support conditions; the second term varies with the material properties and the shape of the beam; and the third term γ varies with the length of the beam. This equation is regarded as the basic equation for lateral torsional buckling of beams.

Lateral Torsional Failure of Marcy Bridge

A pedestrian footbridge near Marcy, New York, collapsed during construction on October 12, 2002, killing one and injuring nine workers. The bridge was designed as a single-span composite box girder bridge consisting of a trapezoidal steel tub girder with stay-in-place forms and a concrete deck. The Marcy Bridge was designed to span approximately 52 m across an extension of the Utica-Rome expressway (NYS Route 49). At the time of collapse, the concrete deck was being placed, starting at the north end and moving south, using an automatic screeding machine to distribute the concrete. Eyewitnesses reported that the bridge had been noticeably 'bouncy' while the concrete was placed, and when the concrete placement had progressed to about mid-span, the bridge suddenly twisted and rolled to the east, tipping off the abutments.

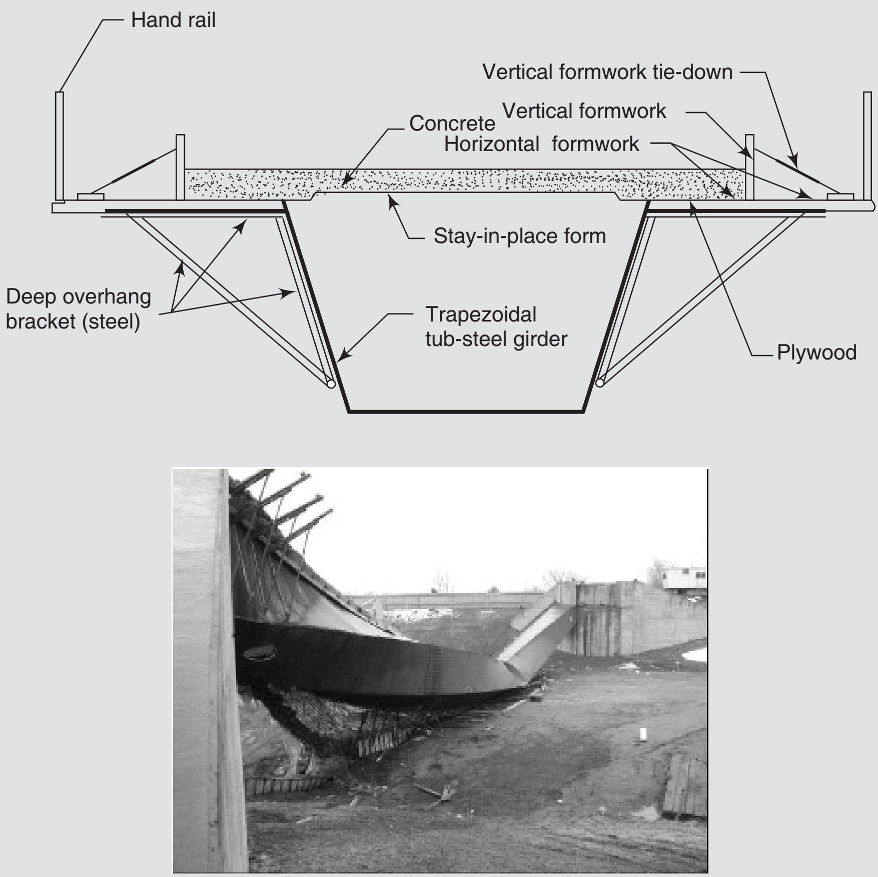


Fig. CS1 Cross-section of Marcy Bridge and the failure due to lateral-torsional buckling (source: Corr, et al, 2009)

Prior to and during the placement of the concrete deck, the tub cross-section was closed only by stay-in-place forms, which were nominally connected to closure angles on each top flange and to each other by using self-tapping screws (see Fig. CS1). The stay-in-place forms do not provide sufficient torsional strength or stiffness, thus the section is open and torsionally flexible. Computer analysis conducted by Exponent Engineering and Scientific Consulting confirmed that the tub girder did not have sufficient strength to carry the wet concrete without collapse. The mode of failure was considered as lateral-torsional buckling of the entire girder cross-section. Such bridges are particularly susceptible to this type of failure, because they are very flexible in a twisting mode before the concrete deck hardens. (After curing, the concrete deck would have provided a closed section with increased torsional stiffness and would have prevented the lateral torsional buckling failure.) The standard practice at that time, as recorded in bridge design specifications and national design standards, did not recognize this failure mode, and there were no requirements in those codes or standards to provide bracing, falsework, or other measures to prevent such a collapse. After the collapse, and a very similar accident in Sweden that occurred just months before this, bridge design standards in New York and the American Association of State Highway and Transportation Officials (AASHTO) were updated with provisions intended to prevent further accidents. Now these codes explicitly require diagonal bracing between the top-flanges in order to suppress this buckling mode.

Reference:

Corr, D.J., McCann, D.M., and McDonald, B.M., Lessons Learned from Marcy Bridge Collapse, Forensic Engineering, 2009, pp. 395–403.

6.4 Effective Length

The concept of effective length of the compression flange incorporates the various types of support conditions. For the beam with simply supported end conditions and no intermediate lateral restraint, the effective length is equal to the actual length between the supports. When a greater amount of lateral and torsional restraints is provided at the supports, the effective length is less than the actual length and alternatively, the length becomes more when there is less restraint. The effective length factor would indirectly account for the increased lateral and torsional rigidities provided by the restraints.

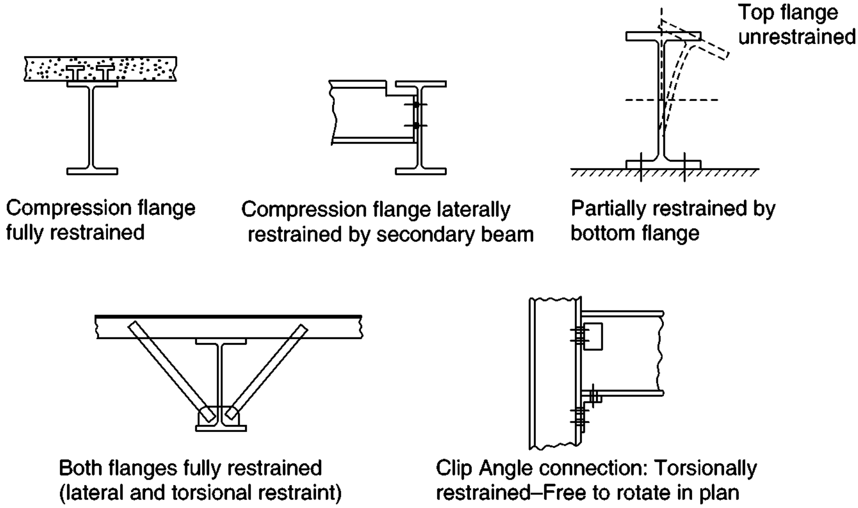
Thus, there are two K factors, K_b (restraint against lateral bending) and K_w (restraint against warping). The values of K_b and K_w are 1.0 for free bending and free warping (simply supported) and 0.5 for bending and warping prevented (fixed) support condition (Kirby & Nethercot 1979). The values of K_b and K_w vary with the proportions of beams and the accurate assessment of the degree of restraint provided by practical forms of connection is difficult. Hence in the code, this problem is treated in an approximate way by considering $K_b = K_w$ and treating the effective length as $L = KL$ (Kirby & Nethercot 1979). The effective lengths KL of the compression flange for different end restraints according to IS 800 : 2007 are given in Table 6.3 (see also Table 15 of the code, which is based on BS 5950–1 : 2000).

For cantilevers, the most severe loading condition is the point load acting at the tip. Nethercot (1983) has shown that for most applications, the simple effective length method (similar to that used for struts) is satisfactory and the critical buckling moment is given by Eqn. (6.16) with L replaced by KL.

Table 6.3 Effective length of compression flanges

Effective length KL for beams, between supports	
Condition at supports (see the following figures)	Effective length, KL
Compression flange at the ends unrestrained against lateral bending (free to rotate in plan)	L*
Compression flange partially restrained against lateral bending (Partially free to rotate in plane at the bearings)	$0.85L$*
Compression flange restrained fully against lateral bending (rotation fully restrained in plan)	$0.7\,L$*

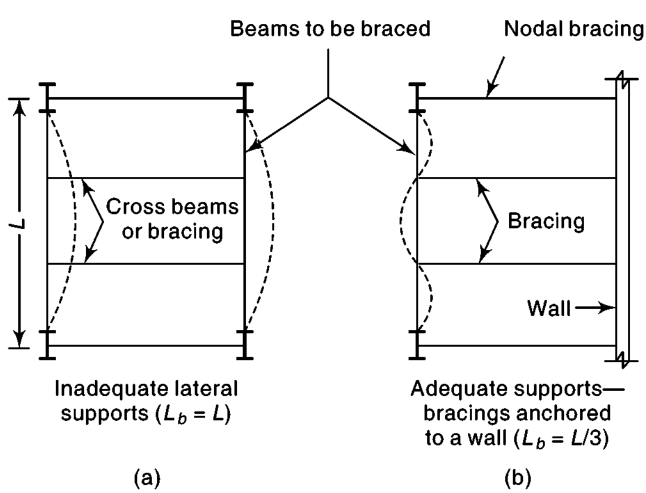
*When the ends of the beam are not restrained against torsion, or where the loading condition is destabilizing or when flanges are free to move laterally, these values have to be increased as per Table 15 of the code.

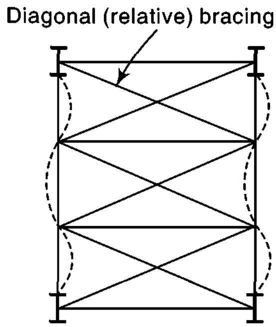


The effective length factors for various restraint conditions at the tip and at the root of the cantilever are given in Table 16 of the code.

6.4.1 Intermediate Braces

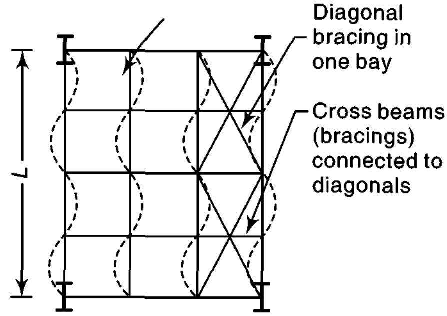
Provision of intermediate lateral supports (bracings) can increase the lateral stability of a beam. For the bracings to be effective, the designer should make certain that the braces themselves are prevented from moving in their axial direction. For example, for the beam shown in Fig. 6.4(a), the lateral buckling of the beams will not be prevented by the braces, and hence the unbraced length should be taken as L. To prevent this type of system-buckling, the bracings should be either anchored into a wall [Fig. 6.4(b)] or provided with diagonal bracings, which effectively transfer the loads to the columns [Fig. 6.4(c)]. It has to be noted that the diagonal bracing need not be provided in all the bays [see Fig. 6.4(d)]. Generally, even a light bracing has the ability to provide substantial increase in stability. Lateral bracing may be either discrete (e.g., cross beams) or continuous [(e.g., beam encased in concrete floors).





Adequate supports—
cross beams with diagonal bracing
($L_b = L/3$)

(c)



$L_b = L/4$

(d)

Cross beams bracing main beams
(e)

Fig. 6.4 Lateral bracing systems for roof and floor beams

It is also important to check the adequacy of beams, during the intermediate stages of construction, as the amount of bracing provided during these stages may be low.

For bracing to be effective, it has to be provided in the compression flange. Bracing provided below the point of application of the transverse load would not be able to resist the twisting and hence the full capacity of the beam cannot be achieved (Kirby & Nethercot 1979). Similarly lateral bracing system, attached near the beam centroid are ineffective. The two requirements for effective bracing are as follows.

- It should have sufficient stiffness so that buckling of the beam occurs in between the braces.
- It should have sufficient strength to withstand the force transferred to it by the beam.

The effective length of the compression flange is the maximum distance centre-to-centre of the restrained members. In the code only the strength aspect is covered. Thus it states that the effective lateral restraints for a beam should be capable of resisting 2.5% of the maximum force in the compression flange taken as divided equally between the numbers of points at which the restraint members occur.

Based on the research done by Winter (1960) and Yura (2001), the American code (ANSI/AISC: 360-2005) suggests the strength and stiffness of lateral bracing as given in Table 6.4.

Table 6.4 Bracing requirements

Requirement	Relative	(Nodal) Discrete
Stiffness k_{br}, N/mm	$(4M_uC_d)/(0.75L_bh_o)$	$(10M_uC_d)/(0.75L_bh_o)$
Strength F_{br}, N	$(0.008\, M_uC_d)/h_o$	$(0.02\, M_uC_d)/h_o$

M_u = required flexural strength, Nmm
h_o = distance between flange centroids, mm
C_d = 1.0 for single curvature, 2.0 for double curvature
L_b = laterally unbraced length, mm

Torsional bracings Torsional bracing may be provided in the form of cross frames or diaphragms at discrete locations or continuous bracing in the form of metal decks and slabs. The factors that have significant effect on lateral bracing, such as the number of bracings, top flange loading and brace location on the cross section, were found to be relatively unimportant while sizing a torsional brace (Yura 2001). However, the position of the braces on the cross section has an effect on the stiffness of the brace itself. More information on torsional bracings may be found in Galambos (1998).

6.4.2 Level of Application of Transverse Loads

The lateral stability of a transversely loaded beam is dependent on the arrangement of the loads as well as the level of application of the loads with respect to the centroid of the cross section. IS 800 takes into account the destabilising effect of the top flange loading by using a notional effective length of 1.2 times the actual span to be used in the calculation of effective length (see Table 15 of the code.)

6.4.3 Influence of Type of Loading

To take into account the non-uniform loading, a simple modifier is applied to Eqn. (6.1) in many codes (Salvadori, 1956) as given below

$$M_{cr} = C_1 M_{ocr} \tag{6.5}$$

where M_{ocr} is the critical moment obtained from Eqn. (6.1) and C_1 is the equivalent uniform moment factor. Various lower-bound formulae have been proposed for C_1.

The Indian code gives various values of C_1 in a table in Annex E for different moment diagrams. This table is based on the Eurocode 3. The values of C_1 given in Table Annex E of the code for end moment loading maty be approximated by the following equation (Gardner & and Nethercot 2005) when $k = 1$

$$C_1 = 1.88 - 1.40\psi + 0.52\psi^2 \text{ but } C_1 \leq 2.70 \tag{6.6}$$

where ψ is the ratio of the end moments defined in Table 42 of the code, It should be noted that the Table 42 given in the Indian code considers the end rotational restraint against lateral bending K and suggests that Kw values be taken as equal to unity.

The Wind-induced Collapse of Tacoma Narrows Bridge

Collapse of the Tacoma Narrows Bridge, (Source: Smithsonian Institution)

The original Tacoma Narrows Bridge opened on July 1, 1940, and dramatically collapsed into Puget Sound on November 7 of the same year. This suspension bridge spanned the Tacoma Narrows strait between Tacoma and the Kitsap Peninsula, in the United States. At the time of its construction (and its destruction), Galloping Gertie (nick named due to its oscillations) was the third longest suspension bridge in the world, behind the Golden Gate Bridge and the George Washington Bridge. 'Longest' is a comparison of the main spans in suspension bridges. It had a total length of 1,810.2 m with a central (longest) span of 853.4 m.

The failure of the bridge occurred due to the twisting of the bridge deck in mild winds of about 64 km/h. This failure mode is termed as the torsional vibration mode (which is different from the transversal or longitudinal vibration mode). Thus, when the left side of the roadway went down, the right side would rise, and vice versa, with the center line of the road remaining still. Specifically, it was the ‘second’ torsional mode. In fact, two men proved this point by walking along the center line, unaffected by the flapping of the roadway rising and falling to each side. This vibration was caused by aeroelastic fluttering (a phenomenon in which aerodynamic forces on an object couple with a structure’s natural mode of vibration to produce rapid periodic motion). The wind-induced collapse occurred on November 7, 1940. The collapse of the bridge was filmed by Mr Barney Elliott (who was travelling on the bridge), which helped engineers to study the behaviour of the bridge.

Suspension bridges consist of cables anchored in heavy foundations at their ends and supported by towers at intermediate points. From these cables, the deck is suspended. Thus, they are more flexible than other types of bridges, and require bracings to reduce vertical and torsional motions. In the Tacoma Narrows bridge, instead of the usual deep open trusses, narrow and shallow solid I-beams were used in the decks, which resulted in the build-up of wind loads.

Fig. CS2 Solid I-beams in the deck of the original bridge and trussed deck of the reconstructed bridge (www.failurebydesign.info)

The bridge collapse boosted research in the field of bridge aerodynamics which resulted in better designs. After the collapse, two bridges were constructed in the same general location. The first one, now called the Tacoma Westbound Bridge, is 1822 m long –12 m longer than the Galloping Gertie. The second one, the Tacoma Eastbound Bridge, opened in 2007.

Reference: http://en.wikipedia.org/wiki/Tacoma_Narrows_Bridge_%281940%29

6.5 Buckling of Real Beams

The theoretical assumptions made in Section 6.3 are generally not realised in practice. In order to understand the behaviour of real beams, it is necessary to consider the combined effects of instability and plasticity and also the role of factors such as residual stress and geometrical imperfections. When all these effects are considered, it may be found that slender beams fail more or less elastically by excessive deformation at loads that are close to M_{cr}, beams of intermediate slenderness fail inelastically by excessive lateral deformation and stocky beams will attain M_p with negligible lateral deformation. This kind of behaviour is shown in Fig. 6.5. The behaviour of beams in bending has already been explained in Section 4.1.1.

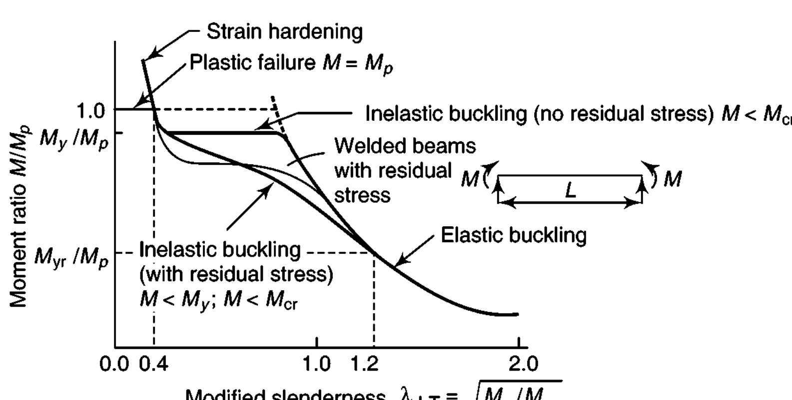


Fig. 6.5 Interaction between instability and plasticity

6.6 Design Strength of Laterally Supported Beams in Bending

For laterally supported beams, the factored design moment M at any section in a beam, due to external actions, satisfy the relationship $M < M_d$, where M_d is the design bending strength of the section. The design bending strength of a laterally supported beam is governed by the yield stress. For a laterally unsupported beam, the design strength is most often controlled by the lateral torsional buckling strength. The above relationship is obtained with the assumption that the beam web is stocky. When the flanges are plastic, compact, or semi-compact but the web is slender (i.e., $d/t_w > 67\varepsilon$), the design bending strength may be calculated using one of the following methods.

- The flanges resist the bending moment and the axial force acting on the section and the web resists only the shear.
- The whole section resists the bending moment and the axial force and therefore the web has to be designed for combined shear and its share of normal stresses. This is done by using simple elastic theory in the case of semi-compact webs and simple plastic theory in the case of compact and plastic webs.

Shear force does not have any influence on the bending moment for values of shear up to $0.6V_d$ (called the low shear load), where V_d is the design shear strength. The provision for determining V_d is explained a little later. When the design shear force V is less than $0.6V_d$, the design bending strength M_d will be taken as

$$M_d = 0.909\beta_b\, Z_p f_y \leq 1.09 Z_e f_y \leq 1.36\, Z_e f_y \text{ (for cantilevers)} \tag{6.7a}$$

where ${}_b = 1.0$, for plastic and compact sections and ${}_b = Z_e/Z_p$, for semi-compact sections.

Thus, for class 3 semi-compact sections,

$$M_d = 0.909 f_y Z_e \tag{6.7b}$$

For class 4 slender sections, $M_d = f_y' Z_e$, where f_y' is the reduced design strength for slender sections. Z_p and Z_e are the plastic and elastic section moduli of the cross section, respectively, and f_y is the yield stress of the material.

The additional check ($M_d < 1.09 Z_e f_y$) is provided to prevent the onset of plasticity under unfactored dead, imposed, and wind loads. For most of the I-beams and channels given in IS 808, Z_{pz}/Z_{ez} is less than 1.2 and hence the plastic moment capacity governs the design. For sections where $Z_{pz}/Z_{ez} > 1.2$, the constant 1.2 may be replaced by the ratio of factored load/unfactored load (γ_f). Thus the limitation $1.2 Z_e f_y$ is purely notional and becomes in practice $\gamma_f Z_e f_y$ (Morris & Plum 1996).

When the design shear force (factored) V exceeds $0.6V_d$ (called the high shear load), where V_d is the design shear strength of the cross section, the design bending strength M_d will be taken as

$$M_d = M_{dv}$$

where M_{dv} is the design bending strength under high shear and it is calculated as follows:

(a) *Plastic or Compact Section*

As the shear force V is increased from zero, no reduction in plastic moment is assumed below the value of M_p until V reaches $0.5V_p$, where V_p is the shear strength of the web given by $A_v f_v$, where A_v is the shear area taken as Dt_w for rolled sections and dt_w for built up sections ($d = D - 2t_f$). The design shear strength f_v is taken as $0.6f_y$, where f_y is the design strength in tension or compression, i.e., slightly greater than the true von Mises value of $f_y/\sqrt{3}$. When the full capacity in shear V_p is reached, the shear area is assumed to be completely ineffective in resisting the moment, and hence the reduced plastic moment M_{dv} becomes M_{fd}, where $M_{fd} = M_p - (D^2t_w/4)f_y$ for rolled sections and $M_{fd} = M_p - (d^2t_w/4)f_y$ for built-up sections. Between $V = 0.5V_p$ and V_p, M_{dv} is assumed to reduce for plastic and compact sections according to the following equation

$$M_{dv} = M_p - \beta\,(M_p - M_{fd}) \leq 1.09\, Z_e f_y \tag{6.8a}$$

where $\beta = (2V/V_p - 1)^2$ (6.8b)

In Eqn (6.8), M_p is the design plastic moment of the whole section disregarding high shear force effect, but considering web buckling effects, V is the factored applied shear force, V_p is the design shear strength as governed by web yielding or web buckling = $0.6Dt_wf_y$, and M_{fd} is the plastic design strength of the area of the cross section excluding the shear area, considering partial safety factor γ_{m0}.

(b) *Semi-compact section*

$$M_{dv} = 0.909\, Z_e f_y \tag{6.8c}$$

where Z_e is the elastic section modulus of the whole section.

6.6.1 Holes in the Tension Zone

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

$$(A_{nf}/A_{gf}) \geq 1.26(f_y/f_u) \tag{6.9}$$

where A_{nf}/A_{gf} is the ratio of net and gross area of the flange and f_y/f_u is the ratio of yield and ultimate strength of the material.

When A_{nf}/A_{gf} does not satisfy Eqn (6.9), the reduced flange area A_{nf} satisfying Eqn (6.9) may be taken as the effective flange area in tension (Dexter and Altstadt 2003).

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 Eqn (6.9) is satisfied for the complete tension zone of the cross section, comprising the tension flange and tension region of the web. Fastener holes in the compression zone of the cross section need not be considered in the design bending strength calculation, except for oversize and slotted holes, or holes without any fastener.

6.6.2 Shear Lag Effects

The simple theory of bending is based on the assumption that plane sections remain plane after bending. In reality, shear strains cause the sections to warp. The effect in the flange is to modify the bending stresses obtained by simple bending theory. Thus higher stresses are produced near the junction of a web and lower stresses at points away from the web as shown in Fig. 6.6. This phenomenon is known as *shear lag*. It results in a non-uniform stress distribution across the width of the flange. The shear lag effects are minimal in rolled sections, which have relatively narrow and thick flanges. For normal dimensions of the flanges, the effects are negligible. But if the flanges are unusually wide, (as in plate girders or box girders), these shear strains influence the normal bending stresses in the flanges.

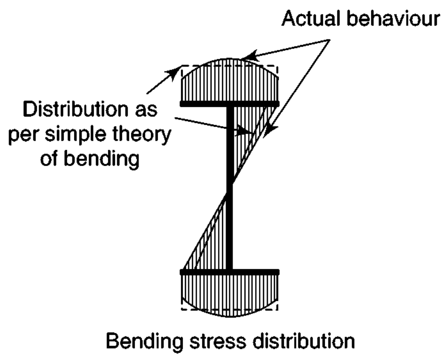


Fig. 6.6 Shear lag effects for an I-section

The shear lag effects in flanges may be disregarded provided, for outstand elements (supported along one edge), $b_o \leq L_0/20$ and for internal elements (supported along two edges), $b_i \quad L_0/10$ where L_0 is the length between points of zero moment (inflection) in the span, b_o is the width of the outstand, and b_i is the width of an internal element.

When these limits are exceeded, the effective width of the flange for design strength may be taken conservatively as the values satisfying the limits given earlier.

6.7 Design Strength of Laterally Unsupported Beams

The effect of lateral torsional buckling on flexural strength need not be considered when $\lambda_{LT} \leq 0.4$ (see Fig. 6.5) where λ_{LT} is the non-dimensional slenderness ratio for lateral torsional buckling. The design bending strength of a laterally unsupported beam as governed by lateral torsional buckling as per the Indian code (IS 800 : 2007) is given by

$$M_d = \beta_b \, Z_p \, f_{\text{bd}} \tag{6.10}$$

with

$$\beta_b = 1.0 \text{ for plastic and compact sections}$$
$$= Z_e/Z_p \text{ for semi-compact sections}$$

where Z_p and Z_e are the plastic section modulus and elastic section modulus with respect to extreme compression fibre and f_{bd} is the design bending compressive stress. The design bending compressive stress is given by

$$f_{\text{bd}} = 0.909 \, \chi_{\text{LT}} \, f_y \tag{6.11a}$$

where χ_{LT} is the reduction factor to account for lateral torsional buckling given by

$$\chi_{\text{LT}} = \frac{1}{\phi_{\text{LT}} + (\phi_{\text{LT}}^2 - \lambda_{\text{LT}}^2)^{0.5}} \leq 1.0 \tag{6.11b}$$

in which $\phi_{\text{LT}} = 0.5[1 + \alpha_{\text{LT}}(\lambda_{\text{LT}} - 0.2) + \lambda_{\text{LT}}^2]$.

The values of imperfection factor α_{LT} for lateral torsional buckling of beams is given by

α_{LT} = 0.21 for rolled section

α_{LT} = 0.49 for welded section

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

$$\lambda_{LT} = \sqrt{\beta_b Z_p f_y / M_{cr}} \le \sqrt{1.2 Z_e f_y / M_{cr}} \tag{6.12}$$

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

where M_{cr} is the elastic critical moment and $f_{cr,\,b}$ is the extreme fibre compressive elastic lateral buckling stress.

The elastic lateral buckling moment M_{cr} is given by Eqn. (6.1a) with L replaced by KL. Kerensky et al. (1956) simplified Eqn (6.1a) by introducing the following approximations for doubly symmetric sections.

$$I_y = b_f^3 t_f/6,\ I_w = I_y h^2/4,\ I_t = 0.9\ b_f t_f^3,\ b_f = 4.2 r_y \text{ and } E = 2.6G$$

Thus, Eqn (6.1a) is reduced to

$$M_{cr} = \frac{\beta_{LT}\pi^2 E I_y h}{2(KL)^2}\left[1 + \frac{1}{20}\left[\frac{KL/r_y}{h/t_f}\right]^2\right]^{0.5} \tag{6.13}$$

$$= \beta_b\, Z_e f_{cr,\,b}$$

where I_t is the torsional constant, I_w is the warping constant, I_y is the moment of inertia about the weak axis, r_y is the radius of gyration of the section about the weak axis, KL is the effective laterally unsupported length of the member, h is the overall depth of the section, t_f is the thickness of the flange, and β_{LT} = 1.20 for plastic and compact sections with $t_f/t_w \le 2.0$ and 1.00 for semi-compact sections or sections with $t_f/t_w > 2.0$.

Using the same approximations, the extreme fibre compressive elastic buckling stress may be obtained as (with $E = 2.0 \quad 10^5$ MPa and $Z_z = 1.1 b_f d_f h$)

$$f_{cr,\,b} = \left(\frac{1625}{KL/r_y}\right)^2\left[1 + \frac{1}{20}\left[\frac{KL/r_y}{h/t_f}\right]^2\right]^{0.5} \tag{6.14}$$

IS 800 : 2007 uses a similar expression but the coefficient of 1625 is replaced by 1473.5. Table 6.5 gives the value of $f_{cr,\,b}$ for various values of KL/r_y and h/t_f based on IS 800 : 2007. Table 6.6 gives the values of design bending compressive strength corresponding to lateral torsional buckling [based on Eqn (6.11)] for α_{LT} = 0.21 and α_{LT} = 0.49 for f_y = 250 MPa. Intermediate values may be obtained by interpolating the values given in these tables.

Table 6.5 Critical Stress $f_{cr,b}$ (in N/mm^2)

KL/r_y	h/t_f 8	10	12	14	16	18	20	25	30	35	40	50	60	80	100
10	22545.42	22249.41	22086.95	21988.41	21924.22	21880.10	21848.49	21799.88	21773.43	21757.47	21747.10	21734.90	21728.27	21721.68	21718.63
20	6218.90	5946.41	5793.01	5698.53	5636.36	5593.33	5562.35	5514.47	5488.28	5472.43	5462.12	5449.97	5443.36	5436.78	5433.73
30	3148.51	2905.13	2763.96	2675.23	2616.04	2574.67	2544.67	2497.92	2472.16	2456.49	2446.27	2434.19	2427.61	2421.04	2418.00
40	2035.61	1820.71	1692.57	1610.39	1554.72	1515.38	1486.60	1441.31	1416.11	1400.69	1390.59	1378.62	1372.07	1365.53	1362.49
50	1492.54	1302.79	1187.08	1111.50	1059.56	1022.43	995.02	951.42	926.88	911.76	901.82	889.98	883.48	876.97	873.94
60	1177.68	1009.25	904.72	835.39	787.13	752.25	726.28	684.51	660.71	645.94	636.17	624.48	618.04	611.57	608.55
70	973.68	823.07	728.32	664.69	619.91	587.23	562.70	522.81	499.81	485.42	475.85	464.33	457.96	451.53	448.52
80	831.04	695.29	609.01	550.48	508.90	478.32	455.18	417.18	395.00	381.01	371.65	360.33	354.03	347.65	344.65
90	725.66	602.40	523.41	469.41	430.74	402.10	380.28	344.13	322.79	309.22	300.08	288.96	282.74	276.42	273.44
100	644.57	531.86	459.18	409.17	373.13	346.27	325.70	291.31	270.81	257.66	248.76	237.86	231.72	225.45	222.49
110	580.18	476.47	409.26	362.77	329.09	303.86	284.44	251.74	232.05	219.33	210.66	199.99	193.94	187.74	184.80
120	527.75	431.79	369.35	325.97	294.42	270.67	252.31	221.20	202.30	190.00	181.57	171.13	165.18	159.04	156.12
130	484.20	394.96	336.71	296.10	266.45	244.05	226.67	197.04	178.90	167.01	158.82	148.62	142.77	136.70	133.80
140	447.43	364.07	309.51	271.36	243.42	222.25	205.77	177.53	160.11	148.63	140.68	130.70	124.95	118.96	116.08
150	415.95	337.76	286.48	250.53	224.14	204.08	188.43	161.48	144.75	133.66	125.94	116.21	110.56	104.64	101.79
160	388.68	315.08	266.72	232.76	207.76	188.72	173.82	148.08	132.01	121.29	113.79	104.29	98.75	92.91	90.08
170	364.82	295.32	249.58	217.40	193.67	175.56	161.36	136.73	121.28	110.92	103.65	94.38	88.94	83.18	80.38
180	343.76	277.93	234.56	203.99	181.42	164.16	150.60	127.01	112.14	102.13	95.07	86.03	80.70	75.02	72.24
190	325.04	262.52	221.28	192.18	170.66	154.18	141.22	118.60	104.27	94.60	87.74	78.93	73.70	68.10	65.35
200	308.27	248.76	209.46	181.70	161.14	145.38	132.97	111.25	97.44	88.08	81.42	72.83	67.70	62.19	59.46
210	293.17	236.39	198.86	172.33	152.66	137.56	125.65	104.77	91.45	82.39	75.93	67.55	62.52	57.09	54.39
220	279.49	225.21	189.30	163.90	145.04	130.56	119.12	99.02	86.16	77.39	71.11	62.93	58.01	52.67	50.00
230	267.04	215.05	180.64	156.27	138.17	124.25	113.25	93.89	81.46	72.96	66.86	58.88	54.06	48.80	46.16
240	255.67	205.78	172.75	149.34	131.94	118.54	107.95	89.27	77.25	69.01	63.08	55.30	50.58	45.39	42.78
250	245.23	197.29	165.53	143.00	126.25	113.35	103.13	85.10	73.47	65.47	59.70	52.11	47.48	42.38	39.80

Table 6.6 Design Bending Stress f_{bd} (in N/mm^2)

		= 0.21			= 0.49		
f_{cr} N/mm^2	λ_{LT}	ϕ_{LT}	χ_{LT}	f_{bd} N/mm^2	ϕ_{LT}	χ_{LT}	f_{bd} N/mm^2
10000	0.1581	0.508	1.000	227.27	0.502	1.000	227.27
9000	0.1667	0.510	1.000	227.27	0.506	1.000	227.27
8000	0.1768	0.513	1.000	227.27	0.510	1.000	227.27
7000	0.1890	0.517	1.000	227.27	0.515	1.000	227.27
6000	0.2041	0.521	0.999	227.07	0.522	0.998	226.79
5000	0.2236	0.527	0.995	226.09	0.531	0.988	224.54
4000	0.2500	0.537	0.989	224.76	0.544	0.975	221.49
3000	0.2887	0.551	0.980	222.76	0.563	0.955	217.03
2000	0.3536	0.579	0.965	219.23	0.600	0.922	209.46
1000	0.5000	0.657	0.924	210.06	0.699	0.843	191.59
900	0.5270	0.673	0.916	208.10	0.719	0.828	188.12
800	0.5590	0.694	0.905	205.65	0.744	0.809	183.95
700	0.5976	0.720	0.891	202.48	0.776	0.787	178.82
600	0.6455	0.755	0.872	198.16	0.817	0.758	172.30
500	0.7071	0.803	0.844	191.90	0.874	0.720	163.70
450	0.7454	0.835	0.825	187.59	0.911	0.696	158.28
300	0.9129	0.992	0.725	164.87	1.091	0.592	134.53
150	1.2910	1.448	0.475	108.05	1.601	0.393	89.24
90	1.6667	2.043	0.310	70.49	2.248	0.266	60.49
80	1.7678	2.227	0.279	63.45	2.447	0.242	54.92
70	1.8898	2.463	0.247	56.22	2.700	0.216	49.11
60	2.0412	2.777	0.215	48.78	3.034	0.189	43.05
50	2.2361	3.214	0.181	41.16	3.499	0.162	36.72
40	2.5000	3.867	0.147	33.34	4.189	0.132	30.11
30	2.8868	4.949	0.112	25.34	5.325	0.102	23.19
20	3.5355	7.100	0.075	17.14	7.567	0.070	15.94
10	5.0000	13.504	0.038	8.73	14.176	0.036	8.28

6.7.1 Elastic Critical Moment of a Section Symmetrical about Minor Axis

In the case of a beam which is symmetrical only about the minor axis (see Fig. 6.7) with bending about major axis, the elastic critical moment for lateral torsional buckling can be calculated by the equation given in Section E-1.2 (Annexe) of the code, which also gives the values of c_1, c_2, and c_3 which are factors to take into account the loading and end restraint conditions.

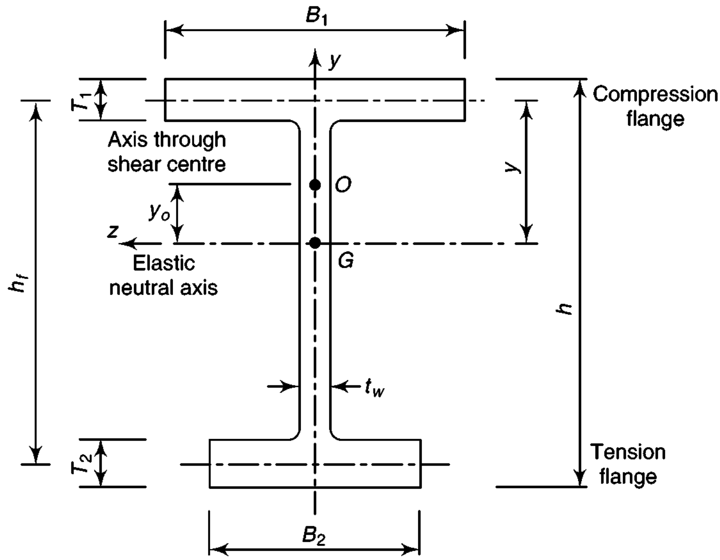


Fig. 6.7 Monosymmetric I-beams

The effective length factors K and K_w vary from 0.5 for full fixity (against warping) to 1.0 for free (to warp) case and 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.

The K_w factor refers to the warping restraint. Unless special provisions to restrain warping of the section at the end lateral supports are made, K_w should be taken as 1.0.

The torsion constant I_t is given by

$$I_t = \sum b_i t_i^3/3 \quad \text{for open sections} \tag{6.15a}$$

$$= \frac{4A_e^2}{\sum(b/t)} \quad \text{for hollow sections} \tag{6.15b}$$

where A_e is the area enclosed by the section and b and t are the breadth and thickness of the elements of the section respectively.

The warping constant I_w, for I-sections mono-symmetric about the weak axis, is given by

$$I_w = (1 - \beta_f)\beta_f I_y h_f^2 \tag{6.16}$$

= 0 for angle, T-, narrow rectangle sections, and approximately for hollow sections

$$\beta_f = I_{fc}/(I_{fc} + I_{ft}) \tag{6.17}$$

where I_{fc} and I_{ft} are the moment of inertia of the compression and tension flanges, respectively, about the minor axis of the entire section. Note that for equal flange beams $\beta_f = 0.5$.

6.7.2 Beams with Other Cross Sections

Channels Unless the loads pass through the shear centre, a channel is subjected to combined bending and torsion. Usual loadings through the centroid or in the plane of the web give rise to such combined stress. For loads in the plane parallel to the web, lateral buckling must be considered (Salmon & Johnson 1996).

For design purposes, lateral torsional buckling equations of symmetrical I-shaped sections may be applied, which are found to err on the unsafe side by about 6% only in the extreme cases (Hill 1954).

Zees For Zees, the loading in the plane of the web causes unsymmetrical bending. The effect of biaxial bending on Z-section was found to reduce the critical moment M_z to about 90% of the value given by Eqn (6.1) (Hill 1954). Salmon and Johnson (1996) suggest using one half of the values of M_{cr} obtained by using Eqn (6.1).

The American code ANSI/AISC: 360-05 gives guidance for calculating the lateral-torsional buckling strength of channels, circular, rectangular, and square hollow sections, Tees and double angles, single angles, and other unsymmetrical shapes. Trahair (2001) provides an extensive coverage on the moment capacities of angle sections.

6.7.3 Compound Beams

(a) *Section classification* Compound sections are classified into plastic, compact, and semi-compact in the same way as discussed for rolled beams in Section 6.2. However, compound beams are treated as a section built-up by welding in the British code. (The Indian code has not specified this clearly.) The limiting width-to-thickness ratios have to be checked as follows (see Fig. 6.8).
   (i) Whole flange consisting of flange plate and rolled beam flange is checked using b_1/t_f, where b_1 is the total outstand of the compound beam flange and t_f is the thickness of the flange of the rolled section.
   (ii) The outstand b_2 of the flange plate from the rolled beam flange is checked using b_2/t_p, where t_p is the thickness of the flange plate.
   (iii) The width-to-thickness ratio of the flange plate between welds b_3/t_p is checked, where b_3 is the width of the flange of rolled sections.
   (iv) The rolled beam flange itself and the web must be checked.

(b) *Moment capacity* The area of flange plates to be added to a given rolled sections to increase the strength by the required amount may be determined as below, for a laterally restrained beam.

   Total plastic modulus required

$$Z_{p,z} = M/f_y \tag{6.18}$$

   where M is the applied factored moment. If $Z_{p,\text{rb}}$ is the plastic modulus of the rolled beam, the additional plastic modulus required is

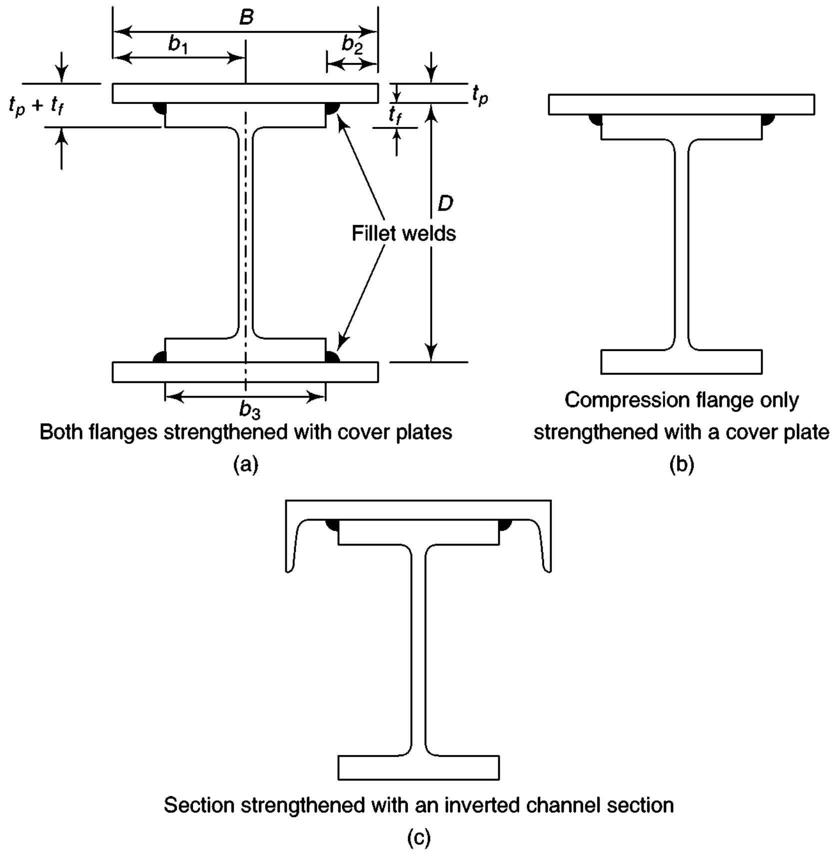


Fig. 6.8 Typical compound beam cross sections

$$Z_{p,\,\mathrm{az}} = Z_{p,\,z} - Z_{p,\,\mathrm{rb}}$$
$$= 2\ Bt_p(D + t_p)/2 \tag{6.19}$$

where Bt_p is the area of the flange plate and D is the depth of the rolled beam. Using Eqn. (6.19), dimensions of the flange plates can be quickly obtained. If the beam is not restrained, successive trails are required.

(c) *Curtailment of flange plates* For a restrained beam with a uniformly distributed load, the theoretical cut-off points for the flange plates can be determined as follows [see Fig. 6.9(a)]. The moment capacity of the rolled beam

$$M_d = 0.909\ f_y Z_{p\ \ b} \quad 1.09 Z_e f_y \tag{6.20}$$

Equate M_d to the moment at P at a distance x from the support

$$wLx/2 - wx^2/2 = M_d$$

where w is the factored uniform load and L is the span of the beam. Solving for x, we will get the theoretical cut-off point. The flange plate should be continued beyond this point so that the weld on the extension can develop the load in the plate at the cut-off point.

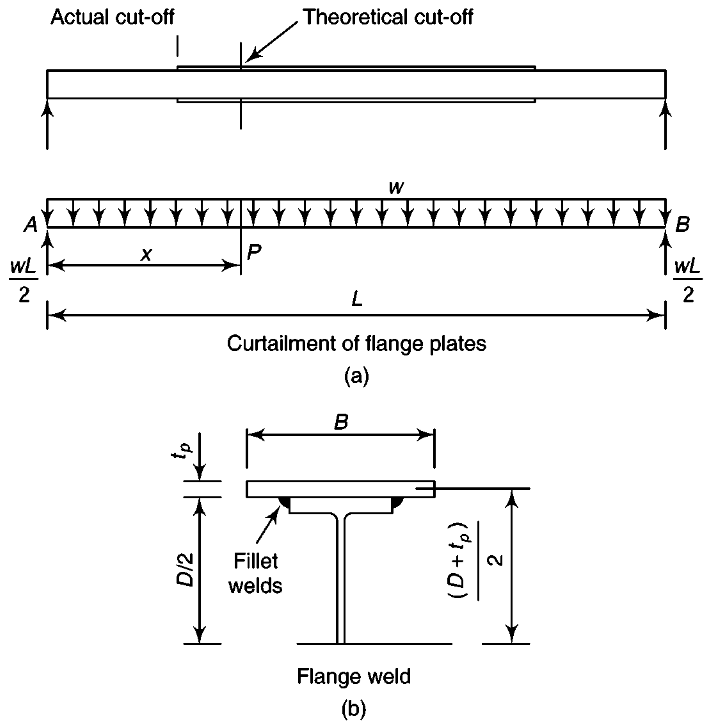


Fig. 6.9 Compound beam design

(d) *Web* The web of the beam should be checked for shear, web buckling, and crippling at support and at points of concentrated loads.

(e) *Welds connecting flange plates and beam flange* The fillet welds between flange plates and rolled beams should be designed to resist horizontal shear using elastic theory [Fig. 6.9(b)].

$$\text{Horizontal shear in each fillet weld} = V_u B t_p (D - t_p)/4I_Z \tag{6.21}$$

where V_u is the factored shear force, I_Z is the moment of inertia about the *Z-Z* axis. The other terms have been defined earlier. The leg length can be selected using the minimum recommended size. Intermittent welds may be specified, but continuous automatic welding considerably reduces the likelihood of failure due to fatigue or brittle fracture (MacGinley and Ang 1992).

Slim Floor Construction

In the early 1990s, engineers in Scandinavia developed the 'slim floor system', which is similar to concrete flat slabs, with 5 to 9 m spans. The essential feature of this system is that the steel beam is contained within the depth if the slab. In the earlier systems, precast concrete slabs were used. Later, deep composite slabs have been developed in UK, Sweden, and Germany.

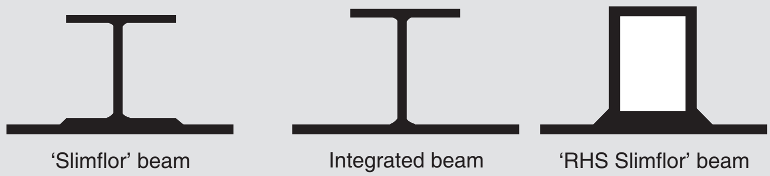


Fig. CS3

The various forms of slim floor beams are shown in Fig. CS3. The 'Slimflor' system developed by British Steel uses an I-section with welded bottom plate. Another system developed by Arbed consists of a section cut from an I-section to which bottom flange plate is welded. Even hollow sections, which have better torsional resistance, with a welded bottom plate have been used. The Corus Corporation developed another system called SLIMDEK®, which uses an asymmetric beam (see Fig. CS4).

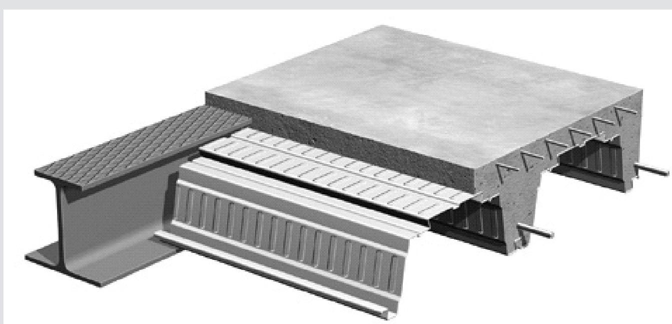


Fig. CS4

Almost the whole steel section is protected from the fire by the floor slab. Hence periods of fire resistance up to 60 minutes are achievable without any protection to the exposed bottom plate. Service ducts up to 160 mm deep × 320 mm long can be accommodated within the depth of Slimdek, by penetrating the web with circular or elongated openings. Extensive tests conducted at City University, London, has shown that a 30-mm concrete cover to the top flange provides sufficient shear bond and hence, welded shear studs are not necessary.

The Slimfloor system optimizes the effective volume of the building and offers the following advantages:

1. *The floor thickness is reduced:* This can be advantageous in tall buildings, where extra floors can be added for the same total height.
2. *Easy installation of equipment:* The integrated beam makes it easier to build under-floor equipment (air-conditioning, piping, electrical networks, etc.) and simplify the fitting of false ceilings.
3. *Increased fire resistance:* It eliminates the need for passive fire protection, resulting in savings in cost and time. Built-in fire resistance of up to 60 minutes, for beams without web-opening.

4. *Economy:* The amount of steel per square metre of floor is relatively low (in the range of 15 to 25 kg/m^2 for beam spans from 5 to 7.5 m)

References

1. www.corusconstruction.com
2. Lawson R.M., Mullett, D.L., and Rackham, J.W. Design of asymmetric Slimflor beams using deep composite decking, The Steel Construction Institute, U.K., SCI-P-175-1997

6.8 Shear Strength of Steel Beams

Since shear force generally exists with bending moments, the maximum shear stress in a beam is to be compared with the shear yield stress. Though bending will govern the design in most steel beams, shear forces may control in cases where the beams are short and carry heavy concentrated loads. The pattern of shear stress distribution in I-section is shown in Fig. 6.10. It may be seen that shear stress varies parabolically with depth, with the maximum occurring at the neutral axis.

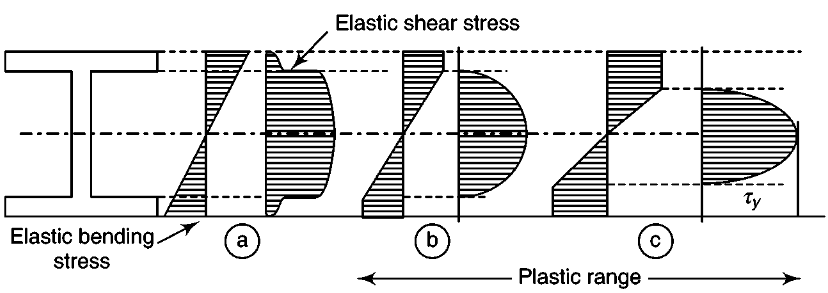


Fig. 6.10 Combined bending and shear in beams

Let us take the case of an I-beam subjected to the maximum shear force (at the support of a simply supported beam). The external shear V varies along the longitudinal axis x of the beam with bending moment as $V = dM/dx$. For beams of open cross section subjected to no twisting, the internal shear stresses τ which resist the external shear V can be written as,

$$\tau = \frac{VQ}{I_z t} \tag{6.22}$$

where V is the shear force at the section, I_z is the moment of inertia of the entire cross section about Z-Z axis about the neutral axis, Q (= $A\bar{y}$) is the static moment of the cross section (above the location at which the stress is being determined) about the neutral axis and t is the thickness of the portion at which τ is calculated.

Using the above equation, the maximum shear stress at the centroidal axis can be evaluated. For the purpose of design, we can assume without much error, the average shear stress for most commonly adopted sections (such as I, channel, T, etc.) as

$$\tau_{\text{av}} = \frac{V}{t_w d_w} \tag{6.23}$$

where t_w is the thickness of the web and d_w is the depth of the web. Whenever there are bolt holes in the web, this stress is multiplied by the ratio of gross web area/net web area. The nominal shear yielding strength of webs (based on the von Mises yield criterion) is given by

$$\tau_y = \frac{f_y}{\sqrt{3}} = 0.58 f_y \tag{6.24}$$

where f_y is the yield stress.

Taking the shear yield stress as 60% of the tensile yields stress, V_p can be written as

$$V_p \approx 0.6 f_y t_w d_w \tag{6.25}$$

This expression gives the nominal shear strength provided by the web when there is no shear buckling of the web. Whether that occurs will depend on d_w/t_w, the depth-thickness ratio of the web. If this ratio is too large, i.e., the web is too slender, the web may buckle in shear either inelastically or elastically.

When the shear capacity of the beam is exceeded, *shear failure* occurs by excessive shear yielding of the gross area of the webs. Shear yielding is very rare in rolled steel beams. Shear is rarely a problem in rolled steel beams; the usual practice being to design the beam for flexure and then to check it for shear.

6.8.1 Shear Buckling of Beam Webs

Since the web of an I-beam is essentially a plate, it may buckle due to shearing stresses which are less than the shearing yield strength of steel. In a plate subjected to pure shear, the shear stresses are equivalent to principal stresses of the same magnitude, one tension and another compression, acting at 45° to the shear stresses. This is shown in the Fig. 4.8(c) given in Chapter 4. Buckling takes place in the form of waves or wrinkles inclined at around 45°.

As discussed in Chapter 4, the shear stress at which buckling of a perfect plate takes place is given by

$$\tau_{\text{cr},\,v} = \frac{k_v \pi^2 E}{12(1 \quad {}^2)(d/t_w)^2} \tag{6.26}$$

where $\tau_{\text{cr},\,v}$ is the elastic critical shear buckling stress of the web and k_v is the buckling coefficient. For a plate with all four edges simply supported, $k_v = 5.35$. When stiffeners are provided only at supports

$$k_v = 4 + \frac{5.34}{(L/d)^2} \qquad L/d \le 1 \tag{6.27a}$$

$$= 5.35 + \frac{4}{(L/d)^2} \qquad L/d \ge 1 \tag{6.27b}$$

where (L/d) is the aspect ratio of the plate with L being the length and d the depth of the beam.

There is some approximation in using Eqn (6.27) as some bending stresses are always present. But these stresses will be very small at the ends of simply supported beams. Note that web buckling due to shear is not a design consideration for rolled beams. However, shear strength of thinner webs of beams of high-strength steels may be less than the yield strength.

6.8.2 Bend Buckling of Webs

The web may undergo local buckling due to the compressive part of the bending stresses along the depth of the beam. The buckling may occur overall or in multiple lengthwise waves. Tests have shown that beam webs are partially restrained against rotation by the flanges and this restraint raises the critical stress by at least 30%. There is no likelihood of bend buckling of the webs of rolled steel I shapes as the d/t ratio is less than 67. However, in plate girders having much thinner webs than rolled sections, bend buckling deserves attention. This is discussed in Chapter 7.

6.8.3 Design for Shear

As per IS 800 the factored design shear force V in a beam due to external actions should satisfy

$$V \leq V_d \tag{6.28a}$$

where V_d, the design strength, is given by

$$V_d = 0.909\ V_n \tag{6.28b}$$

The nominal shear strength of a cross section V_n may be governed by plastic shear resistance or the strength of the web governed by shear buckling. The nominal plastic shear resistance under pure shear is given by $V_n = V_p$, where

$$V_p = \frac{A_v f_{yw}}{\sqrt{3}} \tag{6.29}$$

A_v is the shear area and f_{yw} is the yield strength of the web. The shear area may be calculated as given in Table 6.7 for different cross sections (Nethercot 1991).

Note Fastener holes need not be accounted for in the plastic design shear strength calculation provided that

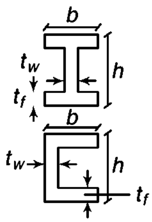
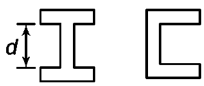
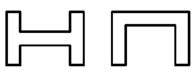
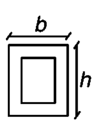
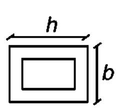
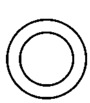
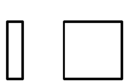
$$A_{vn} \geq 1.26\ (f_y/f_u)A_v$$

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.

6.9 Maximum Deflection

A beam designed to have adequate strength may become unsuitable if it deflects excessively under the loads. Beams that deflect too much may not normally lead to a structural failure, but nevertheless endanger the functioning of the structure. For

Table 6.7 Shear areas

Section	Shear area A_v
 Hot-rolled (a)	ht_w ht_w
 Built-up (b)	dt_w
 Built-up sections or rolled (minor axis bending) (c)	$2bt_f$
 Rectangular hollow section (d)	$Ah/(b+h)$
 Rectangular hollow section (loaded parallel to width) (e)	$Ab/(b+h)$
 Circular hollow section (f)	$2A/\pi$
 Plates and solid bar (g)	A

example, excessive deflection in a floor not only gives a feeling of insecurity, but also damages the non-structural components (cracking of plaster ceilings) attached to it. Excessive deflections in industrial structures often cause misalignment of the supporting machinery and cause excessive vibration. Similarly high deflections in purlins may cause damage to the roofing material. Excessive deflection in the case of a flat roof results in accumulation of water during rainstorms called *ponding*.

Crane misalignment is another consequence of excessive deflection. Since these affect the performance of the structure in its working conditions, the serviceability check is done at working load levels. The deflection in beams is restricted by codes of practice by specifying deflection limitations which are usually in terms of deflection to span ratio (see Table 6 of the code).

The code also recommends that when the deflection, due to dead load plus the live load combination, is likely to be excessive, it is desirable to pre-camber the beams, trusses, and girders. Often larger beam sections are used instead of cambering.

Deflections for some common load cases for simply supported beams together with the maximum moments are given in Fig. 6.11. Some guidelines about design against floor vibration are given in Annex C of the code.

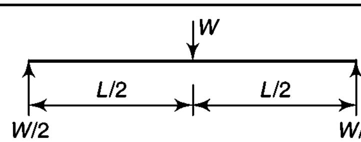
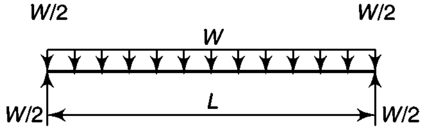
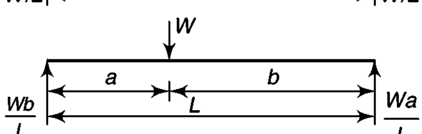
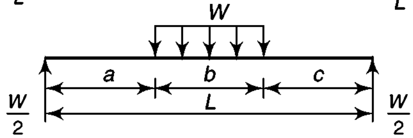
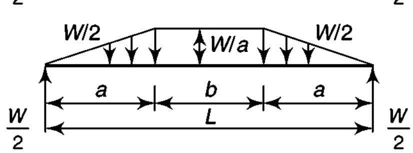
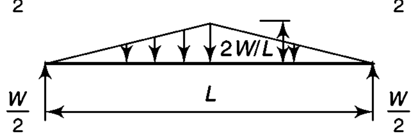
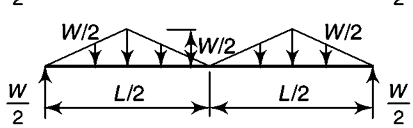
Beam and Load	Maximum Moment	Deflection at Centre
	$WL/4$	$\frac{WL^3}{48EI}$
	$WL/8$	$\frac{5WL^3}{384EI}$
	Wab/L	$\frac{WL^3}{48EI}\left[\frac{3a}{L} - 4\left(\frac{a}{L}\right)^3\right]$
	$W\left(\frac{a}{2}+\frac{b}{8}\right)$	$\frac{W}{384EI}[8L^3 - 4Lb^2 + b^3]$
	$Wa/3$	$\frac{W}{120EI}[16a^2 + 20ab + 5b^2]$
	$WL/6$	$\frac{WL^3}{60EI}$
	$WL/8$	$\frac{WL^3}{73.14EI}$

Fig. 6.11 Simply supported beams: maximum moments and deflections

6.10 Web Buckling and Web Crippling

A heavy load or reaction concentrated on a short length produces a region of high compressive stresses in the web either under the load or at the support. This may cause web failures such as web crippling, web buckling, or web crushing, as shown in Fig. 6.12. Web buckling or vertical buckling occurs when the intensity of vertical compressive stress near the centre of the section becomes greater than the critical buckling stress for the web acting as a column. Tests indicate that for rolled steel beams, the initial failure is by web crippling rather than by buckling. But for built-up beams having greater ratios of depth to thickness of web, failure by vertical buckling may be more probable than failure by web crippling. Provision of web stiffeners at points of load, and reaction or thickening of the web will solve this problem. Since web crippling occurs only at few sections, it is economical to provide stiffeners at these sections.

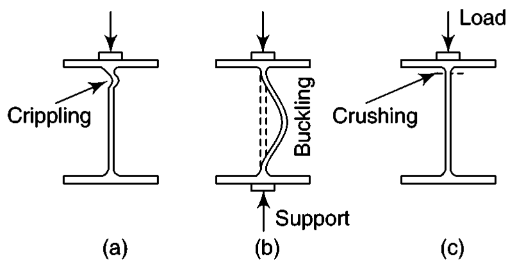


Fig. 6.12 Three possible modes of failures of the web

In the case of web buckling, the web may be considered as a strut restrained by the beam flanges. Such *idealised struts* should be considered at the points of application of concentrated load or reactions at the supports as shown in Fig. 6.13 and Fig. 6.14.

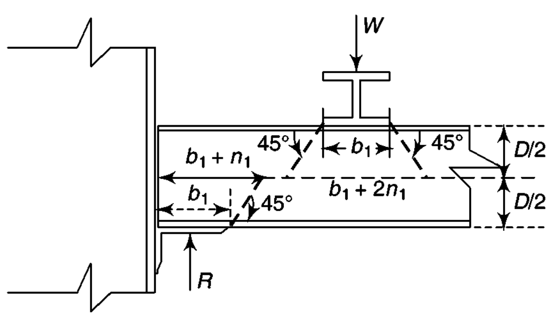


Fig. 6.13 Dispersion of concentrated loads and reactions for evaluating web buckling

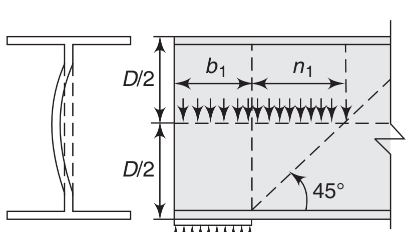


Fig. 6.14 Effective width for web buckling

In both the cases, the load is spread out over a finite length of the web called despersion length, as shown in Fig. 6.13. The dispersion length is taken as $(b_1 + n_1)$ where b_1 is the stiff bearing length and n_1 is the dispersion of 45° line at the mid depth of the section as shown in Fig. 6.14. Hence the *web buckling strength* at the support is given by

$$F_{wb} = (b_1 + n_1)tf_c \tag{6.30}$$

where t is the web thickness and f_c is the allowable compressive stress corresponding to the assumed *web strut*. The effective length of the strut is taken as $L_E = 0.7d$ where d is the depth of the *strut* in between the flanges. Thus, the slenderness ratio is

$$= \frac{L_E}{r_y} \quad \frac{0.7d}{r_y}$$

Since,
$$r_y = \sqrt{\frac{I_y}{A}} \quad \sqrt{\frac{t^3}{12t}} \quad \frac{t}{2\sqrt{3}} \tag{6.31}$$

$$\frac{L_E}{r_y} = 0.7d\frac{2\sqrt{3}}{t} \approx 2.5\frac{d}{t}$$

Hence, the slenderness ratio of the idealized strut is taken as $= 2.5d/t$. For web crippling, an empirical dispersion length of $b_1 + n_2$ is assumed, where n_2 is the length obtained by dispersion through the flange, to the flange to web connection (web toes of fillets), at a slope of 1:2.5 to the plane of the flange (i.e. $n_2 = 2.5d_1$) as shown in Fig. 6.15. As before, the *crippling strength* of the web (also called as the *web bearing capacity*) at supports is calculated as

$$F_{crip} = (b_1 + n_2)\ tf_{yw} \tag{6.32}$$

where f_{yw} is the design yield strength of the web. At an interior point where concentrated load is acting, the crippling strength is given by

$$F_{crip} = (b_1 + 2n_1)\ tf_{yw} \tag{6.33}$$

If the above bearing capacity or crippling strength of the beam web is exceeded, stiffeners must be provided to carry the load.

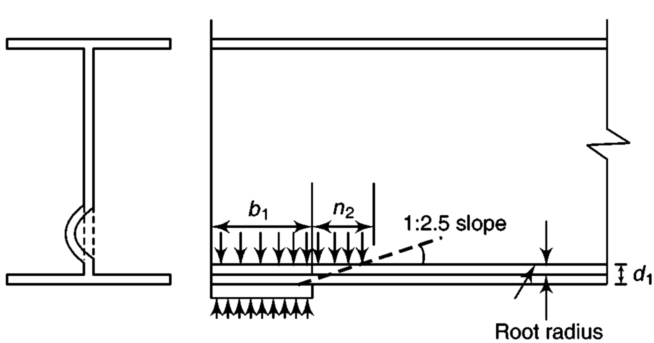


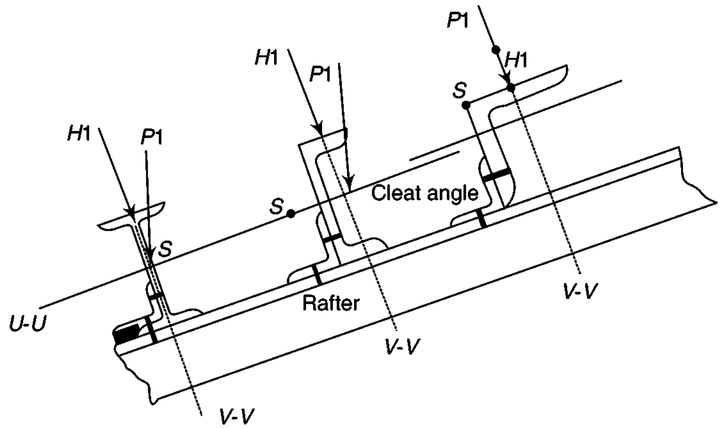
Fig. 6.15 Effective width of web bearing

6.11 Purlins

Purlins are beams used on trusses to support the sloping roof system between the adjacent trusses. Channels, angle sections, and cold formed C- or Z-sections are widely used as purlins. They are placed in an inclined position over the main rafters of the trusses. To avoid bending in the top chords of roof trusses, it is theoretically desirable to place purlins only at panel points. For larger trusses, however, it is more economical to space purlins at closer intervals. In India, where asbestos cement (AC) sheets are used, the maximum spacing of purlins is also restricted by the length of these sheets. AC sheets (though banned in many countries for the risk of lung cancer while working with them) provide better insulation to sun's heat (compared to GI sheets), which can be further improved by painting them white on the top surface. The maximum permissible span for these sheets is 1.68 m. A longitudinal overlap of not less than 150 mm is provided for AC sheets. The purlin spacing is so adjusted with lengths of the sheets that the longitudinal overlaps fall on the purlins to which they are directly bolted. Spacing of purlins should be so fixed that the cutting of sheets is avoided. Hence in practice when AC sheets are used, the purlin spacing is kept between 1.35 to 1.40 m. But in general, purlins are spaced from 0.6 m to about 2 m and their most desirable depth to span ratio is about 1/24. While the dead loads act through the centre of gravity of the purlin section, the wind loads act normal to the roof trusses. Thus, the purlin section is subjected to bending and twisting resulting in unsymmetrical bending.

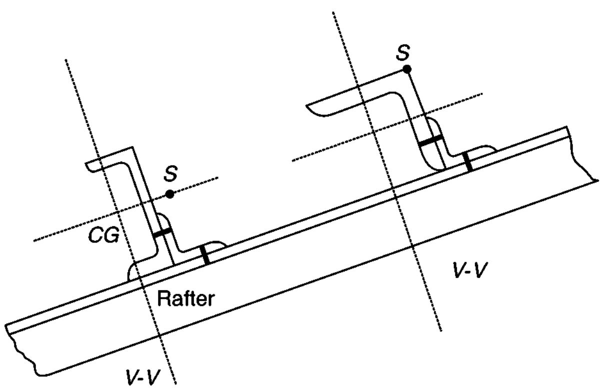
Purlins may be designed as simple, continuous, and cantilever beams. The simple beam design yields the largest moments and deflections. For simply supported purlins the maximum moment will be $WL^2/8$ and if they are assumed as continuous, the moment will be $WL^2/10$. While erecting angle, channel- or I-section purlins, it is desirable that they are erected over the rafter with their flange facing up slope [see Fig. 6.16(a)]. In this position, the twisting moment does not cause instability. If the purlins are kept in such a way that the flanges face the downward slope, then the twisting moment will cause instability [Fig. 6.16(b)].

As we know channels are very weak about their web axes and have a tendency to sag in the direction of the sloping roof and often sag rods are provided midway or at one-third points between the roof trusses to take up the sag. If sag rods are used they will provide lateral support with respect to y-axis bending. Consequently, the moment M_{yy} is reduced and thereby the required purlin section is smaller. The code also permits to take advantage of the continuity of purlins over supports (clause 8.9.1). Note that if sag rods are not used, the maximum moment about the web axis would be $w_{uy}L^2/8$. Thus, when one sag rod is used the moments are reduced by 75% $w_{uy}L^2/32$ and when two sag rods are used at one-third points, the moments are reduced by 91% $w_{uy}L^2/90$. In addition to providing lateral supports to purlins, sag rods also help to keep the purlins in proper alignment during erection until the roof deck is installed and connected to the purlins. Sag rods are often used with channel and I-section purlins, but are very rarely used with angle purlins.



Recommended

(a)



Not recommended

(b)

Fig. 6.16 Orientation of purlins

The design of a purlin is a trail and error process (see next section). Taking advantage of the continuity of the purlin over the supports, the maximum bending moments about the two axes M_u and M_v are calculated separately and checked according to the biaxial bending requirements. This is applicable to channel and I-sections. The recommended bending moments are

$$M_u = PL/10 \text{ and } M_v = HL/10$$

where P acts along v-v axis, H acts along u-u axis, and L is the span of the purlin. It has to be noted that the purlins at the edges or end spans be designed considering local wind effects.

Various linear and non-linear formulae have been suggested for different types of cross sections. Horne and Morris (1981), recommend the following formulae which are safe and sufficiently accurate.

(a) for I-sections

$$[M_z/(Z_{pz}f_y)]^2 + M_y/(Z_{py}f_y) \le 1.0 \tag{6.34a}$$

(b) For all solid sections and closed hollow sections

$$[M_z/(Z_{pz}f_y)]^{5/3} + [M_y/(Z_{py}f_y)]^{5/3} \le 1.0 \tag{6.34b}$$

(c) For all other sections

$$M_z/(Z_{pz}f_y) + M_y/(Z_{py}f_y) \le 1.0 \tag{6.34c}$$

However, the following equation may be used conservatively for biaxial moment of channel and I-sections.

$$\frac{M_u}{M_{du}} + \frac{M_v}{M_{dv}} \le 1.0 \tag{6.34d}$$

6.11.1 Design Procedure for Channel/I-section Purlins

As mentioned earlier, the design of purlins is a trial and error procedure and the various steps involved in the design are as follows:

1. The span of the purlin is taken as the centre-to-centre distance of adjacent trusses.
2. The gravity loads P, due to sheeting and live load, and the load H due to wind are computed. The components of these loads in the direction perpendicular and parallel to the sheeting are determined. These loads are multiplied with partial safety factors γ_f for loads (see Table 4 of the code) for the various load combinations.
3. The maximum bending moments (M_z and M_y) and shear forces (F_z and F_y) using the factored loads are determined.
4. The required value of plastic section modulus of the section may be determined by using the following equations

   $$Z_{pz} = M_z\,\gamma_{m0}/f_y + 2.5(d/b)[M_y\,\gamma_{m0}/f_y] \tag{6.35}$$

   where γ_{m0} is the partial safety factor for material = 1.1, d is the depth of the trial section, b is the breadth of the trial section, M_z and M_y are the factored bending moments about the Z and Y axes, respectively, and f_y is the yield stress of steel.

   Since the above equation involves b and d of a section, we must use a trial section and from the above equation find out whether the chosen section is adequate or not.
5. Check for the section classification (Table 2 of the code).
6. Check for the shear capacity of the section for both the z and y axes (for purlins shear capacity will always be high and may not govern the design. The shear capacity in z and y axes is taken as (Morris & Plum 1996)

   $$V_{dz} = f_y/(\sqrt{3}\,\gamma_{m0})A_{vz}$$

   $$V_{dy} = f_y/(\sqrt{3}\,\gamma_{m0})A_{vy}$$

   and

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$A_{vz} = ht_w$
$A_{vy} = 2b_f t_f$

where h is the height, t_w is the thickness of the web, b_f is the breadth of the flange, and t_f is the thickness of the flange of I-channel section, respectively.

7. Compute the design capacity of the section in both the axes.

$M_{dz} = Z_{pz} f_y / \gamma_{m0} \leq 1.2 Z_{ez} f_y / \gamma_{m0}$
$M_{dy} = Z_{py} f_y / \gamma_{m0} \leq \gamma_f Z_{ey} f_y / \gamma_{m0}$

Note that in the second equation 1.2 is replaced by γ_f. It is because, in the y-direction, the shape factor Z_p/Z_e will be greater than 1.2 and hence if we use the factor as 1.2 we cannot prevent the onset of yielding under unfactored loads.

8. Check for local capacity by using the interaction equation

$(M_z/M_{dz}) + (M_y/M_{dy}) \leq 1.0$

9. Check whether the deflection is under permissible limits (Table 6 of the code).
10. Under wind section (combined with dead load), the bottom flange of the purlin, which is laterally unsupported will be under compression. Hence, under this loading case, the lateral-torsional buckling capacity of the section has to be calculated using Eqns (6.11) to (6.14) and an overall member buckling check is to be made by using the interaction equation

$(M_z/M_{dz}) + (M_y/M_{dy}) \leq 1.0$

where M_{dz} is the lateral-torsional buckling strength of the member.

6.11.2 Empirical Design of Angle Purlins

An angle purlin section is unsymmetrical about both axes. Angle purlin may be used wherever the slope of the roof is less than 30°. The vertical and normal loads acting on the purlin are determined and the maximum bending moment is calculated as $PL/10$ and $HL/10$, where P and H are the respective vertical and horizontal loads. A trial angle section may be assumed with depth as 1/45 of the span and width as 1/60 of the span. The trial depth and width are arrived to ensure that the deflection is within limits. Equation (6.34d) is again used to satisfy the adequacy of the section.

BS 5950-1:2000 gives the following empirical design method for purlins with the following general requirements.

(a) Unfactored loads are used in the design.
(b) The span should not exceed 6.5 m.
(c) If the purlin spans one bay only, it should be connected at the ends by at least two bolts.
(d) If the purlins are continuous over two or three spans, with staggered joints in adjacent lines of members, at least one end of any single bay member should be connected by not less than two bolts.

The rules for empirical design of angle purlins are as follows:

(a) The roof slope should not exceed 30°.

CHAPTER 7

Design of Plate Girders

Introduction

A plate girder is basically an I-beam built up from plates using riveting or welding. It is a deep flexural member used to carry loads that cannot be economically carried by rolled beams. Standard rolled sections may be adequate for many of the usual structures; but in situations where the load is heavier and the span is also large, the designer has the following choices (Fig. 7.1).

- Use two or more regularly available sections, side-by-side.
- Use a cover-plated beam, i.e., weld a plate of adequate thickness to increase the bending resistance of the flange.
- Use a fabricated plate girder, which provides the freedom (within limits) to choose the size of web and flanges.
- Use a steel truss or a steel-concrete-composite truss.

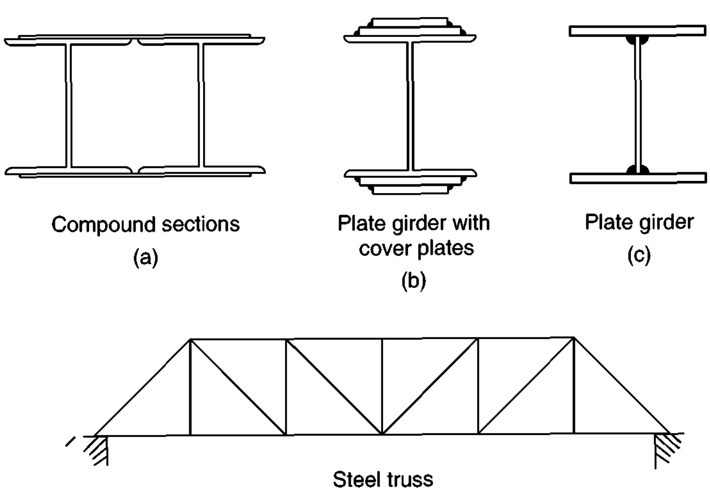


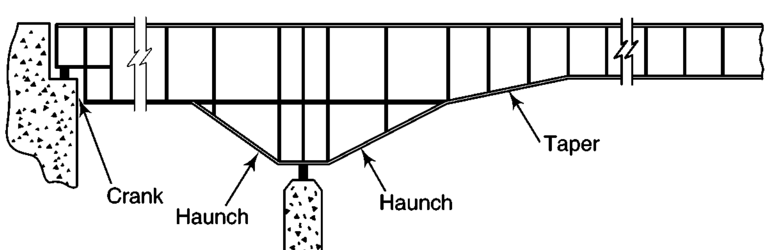
Fig. 7.1 Options for beams of long spans to carry heavy loads

Among the options available, the first is usually uneconomical and does not satisfy deflection limitation. The second option is advantageous where the rolled section is marginally inadequate. Therefore, the real choice lies between the plate girder and the truss girder. The moment resisting capacity of plate girders lies between rolled I-sections and truss girders. Truss girders involve higher costs of fabrication and erection, problems of vibration and impact, and require higher vertical clearance. For short spans (<10 m), plate girders are uneconomical due to higher connection costs and rolled I-sections are the preferred choice.

7.1 Plate Girders

As stated earlier, plate girders are deep flexural members used to carry loads that cannot be carried economically by rolled beams. Plate girders provide maximum flexibility and economy. In the design of a plate girder, the designer has the freedom to choose components of convenient size, but has to provide connections between the web and the flanges. Plate girders offer a unique flexibility in fabrication and the cross section can be uniform or non-uniform along the length. It is possible to provide the exact amount of steel required at each section along the length of the girder by changing the flange areas and keeping the same depth of the girder. In other words, it can be shaped to match the bending moment curve itself. Thus, a plate girder offers limitless possibilities to the creativity of the engineer.

It is a normal practice to fabricate plate girders by welding together three plates, though (see Fig. 7.6) in the past, plate girders were constructed by riveting or bolting. It is possible to have tapered, cranked, and haunched girders (see Fig. 7.2). We can also have holes in the webs to accommodate the services (see Fig. 7.3). The designer may choose to reduce flange thickness (or breadth) in the zone of low applied moment, especially when a field-splice facilitates the change. Similarly in the zone of high shear, the designer might choose a thicker web plate (see Fig. 7.4). Alternatively, higher grade steel may be employed in zones of high applied moment and shear, while standard grade steel (Fe 410) could be used elsewhere. Hybrid girders, with different strength material in the flanges and web, offer a method of closely matching resistance of the section to the requirements (Fan et al. 2006; Veljkovic & Johansson 2004; Schilling 1968).



Note Splices, camber, variation in flanges, web and material strength are not shown

Fig. 7.2 Plate girder with haunches, tapers, and cranks

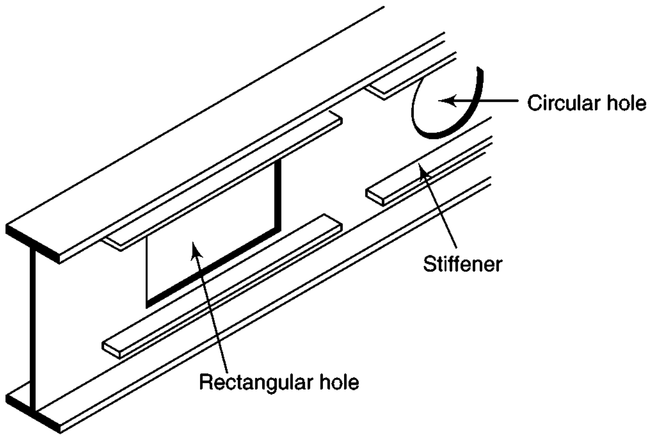


Fig. 7.3 Plate girder with holes for services

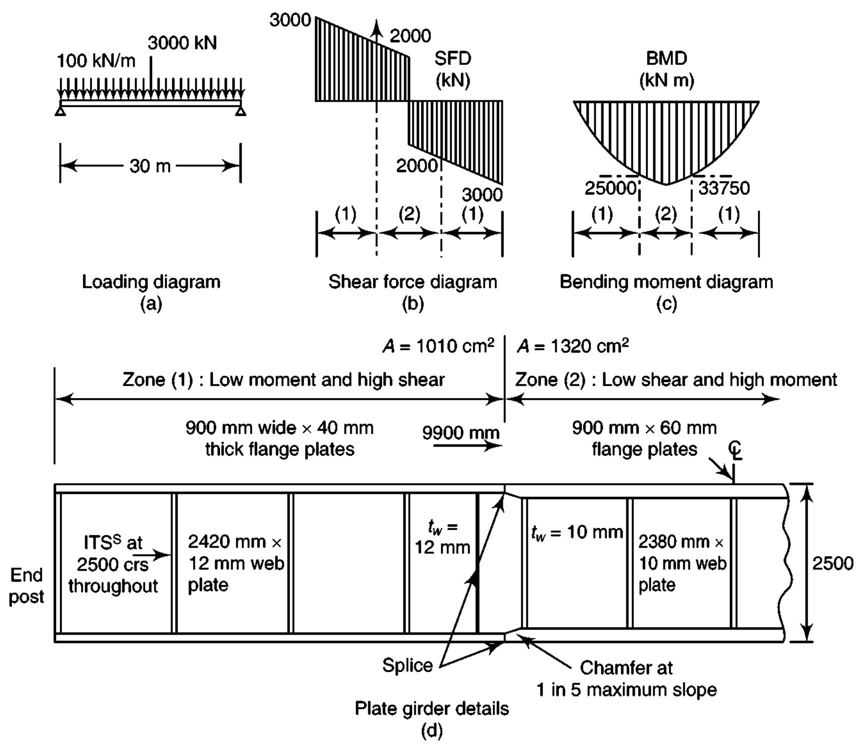


Fig. 7.4 Plate girder with splice and variable cross section (ESDEP 2006)

For making the cross section efficient in resisting the in-plane bending, it is required that maximum material is placed as far away from the neutral axis as possible. From this point of view it is economical to keep the flanges as far apart as possible. The axial forces in the flanges decrease, as the depth of the girder increases. Thus a smaller area of cross section would suffice than would be the case if a smaller depth were chosen. However, this would also mean that the web would be thin and deep. In such a situation, premature failure of the girder due to web buckling in shear might occur. Here there is a choice between thin web provided with vertical and horizontal stiffeners, and a thicker web requiring no stiffening (and therefore avoiding costly fabrication). The choice between the two depends upon a careful examination of the total costs of both forms of construction.

7.1.1 Examples of Plate Girders

Most steel highway bridges in USA for spans less than about 24 m are steel rolled beam bridges. For longer spans, plate girders compete very well economically. Similarly, when loads are extremely large, as in railway bridges, plate girders may prove to be economical even for smaller spans. The upper economical limit of plate girder spans depend on the following factors: (a) whether the bridge is simple or continuous, (b) whether it is a highway or a railway bridge, and (c) the length of the section which can be transported in one piece. In general, plate girders are economical for railway bridges of spans 15–40 m and for highway bridges of spans 24–46 m. They may be very competitive for much longer spans, when they are continuous.

Plate girders are also used in buildings when it is required to support heavy concentrated loads. Such situations may arise when a large hall with no interfering columns is desired on a lower floor of a multi-storey building.

7.1.2 Types of Sections

Several possible plate girder arrangements are shown in Fig. 7.6. Figure 7.6(a) shows the simplest type of plate girder in which the flange plates are made of a pair of angles and they are connected to a solid web plate to form the plate girder. For larger moments, flange area can be increased by riveting/bolting additional plates, also known as cover plates, as shown in Fig. 7.6(b).When there are head room restrictions in buildings, and larger depths of plate girders are not possible, box girders may be an option as shown in Fig. 7.6(c). Box girders also provide greater lateral stability. If the number of cover plates is to be reduced, the flange area can be compensated by providing plates inserted between the web and flange angles [Fig. 7.6(d)]. These arrangements are sometimes adopted to keep the rivets/bolts both connecting flange angles and cover plates shorter. Figures 7.6(e) and 7.6(f) show typical welded plate girders. The form shown in Fig. 7.6(e) is the most commonly used type of welded plate girder. Here, flange angles are not required and instead of using a number of cover plates, a thick plate is used as the flange. As discussed earlier, a thinner cover plate can be used where lesser flange area is desired such as at regions away from the maximum bending moment and the plates of different thicknesses can be joined by butt welding.

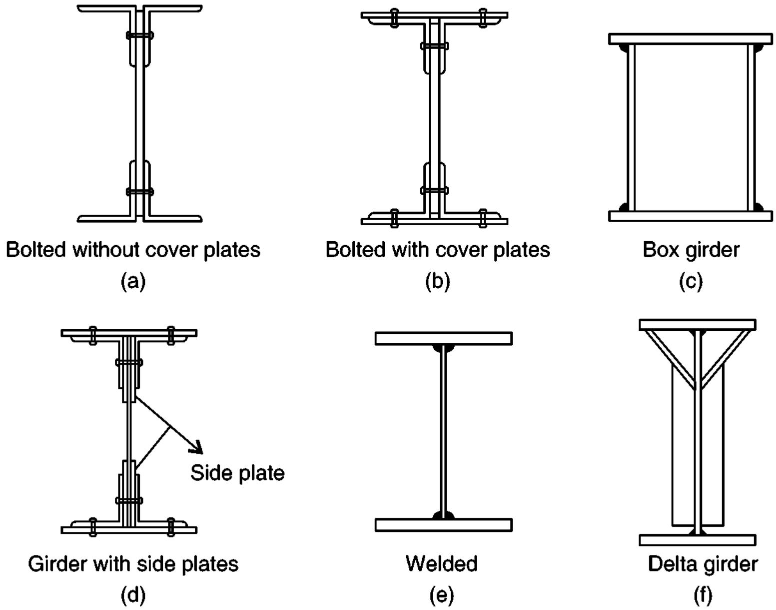


Fig. 7.6 Types of plate girders

7.1.3 Elements of a Plate Girder

The various components of welded stiffened and unstiffened plate girders, shown in Fig. 7.7, are as follows:

(a) Web plate
(b) Flange plate with or without cover plates (with curtailment of cover plate)
(c) Bearing stiffeners or end post (EP)
(d) Intermediate transverse stiffeners (ITS)
(e) Longitudinal stiffeners (LS)
(f) Web splices
(g) Flange splices
(h) Connection between flange and the web
(i) End bearing or end connections

Slender webs (with large *d*/*t* values) would buckle at relatively low values of applied shear loading.

A girder of high strength to weight ratio can be designed by incorporating the post buckling strength of the web in the design method employed. This would be particularly advantageous where the reduction of self-weight is of prime importance. Examples of such situations arise in long span bridges, ship girders, transfer girders in buildings, etc.

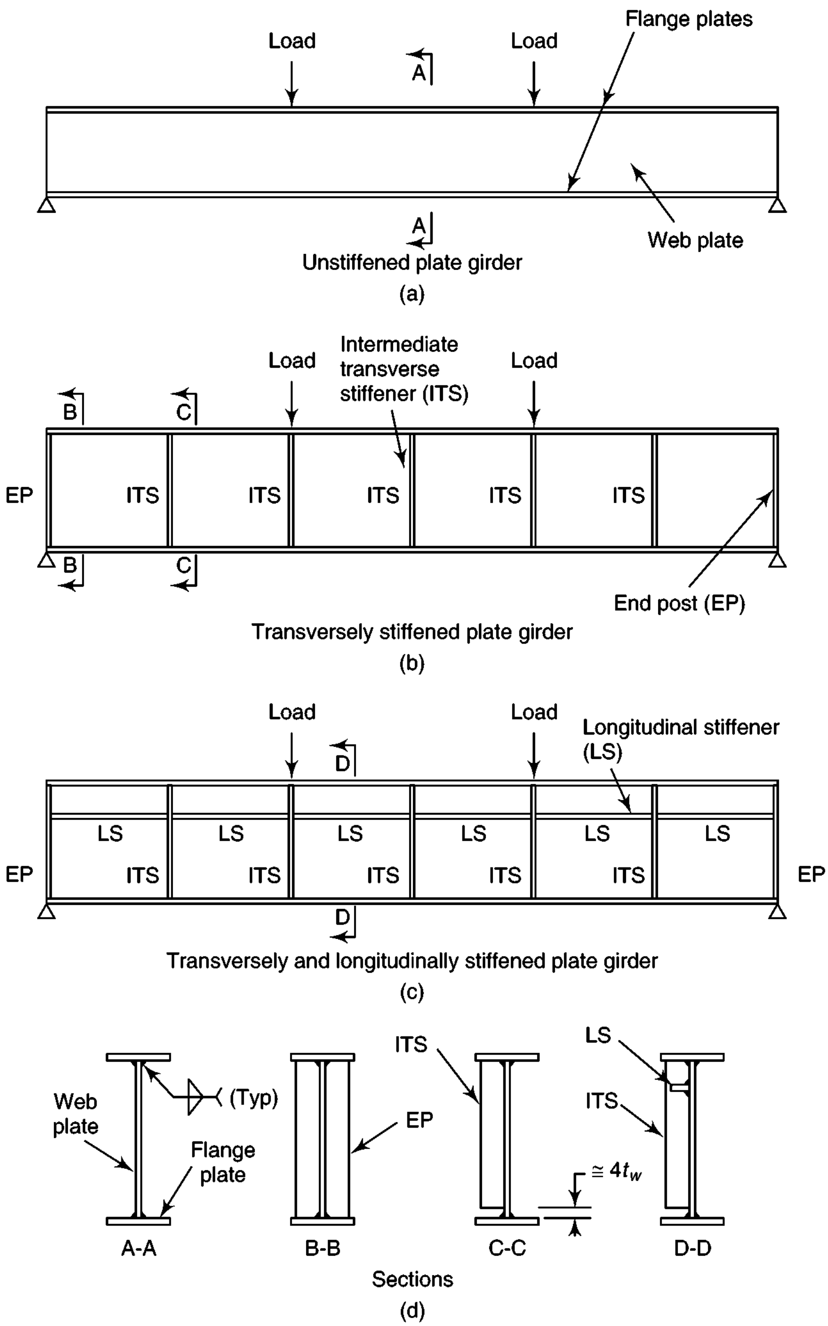


Fig. 7.7 Stiffened and unstiffened plate girders

7.2 General Considerations

Problems arise in plate girders because of the deep thin webs. One way of improving the load carrying capacity of a slender web plate is to provide stiffeners; selection of the right kind of stiffening is an important consideration in a plate girder design.

The modes of failure of a plate girder are by yielding of the tension flange and buckling of the compression flange. The compression flange buckling can take place in various ways, such as vertical buckling into the web, flange local buckling, or lateral torsional buckling.

Plate girders depend on the post-buckling strength of the webs. At high shear locations in the girder web, normally near the supports and the neutral axis, the principal planes would be inclined to the longitudinal axis of the member. Along the principal planes, the principal stresses would be diagonal tension and diagonal compression. Though diagonal tension does not cause any problem, diagonal compression causes the web to buckle in a direction perpendicular to its action. This problem can be solved by any of the following three ways:

- Reduce the depth to thickness ratio of the web such that the problem is eliminated.
- Provide web stiffeners to form panels that would enhance the shear strength of the web.
- Provide web stiffeners to form panels that would develop tension field action to resist diagonal compression.

Figure 7.8(a) shows the tension field action in a panel. As the web begins to buckle, the web loses its ability to resist the diagonal compression. The diagonal compression is then transferred to the transverse stiffeners and the flanges. The vertical component of the diagonal compression is supported by the stiffeners and the flanges resist the horizontal component. The web resists only the diagonal tension and this behaviour of the web is called *tension field action*. The behaviour is very similar to a Pratt truss, in which the vertical members carry compression and the tension is carried by the diagonals, as shown in Fig. 7.8(b). The contribution of the tension field action is realized only after the web starts to buckle. The total strength of the web is made up of the web strength before buckling and the post buckling strength developed due to tension field action. Thus, the provision of suitably spaced stiffeners, called *intermediate stiffeners,* can be used to develop tension field action and improve the shear capacity of the web. The main purpose of these stiffeners is to provide stiffness to the web rather than to resist the applied loads.

Additional stiffeners, called *bearing stiffeners*, are provided at points of concentrated loads, to protect the web from the direct compressive loads. They can simultaneously act as intermediate stiffeners also.

Application of concentrated loads on the top flange may produce web yielding, web crippling, and side-sway web buckling. This form of buckling occurs when the compression in the web causes the tension flange to buckle laterally. In order to prevent this, the relative movements of the flanges should be restrained by lateral bracing. General welding procedures are followed for connecting the components during the design of welds.

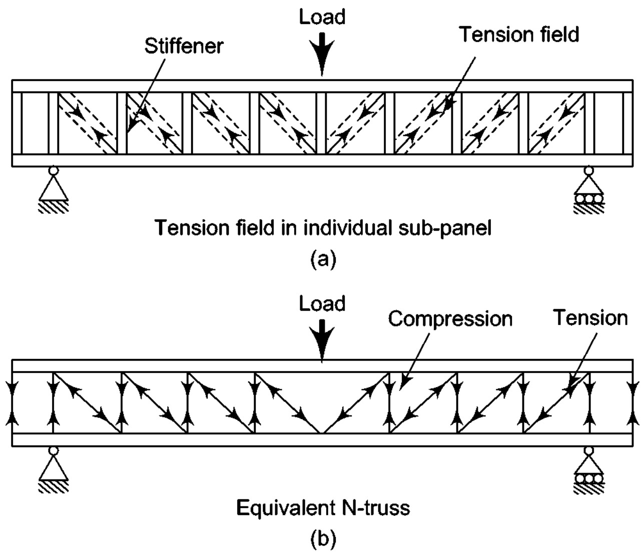


Fig. 7.8 Tension field action and the equivalent N-truss

7.3 Preliminary Design Procedure

General dimensions of a plate girder are fixed based on its optimum behaviour. Some guidelines for proportioning the flanges and webs are given in Section 7.8. One condition that may limit the proportions of the girder is the largest size that can be fabricated in shop and transported to the site. There may be transportation problems such as clearance requirements (of overhead bridges) that limit maximum depths to about 3–3.6 m. A plate girder design begins with values of live load shears and bending moments and estimated dead loads. After the selection of the girder depth, the web may be designed to resist the shear, taking into account the minimum thickness requirements.

A major portion of the moment of resistance in a I-section plate girder is provided by the flanges; the contribution of the web being very small. On the tension side, provision of bolt holes may reduce the effective flange area, which may be compensated by adding more flange area. In the design of a plate girder, a trial section needs to be assumed initially and the amount of load to be carried includes the self weight of the girder also. A preliminary estimate of the girder weight can be obtained using the simplified flange area method.

The design requirements for plate girders are addressed in Section 8 of IS 800 : 2007 specification. The identification of a laterally supported member as a beam or a plate girder is decided based on the d/t_w ratio, where d is the depth of the beam and t_w is the thickness of the web. If d/t_w is less than 67 , the member is classified as a beam and the respective clauses applicable to beams are used, irrespective of whether the member is made up of plates or a rolled section. If the ratio is greater than 67ε, the clauses relating to plate girders are invoked.

7.3.1 Minimum Web Thickness

The thickness of the web in a section should satisfy the following requirements with respect to serviceability (clause 8.6.1 of the code):

(a) When transverse stiffeners are not provided:

$$\frac{d}{t_w} \leq 200\varepsilon \text{ (web connection to flanges along both longitudinal edges)}$$

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

(b) When only transverse stiffeners are provided:

(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 $c < 0.74d$

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

(iv) When $c > 3d$, the web is considered as unstiffened.

(c) When transverse stiffeners and longitudinal stiffeners are provided at one level only, at $0.2d$ from the compression flange:

(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 there is a second longitudinal stiffener (provided at neutral axis):

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

where d is the depth of the web, t_w is the thickness of the web, c is the spacing of the transverse stiffener (see Fig. 7.9), $\varepsilon_w = \sqrt{250/f_{yw}}$, and f_{yw} is the yield stress of the web.

These criteria are to ensure that the web will not buckle under normal service conditions. It should also be ensured that the web is strong enough so that the flange will not buckle into the web. It is also evident that a thinner web can be used if the c/d ratio is less than one and that such a web panel has a higher strength.

7.3.2 Flexural Strength

The method by which the calculation of the bending capacity of a plate girder is determined depends upon the classification of the flanges and the web as calculated from b/t_f and d/t_w ratios. The limiting ratios are presented in the chapter on beams. As mentioned earlier, if the web slenderness ratio is less than 67ε, then the member should be designed as a rolled beam section. The flexural strength of a plate girder is based on the tension flange yielding or otherwise on the compression flange buckling. The compression flange buckling strength will be decided by either flange local buckling or lateral torsional buckling (see Section 6.7).

For sections with slender webs, when their flanges are plastic, compact, or semi-compact (i.e., $d/t_w > 67\varepsilon$), the design bending strength is calculated using one of the following methods:

(a) The applied moment is resisted by the flanges with a uniform stress distribution and the applied shear is resisted by the web. Using this assumption, it is convenient to make preliminary sizes of the girder but the final design may turn out to be uneconomical. The method is very simple and applicable only to laterally fully restrained beams.

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

For sections with slender flanges, moment capacity is calculated based on the approach similar to that of a slender rolled section. In the design of plate girders, a minimum weight solution by adopting thin webs may not be economical if we consider the fabrication cost of the stiffeners.

7.3.3 Lateral Torsional Buckling of Plate Girders

The same procedure as adopted for rolled sections (Section 6.7) may be used except that different values of the imperfection coefficient α_{LT} have to be used to account for higher residual stresses in welded plate girders. Approximate formulae may be used to arrive at the sections properties of the members.

7.3.4 Shear Strength

The shear strength of a plate girder depends upon the depth to thickness ratio of the web and the spacing of the intermediate stiffeners provided. Webs of plate girders are usually stiffened transversely as shown in Fig. 7.7. This helps to increase the ultimate shear resistance of the webs. The shear capacity of the web has two components, namely, strength before the onset of buckling and strength after post-buckling. As the shear load is increased on a stiffened web panel, the web panel buckles (see Fig. 7.9). This load does not indicate the maximum shear capacity of the web. The load can be still increased and the web panel continues to carry further load relying on the tension field action. Part of the buckled web takes the load in tension. This tension member action is across the web panel in an inclined direction to the web panel diagonal as shown in Fig. 7.10.

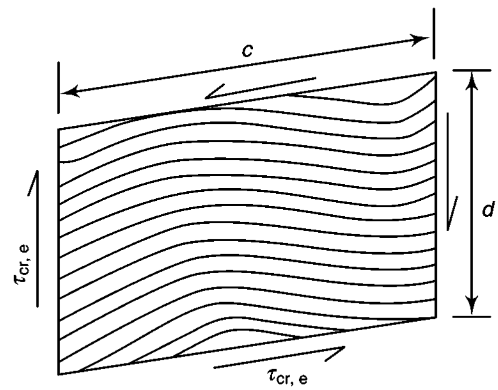


Fig. 7.9 Buckling of the web panel

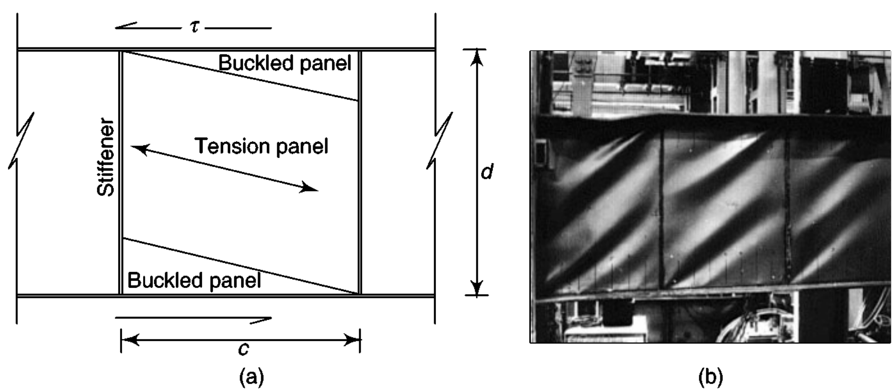


Fig. 7.10 (a) Panel showing tension field and (b) tension field in a plate girder

At this stage, the girder acts like an N-truss (Fig. 7.8) with the compression forces being carried by the flanges and the intermediate stiffeners and the buckled web resisting the tension. This additional reserve strength is termed as *tension field action*. It may be noted here that if no intermediate stiffeners are present or their spacing is large, it is not possible for tension field action to take place and the shear capacity is restricted to the strength before buckling. The stiffener spacing c influences both buckling and post-buckling behaviour of the web under shear. The various stages of shear resistance are explained in the following section.

A long span plate girder may have various web panels, each panel having different combinations of bending moments and shear forces. Panels, which are closer to the support will be predominantly under shear, and those close to the centre will be under predominant bending moments. The effect of the shear alone will be considered first, followed by the case where the effect of combined bending moment and shear forces would be treated.

7.4 Web Panel Subjected to Shear

7.4.1 Shear Resistance Before Buckling (Stage 1)

When a square web plate is subjected to vertical shear, complementary shear stresses are developed to satisfy equilibrium of the plate. As a consequence, the plate develops diagonal tension and diagonal compression.

Consider a small element E in equilibrium inside the web plate subject to a shear stress τ (see Fig. 7.11). The element is subjected to principal compression along the direction AC and principal tension along the direction BD. As the applied loading is gradually increased,   in turn will increase and the plate will buckle along the direction of the compressive diagonal AC. The plate cannot take any further increase in compressive stress. The corresponding shear stress τ in the plate is called the 'elastic critical shear stress' $\tau_{cr,\,e}$ of the panel. The value of $\tau_{cr,\,e}$ can be easily determined from the classical stability theory, if the boundary conditions of the plate can be established. However, it is very difficult to know the true boundary conditions of the plate provided by the flanges and stiffeners. Therefore, conservatively, simply supported conditions are assumed at the boundaries. The critical shear stress in such a case is given by

$$\tau_{cr,\,e} = k_v \frac{\pi^2 E}{12(1-\mu^2)}\left(\frac{t_w}{d}\right)^2 \tag{7.1}$$

where k_v is a coefficient which depends upon the support conditions and μ is Poisson's ratio.

$k_v = 5.35$, when transverse stiffeners are provided only at support
$= 4.0 + 5.35/(c/d)^2$ for $c/d < 1.0$
$= 5.35 + 4.0/(c/d)^2$ for $c/d \geq 1.0$

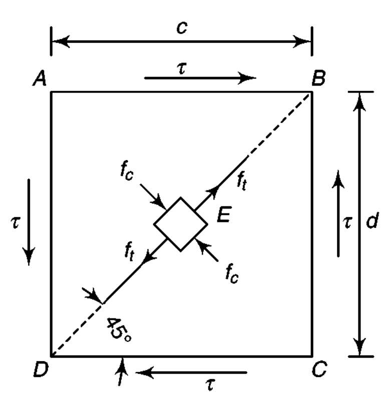


Fig. 7.11 Unbuckled shear panel ($\tau < \tau_{cr}$)

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

The limiting value of d/t_w is given by

$$\frac{d}{t_w}\sqrt{\frac{f_y}{250}} = 82.0$$

Thus, it is seen that for $f_y = 250$ MPa, elastic buckling due to shear will occur when the d/t ratio of the web is greater than 82.

The elastic buckling stress may be significantly increased either by using intermediate transverse stiffeners to decrease the aspect ratio c/d (thereby increasing the value of buckling coefficient k_v), or by using longitudinal stiffeners to decrease the depth-to-thickness ratio d/t_w. The aspect ratio of each panel in the range of 0.5 to 2 is found to be more efficient.

IS 800 : 2007 provisions for Resistance to Shear Buckling Resistance to shear buckling shall be verified when

$$\frac{d}{t_w} > 67\varepsilon \text{ for an unstiffened web}$$

$$> 67\varepsilon\sqrt{\frac{k_v}{5.35}} \text{ for a stiffened web}$$

where k_v is the shear buckling coefficient and $\varepsilon = \sqrt{250/f_y}$.

7.4.2 Shear Buckling Design

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:

- Simple post-critical method
- Tensile field theory

These two methods are discussed in the following sections.

7.4.2.1 Simple Post-critical Method

The simple post-critical method is a general method and can be applied to the design of all girders. The simple post critical method 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

$$V_n = V_{cr} \tag{7.2a}$$

where V_{cr} is the shear force corresponding to web buckling.

$$V_{cr} = A_v \tau_b \tag{7.2b}$$

where A_v is the shear area (see Table 6.7) and τ_b is the shear stress corresponding to web buckling, determined as follows:

Table 7.1 Shear Buckling Strength t_{au} (N/mm^2) of Webs for f_y = 250 MPa

	Stiffener spacing ratio c/d														
d/t	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0	2.5	3.0	Infinity
55	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3
60	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3
65	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3
70	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	143.4	141.9	138.0
75	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	143.7	141.4	139.6	136.8	135.1	131.0
80	144.3	144.3	144.3	144.3	144.3	144.3	144.3	144.3	140.7	137.5	135.0	133.2	130.1	128.3	123.9
85	144.3	144.3	144.3	144.3	144.3	144.3	144.3	139.5	134.7	131.3	128.7	126.7	123.4	121.5	116.9
90	144.3	144.3	144.3	144.3	144.3	144.3	140.7	133.7	128.7	125.1	122.3	120.2	116.8	114.8	109.8
95	144.3	144.3	144.3	144.3	144.3	141.6	135.4	128.0	122.7	118.9	116.0	113.8	110.1	108.0	102.8
100	144.3	144.3	144.3	144.3	143.9	136.6	130.0	122.3	116.7	112.7	109.6	107.3	103.4	101.2	95.7
105	144.3	144.3	144.3	144.3	139.3	131.5	124.7	116.6	110.7	106.5	103.3	100.8	96.8	94.4	87.8
110	144.3	144.3	144.3	143.8	134.7	126.5	119.4	110.9	104.7	100.3	98.4	94.9	89.6	86.6	80.0
115	144.3	144.3	144.3	139.6	130.0	121.5	114.0	105.1	98.7	94.6	90.1	86.9	81.9	79.3	73.2
120	144.3	144.3	144.3	135.4	125.4	116.5	108.7	99.4	92.9	86.8	82.7	79.8	75.3	72.8	67.2
125	144.3	144.3	142.8	131.2	120.7	111.5	103.4	94.1	85.6	80.0	76.2	73.5	69.4	67.1	61.9
130	144.3	144.3	139.1	126.9	116.1	106.5	100.1	87.0	79.1	74.0	70.5	68.0	64.1	62.0	57.3
135	144.3	144.3	135.3	122.7	111.5	101.5	92.8	80.7	73.4	68.6	65.4	63.0	59.5	57.5	53.1
140	144.3	144.3	131.6	118.5	106.8	97.9	86.3	75.0	68.2	63.8	60.8	58.6	55.3	53.5	49.4
145	144.3	142.9	127.8	114.3	102.2	91.2	80.5	69.9	63.6	59.5	56.7	54.6	51.5	49.9	46.0
150	144.3	139.6	124.1	110.0	99.4	85.3	75.2	65.4	59.4	55.6	52.9	51.1	48.2	46.6	43.0
155	144.3	136.4	120.3	105.8	93.1	79.9	70.4	61.2	55.7	52.1	49.6	47.8	45.1	43.6	40.3
160	144.3	133.2	116.6	101.6	87.3	74.9	66.1	57.4	52.2	48.8	46.5	44.9	42.3	40.9	37.8

(*contd*)

165	144.3	129.9	112.8	99.1	82.1	70.5	62.1	54.0	49.1	45.9	43.8	42.2	39.8	38.5	35.6
170	144.3	126.7	109.0	93.4	77.4	66.4	58.5	50.9	46.3	43.3	41.2	39.7	37.5	36.3	33.5
175	143.4	123.5	105.3	88.1	73.0	62.6	55.2	48.0	43.7	40.8	38.9	37.5	35.4	34.2	31.6
180	140.8	120.2	101.5	83.3	69.0	59.2	52.2	45.4	41.3	38.6	36.8	35.5	33.4	32.4	29.9
185	138.1	117.0	99.7	78.9	65.3	56.1	49.4	43.0	39.1	36.5	34.8	33.6	31.7	30.6	28.3
190	135.4	113.8	94.5	74.8	61.9	53.1	46.9	40.7	37.0	34.6	33.0	31.8	30.0	29.0	26.8
195	132.8	110.5	89.7	71.0	58.8	50.5	44.5	38.7	35.2	32.9	31.3	30.2	28.5	27.6	25.5
200	130.1	107.3	85.3	67.5	55.9	48.0	42.3	36.8	33.4	31.3	29.8	28.7	27.1	26.2	24.2
205	127.4	104.0	81.2	64.2	53.2	45.7	40.2								
210	124.8	100.8	77.4	61.2	50.7	43.5	38.4					Not applicable			
215	122.1	99.4	73.8	58.4	48.4	41.5	36.6								
220	119.4	94.9	70.5	55.8	46.2	39.6	34.9								
225	116.8	90.8	67.4	53.3	44.2	37.9	33.4								
230	114.1	86.9	64.5	51.0	42.3	36.3	32.0								
235	111.4	83.2	61.8	48.9	40.5	34.7	30.6								
240	108.8	79.8	59.2	46.9	38.8	33.3	29.4								
245	106.1	76.6	56.8	45.0	37.2	32.0	28.2								
250	103.4	73.5	54.6	43.2	35.8	30.7	27.1								

Note Intermediate values may be obtained by linear interpolation.

(a) When $\lambda_w \leq 0.8$

$$\tau_b = f_{yw}/\sqrt{3} \tag{7.3a}$$

(b) When $0.8 < \lambda_w < 1.2$

$$\tau_b = [1 - 0.8(\lambda_w - 0.8)](f_{yw}/\sqrt{3}) \tag{7.3b}$$

(c) When $\lambda_w \geq 1.2$

$$\tau_b = f_{yw}/(\sqrt{3}\lambda_w^2) \tag{7.3c}$$

where λ_w is the non-dimensional web slenderness ratio for shear buckling stress, given by

$$\lambda_w = \sqrt{f_{yw}/(\sqrt{3}\tau_{cr,e})} \tag{7.4}$$

The elastic critical shear stress of the web $\tau_{cr,\,e}$ is given by Eqn (7.1)

Table 7.1 provides the values of shear buckling strength τ_b for various values of stiffener spacing ratio c/d and depth-to-thickness ratio d/t_w.

7.4.2.2 Web Strength using Tension Field Theory

As stated earlier, a plate girder with intermediate stiffeners ($1.0 \leq c/d \leq 3.0$) derives considerable post-buckling shear strength due to the tension field action. Tension field action can be applied to a certain range of girders only and will produce efficient designs of girders due to its taking advantage of the post-buckling reserve of resistance. According to the theory of tension field action, there are three components contributing to the predicted shear strength: the first is the primary buckling strength of the plate, the second is due to the tension field action of the web, and the third arises from the plastic moment capacity of the flanges. Basler (1961) was the first to formulate a successful model for tension-field action for plate girders of the type used in civil engineering structures. Basler's approach (1961, 1963) disregarded the flexural rigidity of the flanges and hence did not reflect the true anchorage of the tension field along the flanges.

The model as suggested by Porter et al. (1975), is often referred to as the *Cardiff model*. It takes into account the effect of the bending stiffness of the flanges on the width of the diagonal tension band. Thus the diagonal tension field is considered to be composed of a central part that anchors on the transverse stiffeners, and two additional parts anchored on the flanges. It has been found that the shear model as proposed by Porter et al. (1975) provides a fairly good correlation between theory and test results (Maquoi 1992; Valitnat 1982).

7.4.3 Provisions of IS 800 : 2007

The tension field method may be used for girders having transverse stiffeners at the supports and at intermediate points. In the case of end panels, the end posts provide anchorage for the tension fields. It is also necessary that $c/d \geq 1.0$, where c and d are the spacing of transverse stiffeners and the depth of the web, respectively.

In the tension field method, the nominal shear resistance V_n is obtained from $V_n = V_{tf}$ with

$$V_{tf} = [A_v \tau_b + 0.9 w_{tf} t_w f_v \sin\phi] \le V_p \tag{7.5}$$

where τ_b is the buckling strength, as obtained by Eqn (7.3), A_v is the shear area (see Table 6.7), f_v is the yield strength of the tension field obtained from

$$f_v = [f_{yw}^2 - 3\tau_b^2 + \psi^2]^{0.5} - \psi \tag{7.6a}$$

In Eqn (7.6a),

$$\psi = 1.5\tau_b \sin 2\phi \tag{7.6b}$$

$$\phi = \text{inclination of the tension field} = \tan^{-1}\left(\frac{d}{c}\right) \tag{7.6c}$$

The above equation given in the code is correct for θ (slope of the panel diagonal). The value of ϕ ranges between $\theta/2$ to θ. The minimum value of $\theta/2$ applies when the flanges are fully utilized in resisting the applied bending moment, and the maximum value of θ applies when there is complete tension field condtion with $s = c$. Any assumed value between $\theta/2$ and θ is conservative. Eurocode suggests an approximate value of $\phi = \theta/1.5$. We will use this value in the calculations.
The width of the tension field w_{tf} is given by

$$w_{tf} = d\cos\phi - (c - s_c - s_t)\sin\phi \tag{7.7}$$

and f_{yw} is the yield stress of the web, d is the depth of the web, c is the spacing of stiffeners in the web, and s_c and s_t are the anchorage lengths of the tension field along the compression and tension flange respectively, obtained from

$$s = \frac{2}{\sin\phi}\left[\frac{M_{fr}}{f_y t_w}\right]^{0.5} \le c \tag{7.8}$$

In this equation, M_{fr} is the reduced plastic moment of the respective flange plate (disregarding any edge stiffener), after accounting for the axial force N_f in the flange, due to the overall bending, and any external axial force in the cross section, as follows

$$M_{fr} = 0.25 b_f t_f^2 f_{yf} [1 - \{N_f/(0.909\, b_f t_f f_{yf}\}^2] \tag{7.9}$$

where b_f and t_f are the width and thickness of the relevant flange, respectively.

In the case of a stiffened web of a plate girder, it would be of interest to consider the roles played by the stiffeners in the behaviour of the girders. The code provides various methods of designing the end panel depending upon whether tension field action can be developed in the end panel or not. These methods are explained in the next section.

7.4.4 End Panel Design Without Tension Field Action

A tension field cannot normally be fully developed in an end panel. This is evident by looking at the horizontal components of the tension fields shown in Fig. 7.12.

The vertical components are resisted by the stiffeners. Tension field in panel CD is balanced by the tension field in panel BC. Therefore the interior panels are anchored by the neighboring panels. The end panel AB has no anchorage on the left side. Anchorage can be provided by an end stiffener that is specially designed to resist the bending induced by the tension field.

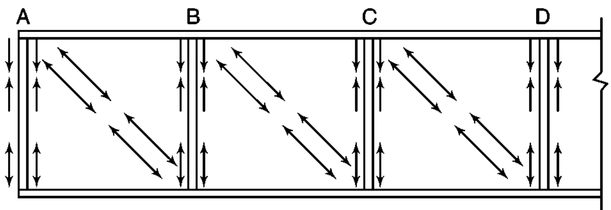


Fig. 7.12 Tension fields in a plate girder

The end panel (panel *B* in Fig. 7.13) may be designed according to the post-critical method, explained earlier. Additionally, the end panel along with the stiffeners should be checked as a beam spanning between the flanges of the girder to resist a shear force R_{tf} and a moment M_{tf} [as given by Eqn. (7.10a)]. The end stiffener should also be capable of resisting the reaction plus a compressive force due to the moment M_{tf}.

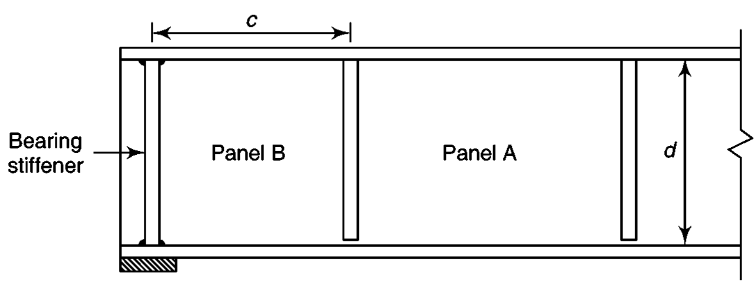


Fig. 7.13 End panel design without tension field action

End panels designed using tension field action (Figs 7.14 and 7.15) In this case, the interior panels are designed as per the provisions explained earlier. The end panel is provided with an end post consisting of a single or double stiffener. If the single stiffener is provided, it should play the roles of a bearing stiffener and end post (Fig. 7.14). These stiffeners need to satisfy the following criteria.

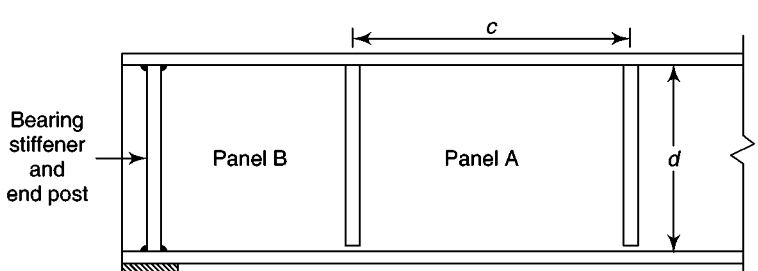


Fig. 7.14 End panel designed with tension field action (single stiffener)

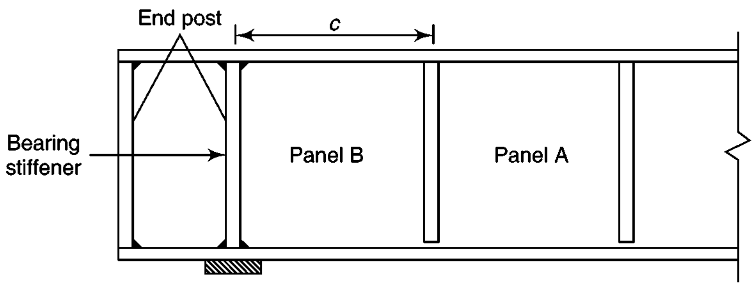


Fig. 7.15 End panel with double stiffener

In the case of a single stiffener, 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 $2/3M_{tf}$, where M_{tf} is obtained by Eqn (7.10a). The width and thickness of the end post should not exceed the width and thickness of the flange.

The anchor field forces developed in the panel A creates a resultant longitudinal shear, R_{tf}, plus a moment M_{tf} on the panel B. They are evaluated as follows.

$$R_{tf} = \frac{H_q}{2} \text{ and } M_{tf} = \frac{H_q d}{10} \tag{7.10a}$$

where $$H_q = 1.25\, V_{dp} \left(1 - \frac{V_{cr}}{V_{dp}}\right)^{1/2} \tag{7.10b}$$

$$V_{dp} = \frac{d t_w f_y}{\sqrt{3}} \tag{7.10c}$$

If the actual factored shear force V in the panel designed using tension field approach is less than the nominal shear strength V_{nt}, as determined earlier, then the values of H_q may be reduced by the ratio

$$\frac{V - V_{cr}}{V_{tf} - V_{cr}} \tag{7.10d}$$

where d is the web depth, V_{tf} is the the basic shear strength for the panel utilizing tension field action [see Eqn (7.5)], and V_{cr} is the critical shear strength for the panel designed without utilizing tension field action [see Eqn (7.2)].

Alternatively, a double stiffener can be provided for the end panel as shown in Fig. 7.15. Panels A and B are designed using tension field action. The bearing stiffener is designed for compressive stresses due to the external reaction. In this case, the two stiffeners and the portion of the web projecting beyond the end support form a rigid end post to provide necessary anchorage for the tension field in the end panel. Adequate space must be available to allow the girder to project beyond its support. The end post needs to act like a beam spanning between the flanges of the girder that should resist the shear force R_{tf} and bending moment M_{tf} due to anchor forces (Martin & Purkiss 1992).

7.5 Behaviour of Transverse Web Stiffeners

As observed from the previous sections, the tension field developed in the web is supported by the flanges and the transverse web stiffeners. Thus, the web stiffeners play a very critical role in achieving the ultimate capacity of the girder. It increases the web buckling stresses, supports the tension field in the web in the post-buckling stages, and finally prevents the flanges from being pulled towards each other, Prior to buckling, stiffeners on initially plane webs are not subjected to any axial loading. But after the plate buckles the axial loads applied to the transverse stiffeners steadily increase, as the web plate develops a membrane tension field (Rockey et al. 1981). Thus, the stiffeners must have sufficient rigidity to remain straight and to contain the web buckling to individual web panels.

From experimental studies, the following set of behaviours is deduced:

1. A portion of the web plates on either side of the stiffener acts with the stiffener, forming a T or a cruciform in resisting the vertical load (see Fig. 7.16), though the plate has theoretically buckled due to tension field action. Rockey et al. (1981) suggested that when a width of $40t_w$ of the web is assumed to act with the stiffener, the design procedure proposed by them are in agreement with the test data.
2. In practical cases, eccentricity of loading is unavoidable. This gives rise to additional bending moment. Further, any imperfections in the stiffener also result in bending moments.

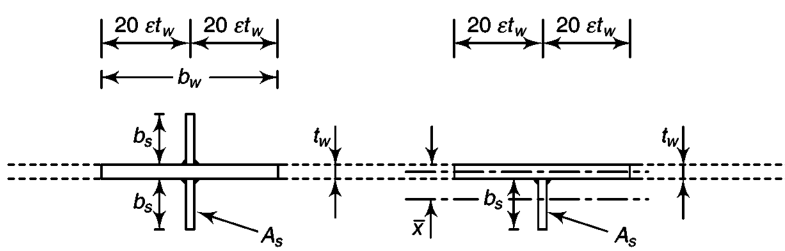


Fig. 7.16 Effective cross section of stiffeners

Design codes simplify this procedure for enabling quick sizing of the stiffeners, by assuming that the compressive force in the stiffener is constant and considering the whole depth of the web as a compression member.

7.6 Design of Plate Girders using IS 800 : 2007 Provisions

In the design of plate girders, required sections are proportioned for various elements. In general, bending moments are assumed to be carried by the flanges and the shear by the web. The depth of the section is chosen in such a way that the flanges are able to carry the design bending moment.

For arriving at a good plate girder design, IS 800 : 2007 has made several provisions with respect to its sectional requirements. The code specifies minimum web thickness requirement from a serviceability point of view for different situations with regard to provision of stiffeners (see Section 7.3.1). Similarly, the code specifies web thickness requirement in order to avoid buckling of the compression flange into the web as follows:

(a) When transverse stiffeners are not provided

$$\frac{d}{t_w} \le 345\varepsilon_f^2 \tag{7.11a}$$

(b) When transverse stiffeners are provided and
  (i) When $c \ge 1.5d$:

$$\frac{d}{t_w} \le 345\varepsilon_f^2 \tag{7.11b}$$

  (ii) When $c < 1.5d$:

$$\frac{d}{t_w} \le 345\varepsilon_f \tag{7.11c}$$

where d is the depth of the web, t_w is the thickness of the web, c is the spacing of the transverse stiffener, $\varepsilon_f = \sqrt{250/f_{yf}}$, and f_{yf} is the yield stress of the compression flange.

It should be noted that when these two sets of criteria are used, the serviceability criteria will be more critical for Fe 410 grade steel and the buckling criteria becomes more important for higher grade steels. This is because buckling strengths are dependent on Young's modulus, which is the same for all grades of steel.

7.6.1 Sectional Properties

In the design of a plate girder, the gross cross-sectional area may not be available for resistance in several cases due to various reasons. In some situations, the plates may be too wide and may be classified as slender. In such cases, the excess width of plates has to be deducted to get the effective area (see Examples 4.13 and 7.1). For some reasons, if open holes are to be provided in the plane perpendicular to the direction of stress, they also have to be deducted for arriving at the effective area. In a similar manner, the effective area of cross section in the tensile flange will have to be determined after deducting the area for holes. This aspect has been elaborately treated in Chapter 6. The calculation of the effective sectional area for shear is also dealt in detail in Chapter 6.

7.6.2 Flanges

When more than one plate is used in the flange, they are to be stitched together by tacking bolts or other means to act as a single piece. At regions where bending moment is less than that at the critical sections, the number of plates can be reduced or curtailed. While curtailing the plates, the curtailed plates have to be extended slightly beyond the cut-off point and the extended plate has to be sufficiently connected to develop the required resistance at the cut-off point. It is also essential that the outstand of the flange plates in a plate girder satisfies the minimum section classification requirement, as described in Chapter 6.

In instances of a plate girder design where the sections are available only in small lengths or where sectional requirements are changing along the span, provision of flange splices are permitted. Splices should connect the two adjoining sections with adequate bolting or welding. It is always prudent to provide splicing only at points away from the locations of maximum stress. The cross section of the splice plate is normally kept 5% more than the area of the flange plates that are spliced. Also the connections need to develop at least 5% more load than the load at the spliced portion or at least 50% of the effective strength of the spliced section. In the case of a welded construction, the flange plate should be welded together using full penetration butt weld that is capable of developing full strength of the plates connected.

In the fabrication of the plate girder, the web and flange plate are joined together with adequate bolts or welds such that they can transmit horizontal shear arising out of the bending moments that develop in the girder and also due to any applied vertical load on the flange. When fillet welding is adopted for the flange-web connection, the web and flange plates are brought as close as possible to each other before welding and the maximum gap between them is limited to 1 mm. The effective sectional area for the web of a plate girder is arrived by considering the full depth of web plate and its uniform thickness. If the thickness of the web is varying in the depth of the section due to provision of tongue plates or in cases where the web contributes 25% or more of its depth to the flange area, the above provision is not applicable. In such cases the exact area has to be calculated on a case-to-case basis.

Similar to splices in flanges, webs also may need splicing or cut outs may become necessary in webs for routing of services. Such web splices and cut outs are best provided at locations away from high shear and large concentrated loads. Splices in the web are designed to withstand the shears and moments at the spliced section. Splice plates are often provided on both sides of the web in equal thickness and they are connected together. If welding is resorted to, complete full penetration butt welding is recommended. In locations where extra plate is added to the web, to increase its strength, they may be provided equally on both sides of the web plate. These plates also extend beyond the length of the requirement as arrived by calculations. The amount of shear force that these reinforcing plates can carry is restricted to the extent of horizontal shear that they transmit to the flanges by way of the fastening provided.

7.6.3 Stiffeners

In a plate girder, the web is often made very thin to derive maximum economy in weight. The webs, when they are inadequate to carry the load, are made strong and stable by the provision of a wide variety of stiffeners. Sometimes double stiffeners are adopted near the bearing [see Fig. 7.15] and in such cases the over hangs beyond the support should be limited to 1/8 of the depth of the girder. The stiffeners are classified based on the role they play in strengthening the web and they are mentioned as follows.

- The function of a *bearing stiffener* is to preclude any crushing of the web at locations of heavy concentrated loads. Thus, bearing stiffeners transfer heavy reactions or concentrated loads to the full depth of the web. They are placed in pairs on the web of plate girders at unframed girder ends and where required for concentrated loads. They should fit tightly against the flanges being loaded and extend out towards the edges of flange plates as far as possible. If the load normal to the girder flange is tensile, the stiffeners must be welded to the loaded flange. If the load is compressive, it is necessary to obtain a snug fit. To accomplish this goal, the stiffeners may be welded to the flange or the outstanding leg of the stiffener may be milled.

- A *load carrying stiffener* prevents local buckling of the web due to any concentrated load.
- An *intermediate transverse web stiffener* mainly improves the buckling strength of the web due to shear. They continue to remain effective after the web buckles to provide anchorage for the tension field and finally they prevent the flanges from moving towards one another. They are also called as non-bearing stiffeners or stability stiffeners.
- *Torsional stiffeners* are provided at supports to restrain the girders against torsional effects.
- Local strengthening of the web under the combination of shear and bending is provided by *diagonal stiffener*.
- The tensile forces from the flange are transmitted to the web through the *tension stiffener.*
- A *longitudinal stiffener* increases the buckling resistance of the web.

The same stiffener may perform more than one function and in such cases, their design should comply with the requirement of those functions.

In the design of various stiffeners, it is important to know the effective dimensions of the stiffener that needs to be considered for the design. The outstand of the stiffener from the face of the web is restricted to a value of $20t_q\varepsilon$. If the outstand value is anywhere between $14t_q\varepsilon$ and $20t_q\varepsilon$, the value of $14t_q\varepsilon$ is chosen as the outstand value. In the above expressions, t_q refers to the thickness of the stiffener plate. Another important parameter of the stiffener is its *stiff bearing length* b_1. It is defined as that length that does not deform appreciably in bending. The stiff bearing length b_1 is determined by considering the dispersion of the load at 45° through a solid material such as the bearing or flange plates as shown in Fig. 7.17.

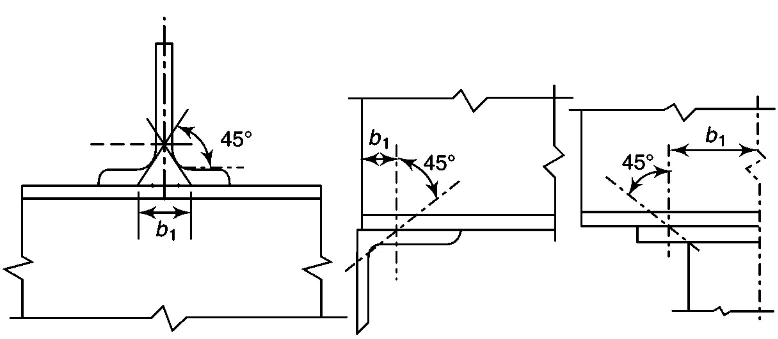


Fig. 7.17 Stiff bearing length (b_1)

The eccentricity of loading with respect to the stiffener centroid is to be considered in the design of a stiffener. The eccentricity arises when the load or reaction is eccentric to the centre line of the web or when the stiffener centroid does not coincide with the centre line of the web. Buckling of the stiffeners is also a matter of serious concern and hence adequate buckling resistance needs to be provided for stiffeners. In order to evaluate the buckling resistance of the stiffener, the stiffener is considered as a strut with its radius of gyration taken about the axis parallel to the web. The effective sectional area of the stiffener is the area of the core section as determined below. The effective width of web acting together with the stiffener is taken as 20 times the web thickness on each side of the centre line of the stiffener, i.e., $40t_w$ for interior stiffeners (see Fig. 7.18), and as $20t_w$ for end stiffeners. The buckling resistance of the stiffener is obtained on the basis of the design compressive stress, f_{cd} of the strut using buckling curve *c* (see Section 5.6.1). In arriving at the buckling resistance, the lower value between the design strength of the web material and the stiffener material is to be adopted.

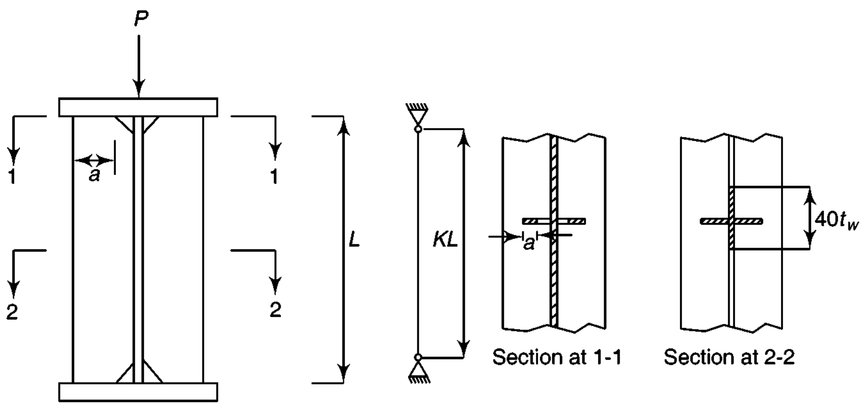


Fig. 7.18 Design of intermediate transverse web stiffener

The effective length of a intermediate transverse web stiffener, which is provided mainly to improve the shear buckling resistance of the web F_{qd} is taken as 0.7 times the length of the stiffener. For other load carrying stiffeners, with the assumptions that the flange, through which the load or reaction is applied, is effectively laterally restrained, the effective length for obtaining the buckling resistance F_{xd} is taken as 0.7. In case the flange is not laterally restrained, the effective length is taken as L, where L is the length of the stiffener.

As mentioned earlier, intermediate transverse web stiffeners mainly provide buckling resistance to the web due to shear. Sometimes intermediate stiffeners are alternated on each side of the web to gain better economy or they are placed all on one side to improve aesthetics. The spacing of these stiffeners is based on the provisions, as explained earlier depending upon the thickness of web (see Sections 7.3.1 and 7.6). The minimum requirement for intermediate transverse stiffener, when they are subjected to external loads or moments is considered in terms of a second moment of area I_s about the centre line of the web (Rockey et al. 1981).

$$\text{if } \frac{c}{d} \geq \sqrt{2};\ I_s \geq 0.75dt_w^3 \tag{7.12a}$$

$$\text{if } \frac{c}{d} < \sqrt{2};\ I_s \geq \frac{1.5d^3t_w^3}{c^2} \tag{7.12b}$$

where d is the depth of the web, t_w is the minimum thickness required for tension field action, and c is the stiffener spacing.

Transverse web stiffeners are also checked for buckling resistance for a stiffener force

$$F_q = 0.909\ (V - V_{cr}) \leq F_{qd} \tag{7.13}$$

where F_{qd} is the design buckling resistance of the intermediate stiffener, V is the factored shear force adjacent to the stiffener, and V_{cr} is the shear buckling resistance of the web panel without considering tension field action [Eqn (7.2b)].

Transverse web stiffeners that are subjected to external load and moments have to satisfy requirements of load carrying stiffeners. They have to also meet the interaction equation

$$\frac{F_q - F_x}{F_{qd}} + \frac{F_x}{F_{xd}} + \frac{M_q}{M_{yq}} \leq 1 \tag{7.14}$$

If $F_q < F_x$, then

$$F_q - F_x = 0$$

where F_q is the stiffener force [in N, see Eqn (7.13)], F_{qd} is the design buckling resistance of intermediate web stiffener about an axis parallel to the web (in N), F_x is the external load or reaction at the stiffener (in N), F_{xd} is the design resistance of load carrying stiffener, buckling about an axis parallel to the web (in N), M_q is the moment at the stiffener due to eccentric load (in N mm), and M_{yq} is the yield moment capacity of the stiffener about an axis parallel to the web (in N mm).

An important aspect of the intermediate transverse stiffener is with respect to their connection with the web. If the stiffeners are not subjected to external load, they may be connected to the web in such a way that they withstand a shear of not less than $t_w^2 / 5b_s$, where t_w is the web thickness and b_s is the stiffener outstand width.

If the stiffeners are subjected to external loading, the resulting shear to be resisted is the total shear due to loads and shear specified earlier. If the stiffeners are not subjected to external load, they may be provided in such a way as to be clear of the tension flange and the gap provided is $4t_w$. With such a gap, the fabrication problems associated with close fit can be avoided.

7.6.4 Load Carrying Stiffeners

As defined previously, these stiffeners are provided at locations where the compressive forces that are applied through the flange by an external load or reactions exceed the buckling resistance F_{cdw} of the web alone. The buckling resistance of the web alone may be calculated by using the effective length and the sectional area of the stiffener. The sectional area of the stiffener is taken as $(b_1 + n_1)t_w$, where b_1 is the stiff bearing length and n_1 is the dispersion length of the load through the web at 45° to the level of half the depth of the cross section. The buckling strength of the web is calculated about an axis parallel to the web thickness using buckling curve c. Although the web can be proportioned to resist any applied concentrated loads, bearing stiffeners are generally provided. If stiffeners are used at each concentrated load, the limit states of web yielding, web crippling, and side sway web buckling need not be checked.

7.6.5 Bearing Stiffeners

Bearing stiffeners are provided where forces, applied through a flange by loads or reaction, are in excess of the local capacity of the web at its connection to the flange, F_w calculated as follows:

$$F_w = 0.909\,(b_1 + n_2)t_w f_{yw} \tag{7.15}$$

where b_1 is the stiff bearing length, n_2 is the 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 is the thickness of the web, and f_{yw} is the yield stress of the web.

7.6.6 Design of Load Carrying Stiffeners

Load carrying stiffeners are checked for their buckling and bearing resistances. For buckling check

$$F_x \leq F_{xd} \tag{7.16}$$

where F_x is the external load or reaction and F_{xd} is the buckling resistance.

In the event that the load carrying stiffeners also act as intermediate transverse web stiffeners, the bearing strength should be checked for the combined loads as given in Eqn (7.14). The bearing strength of the stiffener F_{bsd} is obtained as follows:

$$F_{bsd} = 1.136\,A_q f_{yq} \tag{7.17}$$

where A_q is the area of the stiffener in contact with the flange in mm^2 {the actual area A is generally less than the full cross-sectional area of the stiffener as the corners of the stiffener are often coped to clear the web-to flange fillet weld (see section 1-1 of Fig. 7.18)} and f_{yq} as the yield stress of the stiffener in N/mm^2.

The bearing strength of the web stiffener F_{bsd} as calculated above should be greater than or equal to F_x, the load transferred.

7.6.7 Design of Bearing Stiffener

Bearing stiffeners are designed for the difference in force between the applied load or reaction and the local capacity of the web calculated in Eqn (7.15). In all cases, if the web and stiffener are of materials with different strength, the lower of the two has to be used for the strength equation. A bearing stiffener should be as wide as the overhang of the flange through which the load is applied. The weld connecting the stiffener to the web should have the capacity to transfer the unbalanced shear force. Conversely, the weld can be designed to carry the entire concentrated load.

7.6.8 Design of Diagonal and Tension Stiffeners

As in the case of bearing stiffeners, the diagonal stiffeners are designed for the portion of the combination of applied shear and bearing that exceeds the capacity of the web. Tension stiffeners are designed to carry the portion of the applied load or reaction that exceeds the capacity of the web.

7.6.9 Design of Torsional Stiffener

Torsional stiffeners provide torsional restraint at the supports and they should satisfy the following:

(a) the local capacity of the web is exceeded as calculated in Eqn (7.15)

(b) second moment of area of the stiffener section about the centre line of the web I_s should satisfy the following relation

$$I_s \geq 0.34\alpha_s D^3 t_{cf} \tag{7.18}$$

where $\alpha_s = 0.006$ for $KL/r_y \leq 50$

$\alpha_s = 0.3/(KL/r_y)$ for $50 < KL/r_y \leq 100$

$\alpha_s = 30/(KL/r_y)^2$ for $KL/r_y > 100$

and D is the depth of the beam at support (in mm), t_{cf} is the maximum thickness of the compression flange of the girder (in mm), KL is the effective length of the laterally unsupported compression flange of the girder (in mm), and r_y is the radius of gyration about minor axis (in mm).

7.6.10 Connection of Load Carrying Stiffeners and Bearing Stiffeners to Web

Stiffeners which are subjected to load or reaction applied through a flange have to be connected to the web adequately. This connection is designed to transmit a force equal to the lesser of

(a) the tension capacity of the stiffener or

(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 different directions.

It should also be ensured that the shear stress in the web due to the design force transferred by the stiffener is less than the shear strength of the web. The stiffeners which resist tension should be connected to the flange with continuous welds or non-slip fasteners. Stiffeners resisting compression can either be fitted against the loaded flange or be connected by continuous welds or non-slip fasteners. The stiffener has to be fitted or connected to both flanges if

- the load is applied directly over the support
- the stiffener is an end stiffener of a stiffened web
- the stiffener also acts as torsional stiffener

7.6.11 Longitudinal Stiffeners

In order to obtain greater economy and efficiency in the design of plate girders, slender webs are often stiffened both longitudinally and transversely. The longitudinal stiffeners are generally located in the compression zone of the girder. The main function of the longitudinal stiffeners is to increase the buckling resistance of the web. The longitudinal stiffener remains straight, thereby, subdividing the web and limiting the web buckling to smaller web panels (see Fig. 7.19). In the past it was usually thought that the resulting increase in the ultimate strength could be significant, but recent studies have shown that this is not always the case, as the additional cost of welding the longitudinal stiffeners invariably offsets any economy resulting in their use. Longitudinal stiffeners, thus, are not as effective as the transverse ones, and are frequently used in bridge girders, because many designers feel that they are more attractive. Clause 8.7.13 of the code contains provisions for horizontal stiffeners.

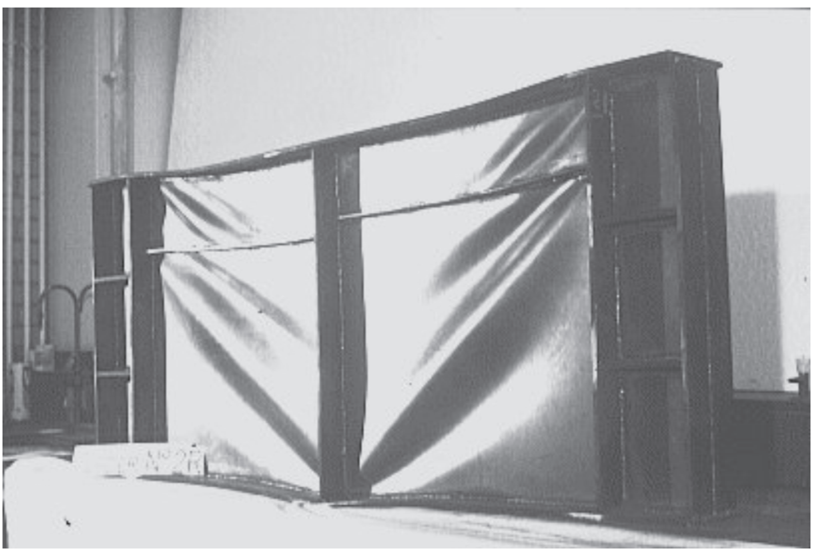


Fig. 7.19 Tension field in a plate girder with longitudianl stiffener (EDEP 2006)

7.7 Welding of Girder Components

The fillet weld between the flange and the web is generally kept as small as possible depending upon the thickness of the flange and the allowable stresses. Continuous welds are desirable preferably with automatic welding equipment. Bearing stiffeners must fit tightly against the flange that transmits the load and must be attached to the flange by full penetration groove welds. The connection of the stiffener to the web may be by intermittent or continuous welds. Appropriate small openings are provided at the weld junctions to avoid interaction of different fillet welds. Transverse stiffeners are solely provided to increase the shear buckling resistance of the web and hence are fitted tightly with the compression flange but welded only to the web. But one-sided transverse stiffeners must be welded to the compression flanges also. Transverse intermediate stiffeners are stopped short of the tension flange by a distance of about $4t_w$. Welding transverse stiffeners to tension flanges may reduce the fatigue strength and may lead to brittle fracture.

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7.8 Proportioning of the Section

The cross section of the girder must be selected such that it adequately performs its functions and requires minimum cost. The functional requirements are as follows:

(a) Strength to carry bending moment (adequate Z_p)
(b) Vertical stiffness to satisfy any deflection limitation (adequate moment of inertia I_z)
(c) Lateral stiffness to prevent lateral-torsional buckling of compression flange (adequate lateral bracing or low values of (L_b/r_t))
(d) Strength to carry shear (adequate web area)
(e) Stiffness to improve buckling or post-buckling strength of the web (related to d/t_w and c/d ratios)

One method of determining the initial dimensions is to use a minimum weight analysis (Schilling 1974). However, it has to be emphasized that a minimum weight design may not result in minimum cost, unless fabrication cost is also included in the optimization process.

7.8.1 Optimum Girder Depth

If the moment M is assumed to be resisted entirely by the flanges, then for a I-section beam,

$$M = f_y b_f t_f d \tag{7.19}$$

where f_y is the design strength of the flanges, b_f and t_f are the width and thickness, respectively of the flanges, and d is the depth of the web.
The gross-sectional area of the beam is given by

$$A = 2\, b_f t_f + d t_w \tag{7.20}$$

where t_w is the thickness of web. Eliminating $b_f t_f$ using Eqn (7.19), we get

$$A = 2M/(d f_y) + d t_w \tag{7.21}$$

Defining the web slenderness as $k = d/t_w$, we get

$$A = 2M/\,(k t_w f_y) + k t_w^2 \tag{7.22}$$

Differentiating the above equation with respect to t_w and setting the result equal to zero, we get the optimum value of t_w as

$$t_w = [M/(f_y k^2)]^{0.33} \tag{7.23}$$

The optimum value of the depth is

$$d = (Mk/f_y)^{0.33} \tag{7.24}$$

Thus the optimum values produce a beam which has the area of single flange and the web as equal (Martin & Purkiss 1992). Extended treatment of this subject has been provided by Salmon and Johnson (1996), who also propose the following simple expression for the required area of one flange plate

$$A_f = M/(f_y d) - A_w/6 \tag{7.25}$$

where A_w is the area of web plate. The above equation may be used for preliminary design purposes.

CHAPTER 8

Design of Gantry Girders

Introduction

In mills and heavy industrial buildings such as factories and workshops, gantry girders supported by columns and carrying cranes are used to handle and transport heavy goods, equipment, etc. There are several types of cranes; overhead travelling, under-slung, jib, gantry, and monorail are among the most common. A building may have one or several of these, either singly or in combinations. Hand-operated overhead cranes have lifting capacities of up to 50 kN and electrically operated overhead travelling cranes, called EOT cranes, can have capacities in the range of 10–3000 kN. The overhead travelling crane runway system consists of the following components (see Figs 8.1 and 8.2):

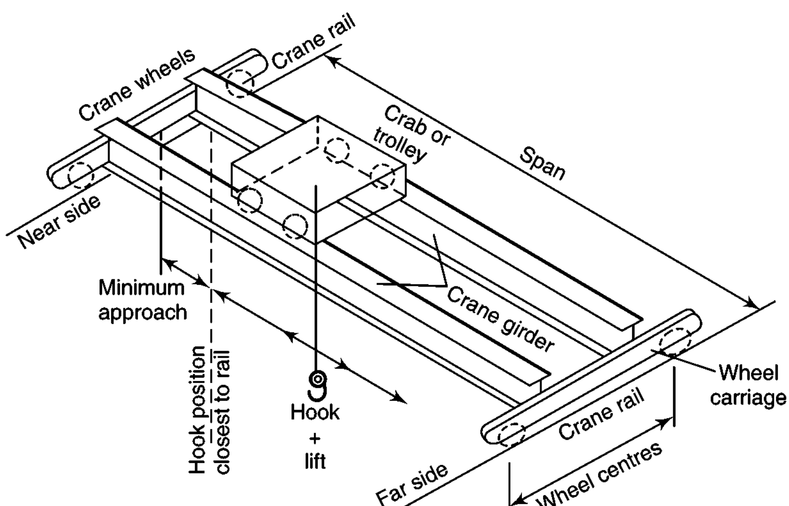


Fig. 8.1 Components of an overhead crane

1. The crane, comprising the crane girder (crane frame), crab or trolley, hoist, power transmitting devices, and a cab which houses the controls and operator
2. The crane rails and their attachments
3. The gantry girder
4. The gantry girder supporting columns or brackets
5. The crane stops

The loads are lifted using a hook and moved longitudinally and transversely anywhere in the building through the movement of a crab car or trolley on the crane frame (girder) or truss and the crane wheels on the crane rails (see Figs 8.1 and 8.2). The gantry girder is supported by brackets attached to the main columns of the building or by stepped columns.

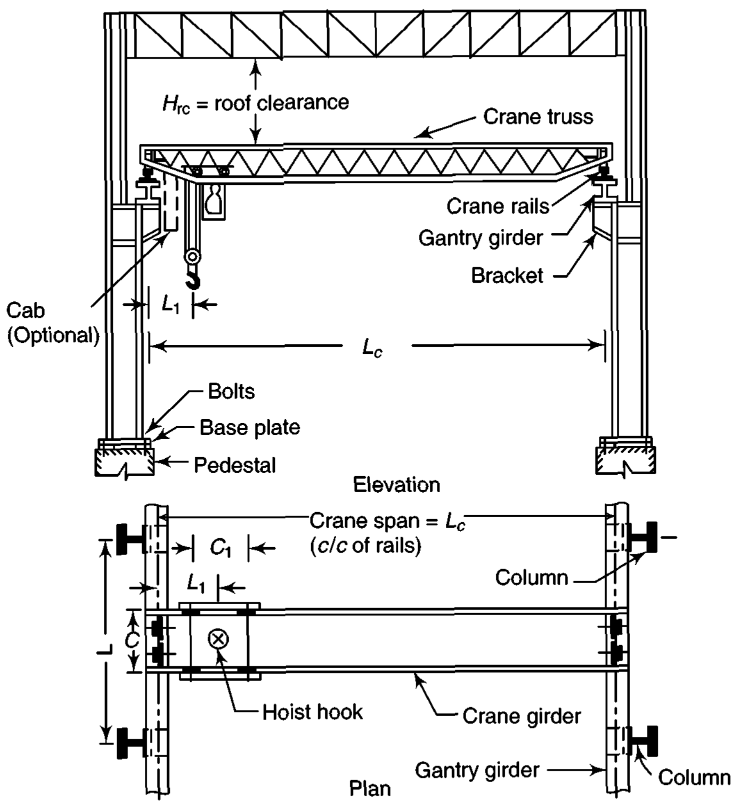


Fig. 8.2 Typical crane notations

The rating of the cranes is based on the hoisting capacity, which in turn depends on the size of the crab, wheel spacing, roof clearance, etc. Usually this data is supplied by the manufacturer of cranes. Power and other requirements of the crane system are also related to the capacity of the crane. Typical data for some cranes is given in Table 8.1 for guidance. The designer needs to check out this data from the manufacturer before carrying out the design of gantry girders.

Table 8.1 Typical data for cranes (see Fig. 8.2)

Capacity of crane (kN)	Auxiliary weight (kN)	Span L_c (m)	Wheel base c (m)	Minimum hook distance of main load, L_1 (m)	Weight of crane bridge (kN)	Vertical clearance (m)	Weight of trolley/crab (kN)	Crane rail Head width (mm)	Crane rail Weight (kN/m)	Crane rail Base width (mm)
50	—	10.5–22.5	3.0–4.8	0.65–1.00	50–150	1.83	15	CR50	298	90
100	30	10.5–22.5	3.2–4.8	0.65–1.10	80–210	1.83	35	CR50	298	90
150	30	10.5–31.5	3.2–5.3	0.80–1.10	210–250	2.13	60	CR60	400	105
200	50	10.5–31.5	3.5–5.3	0.80–1.10	160–275	2.13	75	CR60	400	105
250	50	10.5–31.5	3.5–5.3	0.80–1.10	275–320	2.44	85	CR80	642	130
300	80	10.5–31.5	3.8–5.3	0.80–1.15	300–360	2.44	100	CR80	642	130
400	80	10.5–31.5	3.8–5.3	0.85–1.15	350–400	2.74	120	CR80	642	130
500	125	10.5–31.5	4.0–5.3	0.85–1.20	400–470	2.74	135	CR100	890	150
600	125	10.5–31.5	4.5–5.3	1.00–1.20	600–750	2.74	250	CR120	1180	170

Note: 1. Exact crane data must be obtained from the manufacturer.
2. The auxiliary load need not be considered in the design of the girder.

While deciding on the choice of the crane, one must consider the load capacity, space limitations, and the class of service required. While designing the crane supporting structures, the engineer should take into account these requirements and other factors such as the likely future changes in the load capacity, addition of other cranes, various load combinations, and possible future extensions. Few other structures suffer such an extreme range of stresses and as high an incidence of maximum loadings and fatigue as crane runways, and this must also be considered by the engineer. In addition, one must be aware of the infinite variety of abuses inflicted on crane systems, such as hoisting loads which exceed the crane capacity, swinging loads as a pendulum, dragging loads laterally or longitudinally, and ramming the crane against the crane stops at an excessive speed. The crane is often one of the most important parts of an industrial operation, so any significant 'downtime' required for repairs and maintenance of the same can be disastrous to the owner.

Cranes may be classified on the basis of the load carrying capacity (load lifted), the height to which the load is lifted, and the frequency of lifting the loads. For example, the Crane Manufacturers Association of America (CMAA) have classified the cranes as follows (Ricker 1982):

1. Class A1 (standby service)
2. Class A2 (infrequent use)
3. Class B (light service)
4. Class C (moderate service)
5. Class D (heavy duty)
6. Class E (severe duty cycle service)
7. Class F (Steel Mill)

Table 8.2 gives the representative crane speeds.

Table 8.2 Typical speeds of overhead cranes (Weaver 1979)

Capacity (kN)	Slow (m/min)	Medium (m/min)	Fast (m/min)
10	60	90	120
50	60	75	90
100	30	45	60
150	30	38	45
200	30	38	45
250	30	38	45

It should be remembered that the design aspects for light cranes may be different from those for heavy cranes: cranes with long spans (over 15 m) should be treated differently than those with shorter spans; fast, heavy service cranes require special consideration not required for slower, lighter cranes.

8.1 Loading Considerations

The following important quantities need to be considered by the designer while estimating the loads on the gantry girder (see Fig. 8.3).

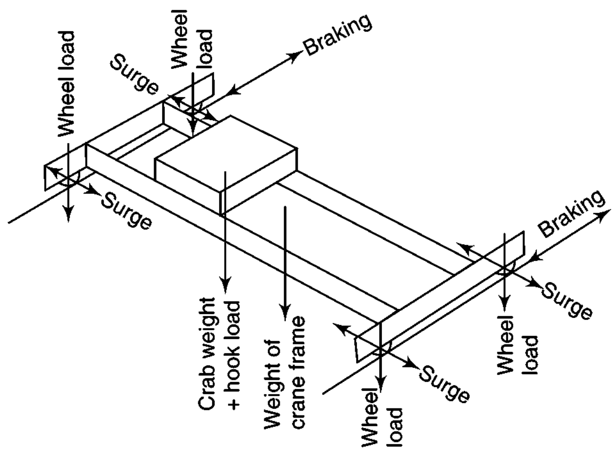


Fig. 8.3 Loads to be considered on gantry girders

Weight of the trolley or crab car (W_t) Since the trolley moves on the crane girder (see Figs 8.1 and 8.3) along the span of the truss, its weight is transferred to the crane wheels as the axle load and finally to the gantry girder. The load transferred to the gantry will be maximum when the trolley wheels are closest to the gantry girder. The wheel load that is transferred from the trolley to the gantry girder is

$$W_1 = [W_t(L_c - L_1)]/(2L_c) \tag{8.1}$$

where W_1 is load of each wheel on the gantry girder (note that there are 2 wheels), W_t is the weight of the trolley or crab car, L_c is the distance between the gantry girders (span of truss/crane span), and L_1 is the distance between the CGs of the trolley and gantry.

Weight of the crane girder (W_c) The crane manufacturers also supply a pair of crane girders, which may be in the form of open web trusses or solid I-beams on which the trolley moves. These I-beams will be mounted on a crane rail using wheel carriages having four wheels, two wheels on either end (see Figs 8.1–8.3). These wheels move on rails mounted on the gantry girders. The crane rails are attached to the gantry girders with bolted clamps or hook bolts spaced about 500–1000 mm apart, depending on the lifting capacity of the crane. Rails are generally not welded to the gantry girder because of the difficulty in the readjustment of the alignment and replacement of defective or worn out rails. The hook bolts (used with slow-moving cranes having a capacity of 50 kN and bridge span of 15 m) and clamps are shown in Fig. 8.4. More details about rails and the methods of attaching them to gantry girders may be found in Ricker (1982). The weight of the crane girder is equally distributed on all the four wheels. Hence, the weight of the crane girder on each wheel is

$$W_2 = W_c/4 \tag{8.2}$$

where W_2 is the load due to the weight of the crane truss/girder and W_c is the weight of the crane truss/girder.

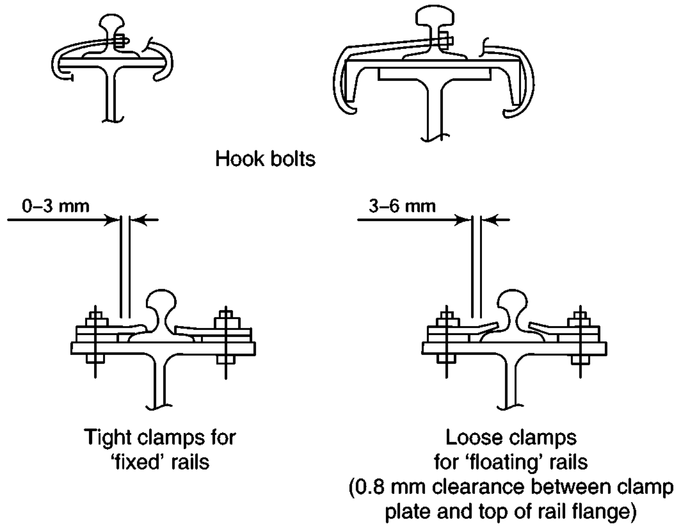


Fig. 8.4 Methods of attaching rails to gantry girders (Ricker 1982)

Impact loads As the load is lifted using the crane hook and moved from one place to another, and released at the required place, an impact is felt on the gantry girder. It is due to the sudden application of brakes (frictional force), moving loaded crane's acceleration, retardation, vibration, or possible slip of slings, etc. To account for these, suitable impact factors should be applied to the loads. As per IS 875 and IS 807, additional impact loads as listed in Table 8.3 should be considered.

Table 8.3 Additional impact loads on cranes

Type of load	Impact allowance (percentage)
Vertical forces transferred to wheels	
(a) For EOT cranes	25% of the maximum static wheel load
(b) For hand-operated cranes	10% of the maximum static wheel load
Horizontal forces transverse to the rails:	
(a) For EOT cranes	10% of the weight of the crab and the weight lifted on the crane
(b) For hand-operated cranes	5% of the weight of the crab and the weight lifted on the crane
Horizontal forces along the rails	5% of the static wheel loads for EOT or hand-operated cranes

Lateral load (surge load) As the crane moves with the load, a lateral load (transverse to the rail) is developed, as shown in Fig. 8.5, due to the application of brakes or the sudden acceleration of the trolley. Suppose that the total weight including the lifted weight and the trolley weight is W, the coefficient of friction is 0.1, the number of wheels is 4, and number of breaking wheels is 2. Then,

$$\text{Horizontal force} = W \times 0.1 \times (2/4) = 0.05\text{W}.$$

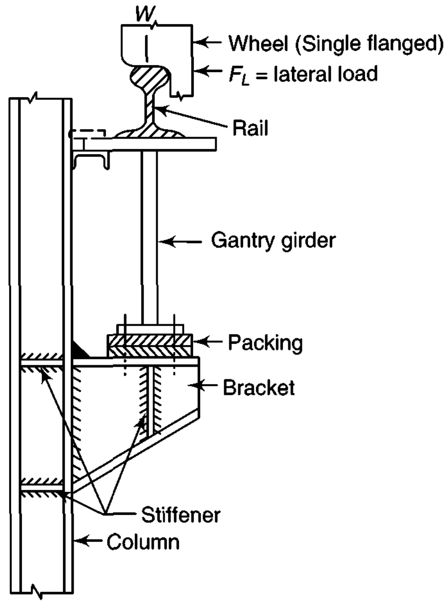


Fig. 8.5 Application of lateral load and typical bracket

However, the Indian code IS 875 recommends 10%W for EOT cranes (see Table 8.3). These horizontal loads are also called *surge loads*. In the design of the girder, it is usually assumed that the lateral or longitudinal surge is resisted by the compression flange alone. Note that it is customary to assume that the entire horizontal force acts on one gantry girder at the position of the wheels. But if the wheels have guides on both sides of the flange of the wheel, the force is resisted by the two opposite gantry girders and only half the horizontal load as specified in Table 8.3 acts on one girder. Thus,

$$W_L = 10/100(W_t + W_k) \text{ for EOT crane} \tag{8.3a}$$

$$W_L = 5/100(W_t + W_k) \text{ for hand-operated crane} \tag{8.3b}$$

where W_k is the hook load (the maximum capacity of the crane is denoted by this load) and W_t is the weight of the trolley.

Longitudinal load (drag load) As the crane moves longitudinally, loads parallel to the rails are caused due to the braking (stopping) or acceleration and swing (starting) of the crane. This load is called the longitudinal load or *drag load* and is transferred at the rail level. Figure 8.6 illustrates the action of the longitudinal load and its reactions. Due to the vertical loads, the reactions R_{a1} and R_{b1} are developed. These reactions can be calculated easily.

Due to the braking action, a longitudinal load is developed at the top of the rails, which acts at a height e from the centre of gravity of the beam. This eccentric load, in turn, generates equal and opposite reactions. Thus, the gantry girder is subjected to bending moments in addition to the

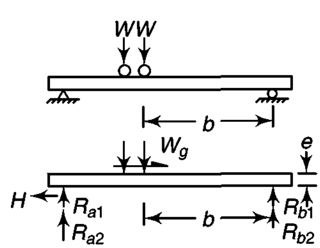


Fig. 8.6 Braking force

axial force. The axial force could be tensile or compressive depending on the direction of motion in relation to the hinge support. The longitudinal load per wheel as per Table 8.3 is

$$W_g = 5W/100 \tag{8.4}$$

where W is the wheel load. The reactions due to the longitudinal loads are

$$R_{b2} = -R_{a2} = W_g e/L \tag{8.5}$$

$$\text{Axial force } P = W_g \text{ (tension or compression)} \tag{8.6}$$

The bending moment under the load is

$$M = R_{b2} b = W_g eb/L \tag{8.7}$$

All these loads (vertical wheel load, surge load, and breaking load are shown schematically in Fig. 8.3. During preliminary design studies, the specific crane information necessary for the final design may not be available, and it becomes often necessary to estimate the loadings. For such studies, the information given in Table 8.1 or the data found in Weaver (1979) and Gaylord et al. (1996) may be useful. When the crane information does become available, it should be carefully compared with the estimated loadings and necessary adjustments should be made to the preliminary design.

8.2 Maximum Load Effects

Moving loads, such as crane wheels, result in bending moments and shear forces, which vary as the load travels along the supporting girder. In simply supported beams the maximum shear force will occur immediately adjacent to the support, while the maximum bending moment will occur in the region of mid-span. In general, influence lines should be used to find the load positions that produce maximum values of shear forces and bending moments (see Marshall & Nelson 1990; Coates et al. 1988 for a discussion and calculation for influence lines).

For a simply supported beam with two moving loads, the load positions which produce maximum shear force and bending moments are shown in Fig. 8.7.
(The wheel load bending moment is maximum when the two loads are in such a position that the centres of gravity of the wheel loads and one of the wheel loads are equidistant from the centre of gravity of the girder. The shear due to the wheel load is maximum when one of the wheels is at the support.) From these, the maximum shear force and bending moments are (Marshall & Nelson 1990)

$$\text{Shear force (max)} = W(2 - c/L) \tag{8.8}$$

$$\text{Bending moment (max)} = WL/4 \text{ or } 2W(L/2 - c/4)^2/L \tag{8.9}$$

The greater of the bending moment values should be used.

The design of the bracket supporting a crane girder uses the values of maximum reaction from the adjacent simply supported beams as in Fig. 8.7. When the adjacent spans are equal, the reaction is equal to the shear force, i.e,

$$\text{Reaction (max)} = W(2 - c/L) \tag{8.10}$$

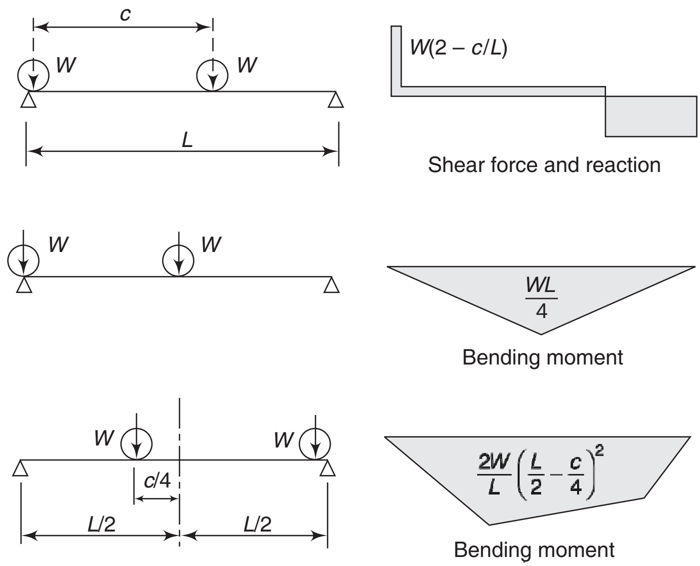


Fig. 8.7 Maximum BM, SF, and reaction for two moving loads

Note that in all the cases the effect of the self-weight of the gantry girder must also be considered as a uniformly distributed load.

The deflection at the mid-span due to placing the two loads symmetric with respect to it is given by

$$\Delta_c = W_c L^3[(3a/4L) - (a^3/L^3)]/(6EI) \tag{8.11}$$

where $a = (L - c)/2$, E is Young's modulus of rigidity, and I is the moment of inertia of the cross section.

When two cranes are operating in tandem on the same span, these have to be located closest to each other towards the mid-span to produce maximum bending moment. In such a situation, there will be four wheels, two wheels from each crane. These four wheels must be adjusted on the span such that the centroid of the load and the nearest wheel load to the CG from the centre line of the beam are at the same distance (see Fig. 8.8). Let x be the distance from left to the load under which maximum bending moment occurs.

$$x = (L/2) - (b/4) \tag{8.12}$$

The reaction and bending moment are

$$R_A = 4W(x/L) \tag{8.13}$$

$$\begin{aligned} M_1 &= R_A x - W_c \\ &= (4Wx^2/L) - W_c \\ &= W/L[4x^2 - cL] \end{aligned} \tag{8.14}$$

The deflection at mid span may be obtained approximately as (see Fig. 8.8)

$$\Delta = W/(24EI)[a(3L^2 - 4a^2) + b(3L^2 - 4b^2)] \tag{8.15}$$

These deflections at working loads should be less than the allowable deflections as specified in Table 8.4 (see Table 6 of IS 800 : 2007).

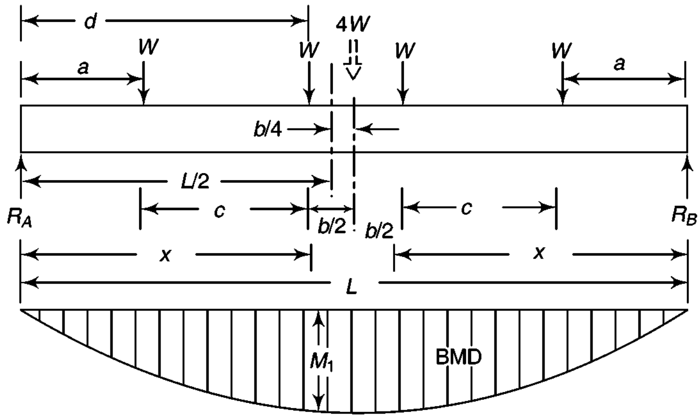


Fig. 8.8 Maximum bending moment for four moving loads (two cranes)

Table 8.4 Limiting deflection of gantry girders

Category	Maximum deflection
Vertical deflection	
Manually operated cranes	$L/500$
EOT cranes with a capacity of less than 500 kN	$L/750$
EOT cranes with a capacity of greater than 500 kN	$L/1000$
Lateral deflection	
Relative between rails	10 mm or $L/400$

8.3 Fatigue Effects

Gantry girders are subjected to fatigue effects due to the moving loads. Normally, light- and medium-duty cranes are not checked for fatigue effects if the number of cycles of load is less than 5×10^6. For heavy-duty cranes, the gantry girders are to be checked for fatigue loads (see also IS 1024 and IS 807). See section 13 of the code for design provisions for fatigue effects. Note that fatigue strength is checked at working loads!

8.4 Selection of Gantry Girder

The gantry girder is subjected to vertical loading including impact loading, lateral loading, and longitudinal loading from traction, braking, and impact on crane stops. In addition, the crane beams must withstand local buckling under wheel loads and at the bottom flange over the column (in the common case where the beam bears on a column cap plate). Figure 8.9 illustrates typical profiles used for gantry girders.

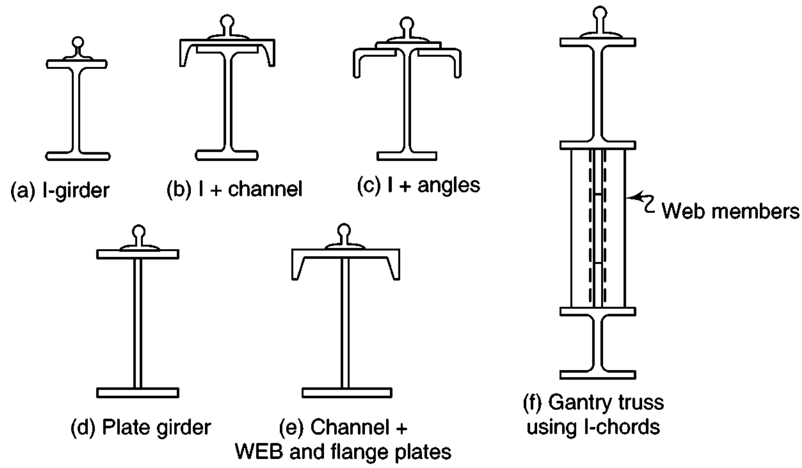


Fig. 8.9 Profiles used for gantry girders

- Rolled beams with or without plates, channels or angles (for spans up to 8 m and usually for 50 kN cranes), as shown in Figs 8.9(a–c).
- Plate girders (for spans from 6 m to 10 m) [Fig. 8.9(d)]
- Plate girder with channels, angles, etc. (for spans more than 10 m) [Fig. 8.9(e)].
- Box girders with angles (for spans more than 12 m)
- Crane truss using I-sections as chord members and angles as web members [Fig. 8.9(f)]

Single-span gantry girders are desirable. Two-span gantry girders can result in uplift in columns at certain loading positions, and differential settlement of columns may result in undesirable additional stresses. Moreover, making reinforcement or replacement of worn out gantry girders is more complicated and expensive in two-span gantry girders. Hence they are not adopted in practice.

Abrupt changes in cross sections of gantry girders should be avoided. Gantry girders or trusses approximately 20 m or more in length should be cambered for deflection due to the dead load plus one-half the live load without impact. (Brockenbrough & Merritt 1999). Cantilevered gantry girders should be avoided. If high-strength steel is used, the deflection should be checked, since the resulting section will be considerably small.

The major cause of problems in crane runs is the deflection of gantry girders and the accompanying end rotation. Stretching of rails, opening of splice joints, column bending, skewing of the crane girders, and undulating crane motion are among the problems created by excessive deflection. Hence it is important to limit the vertical deflection of the gantry girder as per the limits given in Table 8.4. In general, it is a good idea to keep the spans as short as possible and beam depths as large as possible.

Vertical loads are delivered to the gantry girder via the crane rail. (The depth of crane rails varies from 70 mm to 150 mm and their bottom width from 70 mm to 150 mm.) The beam must be capable of withstanding this localized load. It is recommended to use a full penetration groove weld between the web and the top flange of welded plate gantry girders in order to maximize the fatigue life of the member. The length of the web that is affected by the concentrated wheel load is taken by considering a dispersion angle of 30° as shown in Fig. 8.10. The angle of 30° is a logical average between the 45° pure shear angle and the 22° angle used in the column stiffener analysis (Ricker 1982). Using this value of 30°, the affected length is given by

$$\text{Affected length} = 3.5 \times (\text{rail depth} + k) \tag{8.16}$$

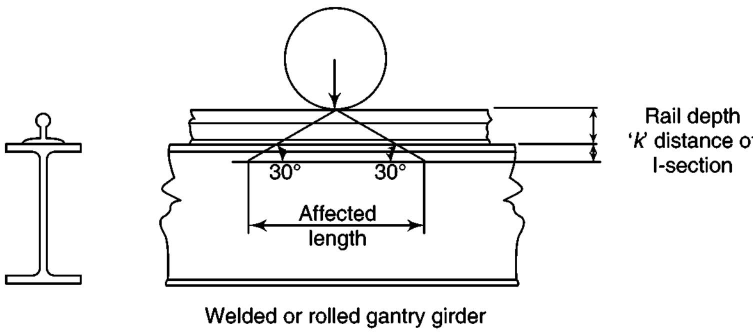


Fig. 8.10 Angle of dispersion for the concentrated load (Ricker 1982)

In the case of a plate girder, it becomes

$$\text{Affected length} = 3.5 \times (\text{rail depth} + \text{flange thickness}) \tag{8.17}$$

However, note that a dispersion angle of 45 degrees has been assumed in the Indian code and the dispersion is taken upto the mid-depth of the girder (see Section 6.10). It has been found that web crushing is not critical in most of the cases (Ricker 1982).

The effects of an off-centre crane rail must be considered. Excessive rail eccentricity must be avoided, because it causes local flange bending and subjects the gantry girder to torsional moments. A limit of $0.75t_w$ is often specified for this eccentricity (see Fig. 8.11) for both wide flange beams and plate girders (Ricker 1982). Intermediate stiffeners may be welded to the underside of the top flange and down the web with a continuous weld to counteract the effect of rail eccentricity.

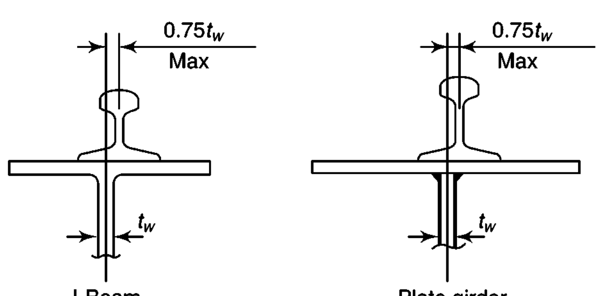


Fig. 8.11 Maximum allowable eccentricity for rails

While designing the gantry girder to resist lateral loading, the strength of the top flange alone should be considered. If this strength is inadequate, it may be reinforced by adding a channel, plate or angles, or by making a horizontal truss or girder in the case of large lateral loads (see Figs. 8.9). These reinforcing members are often attached by welding. Due to the fatigue factor associated with intermittent welds, it is advisable to use continuous welds. In designing gantry girders which require channels, plates, or angles to resist lateral loads, a simplified design method which considers that the beam resists the vertical forces and the lateral forces are resisted only by the channel (or plate or angle) may be used; but this will result in uneconomical designs. Hence most designers assume that the lateral load is resisted by the channel (or plates or angles) plus the top flange of the beam and that the vertical load is resisted by both the beam and the channel (or plate or angle). If clamps are used to fasten the rails above the girder, it is necessary to select member sizes which will accept the required hole spacing (see Fig. 8.12).

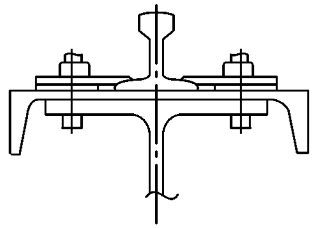


Fig. 8.12 Clamping of rails with bolts

8.4.1 Section Properties

With reference to Fig. 8.13, if A_p is the area of the top plate and A_s is the area of the rolled I-section, then the total area is given by

$$A = A_s + A_p$$

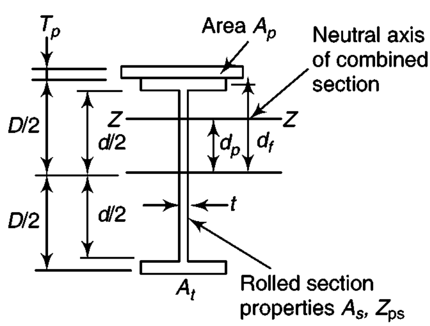


Fig. 8.13 Rolled I-beam with top plate

To find out the plastic section modulus, the neutral axis should be located at a location that divides the total area into two equal parts. Thus,

$$A_s/2 + d_p t = A_s/2 - d_p t + A_p$$

Hence $d_p = A_p/2t$. Ignoring the effect of the root fillets associated with rolled I-sections,

$$Z_{ps} = 2A_f d_f + td^2/4 \tag{8.18}$$

where A_f is the area of the flange and d_f is the distance of the flange from the neutral axis:

$$Z_{pz} = A_f(d_f - d_p) + A_f(d_f + d_p) + t(d/2 - d_p)^2/2 + t(d/2 + d_p)^2/2$$
$$+ A_p(D/2 + T_p/2 - d_p)$$
$$= 2A_f d_f + td^2/4 + td_p^2 + A_p(D/2 + T_p/2 - d_p)$$
$$= Z_{ps} + td_p^2 + A_p(D/2 + T_p/2 - d_p) \tag{8.19}$$

Note that the above formula is applicable only if the neutral axis of the combined section lies within the web depth. If it lies within the flange of the section, the section properties should be determined from first principles.

8.4.2 Columns

Figure 8.14 shows the various gantry girder column profiles. If the gantry girder is supported on a bracket attached to a column, then the impact must be considered in the design of the bracket. Slots are provided in the bracket seat plate for lateral adjustment. Stiffeners are placed at the end of the beam to prevent web buckling. The bolts connecting the beam to the bracket must be strong enough to resist the longitudinal forces.

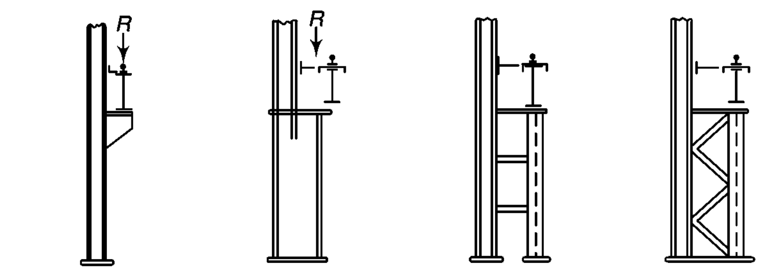


Fig. 8.14 Typical profiles of columns to support gantry girders

8.4.3 Bracings

Columns supporting gantry girders should be braced laterally and longitudinally. The simplest and most effective bracing is the X-bracing system. It is better to limit the L/r ratio of such bracings to 200, due to the abrupt reversal of stresses in crane runways. Bracing members should be made of double angles, wide flange beams, tubes, or pipe sections; these should never be made of rods. Single-angle bracings may be used on light cranes. Bracings should never be connected directly to the underside of gantry girders.

It is better to locate the braces near the centre of the runway, since it will allow thermal expansion and contraction to advance or retreat from a centrally 'anchored' area of the runway towards the ends. Knee braces should never be used, since these are the source of many crane run problems (causing undesirable restraint, column bending, and secondary stresses).

8.5 Design of Gantry Girder

The design of the gantry girder subjected to lateral loads is a trial-and-error process. As already mentioned, it is assumed that the lateral load is resisted entirely by the compression top flange of the beam and any reinforcing plates, channels, etc. and that the vertical load is resisted by the combined beam. The various steps involved in the design are given below.

1. The first step is to find the maximum wheel load. As discussed in Section 8.1, this load is maximum when the trolley is closest to the gantry girder. It can be calculated by using Eqn (8.1) and increased for the impact as specified in Table 8.3.
2. The maximum bending moment in the gantry girder due to vertical loads needs to be computed. This consists of the bending moment due to the maximum wheel loads (including impact) and the bending moment due to the dead load of the girder and rails. Equation (8.9) gives the maximum bending moment due to wheel loads when only one crane is running over the girder and Eqn (8.14) gives the maximum bending moment when two cranes are placed on one gantry girder. The bending moment due to dead loads is maximum at the centre of the girder, whereas the bending moment due to the wheel load is maximum below one of the wheels. However, for simplifying the calculations, the maximum bending moment due to the dead load is directly added to the maximum wheel load moment.
3. Next the maximum shear force is calculated. This consists of the shear force due to wheel loads and dead loads from the gantry girder and rails. The shear force due to wheel loads can be calculated using either Eqn (8.8) or (8.13) depending on whether one or two cranes are operating on the gantry girder. Generally an I-section with a channel section is chosen, though an I-section with a plate at the top flange may be used for light cranes. When the gantry is not laterally supported, the following may be used to select a trail section:

   $$Z_p = M_u/f_y \tag{8.20}$$

   $$Z_p\ (\text{trial}) = kZ_p \quad (k = 1.40\text{–}1.50)$$

   Generally, the economic depth of a gantry girder is about (1/12)th of the span. The width of the flange is chosen to be between (1/40) and (1/30)th of the span to prevent excessive lateral deflection.
4. The plastic section modulus of the assumed combined section is found out by considering a neutral axis which divides the area in two equal parts, at distance y to the area centroid from the neutral axis. Thus

   $$M_p = 2f_y A/2\bar{y} = A\bar{y}f_y \tag{8.21}$$

   where $A\bar{y}$ is equal to the plastic modulus Z_p.

5. When lateral support is provided at the compression (top) flange, the chosen section should be checked for the moment capacity of the whole section (clause 8.2.1.2 of IS 800):

$$M_{dz} = \beta_b Z_p f_y / \gamma_{m0} \leq 1.2 Z_e f_y / \gamma_{m0} \qquad (8.22)$$

The above value should be greater than the applied bending moment. The top flange should be checked for bending in both the axes using the interaction equation

$$(M_y/M_{ndy}) + (M_z/M_{ndz}) \leq 1.0 \qquad (8.23)$$

6. If the top (compression) flange is not supported, then the buckling resistance is to be checked in the same way as in step 4 but replacing f_y with the design bending compressive stress f_{bd} (calculated using Section 8.2.2 of the code).
7. At points of concentrated load (wheel load or reactions) the web of the girder must be checked for local buckling and, if necessary, load-carrying stiffeners must be introduced to prevent local buckling of the web.
8. At points of concentrated load (wheel load or reactions) the web of the girder must be checked for local crushing. If necessary, bearing stiffeners should be introduced to prevent local crushing of the web.
9. The maximum deflection under working loads has to be checked.

Examples

Example 8.1 *Determine the moments and forces due to the vertical and horizontal loads acting on a simply-supported gantry girder given the following data.*

1. *Simply-supported span = 6 m*
2. *Crane's wheel centres = 3.6 m*
3. *Self-weight of the girder (say) = 1.6 kN/m*
4. *Maximum crane wheel load (static) = 220 kN*
5. *Weight of crab/trolley = 60 kN*
6. *Maximum hook load = 200 kN*

Calculate also the serviceability deflection (working load).

Solution

1. *Moments and forces due to self-weight*

Factored self-weight $W_d = 1.5 \times 1.5 \times 6 = 13.5$ kN

Ultimate mid-span BM, $M_1 = W_d L/8 = 13.5 \times 6/8 = 10.125$ kNm

Ultimate reaction = $R_{A1} = R_{B1} = W/2 = 13.5/2 = 6.75$ kN

2. *Moments and forces due to the vertical wheel load*

Wheel load (including γ_f and 25% impact)

$$W_c = 1.5 \times 1.25 \times 220 = 412.5 \text{ kN}$$

Ultimate BM under wheel (case 1) as per Eqn (8.9)

$$= 2W_c(L/2 - c/4)^2/L$$

$$= 2 \times 412.5(6/2 - 3.6/4)^2/6 = 606.375 \text{ kNm}$$

Ultimate BM under wheel (case 2)

$= W_c L/4 = 412.5 \times 6/4 = 618.75$ kNm

Hence, the maximum ultimate BM,

$M_2 = 618.75$ kNm

Ultimate reaction [Eqn (8.10)]

$R_{A2} = W_c(2 - c/L) = 412.5(2 - 3.6/6.0) = 577.5$ kN

3. *Moment and forces due to horizontal wheel loads*

Horizontal surge load (including γ_f)

$W_{hc} = 1.5 \times 0.10(200 + 60) = 39$ kN

This is divided among the 4 wheels (assuming double-flanged wheels).

Horizontal wheel load

$W_{hc} = 39/4 = 9.75$ kN

Using calculations similar to those for vertical moments and forces, ultimate horizontal BM (case 2)

$= 2W_{hc}L/4 = 9.75 \times 6.0/4 = 14.625$ kNm

Ultimate horizontal BM (case 1)

$= 2W_{hc}(L/2 - c/4)^2/L$

$= 2 \times 9.75(6/2 - 3.6/4)^2/6 = 14.33$ kN m

4. *BM and reaction due to drag force*

Assuming that e is 0.15 m and the depth of the girder is 0.6 m,

$R_{A3} = W_g e/L = 1.5 \times (0.05 \times 220 \times 1.25)(0.3 + 0.15)/6 = 1.55$ kN

Ultimate BM due to drag force

$M_3 = R_a(L/2 - c/4) = 1.55(3 - 0.9) = 3.255$ kNm

Hence, the maximum ultimate design BM (vertical)

$= M_1 + M_2 + M_3 = 10.125 + 618.75 + 3.255 = 632.13$ kNm

Maximum design reaction (vertical)

$R_{A1} + R_{A2} + R_{A3} = 6.75 + 577.5 + 1.55 = 585.8$ kN

5. Serviceability deflection due to vertical wheel load excluding impact

$W_c = 220$ kN

$\Delta_c = W_c L^3[(3a/4L) - (a^3/L^3)/(6EI)$ with $a = (L - C)/2$

Assuming ISMB 600 with $I_z = 91{,}800 \times 10^4$ mm^4

$a = (6000 - 3600)/2 = 1200$ mm

$$\Delta_c = \frac{220 \times 1000 \times 6000^3 \left[\dfrac{3 \times 1200}{(4 \times 6000)} - \dfrac{1200^3}{6000^3}\right]}{[6 \times 2 \times 10^5 \times 91{,}800 \times 10^4]}$$

$= 6.125$ mm $< L/750 = 8$ mm

Example 8.2 *Design a gantry girder, without lateral restraint along its span, to be used in an industrial building carrying an overhead travelling crane for the following data (see Fig. 8.15).*

Centre-to-centre distance between columns (i.e., span of the gantry girder) = 7.5 m

Crane capacity = 200 kN

Self-weight of the crane girder excluding trolley = 200 kN

Self-weight of trolley, electrical motor, hook, etc. = 40 kN

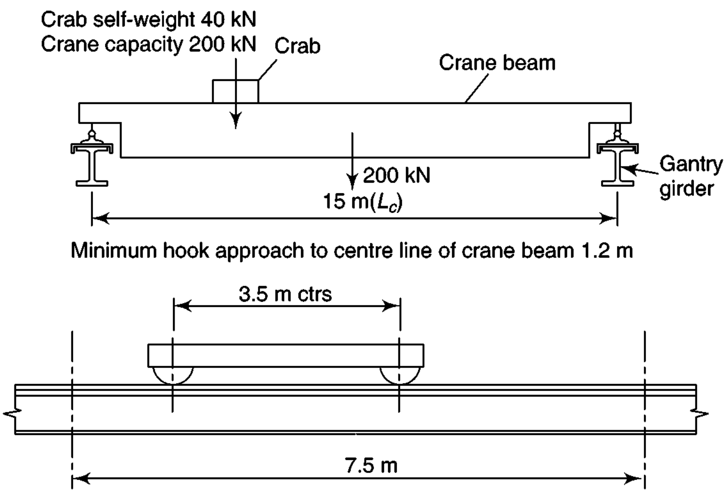


Fig. 8.15 Design data for Example 8.2

Minimum hook approach = 1.2 m
Distance between wheel centres = 3.5 m
Centre-to-centre distance between gantry rails (i.e., span of the crane) = 15 m
Self-weight of the rail section = 300 N/m
Yield stress of steel = 250 MPa

Solution

1. Load and bending moment calculations

(a) Load

(i) Vertical loading

Calculation of maximum static wheel load

Maximum static wheel load due to the weight of the crane = 200/4 = 50 kN
Maximum static wheel load due to crane load

$$W_1 = [W_t(L_c - L_1)]/(2L_c)$$
$$= (200 + 40)(15 - 1.2)/(2 \times 15)$$
$$= 110.4 \text{ kN}$$

Total load due to the weight of the crane and the crane load = 50 + 110.4 = 160.4 kN

To allow for impact, etc., this load should be multiplied by 25% (see Table 8.3).

Design load = 160.4 × 1.25 = 200.5 kN

∴ Factored wheel load on each wheel,

$$W_c = 200.5 \times 1.5 = 300.75 \text{ kN}$$

(ii) Lateral (horizontal) surge load

Lateral load (per wheel) = 10%(hook + crab load)/4
= 0.1 × (200 + 40)/4
= 6 kN

Factored lateral load = 1.5 × 6 = 9 kN

(iii) Longitudinal (horizontal) braking load

Horizontal force along rails (Table 8.3) = 5% of wheel load

$= 0.05 \times 200.5 = 10.025$ kN

Factored load $P_g = 1.5 \times 10.025 = 15.04$ kN

(b) Maximum bending moment

(i) Vertical maximum bending moment

Without considering the self-weight,

$$M_1 = W_c L/4 = 300.75 \times 7.5/4 = 563.90 \text{ kNm}$$

$$M_2 = 2W_c(L/2 - c/4)^2/L$$

$$= 2 \times 300.75(7.5/2 - 3.5/4)^2/7.5$$

$$= 662.90 \text{ kNm}$$

Hence $M = 662.90$ kNm.

Assume that the self-weight of the gantry girder is 1.6 kN/m.

Total dead load = 1600 + 300 (self-weight of rail) = 1.9 kN/m

Factored DL = (1.9 × 1.5) = 2.85 kN/m

BM due to dead load $= wl^2/8 = \left(2.85 \times \dfrac{7.5^2}{8}\right)$

$= 20.04$ kNm

(ii) Horizontal bending moment

Moment due to surge load $= 2 \times 9(7.5/2 - 3.5/4)^2/7.5$

$$M_y = 19.84 \text{ kNm}$$

(iii) Bending moment due to drag (assuming the rail height as 0.15 m and the depth of given as 0.6 m)

Reaction due to drag force $= P_g e/L = 15.04(0.3 + 0.15)/7.5 = 0.903$

$$M_3 = R(L/2 - c/4) = 0.903(7.5/2 - 3.5/4) = 2.59 \text{ kNm}$$

Total design bending moment $M_z = 662.9 + 20.04 + 2.59 = 685.53$ kNm

(c) Shear force

(i) Vertical shear force

Shear force due to wheel load

$$W_L(2 - c/L) = 300.75(2 - 3.5/7.5) = 461.15 \text{ kN}$$

Shear force due to DL $= \dfrac{wl}{2} = 2.85 \times \dfrac{7.5}{2} = 10.69$ kN

Maximum ultimate shear force

$$V_z = 10.94 + 461.15 = 472.09 \text{ kN}$$

(ii) Lateral shear force due to surge load $V_y = 9(2 - 3.5/7.5) = 13.8$ kN

Reactions due to drag force = 0.903 kN

Maximum ultimate reaction

$$R_Z = 472.09 \text{ kN} + 0.903$$

$$= 472.99 \text{ or } 473 \text{ kN}$$

2. Preliminary selection of the girder

Since $L/12 = 7500/12 = 625$, we choose the depth as 600 mm. Therefore,
Approximate width of beam = $L/30 = 250$ mm
Since deflection governs the design, choose I, using the deflection limit of $L/750$,

$$I = \frac{15.6W(L-c)}{LE}[2L^2 + 2Lc - c^2]$$
$$= 15.6 \times 200.5 \times 10^3 \times (7500 - 3500)$$
$$\times [2 \times 7500^2 + 2 \times 7500 \times 3500 - 3500^2]/(2 \times 10^5 \times 7500)$$
$$= 1.274 \times 10^9 \text{ mm}^4$$

Required $Z_p = 1.4 \times M/f_y = 1.4 \times 685.53 \times 10^6/250 = 3.83 \times 10^6$ mm^3

Choose ISMB 600 @ 1230 N/m with a channel ISMC 300 @ 363 N/m @ the top (see Fig. 8.16).

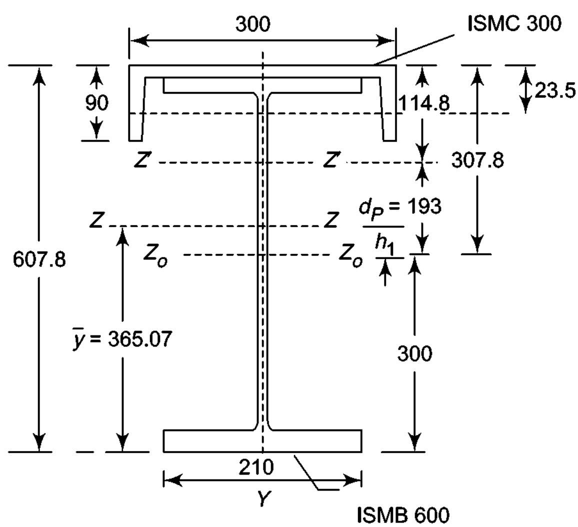


Fig. 8.16

a. Properties of the sections

ISMB 600 @ 1.23 kN/m	ISMC 300 @ 0.363 kN/m
$A = 15600$ mm^2	$A = 4630$ mm^2
$t_f = 20.3$ mm	$t_f = 13.6$ mm
$t_w = 12$ mm	$t_w = 7.8$ mm
$B = 210$ mm	$B = 90$ mm
$I_{zz} = 91800 \times 10^4$ mm^4	$I_{zz} = 6420 \times 10^4$ mm^4
$I_{yy} = 2650 \times 10^4$ mm^4	$I_{yy} = 313 \times 10^4$ mm^4
$R = 20$ mm	$C_y = 23.5$ mm

(i) Elastic properties of the combined section

Total area $A = A_B + A_{ch} = 15{,}600 + 4630 = 20{,}230$ mm^2

The distance of NA of the built-up section from the extreme fibre of tension flange,

$\bar{y}$ = [15,600 × 600/2 + 4630 × (600 + 7.8 – 23.5)]/20,230 = 365.07 mm

$h_1 = \bar{y} - h_B/2 = 365.07 - 600/2 = 65.07$ mm

$h_2 = (h_B + t_{ch}) - \bar{y} - C_y = (600 + 7.8) - 365.07 - 23.5 = 219.23$ mm

$h_3 = 607.8 - 365.07 - 7.8 = 234.93$ mm

$I_Z = I_{ZB} + A_B h_1^2 + (I_y)_{ch} + A_{ch} \times h_2^2$

$= 91{,}800 \times 10^4 + 15{,}600 \times 65.07^2 + 313 \times 10^4 + 4630 \times 219.23^2$

$= 1.2097 \times 10^9$ mm^4 ≈ 1.274×10^9 (required for deflection control)

$Z_{Zb} = 1.2097 \times 10^9/365.07 = 3.31 \times 10^6$ mm^3

$Z_{Zt} = 1.2097 \times 10^9/242.73 = 4.98 \times 10^6$ mm^3

I_{yy} combined = $2650 \times 10^4 + 6420 \times 10^4 = 9070 \times 10^4$ mm^4

I_y for tension flange about the *y-y* axis

$I_{tf} = 20.3 \times 210^3/12 = 1566.6 \times 10^4$ mm^4

For compression flange about the *y-y* axis

$I_{cf} = 1566.6 \times 10^4 + 6420 \times 10^4 = 7986.6 \times 10^4$ mm^4

Z_y (for top flange alone) = $7986.6 \times 10^4/150 = 532{,}443$ mm^3

b. Calculation of plastic modulus (see Section 8.4.1 and Fig. 8.16)

The plastic neutral axis divides the area into two equal areas, i.e., 10,115 mm^2.

$d_p = 4630/(2 \times 12) = 193$ mm

Ignoring fillets, the plastic section modulus below the equal-area axis is

$\Sigma A\bar{y} = 20.3 \times 210 \times (493 - 20.3/2) + (493 - 20.3) \times 12 \times (493 - 20.3)/2$

$= 3399.1 \times 10^3$ mm^3

Above the equal-area axis

$\Sigma A\bar{y} = 4630 \times (114.8 - 23.5) + 210 \times 20.3 \times (114.8 - 7.8 - 10.15)$

$+ 86.7 \times 12 \times 86.7/2 = 880.692 \times 10^3$ mm^3

$Z_{pz} = 3399.1 \times 10^3 + 880.692 \times 10^3 = 4279.792 \times 10^3$ mm^3

For the top flange only

$Z_{py} = 20.3 \times 210^2/4 + (300 - 2 \times 13.6)^2 \times 7.8/4 + 2 \times 90 \times 13.6$

$\times (150 - 13.6/2) = 719{,}479.8$ mm^3

3. Check for moment capacity

Check for plastic section

b/t of the flange of the I-beam = [(210 – 12)/2]/20.3 = 4.87 < 9.4

b/t of the flange of the channel = (90 – 7.8)/13.6 = 6.04 < 9.4

d/t of the web of the I-section = (600 – 2 × 20.3)/12 = 46.6 < 84

Hence the section is plastic.

a. Local moment capacity

$1.2Z_e f_y/1.1 = 1.2 \times 3.31 \times 10^6 \times (250/1.1) \times 10^{-6} = 902.72$ kNm

$M_{dz} = f_y Z_p/1.1 = (250/1.1) \times 4279.792 \times 10^{-3} = 972.68$ kNm > 902.72 kNm

Hence take M_{dz} = 902.72 kNm

$$M_{dz} = (f_y/1.1) \times Z_p(\text{top flange}) = (250/1.1) \times 719{,}479.8 \times 10^{-6} = 163 \text{ kNm}$$

$$1.2Z_e f_y/1.1 = 1.2 \times 532{,}443 \times (250/1.1) \times 10^{-6} = 145.2 \text{ kNm} < 163 \text{ kNm}$$

Hence take $M_{dy} = 145.2$ kNm.

b. Combined local capacity check

$$685.53/902.72 + 19.84/145.2 = 0.759 + 0.137 = 0.896 < 1$$

Hence the chosen section is the right choice.

4. Check for buckling resistance

As per IS 800 (Clause 8.2.2), the design bending strength

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

We have

$$\beta_b = 1.0$$

$$h = 600 + 7.8 = 607.8 \text{ mm}$$

$$KL = 7500 \text{ mm}$$

$$E = 2 \times 10^5 \text{ N/mm}^2$$

$$t_f = 20.3 + 7.8 = 28.1 \text{ mm}$$

$$r_y = \sqrt{I_{yy}/A}$$

$$I_{yy} = (2650 \times 10^4) + (6420 \times 10^4) = 9070 \times 10^4 \text{ mm}^4$$

$$A = 15{,}600 + 4630 = 20{,}230 \text{ mm}^2$$

$$r_y = \sqrt{\frac{9070 \times 10^4}{20{,}230}} = 66.96 \text{ mm}$$

According to clause 8.2.2.1 of (IS 800), elastic lateral buckling moment

$$M_{cr} = C_1 \frac{\pi^2 E I_y h}{2(KL)^2}\left[1 + \frac{1}{20}\left[\frac{KL/r_y}{h/t_f}\right]^2\right]^{0.5},$$

$C_1 = 1.132$ (from Table 42 of IS 800 : 2007)

Note The value of C_1 will be in between 1.046 and 1.132 as both udl and concentrated loads are applied.

$$M_{cr} = 1.132 \times \left(\frac{\pi^2 \times 2 \times 10^5 \times 9070 \times 10^4 \times 607.8}{2 \times 7500^2}\right)$$

$$\left[1 + \frac{1}{20}\left(\frac{7500/66.96}{607.8/28.1}\right)^2\right]^{0.5} = 1675.22 \times 10^6 \text{ N/mm}$$

Note The above formula is valid for I-sections only. For more accurate values of M_{cr} for a compound section, the formula given in E1.2 (Annex E) of the code may be used.

Non-dimensional slenderness ratio

$$\lambda_{LTZ} = \sqrt{\frac{\beta_b Z_{pz} f_y}{M_{cr}}}$$

$$= \sqrt{\frac{1.0 \times 4279.792 \times 10^3 \times 250}{1675.22 \times 10^6}} = 0.7992$$

Along the z-direction

$$\phi_{LTZ} = 0.5[1 + \alpha_{LT}(\lambda_{LTZ} - 0.2) + \lambda_{LTZ}^2]$$

$$= 0.5[1 + 0.21(0.7992 - 0.2) + 0.7992^2] = 0.882$$

Note Since the channel will normally be connected by intermittent welds to the I-section, α_{LT} value has been taken as 0.21. If heavy welding is involved, take α_{LT} as 0.49.

$$\chi_{LTZ} = \frac{1}{[\phi_{LTZ} + (\phi_{LTZ}^2 - \lambda_{LTZ}^2)^{0.5}]} \leq 1.0$$

$$= \frac{1}{0.882 + (0.882^2 - 0.7992^2)^{0.5}}$$

$$= 0.7967$$

$$f_{bd} = f_y \chi_{LT} / \gamma_{m0}$$

$$\gamma_{m0} = 1.10 \text{ (from Table 5 of the code)}$$

$$\therefore \quad f_{bd} = 0.7967 \times 250/1.1 = 181.07 \text{ N/mm}^2$$

$$\therefore \quad M_{dz} = \beta_b Z_{pz} f_{bd}$$

$$= 1.0 \times 181.07 \times 4279.792 \times 10^{-3}$$

$$M_{dz} = 774.9 \text{ kNm} > 685.53 \text{ kNm}$$

Thus the beam is satisfactory under vertical loading. Now it is necessary to check it under biaxial bending.

For top flange only

$$M_{dy} = (f_y/1.1) \times Z_{yt}$$

$$= (250/1.1) \times 719,\ 479 \times 10^{-6} = 163.5 \text{ kNm}$$

$$> \frac{1.2 \times 532{,}433 \times 250}{1.1 \times 10^6} = 145.2 \text{ kNm}$$

Hence $M_{dy} = 145.2$ kNm

a. Check for biaxial bending

In order to check for biaxial bending, we substitute the terms with their values in the following equation:

$$\frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} \not> 1.0$$

We have

$$\frac{658.53}{774.9} + \frac{19.84}{145.2} = 0.85 + 0.137 = 0.987 < 1.0$$

Hence the beam is safe.

5. Check for shear capacity

For vertical load,

$$V_z = 472.09 \text{ kN}$$

Shear capacity $= A_v f_{yw}/(\sqrt{3} \times 1.10)$

$= (600 \times 12) \times 250/(\sqrt{3} \times 1.10) \times 10^{-3}$

$= 944.75 \text{ kN} > 472.09 \text{ kN}$

The maximum shear force is 472.09 kN, which is less than 0.6 times the shear capacity, i.e.,

$$0.6 \times 944.75 = 566.85 \text{ kN}$$

Hence it is safe in vertical shear and there is no reduction in the moment capacity.

a. Weld design

The required shear capacity of the weld is given by

$$q = VA\bar{y}/I_Z$$

$$\bar{y} = h_3 = 234.93$$

$$A = 4630 \text{ mm}^2, \quad V = 472.09 \text{ kN}$$

$$I_Z = 1.2097 \times 10^9 \text{ mm}^4$$

$$q = 472.09 \times 10^3 \times 4630 \times 234.93/(1.2097 \times 10^9)$$

$$= 424 \text{ N/mm}$$

This shear is taken by the welds. Hence use a minimum weld of 4 mm (442 N/mm per weld) connecting the channel to the top flange of the I-beam.

For lateral shear force,

$$F_y = 13.8 \text{ kN}$$

Shear capacity $V_{ny} = A_v f_{yw}/(\sqrt{3} \times 1.10)$

$= 250/(\sqrt{3} \times 1.10) \times (210 \times 20.3 + 300 \times 7.8) \times 10^{-3}$

$= 866.41 \text{ kN} > 13.8 \text{ kN}$

Hence it is safe for resisting lateral shear.

6. Web buckling

At points of concentrated loads (wheel loads or reaction) the web of the girder must be checked for local buckling (see Section 6.10)

The dispersion length under wheel (assuming the diameter of wheel to be 150 mm and assuming an angle of dispersion of 45°)

$$b_1 = 150 \text{ mm}$$

$$n_1 = 600/2 + 2 \times 7.8 = 315.6 \text{ mm}$$

Web slenderness $\lambda = 2.5d/t$

$= 2.5 \times [600 - 2(20.3 + 20)]/12$

$= 108.2$

Stress reduction factor (from Table 8 of IS 800 : 2007) = 0.426

$$f_{cd} = 0.426 \times 250/1.1 = 96.8 \text{ MPa}$$

Buckling resistance $= (b_1 + n_1)tf_{cd}$

$= (150 + 315.6)12 \times 96.8 \times 10^{-3}$

$= 540.8 \text{ kN}$

Maximum wheel load = 300.75 kN < 540.8 kN

Hence buckling resistance is satisfactory.

7. Web bearing (see Section 6.10)

Load dispersion at support with 1: 2.5 dispersion

Minimum stiff bearing = $R_x/(tf_{yw}/1.1) - n_2$

$$n_2 = (20.3 + 20) \times 2.5 = 100.75 \text{ mm}$$

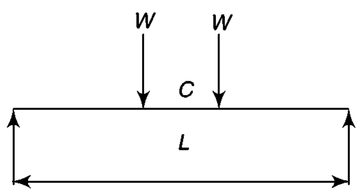
$$R_x = 473 \text{ kN (support reaction)}$$

$$b_1 = 473 \times 10^3/(12 \times 250/1.1) - 100.75 = 72.68 \text{ mm}$$

Web bearing at support requires a minimum stiff bearing of 73 mm.

8. Check for deflection at working load

Serviceability vertical wheel load excluding impact = 160.4 kN



Deflection at mid-span

$$\Delta = WL^3[(3a/4L) - (a^3/L^3)/(6EI)$$

Where

$$a = (L - c)/2 = (7500 - 3500)/2 = 2000$$

(i) Vertical

Combined $I_{zz} = 1.2097 \times 10^9$ mm^4

$$\Delta = \frac{160.4 \times 10^3 \times 7500^3}{6 \times 2 \times 10^5 \times 1.2097 \times 10^9} [3 \times 2000/(4 \times 7500) - 2000^3/7500^3]$$

$$= 8.43 \text{ mm} < L/750 = 10 \text{ mm}$$

(ii) Lateral

Only the compound top flange will be assumed to resist the applied surge load as in the bending check.

$$I = (I_z)_{ch} + I_F = 7986.6 \times 10^4 \text{ mm}^4$$

$$\Delta = \frac{6 \times 10^3 \times 7500^3}{6 \times 2 \times 10^5 \times 7986.6 \times 10^4} [3 \times 2000/(4 \times 7500) - 2000^3/7500^3]$$

$$= 4.78 < 10 \text{ mm (Table 6 of IS 800 : 2007)}$$

Summary

Cranes are often employed in industrial buildings to move stock, finished goods, or new materials from one place to another for processing. These cranes are supported on gantry girders, which are supported on separate columns or brackets attached to steel columns. The design of gantry girders is often complicated since they support moving load and are, hence, subjected to fatigue. Before attempting to design a gantry girder, the engineer should know the loading and clearance details from the crane manufacturer. In this chapter, some guidelines have been provided in the form of tables to do preliminary calculations and arrive at the size of gantry girders. These calculations should be checked, during the final design, with the exact details obtained from the manufacturers. The various loadings that may occur due to the moving loads have been discussed and expressions for calculating the maximum bending moment, shear, and deflection of gantry girders provided. The various factors that may affect the choice of girders are discussed. The steps in the design of gantry girders have been explained with the use of practical examples.

Exercises

1. A 50 kN hand-operated crane is provided in a building and has the following data:

Centre-to-centre distance of the gantry beam (width of the building)	16 m
Longitudinal spacing of columns (span of gantry)	7.5 m
Weight of the crane	40 kN
Wheel spacing	3 m
Weight of the crab	10 kN
Minimum hook approach	1 m
Yield stress of steel	250 MPa

   Design a simply supported gantry girder assuming lateral support to it.

2. Design a gantry girder, without lateral restraint along its span, to be used in an industrial building carrying overhead travelling crane for the following data:

Centre-to-centre distance between columns (span of the gantry girder)	6 m
Crane capacity	50 kN
Self-weight of the crane girder excluding trolley	40 kN
Self-weight of the trolley, electric motor, hook, etc.	10 kN
Minimum hook approach	1.0 m
Wheel centres	3 m
Centre-to-centre distance between gantry rails (span of crane)	12 m
Self-weight of rail section	100 N/m
Yield stress of steel	250 N/mm^2

3. Design a simply supported crane girder to carry an electric overhead travelling crane for the following data:

Crane capacity	300 kN
Weight of the crane and crab	300 kN
Weight of the crane	200 kN
Minimum hook approach	1.2 m
Centre-to-centre distance between wheels	3.2 m
Span of the gantry girder	5 m
Centre-to-centre distance between gantries	15 m
Weight of rail	300 N/m
Height of rails	75 mm
Yield stress of steel	250 MPa

Review Questions

1. What is the main purpose of a gantry girder?
2. What are the components of a crane runway system?
3. What are the requirements to be considered by the designer while selecting a crane and designing a crane supporting structure?
4. List the loads that should be considered while designing a gantry girder.
5. The impact allowance in percentage to be applied to the vertical forces transferred to the wheels of an EOT crane is
   (a) 10% of the maximum static wheel load
   (b) 10% of the weight of the crab and the weight lifted on the crane
   (c) 25% of the maximum static wheel load
6. The impact allowance for horizontal force transverse to the rail (surge load) for an EOT crane is
   (a) 25% of the weight of the crab and the weight lifted on the crane
   (b) 10% of the static wheel load
   (c) 10% of the weight of the crab and the weight lifted on the crane
7. The impact allowance for drag forces along the rail for an EOT crane is
   (a) 10% of the static wheel load
   (b) 5% of the static wheel load
   (c) 10% of the weight of crab and the weight lifted on the crane.
   (d) 5% of the weight of the crab and the weight lifted on the crane.
8. What is the difference between surge load and drag load of cranes?
9. The surge load is assumed to be resisted by the
   (a) whole cross section
   (b) compression flange alone
   (c) compression and tension flanges
   (d) cross section above the neutral axis
10. Write down the expressions for maximum shear force, bending moment, and deflection at mid-span for a simply supported beam with two moving loads, each with a value W.
11. Write down the expressions for maximum shear force, bending moment, and deflection at mid-span for a simply supported beam with four moving loads, each with a value W (two cranes running in tandem).
12. List the different profiles of cross sections which are used for gantry girders.
13. Why are simply supported girders preferred to two-span gantry girders?
14. What is the limiting vertical deflection of the gantry girder for
   (a) manually operated cranes
   (b) EOT cranes with a capacity less than 500 kN
   (c) EOT cranes with a capacity greater than 500 kN
15. List the various steps involved in the design of a gantry girder.

CHAPTER 9

Design of Beam-columns

Introduction

Most columns are subjected to bending in addition to the axial load; considerable care should be taken in a practical situation to load a column under axial load only. When significant bending is present in addition to an axial load in a member, the member is termed as a *beam-column.* The bending moments on a column may be due to any of the following effects.

(a) *Eccentricity of axial force*

(b) *Building frame action* In a multi-storey building, usually columns support beams which have similar identical connection eccentricities at each floor level. In a rigid frame building construction, the columns carry the building load axially as well as end moments from the girders that frame into them. Most building frames are braced against sway by bracings or core walls, but the horizontal wind forces have to be resisted by bending actions in the columns. Due to the wind forces, the columns may bend in double-curvature bending (similar to the shape of the letter 'S') in contrast to the single curvature bending of columns due to moment, created by gravity loads. Single curvature bending is often the most critical design condition than reverse or double curvature bending. These are shown schematically in Fig. 9.1. Wind loads can also produce lateral loading on a column, giving beam-type bending moment distribution.

(c) *Portal or gable frame action* Another common example of a column with bending moments occurs in a portal frame where the columns and rafters are subjected to relatively light axial loads combined with bending.

(d) *Load from brackets* In industrial buildings, column brackets may be used to carry gantry girders on which the cranes move. The resulting eccentricity produces bending moments in addition to the axial loads in the columns. In this case, the column moment is not at the column ends {see Fig. 9.1(d)}.

(e) *Transverse loads* As already discussed, wind pressure on long vertical members may produce bending moments. Similarly earthquakes also produce bending in the columns. Purlins placed between panel joints of a rafter of roof trusses (in order to reduce the size of purlin or to accommodate maximum size of roofing sheets) will produce bending in rafters (see also Chapter 12).

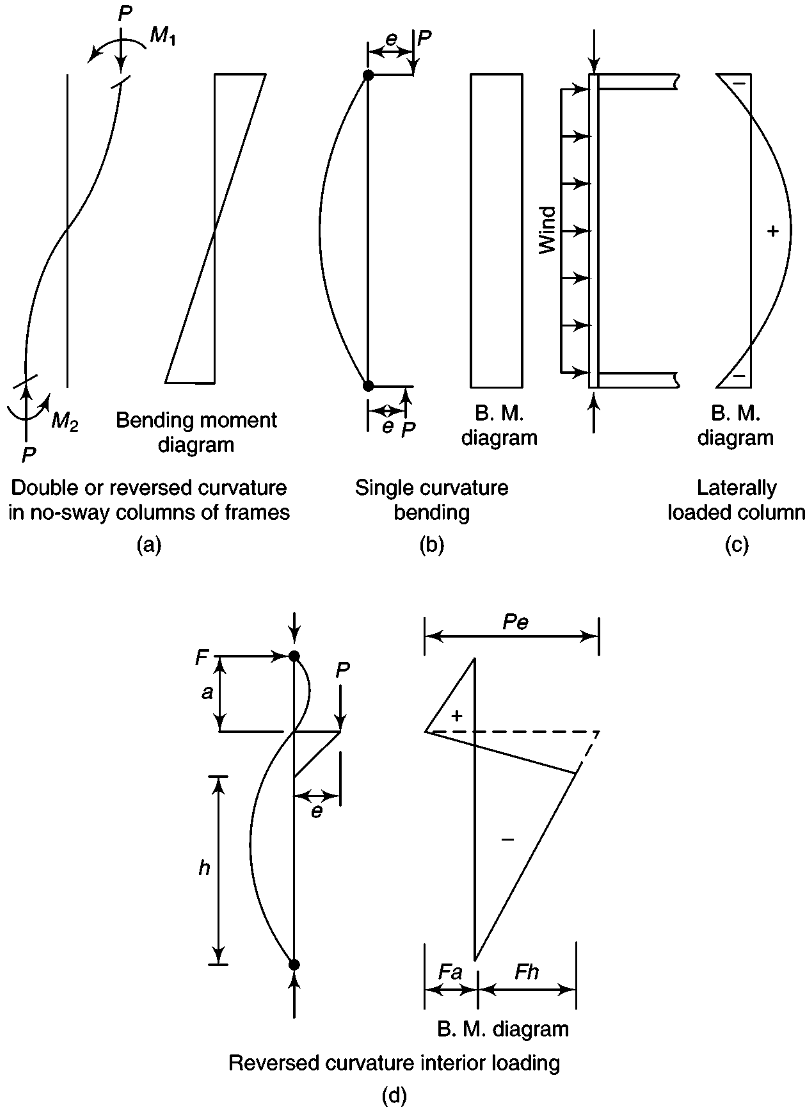


Fig. 9.1 Single and double curvature bending in beam-columns

(f) *Fixed base condition* If the base of the column is fixed due to piles, rafts, or grillage foundation, bending moments will be present at the base of the columns, even though their top ends may be hinged.

Beam columns in steel structures are often subjected to biaxial bending moments, acting in two principal planes, due to the space action of the framing system. The column cross section is usually oriented in such a way to resist significant bending about the major axis of the member. When I-shaped cross sections are used for the columns, the minor axis bending may also become significant, since the minor axis bending resistance of I-section is small compared to the major axis bending resistance.

Thus, in general, beam-columns are subjected to axial forces and bending moments. As the bending moment on a beam-column approaches zero, the member tends to become a centrally loaded column, a problem that has been treated in Chapter 5 on compression members. As the axial forces on a beam-column approaches zero, the problem becomes that of a beam, which has been adequately covered in Chapter 6 on beams. All of the parameters that affect the behaviour of a beam or a column (such as length of the member, geometry and material properties, support conditions, magnitude and distribution of transverse loads and moments, presence or absence of lateral bracing, and whether the member is a part of an unbraced or braced frame), will also affect the behaviour, strength, and design of beam-columns (Vinnakota 2006).

Note that bending may also be produced in tension members such as the bottom chords of bridge trusses, when floor beams frame into them. Bottom chords of roof trusses may support hoisting devices or other temporary loads, thus producing bending moments in addition to the axial loads present.

9.1 General Behaviour of Beam-columns

Beam columns are aptly named, as sometimes they can behave essentially like restrained beams, forming plastic hinges, and under other conditions fail by buckling in a similar way to axially loaded columns or by lateral torsional buckling similar to unrestrained beams. Under both bending moment (M) and axial load (P) the response of a typical beam-column for lateral deflection (δ) or end-joint rotation (θ) would be as shown in Fig. 9.2. However, both the strength attained and the form of curve is dependent on which features dominate the behaviour of the member.

It may be observed that the curve is non-linear almost from the start because of the P-δ effect. The P-δ effect becomes more and more significant as the applied end moments increase. At point A, due to the combined effect of the primary moment M and the secondary P-δ moment, the most severely stressed fibres of the cross sections may yield. This yielding reduces the stiffness of the member and hence the slope of the M-θ curve reduces beyond point A. As the deformation increases, the P-δ moment also increases. Now, this secondary moment will share a proportionally larger portion of the moment capacity of the cross section. Under increasing moment, the plasticity would spread into the section and a local hinge rotation would be developed (point B on the M-θ curve). The hinge would now spread a short distance down the column, which causes the slight downward slope on the M–θ curve and at point C, the moment carrying capacity of the cross section is finally exhausted.

In the preceding discussion, we assumed that other forms of failure do not occur before the formation of a plastic hinge. However, if the member is slender and the cross section is torsionally weak (e.g., open cross sections), lateral torsional buckling may occur. Lateral torsional buckling may occur in the elastic range (curve 1 shown in Fig. 9.2) or in the inelastic range (curve 2 shown in Fig. 9.2) depending on the slenderness of the member. As discussed in Chapter 6, member with a high slenderness ratio will experience elastic lateral torsional buckling, whereas a member with an intermediate slenderness ratio will experience inelastic lateral torsional buckling. Lateral torsional buckling will not occur if the slenderness ratio of the member is low or if the member is bent about the minor principal axis of the cross section. Similarly in members with cross sections having axis symmetry (e.g., circular sections) or equal moment of inertia about both principal axes (e.g., square box sections), lateral torsional buckling will not occur, regardless of the slenderness ratio.

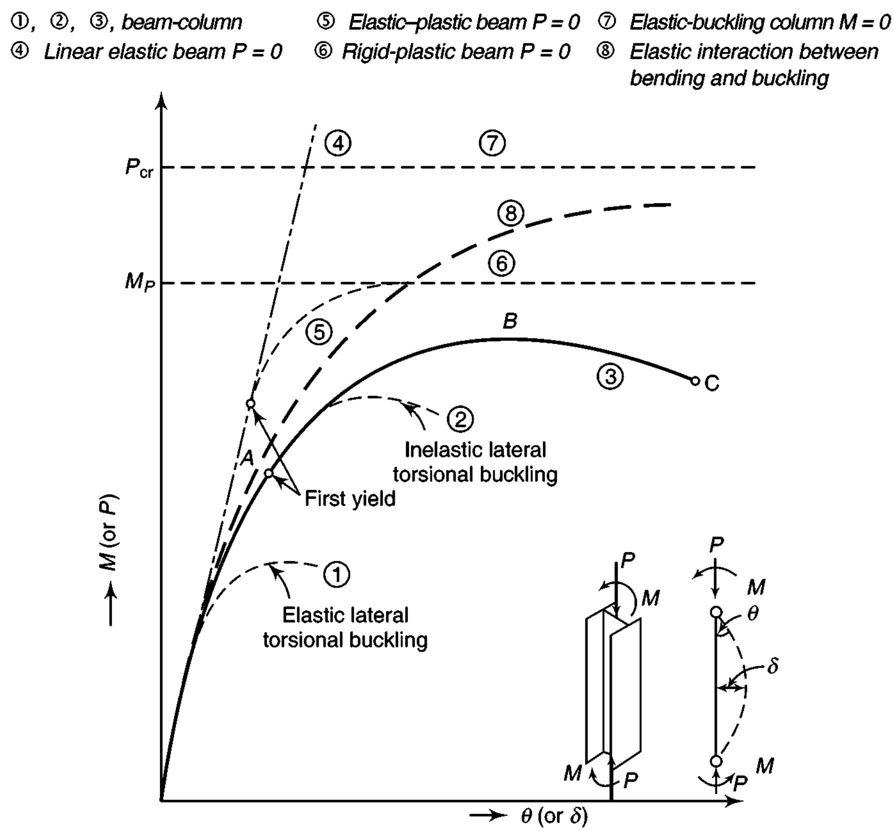


Fig. 9.2 Behaviour of a beam column compared with beams and columns

Another form of failure which may occur in the member is local buckling of component elements of the cross section. As seen in Chapter 6, the element with high width-to-thickness ratio is susceptible to local buckling. Like lateral torsional buckling, local buckling may occur in the elastic or inelastic range. The effect of both lateral torsional buckling and local buckling is to reduce the load carrying capacity of the cross section. Local buckling may be prevented by limiting the width-to-thickness ratios as specified in Table 4.3. When it is not possible to limit the width-to-thickness ratios local buckling may be accounted for in design by using a reduced width for the buckled element (see Example 5.1b).

Based on the earlier discussions, the behaviour of the beam-column may be classified into the following five cases (MacGinley & Ang 1992):

1. A short beam-column subjected to axial load and uniaxial bending about either axis or biaxial bending. Failure occurs when the plastic capacity of the section is reached, with the limitations set in the second case.

2. A slender beam-column subjected to axial load and uniaxial bending about the major axis *z-z*. If the beam-column is supported laterally against buckling about the minor axis *y-y* out of the plane bending, the beam-column fails by buckling about the *z-z* axis. It represents an interaction between column buckling and simple uniaxial bending. If the beam-column is not very slender a plastic hinge forms at the end or point of maximum moment {see Fig. 9.3(a)}. Note that this is not a common case.

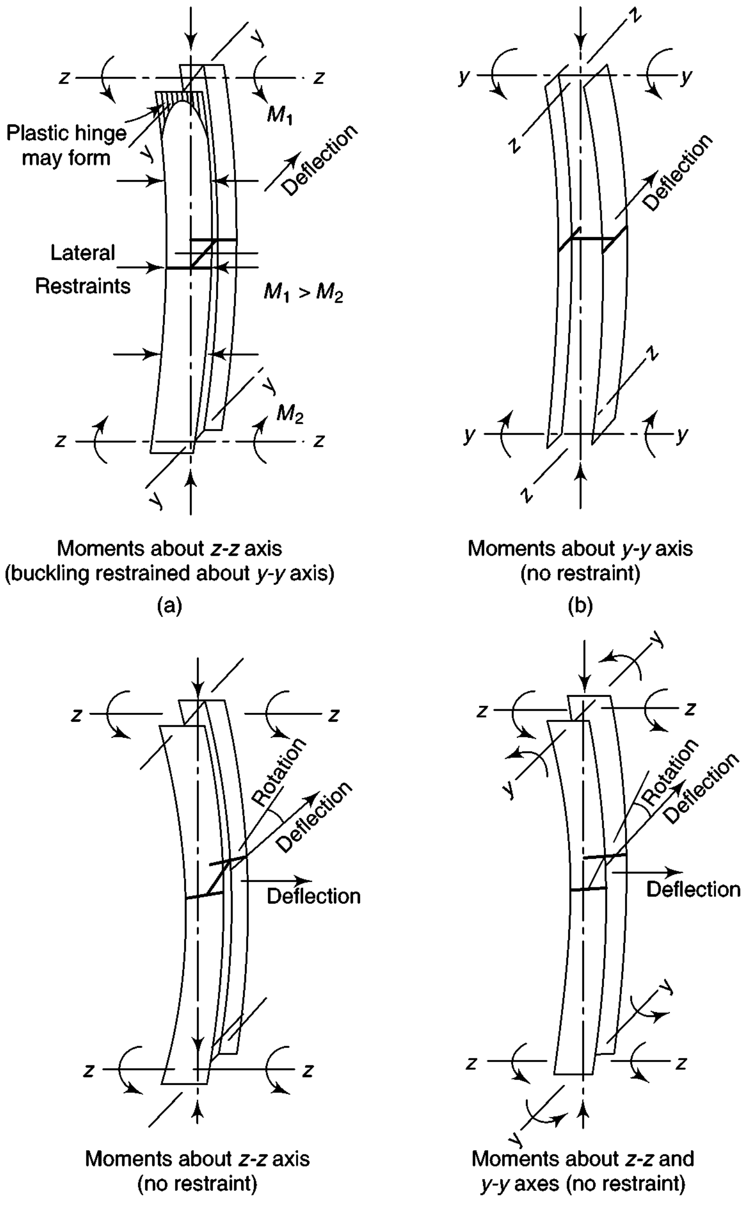


Fig. 9.3 Behaviour of slender beam-columns

3. A slender beam-column subjected to axial load and uniaxial bending about the minor axis y-y. Now there is no need for lateral support and no buckling out of the plane of bending. The beam-column fails by buckling about the y-y axis. This also represents an interaction between column buckling and simple uniaxial bending. At very low axial loads, the beam-column will attain the bending capacity about y-y axis {see Fig. 9.3(b)}.
4. A slender beam-column subjected to axial load and bending about the major axis z-z, and not restrained out of the plane of bending. The beam-column fails due to a combination of column buckling about the y-y axis and lateral torsional buckling. The beam-column fails by twisting as well as deflecting in the z-z and y-y planes {see Fig. 9.3(c)}. Thus it represents an interaction between column buckling and beam buckling.
5. A slender beam-column subjected to axial load and biaxial bending and not having any lateral support. The ultimate behaviour of the beam-column is complicated by the effect of plastification, moment magnification, and lateral torsional buckling. The failure will be similar to the fourth case but minor axis buckling will dominate. This is the general loading case {see Fig. 9.3(d)}.

9.2 Equivalent Moment Factor C_m

Member sizes of beam-columns are generally based on the magnitude of the maximum moment and the location of this maximum moment is not important in the design. Hence, the concept of equivalent moment, schematically shown in Fig. 9.4 is usually adopted in design specifications. Thus, it is assumed that the maximum second-order moment of a beam-column subjected to an axial load P and end moments M_A and M_B with $|M_B| > |M_A|$ as shown in Fig. 9.4(a) is numerically equal to the maximum second-order moment of the same member under axial load P and a pair of equal and opposite moments M_{eq}, as shown in Fig. 9.4(b). Note that the axial load P and second-order moment M_{max} are the same for both the members. The value of M_{eq} is given by (Salvadori 1956; Vinnakota 2006)

$$M_{eq} = C_m|M_B| \tag{9.1}$$

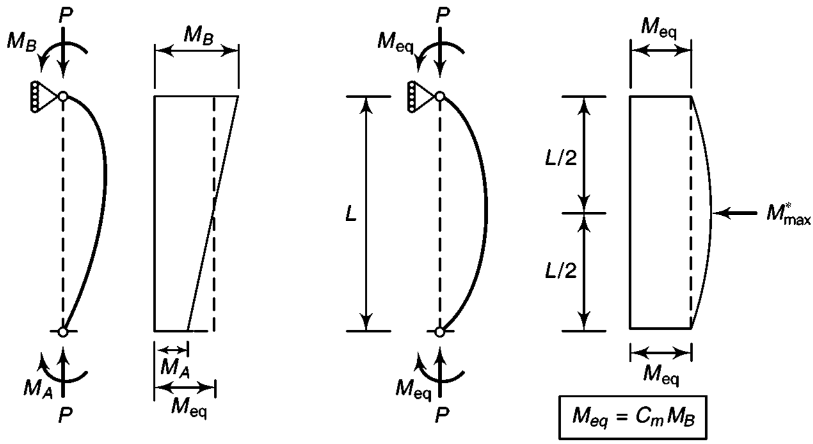
where C_m is called the *equivalent moment factor* or *moment reduction factor*. This factor is a function of the moment ratio $\psi = M_A/M_B$ and also the axial load ratio P/P_{cr}. Many simplified expressions for C_m have been proposed, as a function of the moment ratio only. The following expression given by Austin (1961) has been adopted in the ANSI/AISC code for beam-columns subjected to end moments (Chen & Zhou 1987; Galambos 1998)

$$C_m = 0.6 + 0.4\ \psi\ \geq 0.4 \text{ where } -1.0 \leq \psi \leq 1.0 \tag{9.2}$$

For beam-columns with transverse loadings, the second-order moments can be approximated by (ANSI/AISC: 360-05)

$$C_m = 1 + \beta(\alpha P_u/P_{cr}) \tag{9.3}$$

where $\beta = \pi^2\ \delta_o EI/ML^2 - 1$, $\alpha = 1.0$, δ_o is the maximum deflection due to transverse loading (in mm), M is the maximum first-order moment within the member due to transverse loading (N mm).



Unequal end moments
(a)

Equivalent end moment
(b)

Fig. 9.4 Concept of equivalent uniform moment

Thus, for a pin-ended beam-column of length L with a uniformly distributed load W,

$$M = WL/8,\ \delta = 5WL^3/384EI,\ P_{cr} = \pi^2EI/L^2$$

$$\beta = [(\pi^2EI)/L^2]\ [(5WL^3)/(384EI)]\ (8/WL) - 1 = 1.028 - 1 = 0.028$$

Hence $C_m = 1 + 0.028P/P_{cr}$.

Similarly for a beam-column of length L with a transverse load P at mid length,

$$M = PL/4;\ \delta = PL^3/48EI;\ P_{cr} = \pi^2EI/L^2$$

Thus $\beta = [(\pi^2EI)/L^2]\ (PL^3/48EI)\ (4/PL) - 1 = 0.822 - 1 = -0.178$

and

$$C_m = 1 - 0.178\ P/P_{cr}$$

Note that we must consider signs when using the ratio $\psi = M_A/M_B$, with M_A being the smaller of the two end moment values. The correct value of C_m will be close to 1.0 for all the cases and hence the American code (AISC 360-05) recommends that C_m may be conservatively taken as 1.0 for members with transverse loading.

9.3 Nominal Strength—Instability in the Plane of Bending

The basic strength of beam-columns, where lateral-torsional buckling and local buckling are adequately prevented, and bending is about one axis, will be achieved when instability occurs in the plane of bending (without twisting). The elastic differential equation solution shows that the axial compression effect and the bending moment effect cannot be determined separately and combined by super-position. The relationship is also non-linear.

Residual stresses, which cause premature yielding and consequently a premature reduction in stiffness, may reduce both the initial yield and the maximum strength of beam-columns. This is similar to their effect on axially loaded compression members. Similarly, an initial crookedness, which increases the secondary bending moment caused by the axial load, reduces both the initial yield load and the maximum strength. Moreover, when a beam-column with initial imperfections (displacements u_i, v_i, and initial twist ϕ_i) is loaded by the axial force P and major axis moment M_z, the member exhibits a non-bifurcation type of instability, in which the deformations increase (from u_i, v_i, and ϕ_i), until a maximum moment is reached, beyond which static equilibrium can only be sustained by decreasing the moment. The maximum strength based on the spatial behaviour of such initially crooked beam-columns $M_{cs,max}$ could be lower than the lateral-buckling load M_{cr} of the corresponding initially straight beam-column as shown in Fig. 9.5 (Vinnakota 2006).

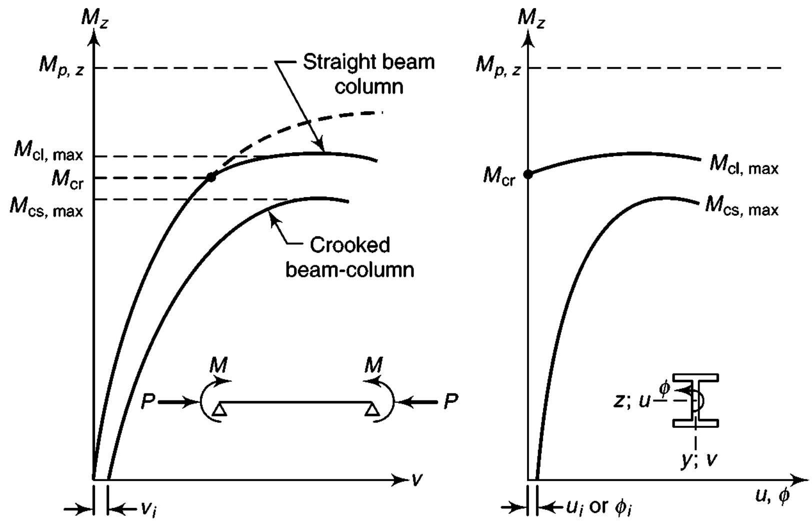


Fig. 9.5 Inelastic lateral-torsional buckling of beam-columns

The inelastic analysis to determine the strength interaction between axial compression P and bending moment M for a beam column is complicated. To trace a load deflection curve similar to that shown in Fig. 9.2, one should use some type of approximation or numerical technique. This is because the differential equations governing the inelastic behaviour of a beam-column are highly non-linear even for the simplest loading case (Chen & Atsuta 1976, 1977).

The analysis of the inelastic behaviour normally proceeds in two steps.

1. Cross section analysis
2. Member analysis

In the cross section analysis, the behaviour of a cross section subjected to the combined action of axial force and bending moment is investigated. The result is expressed as a set of M– –P (moment–curvature–axial compression) relationship. Once M–θ–P relationship is established, member analysis can be done.

In the member analysis, the member is divided into a number of segments whereby equilibrium and compatibility conditions along the length of the member at each division point are enforced for a given set of loadings or deflections. The analysis thus consists of finding successive solutions as the applied load or deflection of the member is increased in steps. When enough of these analyses have been performed, the load-deflection relationship of the beam-column can be traced on a pointwise basis.

9.3.1 Nominal Strength-Interaction Equations

As observed from the earlier discussions the behaviour of beam-columns is affected by a number of parameters. Moreover a real beam-column may receive end moments and axial load from its connections to other members of a structure, such as a rigid frame. Hence the relation of the beam-column to the other elements of the structure is important in determining both the applied forces and the resistance of the member.

The elasto–plastic analysis involving numerical procedures (Chen and Atsuta 1976, 1977) are quite laborious and their direct use in design is rather prohibitive since a structural frame normally consists of numerous beam-columns subjected to a variety of loading conditions. To simplify the design process and bring the problem within practical limits, *interaction equations*, relating a safe combination of axial force and bending moments, are often suggested by codes and specifications. Interaction curves are normally developed based on curve-fitting to existing analytical and experimental data on isolated beam-columns or beam-columns of simple portal frames. They have the general form of (Chen & Lui 1991; Duan & Chen 1989)

$$f\{(P_u/P_n),\ (M_{uz}/M_{nz}),\ (M_{uy}/M_{ny})\} \leq 1.0 \tag{9.4}$$

where P_u, M_{uz}, and M_{uy} are the axial force and bending moments (allowing for the P-Δ and P-δ effects) in the beam-column, and P_n, M_{nz}, and M_{ny} are the corresponding axial and bending moment capacities of the member {determined as discussed in Chapter 5 (Section 5.6.1) on compression members and Chapter 6 (Sections 6.6 and 6.7) on beams}.

The three-dimensional graphical representation of Eqn (9.4) is shown in Fig. 9.6. In this figure each axis represents the capacity of the member when it is subjected to loading of one type only, while the curves represent the combination of two types of loading. The surface formed by connecting the three curves represents the interaction of axial load and biaxial bending. It is this interaction surface that is of interest to the designer.

The end points of the curves shown in Fig. 9.6 are dependent on the capacities of the members described for columns (Chapter 5) and beams (Chapter 6). The shapes of these curves between these end points will depend on (a) the cross-sectional shape and the beam-column imperfections, (b) the variation of moments along the beam-column, and (c) the end restraint conditions of the beam-column. All these variables can only be dealt with on an approximate basis and hence various formulae are given in the codes, which attempt to allow for the effects mentioned earlier.

The basic form of the three-dimensional interaction equation is

$$(P/P_n) + (M_z/M_{nz}) + (M_y/M_{ny}) \leq 1.0 \tag{9.5}$$

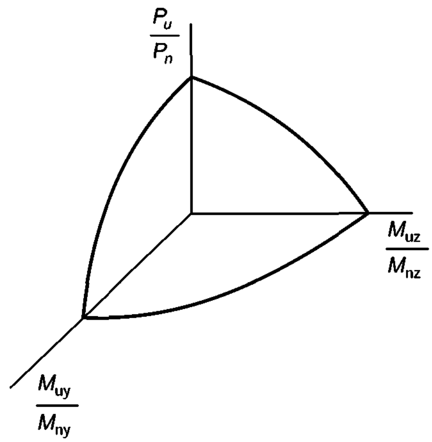


Fig. 9.6 Ultimate interaction surface for beam-columns

This interaction equation results in a straight-line representation of the interaction between any two components shown in Fig. 9.7. This simplified interaction equation gives a conservative design.

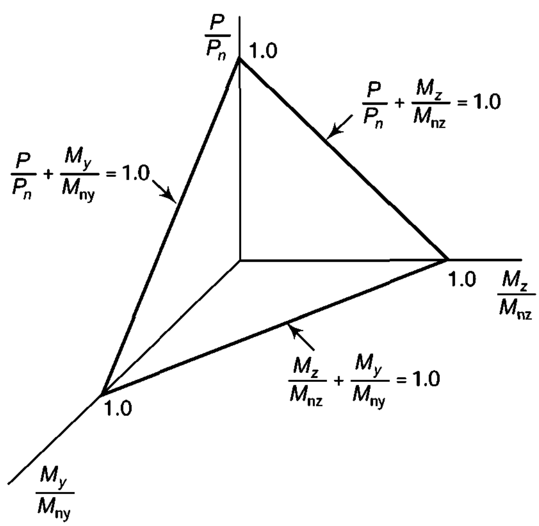


Fig. 9.7 Simplified interaction surface

The behaviour of beam-columns subject to bending moment about minor axis is similar to that subjected to major axis bending but for the following differences.

- In the case of slender members under small axial load, there is very little reduction of moment capacity below M_p, since lateral torsional buckling is not a problem in weak axis bending.
- The moment magnification is larger in the case of beam-columns bending about their weak axis.

- As the slenderness increases, the failure curves in the P/P_n, y-y axis plane change from convex to concave, showing increasing dominance of minor axis buckling.
- The failure of short/stocky members is either due to section strength being reached at the ends (under small axial load) or at the section of larger magnified moment (under large axial load).

9.4 Interaction Equations for Local Capacity Check

The interaction equations discussed till now are for overall buckling check. The beam-column should also be checked for local capacity at the point of the greatest bending moment and axial load. This is usually checked at the ends of the member, but it could be within the length of the beam-column, if lateral loads are also applied. The capacity in these cases is controlled by yielding or local buckling (if it is not prevented by limiting the width-to-thickness ratios specified in the codes). The linear interaction equation for semi-compact and slender cross section is given by

$$(P/P_y) + (M_z/M_{pz}) + (M_y/M_{py}) \leq 1 \tag{9.6}$$

where P is the applied axial load, P_y is the yield load = $A_g f_y$, M_z is the applied moment about the major axis z-z, and M_{pz} is the moment capacity about the major axis z-z in the absence of the axial load,

$$= M_{pz} \text{ if } P/P_y \leq 0.15 \text{ and}$$

$$= 1.18M_{pz}(1 - P/P_y) \quad \text{for } 0.15 \leq P/P_y < 1.0$$

M_y is the applied moment about the minor axis y-y and M_{py} is the moment capacity about the minor axis y-y in the absence of the axial load

$$= 1.19M_{py}[1 - (P/P_y)^2] \leq 1.0.$$

More accurate interaction equations are available for compact cross sections, which are based on the convex failure surface discussed in the previous section, which result in greater economy in design. Chen and Atsuta (1977) and Tebedge and Chen (1974) provide the following non-linear interaction equation for compact I-shapes in which the flange width is not less than 0.5 times the depth of the section

$$(M_z/M_{pz})^{\alpha} + (M_y/M_{py})^{\alpha} \leq 1 \tag{9.7a}$$

in which M_{pz}, M_{py}, M_z, and M_y are as defined earlier.
The value of the exponent is given by

$$\alpha = 1.6 - [(P/P_y)/\{2\ln(P/P_y)\}] \quad \text{for } 0.5 \leq b_f/d \leq 1.0 \tag{9.7b}$$

where ln is the natural logarithm, b_f is flange width (in mm), and d is the member depth (in mm).

A comprehensive assessment of the accuracy of the non-linear interaction equations in predicting the load carrying capacities of biaxially loaded I-sections has been made by Pillai (1981), who found that these equations predict the capacity reasonably well compared to the experimental results.

Interaction equations for a number of sections, including circular tubes, box sections, and unsymmetrical sections such as angles are available in Chen and Lui (1971), Chen and Atsuta (1977), and Shanmugam et al. (1993).

9.5 Code Design Procedures

Modern structural design specifications around the world have retained the generic form of the interaction formula given in Eqn (9.4). In every specification the moment M is always specified as the second-order (amplified) moment obtained either from a second-order structural analysis, where equilibrium is formulated in the deformed configuration of the structure, or from an approximation of using the moments from a first-order elastic analysis, which is then multiplied by an amplification factor. Depending upon whether P_{cr}, the elastic critical load, is evaluated for the member length or the storey effective length, the amplification factor accounts empirically for the member or the frame stability (Galambos 1998). This versatility of the interaction equations approach makes it very useful in design.

Most limit-states design codes use a set of load and resistance factors that are based on probabilistic principles (Bjorhovde et al. 1978).

9.5.1 Indian Code (IS 800 2007) Provisions

The Indian code (IS 800 2007) provisions are based on the Eurocode provisions and the code requires the following two checks to be performed

(a) Local capacity check and

(b) Overall buckling check

Local capacity check For beam-columns subjected to combined axial force (tension or compression) and bending moment, the following interaction equation should be satisfied.

$$(M_y/M_{\mathrm{ndy}})^{\alpha 1} + (M_z/M_{\mathrm{ndz}})^{\alpha 2} \leq 1.0 \tag{9.8}$$

where M_y and M_z are the factored applied moments about the minor and major axis of the cross section, respectively and M_{ndy} and M_{ndz} are the design reduced flexural strength under combined axial force and the respective uniaxial moment acting alone. The approximate value of M_{ndy} and M_{ndz} for plastic and compact I-section is given in Table 9.1.

The constants α_1 and α_2 are taken as 5n and 2 respectively for I or channel sections.

For semi-compact sections, without bolt holes, the code (IS 800 : 2007) suggests the following linear equation, when shear force is low

$$(N/N_d) + (M_y/M_{\mathrm{dy}}) + (M_z/M_{\mathrm{dz}}) \leq 1.0 \tag{9.9}$$

Table 9.1 Approximate value of reduced flexural strength for plastic and compact sections

Element	Equation
Welded I- or H-sections	$M_{ndz} = M_{dz}(1-n)/(1-0.5a) \le M_{dz}$
	$M_{ndy} = M_{dy}\left[1-\left(\dfrac{n-a}{1-a}\right)^2\right] \le M_{dy}$
	$a = (A - 2\,bt_f)/A \le 0.5$
Rolled I- or H-sections without bolt holes	$M_{ndz} = 1.11 M_{dz}(1-n) \le M_{dz}$
	$M_{ndy} = M_{dy}$ for $n \le 0.2$
	$M_{ndy} = 1.56\,M_{dy}(1-n)(n+0.6)$

$n = N/N_d$

N = Factored applied axial force (Tension T or compression P)

N_d = Design strength in tension T_d, or compression P(see Chapter 5)

$= A_g f_y/\gamma_{m0}$ where $\gamma_{m0} = 1.1$

M_{dy}, M_{dz} = Design strength under corresponding moment acting alone (see Chapter 6)

A_g = Gross area of cross section

Overall buckling check The interaction equation for overall buckling check is given by the code as

$$(P/P_{dy}) + (K_y C_{my} M_y/M_{dy}) + (K_{LT} M_z/M_{dz}) \le 1.0 \tag{9.10a}$$

$$(P/P_{dz}) + (0.6 K_y C_{my} M_y/M_{dy}) + (K_z C_{mz} M_z/M_{dz}) \le 1.0 \tag{9.10b}$$

where

C_{my}, C_{mz} = Equivalent uniform moment factor obtained from Table 9.2, which depends on the shape of the bending moment diagram between lateral bracing points in the appropriate plane of bending

P = Factored applied axial compressive load

P_{dy}, P_{dz} = Design compressive strength under axial compression as governed by buckling about minor and major axis respectively (see Chapter 5, Section 5.6.1)

M_y, M_z = Maximum factored applied bending moments about minor and major axis of the member, respectively

M_{dy}, M_{dz} = Design bending strength about minor and major axis considering laterally unsupported length of the cross-section (see Chapter 6, Section 6.7).

K_y, K_z, K_{LT} = Moment amplification factors as defined below

$$K_y = 1 + (\lambda_y - 0.2) n_y \le 1 + 0.8 n_y \tag{9.10c}$$

$$K_z = 1 + (\lambda_z - 0.2) n_z \le 1 + 0.8 n_z \tag{9.10d}$$

$$K_{LT} = 1 - \frac{0.1\lambda_{LT} n_y}{(C_{mLT} - 0.25)} \ge 1 - \frac{0.1 n_y}{(C_{mLT} - 0.25)} \tag{9.10e}$$

where

n_y, n_z = Ratio of actual applied axial-force to the design axial strength for buckling about minor and major axis respectively = (P/P_{dy}) or (P/P_{dz})

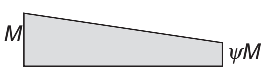
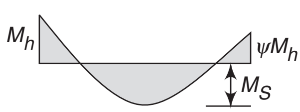
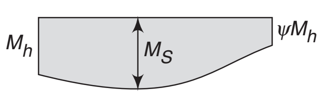
C_{mLT} = Equivalent uniform moment factor for lateral-torsional buckling as per Table 9.2, which depends on the shape of the bending moment diagram between lateral bracing points

λ_y, λ_z = Non-dimensional slenderness ratio about the minor and major axis respectively, for example, $\lambda_y = (f_y/f_{cr})^{0.5}$, where $f_{cr} = \pi^2 E/(KL/r)^2$ (see Section 5.6.1)

λ_{LT} = Non-dimensional slenderness ratio in lateral buckling = $(f_y/f_{cr,b})^{0.5}$ and $f_{cr,b}$ is the extreme fibre bending compressive stress corresponding to elastic lateral buckling moment which may be determined as per Table 14 of the code.

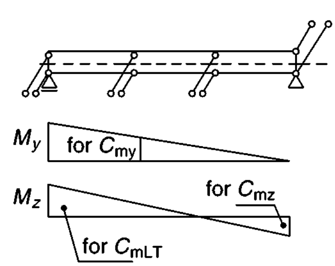
The above Indian code provisions are based on the Eurocode 3 provisions.

Table 9.2 Equivalent uniform moment factor (Greiner & Lindner 2006)

Bending Moment Diagram	Range		C_{my}, C_{mz}, C_{mLT} Uniform Loading	Concentrated Load
		$-1 \le \Psi \le 1$	$0.6 + 0.4\Psi \ge 0.4$	
	$0 \le \alpha_s \le 1$	$-1 \le \Psi \le 1$	$0.2 + 0.8\alpha_s \ge 0.4$	$0.2 + 0.8\alpha_s \ge 0.4$
	$-1 \le \alpha_s \le 0$	$0 \le \Psi \le 1$	$0.1 - 0.8\alpha_s \ge 0.4$	$-0.8\alpha_s \ge 0.4$
$\alpha_s = M_s/M_h$		$-1 \le \Psi \le 0$	$0.1(1 - \Psi) - 0.8\alpha_s \ge 0.4$	$-0.2\ \Psi\ 0.8\alpha_s \ge 0.4$
	$0 \le \alpha_h \le 1$	$-1 \le \Psi \le 1$	$0.95 + 0.05\alpha_h$	$0.90 + 0.10\alpha_h$
	$-1 \le \alpha_h \le 0$	$0 \le \Psi \le 1$	$0.95 + 0.05\alpha_h$	$0.90 + 0.10\alpha_h$
$\alpha_h = M_h/M_s$		$-1 \le \Psi \le 0$	$0.95 + 0.05\alpha_h (1 + 2\Psi)$	$0.90 - 0.10\alpha_h (1 + 2\Psi)$

For members with sway buckling mode the equivalent uniform moment factor $C_{my} = C_{mz} = 0.90$

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



(Contd)

(*Contd*)

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*

9.6 Design of Beam-columns

The design of beam columns involves a trail-and-error procedure. A trail section is selected by some process and is then checked with the appropriate interaction formula. If the section does not satisfy the equation (LHS > 1.0) or if it is too much on the safer side, indicated by LHS much less than 1.0 (that is, if it is over designed), a different section is selected and the calculations are repeated till a satisfactory section is found. Thus, the different steps involved in the design of beam columns are as follows.

1. Determine the factored loads and moments acting on the beam-column using a first-order elastic analysis (though a second-order analysis is recommended by most of the codes)
2. Choose an initial section and calculate the necessary section properties.
3. Classify the cross section (plastic, compact, or semi-compact) as per clause 3.7 of the code.
4. Find out the bending strength of the cross section about the major and minor axis of the member (clause 8.2.1.2).
5. (a) Determine the shear resistance of the cross section (clause 8.4.1). When the design shear force exceeds $0.6V_d$, then the design bending strength must be reduced as given in clause 9.2.2 of the code.
   (b) Check whether shear buckling has to be taken into account (clause 8.4.2).
6. Calculate the reduced plastic flexural strength (clause 9.3.1.2), if the section is plastic or compact.
7. Check the interaction equation for cross-section resistance for biaxial bending (clause 9.3.1.1 for plastic and compact section and clause 9.3.1.3 for semi-compact section). If not satisfied go to step 2.
8. Calculate the design compressive strength P_{dz} and P_{dy} (clause 7.1.2) due to axial force.
9. Calculate the design bending strength governed by lateral-torsional buckling (clause 8.2.2).
10. Calculate the moment amplification factors (clause 9.3.2.2).
11. Check with the interaction equation for buckling resistance (clause 9.3.2.2). If the interaction equation is not satisfied (LHS > 1.0) or when it is over design (LHS $\ll$ 1.0), go to step 2.

9.6.1 Selection of Initial Section

A common method used for selecting sections to resist both moments and axial loads is the *equivalent axial load or effective axial load method.* In this method, the axial load P and the bending moments M_z and/or M_y are replaced with fictitious concentric loads P_{eff}, equivalent to the actual design axial load plus the design moment. This fictitious load is called the equivalent axial load or the effective axial load.

Yura (1988) suggested a simpler approach for the initial sizing and suggested the following equation.

$$P_{eff} = P + 2M_z/d + 7.5M_y/b \tag{9.11}$$

where d and b are the depth and breadth of the selected beam-column.

The equivalent axial load equations were found to yield sections that are generally on the conservative side. The equivalent axial load approaches given by Eqn (9.11) are useful in preliminary sizing of beam-columns under gravity load combinations. However, when bending moment is predominant, those equations may result in considerable error. In this case, the equivalent moment approach, rather than equivalent axial force approach is preferable. Yura (1988), proposed an equation for estimating the equivalent moment as

$$M_{eq} = M_z + P_u d/2 \tag{9.12}$$

where d is the depth of the section. This equation is applicable for initial beam-column sizing in unbraced frames under lateral-load combinations.

9.7 Beam-columns Subjected to Tension and Bending

Bending moments may occur in tension members due to connection eccentricity, self weight of the member or transverse loads such as wind acting along the length of the member. The effect of tension load will always reduce the primary bending moment. Hence, secondary bending effects can be conservatively ignored in the design of members subjected to an axial tensile force and bending.

Local capacity check The following simplified interaction equation is specified in the code for the beam-column subjected to combined axial force and bending moment

$$(N/N_d) + (M_y/M_{dy}) + (M_z/M_{dz}) \le 1.0 \tag{9.13}$$

where M_{dy} and M_{dz} are the design reduced flexural strength under combined axial force and the respective uniaxial moment acting alone, M_y and M_z are the factored applied moments along minor and major axis of the cross section, respectively, N_d is the design strength in tension obtained from Section 6 of the code, and N is the factored applied axial tensile force.

Overall buckling check The code stipulates that the member should be checked for lateral-torsional buckling under reduced effective moment M_{eff} due to tension and bending. The reduced effective moment is given by the code as

$$M_{eff} = [M - \psi\, TZ_{ec}/A] \le M_d \tag{9.14}$$

where M and T are the factored applied moment and tension, respectively, A is the area of cross section, Z_{ec} is the elastic section modulus of the section with respect to extreme compression fibre, $\psi = 0.8$, if T and M vary independently and 1.0 otherwise, and M_d is the bending strength due to lateral-torsional buckling.

9.8 Design of Eccentrically Loaded Base Plates

The design of base plate subjected to concentric compression was considered in Section 5.13. In this section column bases that transmit forces, and moments from the steel column to the concrete foundation are considered. The forces may be axial loads, shear forces, and moment about either axis or any combinations of them. The main function of the base plate is to distribute the loads to the weaker material.

The common design deals with axial load and moment about major axis. With respect to slab and gusseted base (see Fig. 5.21), there may be two separate cases (a) compression over the whole base or compression over part of the base and tension in the *holding-down bolts* (also called *anchor bolts*). The relative values of moment and axial load determine which case will occur in a given instance. Horizontal loads are resisted by shear in the weld between column and base plates, friction and bond between the base plate and concrete, and shear in the holding down bolts. As mentioned in Chapter 5, ANSI/AISC code does not allow the anchor rods to transfer substantial shear, and suggests the use of a shear key or lug to transfer a large horizontal force from the column to the foundation (see Fig. 5.22). Note that the horizontal loads are generally small except when earthquake loads are considered.

(a) (b) (c)

Base plate connections: (a) welded slab base plate, (b) gusseted base plate,(c) moment resisting base plate © American Institute of Steel Construction, Inc., Reprinted with permission. All right reserved.)

9.8.1 Compression Over the Whole Base Plate

A column base and loading are shown in Fig. 9.8. If the area of moment/axial load is less than $L/6$, where L is the base length, then a positive pressure exists over the whole base and may be calculated from equilibrium alone. In this case, nominal holding down bolts (2 to 4) are provided to locate the base plate accurately.

Eccentricity of the load $e = M/P$

where P is the total load on the base plate and M_z is the major axis moment on the base. The area of the base is given by

$$A = BL$$

where B is the breadth of the base plate, and L is the length of the base plate. The maximum pressure on the concrete foundation is

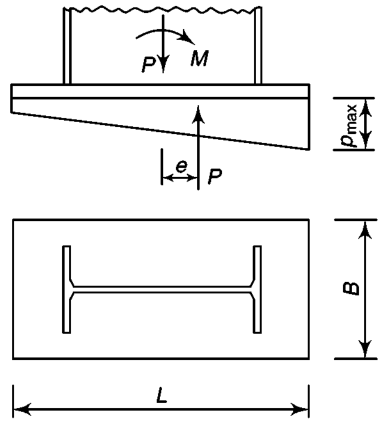


Fig. 9.8 Compression over the whole base plate

$$p_{max} = (P/A) + (M_z/Z_z) \tag{9.15}$$

where Z_z is the modulus of the base plate about Z-axis = $BL^2/6$.

When there are bi-axial moments, then

$$p_{max} = (P/A) + (M_z/Z_z) + (M_y/Z_y) \tag{9.16}$$

where M_y is the minor axis moment on the plate and Z_y is the modulus about the y-y axis = $LB^2/6$.

The maximum pressure must not exceed the bearing strength of the concrete, which is taken as $0.45f_{ck}$, where f_{ck} is the characteristic compressive strength of concrete.

In this case, the size of the base plate is established by successive trials. If the length is fixed, the breadth may be determined so that the bearing strength of the concrete is not exceeded. The weld size between base plate and the column is determined using the same requirements that were set out for the axially loaded base plate in Section 5.13. Example 9.3 shows the calculations required for a base plate with compression over the whole of the base plate.

9.8.2 Gusseted Base Plate

When gussets are provided as shown in Fig. 9.9 they support the base plate against bending and hence the thickness of the base plate could be reduced. Now, a part of the load is transmitted from the column through the gussets to the base plate.

The gussets are subjected to bending from the upward pressure under the base as shown in Fig. 9.9. The top edge of the gusset plate will be in compression and hence must be checked for buckling. To prevent this, we should choose the dimensions of outstand in such a way that they are within the limiting width-to-thickness ratios of welded sections, as given in Table 2 of the code. Thus, for the configuration shown in Fig. 9.9, the limiting width-to-thickness ratios for the gusset plate is given by

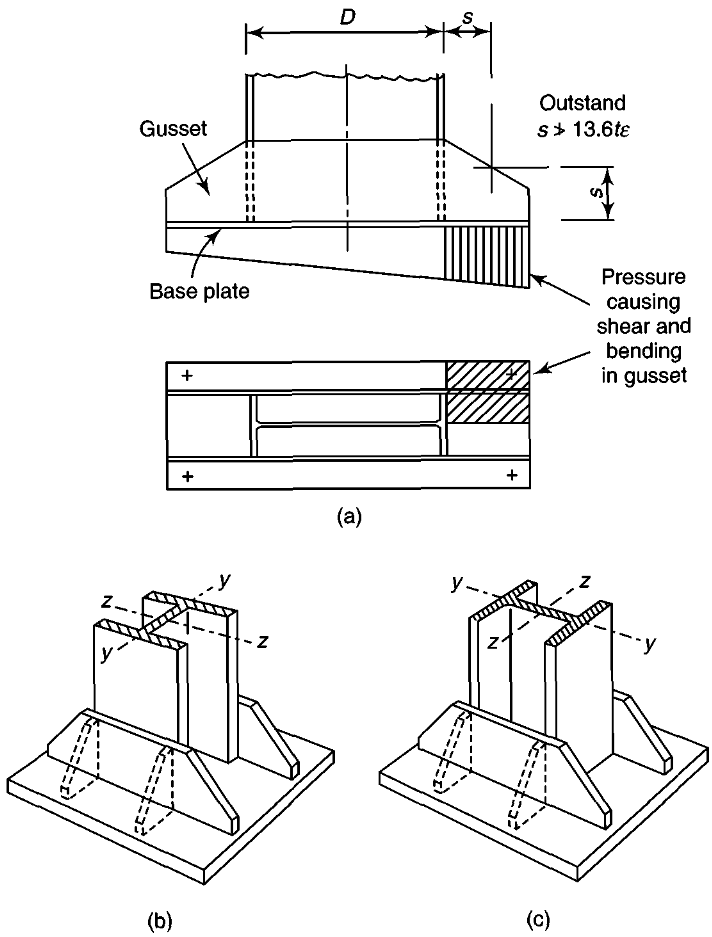


Fig. 9.9 Gusseted base plates

(a) For the portion of gusset plate welded to the flanges of the column

$$D \leq 29.3\varepsilon t \tag{9.17}$$

(b) Outstand of the gusset plate from the column or base plate

$$S \leq 13.6\ \varepsilon\, t \tag{9.18}$$

where $\varepsilon = (250/f_{yg})^{0.5}$, t is the thickness of the gusset plate, and f_{yg} is the design yield strength of the gusset plate.

The gusset plates are designed to resist shear and bending. The moment in the gusset should not exceed $f_{yg}Z_e/\gamma_{m0}$, where Z_e is the elastic modulus of the gusset and γ_{m0} is the partial safety factor for material = 1.1. An example of a gusseted base plate is provided in Example 9.4.

9.8.3 Anchor Bolts and Shear Connectors

The specification for foundation bolts is given in IS 5624-1993. Some typical shank forms of black foundation bolts as per this code are given in Fig. 9.10. The types of anchor bolts include cast-in-place anchors and post-installed anchors. Cast-in-place anchors, placed before concrete is cast, include a bolt with a curved end (used when there is no or low uplift forces) and a headed anchor. The tensile load transfer is by bonding between anchor bolt and concrete filling. Bonding between filling and concrete wall is guaranteed whenever the contact surface is sufficiently rough.

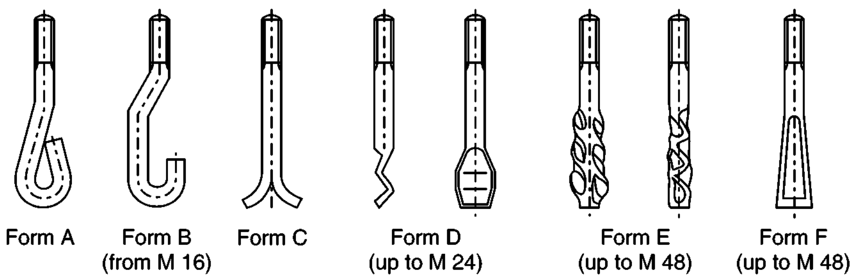


Fig. 9.10 Typical shank forms of foundation bolts

The headed anchors transfer tensile load by mechanical bearing of the head, nut and possibly by bond between anchor shank and the surrounding concrete. As these pre-installed anchors do not allow any clearance, they need very accurate positioning.

Post-installed anchors, installed in hardened concrete, are classified according to their load-transfer mechanisms. *Adhesive or bonded anchors* transfer load through cementitious grout or chemical adhesive. Details of installation and behaviour of these types of anchors are provided by Subramanian and Cook (2002, 2004). Epoxy is the most widely used adhesive though resins such as vinyl esters, polyesters, methacrylates, and acrylics have also been used. *Mechanical anchors* transfer load by friction or bearing and include expansion anchors and undercut anchors.

Fuchs et al. (1995) suggested the concrete capacity design (CCD) approach to fastenings in concrete, which has been adopted by the ACI code (Appendix D of ACI 318-2005). According to this method the tensile capacity of anchors may be determined using the following formula (Subramanian 2000)

$$N_u = k\sqrt{f_{\text{ck}}}\,(h_{\text{ef}})^{1.5} \tag{9.19}$$

where k = 13.5 for post-installed anchors and 15.5 for cast-in-situ headed studs and headed anchor bolts and h_{ef} is the embedment length.

When fastenings are located so close to an edge or to an adjacent anchor, there will not be enough space for complete concrete cone to develop and hence the load-bearing capacity of the anchor has to be reduced [see Subramanian (2000) and Cook (1999) for more details of the CCD method and worked examples].

For grouted anchors bond failure at the grout-concrete interface may occur. This failure mode is best represented by a uniform bond stress model. The strength in this case is given by

$$N_{\text{bond}} = \tau_o\, \pi\, d_o h_{\text{ef}} \tag{9.20}$$

where τ_o is the grout-concrete bond strength of the product (in N/mm^2), d_o is the diameter of the hole (in mm), and h_{ef} is the embedment length.

The predicted mean strength of a headed grouted anchor is determined by the lower value of Eqn (9.19), Eqn (9.20), and the steel failure strength. Swiatek and Whitback (2004) discuss many practical problems connected with anchor rods and provide some easy solutions.

Shear connectors When the shear force to be transmitted to the foundation from the column is high, a shear lug (a short length of a rolled I-section or a plate) is welded perpendicular to the bottom of the base plate (see Fig. 9.11). Failure occurs when a wedge of concrete shears off. The design approaches involve treating the failure as a bearing problem. Bearing is assumed to be uniformly distributed through the height equal to *H-G* where *G* is the depth of grout (see Fig. 9.11). The plate may be sized for bearing and bending as a cantilever beam. The web of the connector must be able to support the shear force in the column.

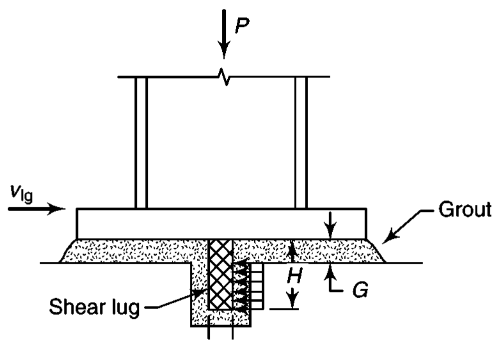


Fig. 9.11 Shear lug

The different steps involved in the design are as follows (DeWolf and Ricker 2006).

1. Determine the portion of the shear which is resisted by friction equal to μ multiplied by the factored dead load and appropriate portion of the live load, which generates the shear force. The shear force to be resisted by the lug is the difference between the factored shear force and this force. μ may be taken as 0.3 (steel friction coefficient, since steel shims are often placed under the base plate).
2. The required bearing area for the shear lug is
$$A_{lg} = V_{lg}/0.45 f_{ck} \tag{9.21}$$
3. Determine the shear lug dimensions, assuming that bearing occurs on the portion of the lug below the concrete foundation.
4. The factored cantilever end moment M_{lg} acting on a unit length of the shear lug is
$$M_{lg} = (V_{lg}/W)(H + G/2) \tag{9.22}$$
where *W* is the total horizontal width of the lug, *H* is the vertical height of the lug, and *G* is the thickness of grout (see Fig. 9.11)

5. The shear lug thickness is determined as follows:

$$t_{lg} = \sqrt{[4M_{lg}/(f_y/\gamma_{m0})]} \leq t_b \tag{9.23}$$

where t_b is the thickness of the base plate. Example 9.5 illustrates the use of these steps.

DeWolf and Ricker (2006) provide more details about the design of base plates. Drake and Elkin (1999) present a step-by-step methodology for the design of base plates and anchor rods using factored loads and rectangular pressure distribution.

Examples

Example 9.1 *A non-sway column in a building frame with flexible joints is 4-m high and subjected to the following load and moment:*

Factored axial load = 500 kN

Factored moment M_z:

at top of column = 27.0 kNm

at bottom of column = 45.0 kNm

Design a suitable beam-column assuming f_y = 250 N/mm². Take the effective length of the column as 0.8L along both the axes.

Solution

1. Trial section

Select ISHB 200 with A = 4750 mm², r_y = 45.1 mm, H = 200 mm, b = 200 mm, and b_f = 9.0 mm.

$$KL = 0.8 \times 4000 = 3200 \text{ mm}$$

$$KL/r_y = 3200/45.1 = 70.95$$

$$P_e = P + 2M_z/d = 500 + 2 \times 45000/200 = 950 \text{ kN}$$

From Table 10 of the code for $h/b_f < 1.2$ and $t_f < 40$ mm, using curve 'c' for minor axis buckling, we have

For KL/r_y = 70.95 and f_y = 250 MPa, from Table 9c of the code, f_{cd} = 150 N/mm²

Hence capacity = 150 × 4750/1000 = 712.5 kN < 950 kN

Hence use ISHB 250 with A = 6500 mm², H = 250 mm, and r_y = 54.9 mm.

$$KL/r_y = 3200/54.9 = 58.29$$

$$P_{eq} = 500 + 2 \times 45{,}000/250 = 860 \text{ kN}$$

For KL/r_y = 58.29 and f_y = 250 MPa,

From Table 9c of the code, f_{cd} = 170.56 N/mm²

Thus capacity = 170.56 × 6500/1000 = 1108 kN > 860 kN

Hence, choose ISHB 250 as the trial section.

Section Properties

ISHB 250 has the following cross-sectional properties:

H = 250 mm	A = 6500 mm²	$I_z = 7740 \times 10^4$ mm⁴
b_f = 250 mm	r_z = 109 mm	$I_y = 1960 \times 10^4$ mm⁴

$t_f = 9.7$ mm $\quad$ $r_y = 54.9$ mm

$t_w = 6.9$ mm $\quad$ $Z_z = 619 \times 10^3$ mm^3

$R = 10$ mm $\quad$ $Z_y = 156 \times 10^3$ mm^3

$$Z_{pz} = 2b_f t_f (H - t_f)/2 + t_w(H - 2t_f)^2/4$$
$$= 2 \times 250 \times 9.7\ (250 - 9.7)/2 + 6.9(250 - 2 \times 9.7)^2/4$$
$$= 674.46 \times 10^3 \text{ mm}^3$$

2. Cross-section classification

$$\varepsilon = \sqrt{(250/f_y)} = \sqrt{250/250} = 1.0$$

Outstand flanges (Table 2 of the code)

$$b/t_f = (250/2)/9.7 = 12.88 < 15.7\varepsilon$$

Hence, the flange is semi-compact.

Web

$$d = H - 2t_f - 2R = 250 - 2 \times 9.7 - 2 \times 10 = 210.6 \text{ mm}$$
$$d/t_w = 210.6/6.9 = 30.5 < 42\varepsilon$$

Hence, the cross-section is semi-compact.

3. Check for resistance of cross section to the combined effects (clause 9.3.1.3)

The interaction equation is

$$(N/N_d) + (M_z/M_{dz}) \leq 1.0$$
$$N_d = A_g f_y/\gamma_{m0} = 6500 \times 250/(1.1 \times 1000) = 1477.27 \text{ kN}$$
$$M_{dz} = \beta_b Z_p f_y/\gamma_{m0}$$

where $\beta_b = Z_e/Z_p$ for a semi-compact section. Hence,

$$M_{dz} = Z_e f_y/\gamma_{m0} = 619 \times 10^3 \times 250/(1.1 \times 10^6)$$
$$= 140.68 \text{ kN m}$$

Thus,

$$(500/1477.27) + (45/140.68) = 0.338 + 0.320 = 0.658 < 1.0$$

Hence, the section is safe.

4. Member buckling resistance in compression (clause 7.1.2)

$$\text{Effective length} = 0.8L = 0.8 \times 4000 = 3200 \text{ mm}$$
$$KL_z/r_z = 3200/109 = 29.35$$
$$KL_y/r_y = 3200/54.9 = 58.29$$

From Table 10 of the code,

$$h/b = 250/250 = 1.0 \text{ and } t_f < 40 \text{ mm}$$

Major axis buckling, use curve *b*

Minor axis buckling, use curve *c*

$$f_{cr,z} = \pi^2 \times 2 \times 10^5/(29.35)^2 = 2291.5 \text{ N/mm}^2$$
$$\lambda_z = \sqrt{(250/2291.5)} = 0.33$$

From Table 9c of the code, for $KL/r = 58.29$ and $f_y = 250$ N/mm^2,

$$f_{cd} = 170.56 \text{ N/mm}^2 \text{ and}$$
$$P_{d,y} = 170.56 \times 6500/1000 = 1108 \text{ kN} > 500 \text{ kN}.$$

From Table 9b of the code, for $KL/r = 29.35$ and $f_y = 250$ N/mm^2,

$$f_{cd} = 215.6 \text{ N/mm}^2,$$

$$P_{d,z} = 215.6 \times 6500/1000 = 1401 \text{ kN} > 500 \text{ kN}.$$

Hence, the section is safe.

5. Member buckling resistance in bending (clause 8.2.2)

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

$$\beta_b = Z_e/Z_p \text{ for semi-compact section} = 619/674.76 = 0.918$$

Hence $M_d = Z_e f_{bd}$

From Table 42 of the code (assuming $k = 1$)

For $\psi = 0.75$, $C_1 = 1.141$ and for $\psi = 0.5$, $C_1 = 1.323$. Hence, for $\psi = 0.6$, $C_1 = 1.25$,

Note More refined value of $C_1 = 1.345$ may be obtained by considering $k = 0.8$ in this table.

$L_y = 4$ m, $h/t_f = 250/9.7 = 25.77$

Determination of M_{cr}

$$f_{cr,b} = C_1 \, [1473.5/(KL/r_y)]^2 \, \{1 + (1/20)[(KL/r_y)/(h/t_f)]^2\}^{0.5}$$

$$= 1.25(1473.5/58.29)^2 \, [1 + (1/20)(58.29/25.77)^2]^{\,0.5}$$

$$= 895.1 \text{ N/mm}^2$$

$$M_{cr} = 895.1 \times 619 \times 10^3/10^6 = 554 \text{ kNm}$$

Non-dimensional lateral-torsional slenderness ratio

$$\lambda_{LT} = \sqrt{(\beta_b Z_{pz} f_y / M_{cr})}$$

$$= \sqrt{0.918 \times 674.46 \times 10^3 \times 250/(554 \times 10^6)} = 0.527$$

$$\alpha_{LT} = 0.21 \text{ for rolled sections}$$

Reduction factor for lateral torsional buckling

$$\chi_{LT} = 1/[\phi_{LT} + (\phi_{LT}^2 - \lambda_{LT}^2)^{0.5}]$$

where $\phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2]$

$$= 0.5[1 + 0.21(0.527 - 0.2) + 0.527^2]$$

$$= 0.6732$$

Thus $\chi_{LT} = 1/[0.6732 + (0.6732^2 - 0.527^2)^{0.5}]$

$$= 0.9157$$

Lateral torsional buckling resistance

$$= \chi_{LT} \, (f_y/\gamma_{m0}) Z_e = 0.9157 \times (250/1.1) \times 619 \times 10^3/10^6$$

$$= 128.82 \text{ kNm} > 45 \text{ kNm}$$

Hence, the section is safe.

6. Member buckling resistance in combined bending and axial compression

Determination of moment amplification factors

$$K_z = 1 + (\lambda_z - 0.2)P/P_{dz} \le 1 + 0.8P/P_{dz}$$

$$K_z = 1 + (0.33 - 0.2)500/1401$$

$$= 1.0463 < 1 + 0.8 \times 500/1401 = 1.285$$

$$\psi_z = M_2/M_1 = 27/45 = 0.6,$$
$$C_{mz} = 0.6 + 0.4\psi$$
$$= 0.6 + 0.4 \times 0.6 = 0.84 > 0.4$$

Check with interaction formula (Clause 9.3.2.2)

$$P/P_d + [(K_z C_{mz} M_z)/M_{dz}] < 1$$

Thus, $(500/1401) + (1.0463 \times 0.84 \times 45)/128.82 = 0.357 + 0.307 = 0.664 < 1.0$

Hence section is safe against combined axial force and bending moment.

Example 9.2 *An I-section beam-column of length 4 m has to be designed as a ground floor column in a multi-storey building. The frame is moment-resisting in-plane and pinned out-of plane, with diagonal bracing provided in both directions. The column is subjected to major axis bending due to horizontal forces and minor axis bending due to eccentricity of loading from the floor beams. The design action effects for this column from a linear analysis program are as follows (see also Fig. 9.12).*

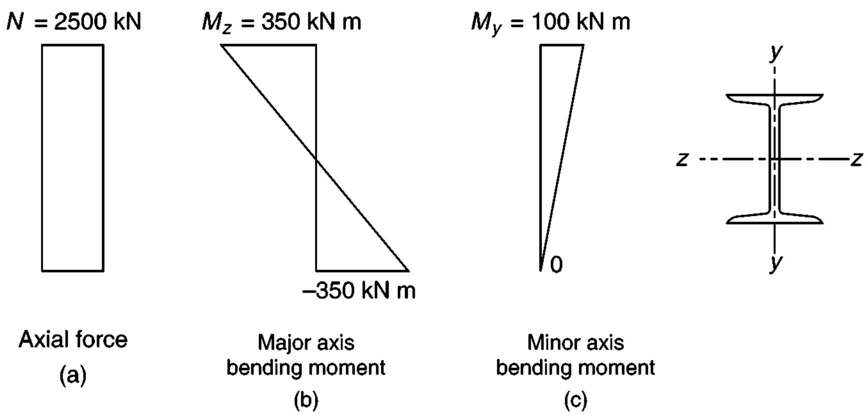


Fig. 9.12 Design action effects on the beam-column

$$N = 2500\ kN$$

At the base of column: $M_z = -350$ *kNm,* $M_y = 0$

At the top of column: $M_z = 350$ *kNm,* $M_y = 100$ *kNm*

Determine whether a hot-rolled wide flange section W 310 × 310 × 226 will be suitable to resist the design action effects. Use Fe 410 grade steel.

Solution

1. Section properties

The section properties of W 310 × 310 × 226 are as follows:

$b_f = 317$ mm

$H = 348$ mm

$t_f = 35.6$ mm $\qquad I_z = 59560 \times 10^4$ mm^4

$t_w = 22.1$ mm $\qquad I_y = 18930 \times 10^4$ mm^4

$R = 15.0$ mm $\qquad r_z = 143.6$ mm

$A = 28{,}880$ mm^2 $\qquad r_y = 81.0$ mm

$Z_{ez} = 3423 \times 10^3$ mm^3 $\qquad Z_{pz} = 3948{,}812$ mm^3

$Z_{ey} = 1194 \times 10^3$ mm^3 $\qquad Z_{py} = 1822{,}502$ mm^3

The calculations for I_t, I_w, Z_{pz}, and Z_{py} are as follows:

$$Z_{pz} = 2b_f t_f(H - t_f)/2 + t_w(\mathrm{H} - 2t_f)^2/4$$
$$= 2 \times 317 \times 35.6\ (348 - 35.6)/2 + 22.1 \times (348 - 2 \times 35.6)^2/4$$
$$= 3948{,}812 \text{ mm}^3$$
$$Z_{py} = 2 \times t_f b_f^2/4 + (\mathrm{H} - 2t_f)t_w^2/4$$
$$= 2 \times 35.6 \times 317^2/4 + (348 - 2 \times 35.6)\ 22.1^2/4$$
$$= 1{,}822{,}502 \text{ mm}^3$$
$$E = 2 \times 10^5 \text{ N/mm}^2$$
$$G = 76923 \text{ N/mm}^2$$

2. Cross-section classification (clause 3.7)

$$\varepsilon = \sqrt{(250/f_y)} = \sqrt{(250/250)} = 1$$

Outstand flanges plastic.

$$b/t_f = (317/2)/35.6 = 4.45$$

Limit for class 1 flange = 9.4 = 9.4 > 4.45

Hence flanges are class 1 (plastic)

Web

$$d = H - 2t_f - 2R = 348 - 2 \times 35.6 - 2 \times 15 = 246.8 \text{ mm}$$
$$d/t_w = 246.8/22.1 = 11.1 < 42\varepsilon = 42$$

Hence web is plastic.

The overall cross-section classification is plastic.

3. Compression resistance of the cross section

The design compression resistance of the cross section

$$N_d = A_g f_y/\gamma_{m0} = 28880 \times 250/(1.1 \times 1000) = 6563 \text{ kN} > 2500 \text{ kN}$$

Hence the design compression resistance is alright.

4. Bending resistance of the cross section (clause 8.2.1.2)

Major z-z axis

Maximum bending moment = 350 kN m

The design major axis bending resistance of the cross section

$$M_{dz} = \beta_b Z_p f_y/\gamma_{m0} = (1 \times 3{,}948{,}812 \times 250/1.1) \times 10^{-6}$$
$$= 897.45 \text{ kN m} > 350 \text{ kN m}$$

Minor y-y axis

Maximum bending moment = 100 kN m

The design minor axis bending resistance of the cross section

$$M_{dy} = (1 \times 1822{,}502 \times 250/1.1) \times 10^{-6} = 414.2 \text{ kN m} > 100 \text{ kN m}$$

Hence the bending resistance is fine along both major *z-z* axis and minor *y-y* axis.

5. Shear resistance of the cross-section (clause 8.4.1)
The design plastic shear resistance of the cross section

$$V_p = A_v(f_{yw}/\sqrt{3})/\gamma_{m0}$$

Load parallel to web
Maximum shear force $V = [350 - (-350)]/4.0 = 175$ kN
For a rolled section, loaded parallel to the web the shear are

$$A_v = Ht_w = 348 \times 22.1 = 7690.8 \text{ mm}^2$$

$$V_p = 7690.8 \times (250/\sqrt{3})/(1.1 \times 1000) = 1009.1 \text{ kN} > 175 \text{ kN}$$

Hence the shear resistance of the cross section is alright.
Load parallel to flanges
Maximum shear force $V = 100/4 = 25$ kN

$$A_v = 2b_f t_f = 2 \times 317 \times 35.6 = 22{,}570.4 \text{ mm}^2$$

$$V_p = 22570.4 \times (250/\sqrt{3})/(1.1 \times 1000) = 2961.5 \text{ kN} > 25 \text{ kN}$$

Hence the shear force is alright.
Shear buckling (clause 8.4.2)
Shear buckling need not be considered, provided

$$d/t_w < 67\varepsilon, \text{ for unstiffened webs}$$

$$d/t_w = 246.8/22.1 = 11.1 < 67 \ (\varepsilon = 1)$$

Hence no shear buckling check is required.
6. Cross-section resistance (clause 9.3.1)
Provided the shear force is less than 60% of the design plastic shear resistance and provided shear buckling is not a concern, the cross section needs only satisfy the requirements of bending and axial force (clause 9.2.1). In this case shear force is less than 60% of design plastic shear resistance and hence the cross section needs to be checked for bending and axial force only.
Reduced plastic moment resistances
Major z-z axis
For rolled I- or H-sections,

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

where $n = N/N_d = 2500/6563 = 0.381$

Hence $M_{ndz} = 1.11 \times 897.45\ (1 - 0.381)$
$= 616.6$ kNm > 350 kNm

Minor y-y axis
For $n > 0.2$, $M_{ndy} = 1.56 M_{ndy}(1 - n)(n + 0.6)$
$= 1.56 \times 414.2(1 - 0.381)(0.381 + 0.6)$
$= 392.4$ kNm > 100 kNm

Hence the moment resistances for major *z-z* axis and minor *y-y* axis are alright.
Cross-section check for biaxial bending (with reduced moment resistances)

$$(M_y/M_{ndy})^{\alpha_1} + (M_z/M_{ndz})^{\alpha_2} \le 1.0$$

For I- and H-sections

$$\alpha_1 = 5n \geq 1 \text{ and } \alpha_2 = 2$$
$$= 5 \times 0.381 = 1.905$$

Thus, $(100/392.4)^{1.905} + (350/616.6)^2 = 0.396 < 1.0$

7. Member buckling resistance in compression (clause 7.1.2)

$$N_d = A_e f_{cd}$$
$$f_{cd} = \{(f_y/\gamma_{m0})/(\phi + [\phi^2 - \lambda^2]^{0.5}\} \leq f_y/\gamma_{m0}$$

where $\phi = 0.5[1 + \alpha(\lambda - 0.2) + \lambda^2)]$

$$\lambda = \sqrt{(f_y/f_{cr})}$$
$$f_{cr} = \pi^2 E/(KL/r)^2$$

For buckling about the major z-z axis (Table 11 of the code)

$$KL_z = 0.65L = 0.65 \times 4 = 2.6 \text{ m}; KL_z/r_z = 2600/143.6 = 18.1$$

Note If the stiffness of the beams are known, the effective length should be calculated by using Wood's curves. For buckling about the minor y-y axis,

$$KL_y = 1.0L = 1 \times 4 = 4 \text{ m}; KL_y/r_y = 4000/81 = 49.38$$
$$f_{cr,z} = \pi^2 E/(KL_z/r_z)^2 = \pi^2 \times 2 \times 10^5/(18.1)^2 = 6025 \text{ N/mm}^2$$
$$\lambda_z = \sqrt{(250/6025)} = 0.2037$$
$$f_{cr,y} = \pi^2 \times 2 \times 10^5/(49.38)^2 = 810 \text{ N/mm}^2$$
$$\lambda_y = \sqrt{(250/810)} = 0.5557$$

Selection of buckling curve and imperfection factor α

For hot rolled H-section (with $h/b = 348/317 = 1.09 < 1.2$ and $t_f < 100$ mm):

- For buckling about the z-z axis, use curve b (Table 10 of the code)
- For buckling about the y-y axis, use curve c (Table 10 of the code)
- For curve b, $\alpha = 0.34$ and for curve c, $\alpha = 0.49$ (Table 7 of the code)

Buckling resistance

$$\phi_z = 0.5\ [1 + 0.34(0.2037 - 0.2) + 0.2037^2]$$
$$= 0.5213$$
$$\chi_y = 1/[0.5213 + \sqrt{(0.5213^2 - 0.2037^2)}\ ]$$
$$= 0.9988$$
$$P_{d,z} = 0.9988 \times (250/1.1) \times 28{,}880/1000 = 6556 \text{ kN} > 2500 \text{ kN}$$

Buckling resistance is, thus, fine.

Buckling resistance: Minor y-y axis

$$\phi_y = 0.5\ [1 + 0.49(0.5557 - 0.2) + 0.5557^2] = 0.7415$$
$$\chi_z = 1/[0.7415 + \sqrt{(0.7415^2 - 0.5557^2)}\ ] = 0.8113$$
$$P_{d,y} = 0.8113 \times (250/1.1) \times 28{,}880/1000 = 5325.4 \text{ kN} > 2500 \text{ kN}$$

Hence the buckling resistance about the minor axis is fine.

8. Member buckling resistance in bending (clause 8.2.2)

The 4-m long column is unsupported along its length with no-torsional or lateral restraint. Equal and opposite design end moments of 350 kNm are applied about the major axis. Hence the full length of the column must be checked for lateral torsional buckling.

$$M = 350 \text{ kNm}$$

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

where β_b = 1.0 for plastic and compact sections.

Determination of M_{cr} (L_y = 4000 mm)

$$M_{cr} = C_1[(\pi^2 EI_y h)/2(KL_y)^2] \{1 + (1/20) [(KL_y/r_y)/(h/t_f)]^2\}^{0.5}$$

For equal and opposite end moments ($\psi = -1$) and $K = 1.0$, $C_1 = 2.752$ from Table 42 of the code, $h/t_f = 348/35.6 = 9.78$.

$$M_{cr} = 2.752[(\pi^2 \times 2.0 \times 10^5 \times 18930 \times 10^4 \times 348)/(2 \times 4000^2)] \{1 + (1/20) [49.38/9.78]^2\}^{0.5}$$

$$= 16{,}866 \times 10^6 \text{ Nmm} = 16866 \text{ kNm}$$

Non-dimensional lateral-torsional slenderness ratio,

$$\lambda_{LT} = \sqrt{(\beta_b Z_{pz} f_y / M_{cr})}$$

$$= \sqrt{[1 \times 3948{,}812 \times 250/(16866 \times 10^6)]} = 0.2419$$

$$\alpha_{LT} = 0.21 \text{ for rolled section}$$

Reduction factor for lateral torsional buckling

$$\chi_{LT} = 1/[\phi_{LT} + (\phi_{LT}^2 - \lambda_{LT}^2)^{0.5}]$$

where $\phi_{LT} = 0.5 [1 + \alpha_{LT} (\lambda_{LT} - 0.2) + \lambda_{LT}^2]$

$$= 0.5 [1 + 0.21(0.2419 - 0.2) + 0.2419^2]$$

$$= 0.5337$$

Thus, $\chi_{LT} = 1/[0.5337 + (0.5337^2 - 0.2419^2)^{0.5}]$

$$= 0.9907$$

Lateral torsional buckling resistance

$$= \chi_{LT}(f_y/\gamma_{m0}) \beta_b Z_{pz}$$

$$= 0.9907 \times 250/1.1 \times 1 \times 3948{,}812$$

$$= 889.1 \times 10^6 \text{ Nmm} = 889.1 \text{ kNm}$$

$$M/M_d = 350/889.1 = 0.394 \leq 1.0$$

Hence o.k.

9. Member buckling resistance in combined bending and axial compression

Determination of moment amplification factors (clause 9.3.2.2.)

$$K_z = 1 + (\lambda_z - 0.2)P/P_{dz} \leq 1 + 0.8 P/P_{dz}$$

$$K_z = 1 + (0.2037 - 0.2)2500/6556 = 1.0015 < 1 + 0.8 \times 2500/6556 = 1.305$$

$$\psi_z = M_2/M_1 = -350/350 = -1,$$

$$C_{mz} = 0.6 + 0.4\psi = 0.6 + 0.4 \times (-1) = 0.2 < 0.4; \text{ Hence } C_{mz} = 0.4$$

$$K_y = 1 + (\lambda_y - 0.2)P/P_{dy} \leq 1 + 0.8P/P_{dy}$$

$$K_y = 1 + (0.5557 - 0.2)2500/5325.4 = 1.167 < 1 + 0.8 \times 2500/5325.4 = 1.375$$

$$\psi_y = M_2/M_1 = 0,$$

$$C_{my} = 0.6 + 0.4\psi = 0.6 + 0.4 \times (0) = 0.6 > 0.4$$

$$C_{mLT} = 0.4$$

$$n_y = P/P_{dy} = 2500/5325.4 = 0.4694;\ \lambda_{LT} = 0.2419$$

$$K_{LT} = 1 - \frac{0.1\lambda_{LT}n_y}{(C_{mLT} - 0.25)} \geq 1 - \frac{0.1n_y}{(C_{mLT} - 0.25)}$$

$$K_{LT} = 1 - 0.1 \times 0.2419 \times 0.4694/(0.4 - 0.25)$$
$$= 0.9243 \geq 1 - 0.1 \times 0.2419/(0.4 - 0.25) = 0.84$$

Check with interaction formula (clause 9.3.2.2)

$$(P/P_{dy}) + (K_yC_{my}M_y/M_{dy}) + (K_{LT}M_z/M_{dz}) \leq 1.0$$
$$(2500/5325.4) + (1.375 \times 0.6 \times 100)/414.2 + (0.9243 \times 350)/889.1$$
$$= 0.469 + 0.199 + 0.363 = 1.03 \approx 1.0$$
$$(P/P_{dz}) + (0.6K_yC_{my}M_y/M_{dy}) + (K_zC_{mz}M_z/M_{dz}) \leq 1.0$$
$$(2500/6556) + (0.6 \times 1.375 \times 0.6 \times 100)/414.2 + (1.305 \times 0.4 \times 350)/889.1$$
$$= 0.381 + 0.119 + 0.205 = 0.705 < 1.0$$

Hence the section is suitable to resist the design action effects.

Example 9.3 *Design the base plate for the column in Example 9.1 subjected to a factored moment of 45 kNm and a factored axial load of 500 kN. The column size is ISHB 250. The cube compressive strength of concrete in the foundation is f_{ck} = 25 N/mm^2. Use grade 410 steel.*

Solution

1. Size of the base plate

$$e = 45 \times 10^3/500 = 90 \text{ mm}$$

If the base plate is made $6e$ in length there will be compressive pressure over the whole of the base.

$$6e = 6 \times 90 = 540 \text{ mm}$$

The required breadth to limit the bearing pressure to $0.45f_{ck}$ (= 11.25 N/mm^2) is

$$B = 2P/(L \times 0.45f_{ck}) = (2 \times 500 \times 10^3)/(540 \times 0.45 \times 25) = 164.6 \text{ mm}$$

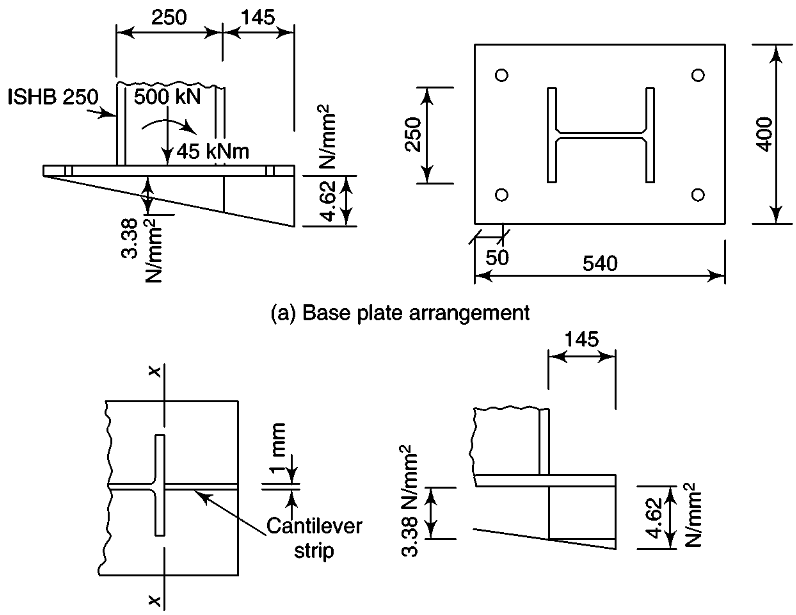
Provide a rectangular base plate of size 540 × 400 mm. The arrangement of the base plate is shown in Fig. 9.13.

$$\text{Area} = 540 \times 400 = 216 \times 10^3 \text{ mm}^2$$

$$\text{Modulus } Z = 400 \times 540^2/6 = 19.44 \times 10^6 \text{ mm}^3$$

Maximum pressure:

$$p_{max} = 500 \times 10^3/(216 \times 10^3) + 45 \times 10^6/(19.44 \times 10^6)$$
$$= 2.31 + 2.31$$
$$= 4.62 \text{ N/mm}^2$$
$$p_{min} = 2.31 - 2.31 = 0$$



(b) Base plate design for thickness

Fig. 9.13

2. Thickness of base plate

Consider a 1-mm wide strip as shown in Fig. 9.13(b). This acts as a cantilever from the face of the column with the loading caused by pressure on the base. This method gives a conservative design for the thickness of base plate, since the plate action due to bending in two directions at right angles is not considered.

$$\text{Base pressure at section } XX = [(540 - 145)/540] \times 4.62$$
$$= 3.38 \text{ N/mm}^2$$

For the trapezoidal pressure loading on the cantilever strip as shown in the figure, the moment at XX is calculated as follows

$$M_x = (3.38 \times 145^2/2) + (4.62 - 3.38) \times 145/2 \times 2/3 \times 145$$
$$= 44.22 \times 10^3 \text{ Nmm}$$

Moment capacity of plate = $1.2 f_y Z_e/\gamma_{m0}$

where $Z_e = t^2/6$

Hence $44.22 \times 10^3 = 1.2 \times 250 \times t^2/(6 \times 1.1)$

$$= 45.45\, t^2$$

Thickness of plate $t = \sqrt{(44.22 \times 10^3/45.45)} = 31.18$ mm

Hence, use a 32-mm thick plate.

3. Weld connecting beam-column to base plate

The base plate has been designed on the basis of linear distribution of pressure. For consistency the weld will be designed on the same basis.

Beam-column size: ISHB 250; $A = 6500$ mm²; $Z_z = 619 \times 10^3$ mm³

Axial stress = $500 \times 10^3/6500 = 76.92$ N/mm^2

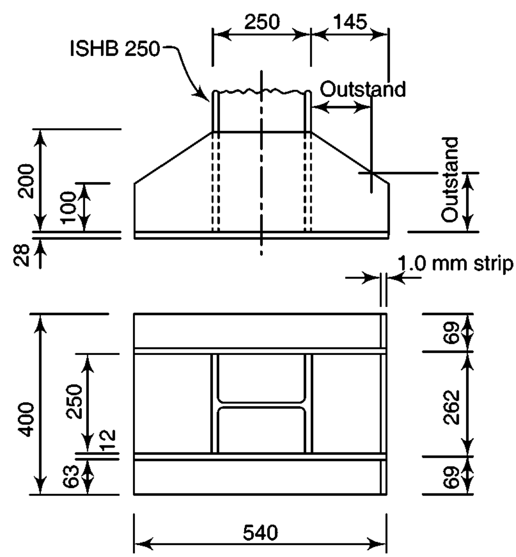
Bending stress = $45 \times 10^6/619 \times 10^3 = 72.70$ N/mm^2

On the basis of elastic stress distribution, there is compressive stress over the whole of the base. The base plate and column are to be machined for tight contact so that the weld is required only to hold the base plate in position. Use a 6 mm continuous fillet weld around the column profile.

Example 9.4 *Redesign the base plate in Example 9.3 using gusset plates. Assume that the parts are not machined for tight contact in bearing, so the welds have to be designed to transmit the column load and bending moment to the base plate.*

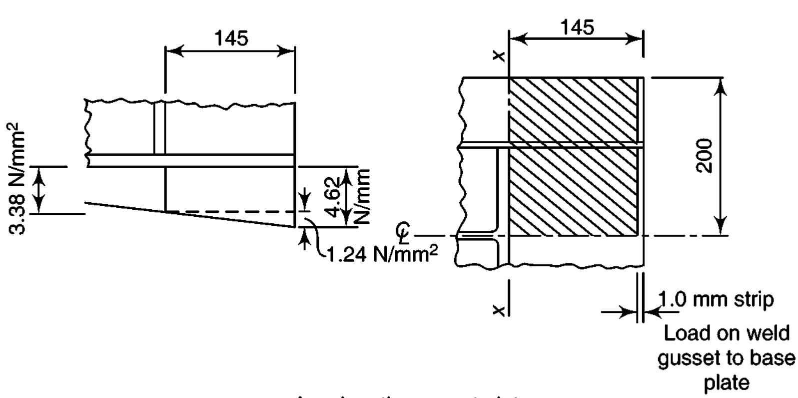
Solution

Let us consider the arrangement of gusset plate as shown in Fig. 9.14.



Arrangement of base

(a)



Load on the gusset plate

(b)

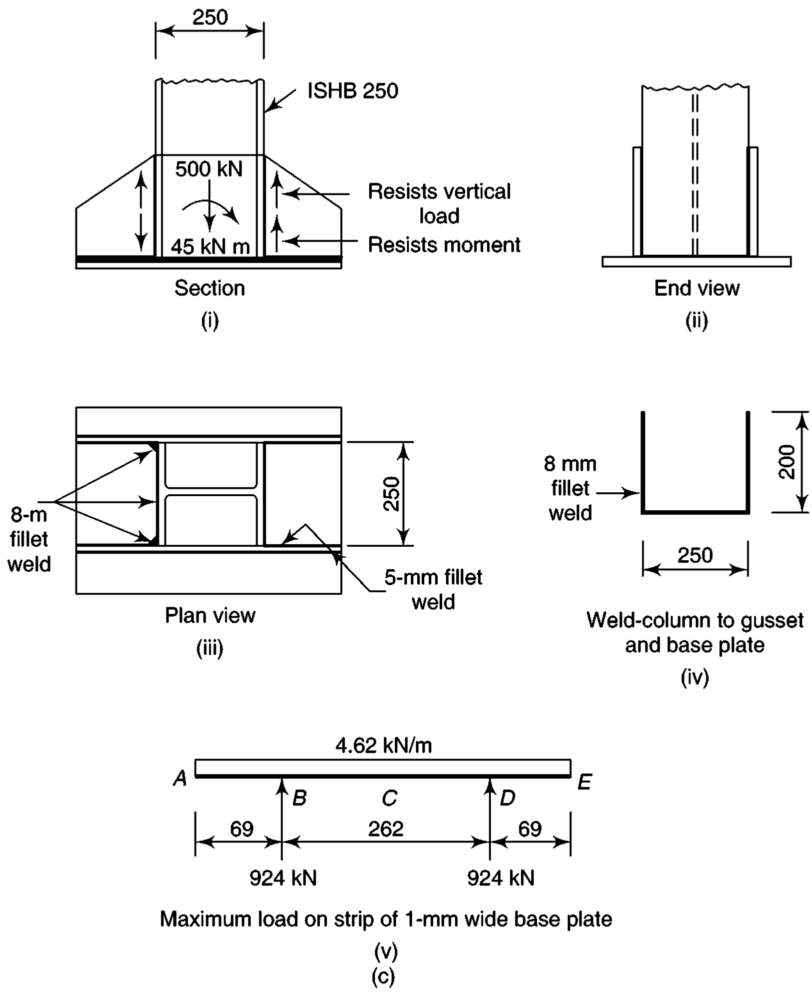


Fig. 9.14

1. Gusset plate

The gusset plate is first checked against local buckling. Assuming a 12-mm thick gusset plate,

(a) Gusset between the column flanges, with $\varepsilon = \sqrt{(250/250)} = 1$

Length = 250; D/t = 250/12 = 20.83 < 29.3 (Table 2 of the code)

(b) Gusset outstand

This should not exceed 13.6 $\varepsilon\, t$ = 13.6 × 1 × 12 = 163.2 mm > 145 mm

Provide the gusset plate as shown in Fig. 9.14.

Average height of the outstand from the base plate = (100 + 200)/2 = 150 mm < 163.2 mm

The gusset is a semi-compact section. The pressure under the base plate from Example 9.6 is shown in Fig. 9.14(b). The shear on one gusset at section *X-X* is

$$V = (4.62 + 3.38)/2 \times 145 \times 200$$
$$= 116{,}000 \text{ N} = 116 \text{ kN}$$

The bending moment at X-X axis is

$$M_x = 3.38 \times 145 \times 200 \times 145/2 + 1.24 \times 145 \times 200/2 \times 2/3 \times 145$$
$$= 8.84 \times 10^6 \text{ Nmm} = 8.84 \text{ kNm}$$

The shear capacity

$$V_n = V_p = A_v f_{yg}/(\sqrt{3}\,\gamma_{m0}) = 200 \times 12 \times 250/(\sqrt{3} \times 1.1 \times 1000)$$
$$= 314.9 \text{ kN} < 116 \text{ kN}$$
$$V = 116 \text{ kN} < 0.6 \times 314.9 = 188.95 \text{ kN}$$

The moment capacity is not reduced by the effect of shear.

$$M_g = Z_e f_y/\gamma_{m0} = (12 \times 200^2/6) \times 250/(1.1 \times 10^6)$$
$$= 18.18 \text{ kNm} > 8.84 \text{ kNm}$$

Hence the size of the gusset plate is satisfactory.

2. Gusset plate to column weld

The welds between the column, gussets, and base plate have to transmit all the load to the base plate. These welds are shown in Fig. 9.14(c).

Weld connecting column-gusset-base plate:

$$\text{Load per weld} = 500/2 + 45/0.25 = 430 \text{ kN}$$

Assuming an 8-mm weld,

Length of the weld {see Fig. 9.14(c)(iv)} = $250 + 200 \times 2 - 2 \times 8 = 634$ mm

$$\text{Load per mm} = 430/634 = 0.678 \text{ kN/mm}$$

Use an 8-mm fillet weld (site weld) which has strength of 0.884 MPa (from Appendix D). The weld between one gusset plate and the base must support the maximum pressure under the base. Considering a 1-mm wide strip at the edge of the base plate, load on one weld {see Figs 9.14(a) and (b)}

$$= 4.62 \times 200/(2 \times 10^3) = 0.462 \text{ kN/mm}$$

Provide a 5-mm fillet weld with strength = 0.553 kN/mm (from Appendix D).

3. Thickness of the base plate

Consider a 1-mm wide strip, at the edge of the base plate, as shown in Fig. 9.14(a). It is assumed to act as a beam with overhanging ends as shown in Fig. 9.14(c)(v). The bending moments are

$$M_B = 4.62 \times 69^2/2 = 10998 \text{ Nmm}$$
$$M_C = 924 \times (262/2) - 4.62 \times 200^2/2 = 28644 \text{ Nmm}$$

Moment capacity of base plate = $1.2 f_y Z_e/\gamma_{m0} = 1.2 \times (250/1.1) \times t^2/6$

Hence $28644 = 1.2 \times (250/1.1) \times t^2/6$

$$t^2 = 630.2$$

or $\quad t = 25.1$ mm

Hence, a 28-mm thick base plate is required.

Example 9.5 *Design a shear lug for a 350 mm square base plate (thickness = 30 mm) subjected to an axial dead load of 450 kN, live load of 500 kN, and shear of 240 kN resulting from wind loading. Assume that the base plate and shear lug are of Fe 410 grade steel and the foundation is M25 grade concrete.*

Solution

Steel shims are often placed under the base plates. Hence let us assume a steel-steel friction coefficient of 0.3 (Note that IS 800 in clause 7.4.1 suggests a steel concrete friction coefficient of 0.45)

1. $V_{lg1} = 1.2(240) - 0.3[1.2\,(450 + 500)] = -54$ kN

   $V_{lg2} = 1.5(240) - 0.3[1.5(450)] = 157.5$ kN
2. $A_{lg} = V/(0.45 f_{ck}) = 157.5 \times 10^3/(0.45 \times 25) = 14 \times 10^3$ mm^2
3. Assume a shear lug with width $W = 200$ mm. The height of the bearing portion is

   $$H - G = 14 \times 10^3/200 = 70 \text{ mm}$$

   Assuming a grout depth of 25 mm,

   Required depth of shear lug = 70 + 25 = 95 mm
4. The cantilever end moment

   $$M_{lg} = (V/W)(H + G)/2 = (157.5 \times 10^3/200)\,(95 + 25)/2$$
   $$= 47250 \text{ Nmm}$$
5. The required thickness $t_{lg} = \sqrt{[4M_{lg}/(f_y/1.1)]}$

   $$= \sqrt{[4 \times 47250/(250/1.1)]}$$
   $$= 28.83 \text{ mm} < 30 \text{ mm}$$

   Use a 200-mm wide, 95-mm high, and 30-mm thick shear lug.

Summary

Bending moments will be present in many practical situations in columns, in addition to the axial loads. These bending moments may be due to (a) eccentricity of axial force, (b) building frame action, (c) portal or gable frame action, (d) load from brackets, (e) transverse loads, and (f) fixed base condition. When a member is subjected to bending moment and axial force, it is called as a beam-column. The cross section of such beam-columns is oriented in such a way to resist significant bending in the major axis of the member. In general, beam-columns may be subjected to axial forces and biaxial bending moments.

All of the parameters that affect the behaviour of a beam or a column (such as the length of the member, geometry and material properties, support conditions, magnitude and distribution of transverse loads and moments, presence or absence of lateral bracing, and whether the member is a part of an unbraced or braced frame) also affect the behaviour and strength of a beam-column. The general behaviour of beam-columns is described. Thus a beam-column may fail by local buckling, overall buckling (similar to axially loaded columns), lateral-torsional buckling (similar to beams), plastic failure (short beam-columns), and by a combination of column buckling and lateral torsional buckling. Moreover, the ultimate behaviour of beam-columns subjected to axial load and biaxial bending moments is complicated by the effect of plastification, moment magnification, and lateral-torsional buckling.

The inelastic analysis to determine the strength of beam-column is complicated and is carried out in two steps: cross-section analysis and member analysis. These methods require extensive numerical analysis procedures and hence are not suitable for design office use.

Hence codes and specifications of many countries suggest the use of interaction equations, which are developed based on curve-fitting the existing analytical and experimental data on isolated beam-columns or beam-columns of simple portal frames.

The generic form of the interaction equations have been retained in the codes of practices of several countries. The interaction equations given in the Indian code (which are based on the Eurocode provisions) are discussed.

The various steps involved in the design of beam-columns, are given. The equivalent axial load method is described, which will be useful while selecting the initial cross section of the iterative design process. Though the codal equations are based on the first-order elastic analysis and hence include the moment amplification term, it is possible to use a second-order analysis and get the second-order moments directly. In such a case, the amplification factors should not be used in the interaction equations. Similarly the advanced analysis methods may eliminate the determination of K factors. (It is of interest to note that an error of 20% in the determination of the effective length factor may result in an error of 50% in the load carrying capacity of a slender column.)

Bending moments may occur in members subjected to tension; however, the effect of tension load will always reduce the primary bending moments. Hence in the code, a reduced effective bending moment is specified to be used with the interaction equations.

The design of base plates subjected to bending and axial load are covered. The provision of gussets will result in reduced thickness of base plates, though increasing the fabrication cost. Such gusseted base plates are also discussed. Information on the design of anchor bolts and shear lugs (to resist heavy shear forces) are also included. All the design concepts are explained with illustrative examples.

Exercises

1. A beam-column of length 5 m is subjected to a compression of 800 kN and a major axis moment of 4.5 kN m. The weaker plane of the column is strengthened by bracing. If the effective length factor is 0.8, design the beam-column, assuming Fe 410 grade steel.

2. A wide flange W $310 \times 310 \times 143$ beam-column has a height of 3.54 m and is pinned at both ends. Check whether it can support a design axial load of 600 kN together with a major axis bending moment of 300 kN m applied at the top of the beam-column. Assume grade Fe 410 steel.
3. Design a beam-column of length 3.75 m if it carries a compressive load of 500 kN, a major axis moment of 5 kN m and a minor axis moment of 2 kN m. Assume that the effective length factor is 1.2 and the column is free to buckle in any plane.
4. Redesign the beam-column given in Exercise 1 assuming that the axial load is tensile.
5. Design the base plate for an ISHB 300 column subjected to a factored axial load of 800 kN and a factored moment of 40 kN m in the major axis. Assume M25 concrete for the foundation and grade Fe 410 steel.
6. Redesign the base plate mentioned in Exercise 9 using gusset plates.
7. Design a base plate for a ISHB 300 column subjected to a factored axial load of 300 kN, bending moment of 60 kN m and a shear force of 300 kN. Assume that the base plate and shear lug are of Fe 410 grade steel and the foundation is M25 concrete.

Review Questions

1. What are beam-columns?
2. How are bending moments introduced in columns?
3. When bending moments are acting on a column in addition to axial loads, how are the columns oriented?
4. What are the parameters that affect the behaviour of beam-columns?
5. Describe the general behaviour of a beam-column.
6. Slender beam-columns may fail by
   (a) lateral torsional buckling
   (b) buckling similar to columns
   (c) local buckling
   (d) combination of (a) and (b)
   (e) combination of (a), (b), and (c).
7. Under which five cases can the behaviour of a beam-column be classified?
8. What is equivalent moment factor?
9. What is the general form of the interaction equation?
10. Identify the difference in behaviour of beam-columns subject to bending moment about minor axis compared to applied bending moment in major axis.
11. What is the form of interaction equation for biaxially loaded beam-column (state the linear format only)?
12. Interaction equations are specified in the codes for
    (a) overall buckling check
    (b) local capacity check
    (c) both (a) and (b)
13. State the general form of interaction equation specified in the code (IS 800) for
    (a) overall buckling check
    (b) local capacity check

14. What are the different steps to be followed while designing a beam column?
15. Describe the equivalent axial load method for selecting the initial section of a beam-column.
16. When equivalent moment method has to be used to select the initial section of a beam-column?
17. State the interaction equation used for the local capacity check of beam-columns subject to axial tension and bending moment.
18. What is the purpose of anchor bolts in a base plate having compression over the whole area?
19. What are the criteria for design in the case of base plate having compression over the whole area?
20. How are the gusset plates sized in the gusseted base plate?
21. How do the headed anchors transfer tensile load to the foundation?
22. State the advantage of using post-installed anchors.
23. List the different types of post-installed anchors.
24. What is a shear lug? When is it used?
25. What is the suggested value of μ for base plates? Why is this value suggested?
26. List the different design steps for sizing the shear lug.

CHAPTER 10

Bolted Connections

Introduction

Any steel structure is an assemblage of different members such as beam, columns, and tension members, which are fastened or connected to one another, usually at the member ends. Many members in a steel structure may themselves be made of different components such as plates, angles, I-beams, or channels. These different components have to be connected properly by means of *fasteners*, so that they will act together as a single composite unit. Connections between different members of a steel framework not only facilitate the flow of forces and moments from one member to another but also allow the transfer of forces up to the foundation level. It is desirable to avoid connection failure before member failure due to the following reasons.

(a) A connection failure may lead to a catastrophic failure of the whole structure.
(b) Normally, a connection failure is not as ductile as that of a steel member failure.
(c) For achieving an economical design, it is important that connectors develop full or a little extra strength of the members it is joining.

Connection failure may be avoided by adopting a higher safety factor for the joints than the members.

The basic goal of connection design is to produce a joint that is safe, economical, and simple (so that it can be manufactured and assembled at site without any difficulty). It is also important to standardize the connections in a structure and to detail it in such a way that it allows sufficient clearance and adjustment to accommodate any lack of fit, resists corrosion, is easy to maintain, and provides reasonable appearance.

Connections (or structural joints) may be classified according to the following parameters:

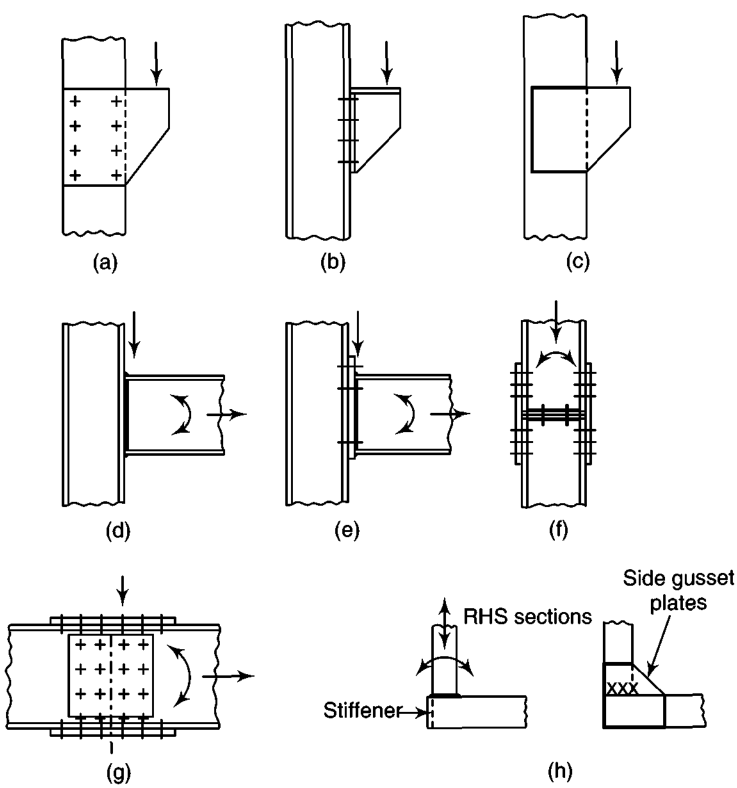
(a) Method of fastening such as rivets, bolts, and welding—connections using bolts are further classified as *bearing* or *friction type connections*
(b) Connection rigidity—simple, rigid (so that the forces produced in the members may be obtained by using an indeterminate structural analysis), or semi-rigid

(c) Joint resistance—bearing connections and friction connections (these are explained in subsequent sections)
(d) Fabrication location—shop or field connections
(e) Joint location—beam column, beam-to-beam, and column-to-foundation
(f) Connection geometry—single web angle, single plate, double web angle, top and seat angles (with and without stiffeners), end plates, header plate, welded connections using plates and angles, etc.
(g) Type of force transferred across the structural connection—shear connections, shear and moment connection or simply moment connection, tension or compression, and tension or compression with shear.

Structural connections transmit forces which result in linear and rotational movements. The linear movements at a joint are generally small but the rotational movement depends on the stiffness of the type of connection.

According to the IS code, based on connection rigidity, the joints can be defined as follows:

Rigid Rigid connections develop the full moment capacity of connecting members and retain the original angle between the members under any joint rotation, that is, rotational movement of the joint will be very small on these connections. Examples of rigid connections are shown in Fig. 10.1.



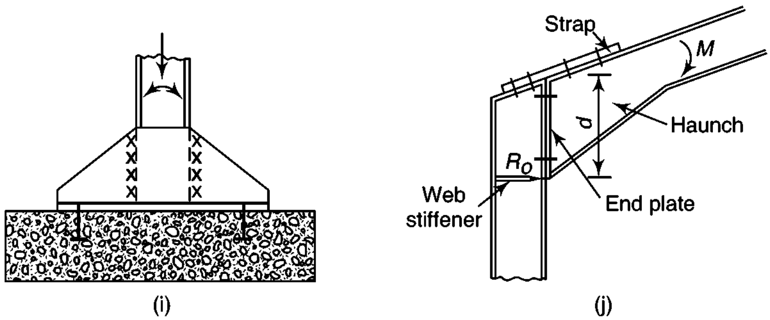
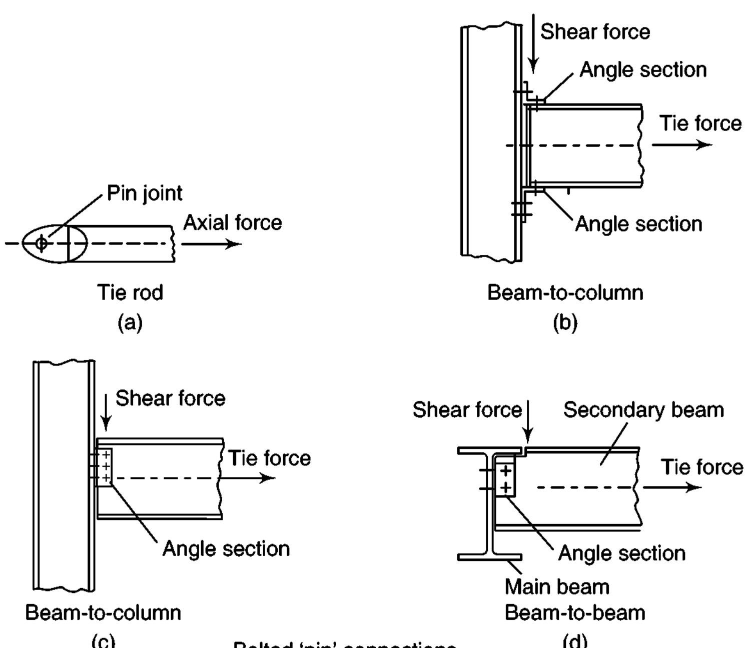


Fig. 10.1 Examples of 'rigid' connections (Martin & Purkiss 1992)

Simple In simple connections no moment transfer is assumed between the connected parts and hence are assumed as hinged (pinned). The rotational movement of the joint will be large in this case. Actually, a small amount of moment will be developed but is normally ignored in the design. Any joint eccentricity less than about 60 mm is neglected. Examples of hinged (pinned) connections are shown in Fig. 10.2. Some simple connections, for example, tie bars, are connected by real pins as shown in Fig. 10.2(a). If the pins are not corroded or blocked with debris, they will act as pin joints. Tie bars are rarely used now since the safety depends on a single pin and also due to the cost of manufacture and malfunctioning of pins due to corrosion or debris (Martin & Purkiss 1992).



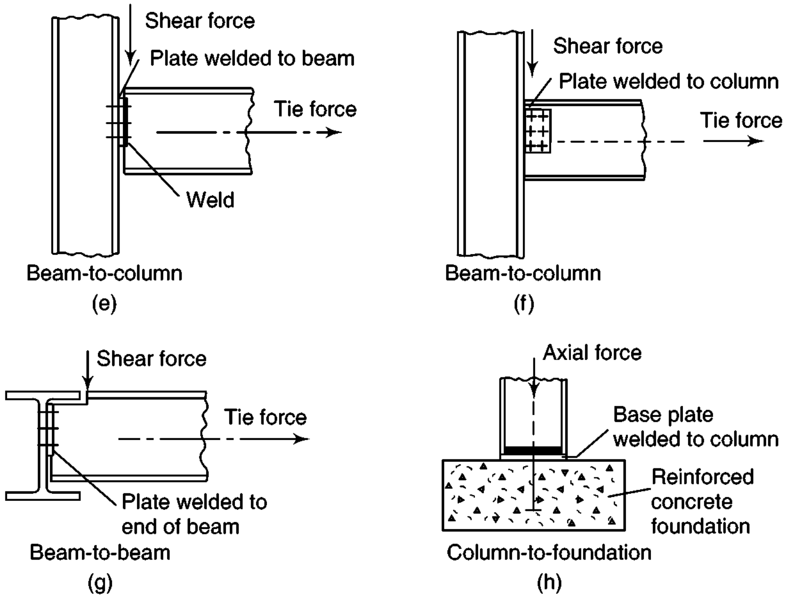


Fig. 10.2 Examples of 'pinned' connections (Martin & Purkiss 1992)

Semi-rigid Semi-rigid connections may not have sufficient rigidity to hold the original angles between the members and develop less than the full moment capacity of the connected members. The design of these connections requires determining the amount of moment capacity (or moment–rotation relationship of the connection) based on test results or rational methods (say 20%, 30%, or 75% of moment capacity).

In reality, all the connections will be semi-rigid. However, for convenience we assume some of them as rigid and some as hinged. We will discuss bolted connections in this chapter. Welded connections are discussed in the next chapter.

10.1 Rivets and Riveted Connections

For many years rivets were the sole practical means of producing safe and serviceable steel connections. A rivet is made up of a round ductile steel bar (mild or high tensile steel as per IS 1929 and IS 2155) called *shank*, with a head at one end (see Fig. 10.3). The head may have different shapes as shown in Fig. 10.3. The snap and pan heads form a projection beyond the plate face, whereas the counter sunk head will be flush with the surface of the plate face.

The length of the rivet to be selected should be longer than the grip of the rivet (see Fig. 10.3), sufficient to form the second head. The installation of a rivet requires the heating of the rivet to a cherry red colour (approximately 980°C), inserting it into an oversize hole (approximately 1.5 mm more than the size of the rivet), applying pressure to the preformed head while at the same time squeezing the plain end of the rivet using a pneumatic driver to form a round head. During this process, the shank of the rivet completely or nearly fills the hole into which it had been inserted. Upon cooling, the rivet shrinks, thereby producing a clamping force. Owing to this, a riveted joint is intermediate between a friction and a bearing type connection. Since the amount of clamping produced is not dependable, (rivets are often inspected after installation, wherein loose rivets are detected and replaced) a bearing type connection is commonly assumed. The riveted joint has had a long history of success under fatigue stresses as in the several railway bridges throughout the world. However, riveting operations require at least four persons—one to heat and toss the rivet to the driving crew, one to catch the hot rivet and insert it in the hole, one to handle the backup bar and one to drive the rivet with a pneumatic hammer.

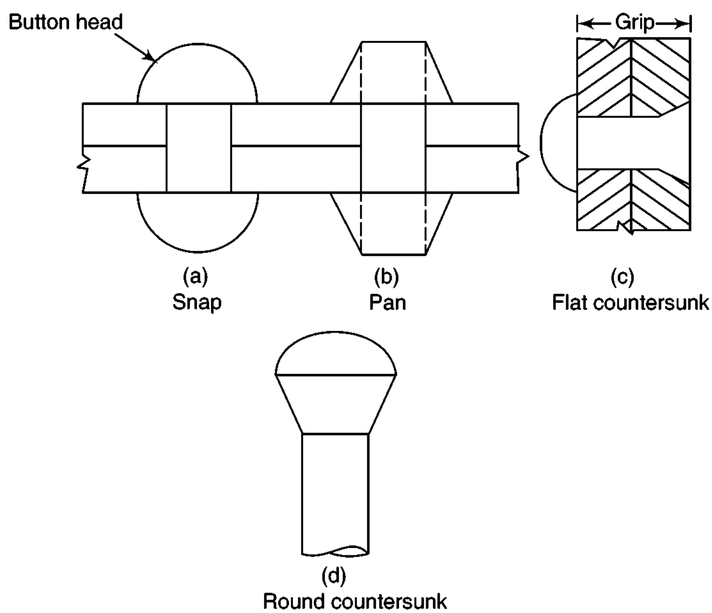


Fig. 10.3 Types of rivets

Riveting is no longer used in engineering structures for the following reasons:

(a) The necessity of pre-heating the rivets prior to driving
(b) The labour costs associated with large riveting crews
(c) The cost involved in careful inspection and removal of poorly installed rivets
(d) The high level of noise associated with driving rivets

Readers should be aware that the design of riveted connection is similar to the design of bolted connection, except that the diameter of the rivet is taken as the diameter of the hole (diameter of rivet + clearance) in place of the nominal diameter of the bolt.

10.2 Bolted Connections

There are several types of bolts used to connect structural members. Some of them are listed as follows:

- Unfinished bolts or black bolts or C grade bolts (IS 1363 : 2002)

- Turned bolts
  - Precision bolts or A grade bolts (IS 1364 : 2002)
  - Semi-precision bolts or B grade bolts (IS 1364 : 2002)
- Ribbed bolts
- High strength bolts (IS 3757 : 1985 and IS 4000 : 1992)

10.2.1 Black Bolts

Black bolts are also referred to as ordinary, unfinished, rough, or common bolts. They are the least expensive bolts. However, they may not produce the least expensive connection since the connection may require a large number of such bolts. They are primarily used in light structures under static loads such as small trusses, purlins, girts, bracings, and platforms. They are also used as temporary fasteners during erection where HSFG bolts or welding are used as permanent fasteners. They are not recommended for connections subjected to impact, fatigue, or dynamic loads. These bolts are made from mild steel rods with a square or hexagonal head and nuts as shown in Fig. 10.4, conforming to IS 1363.

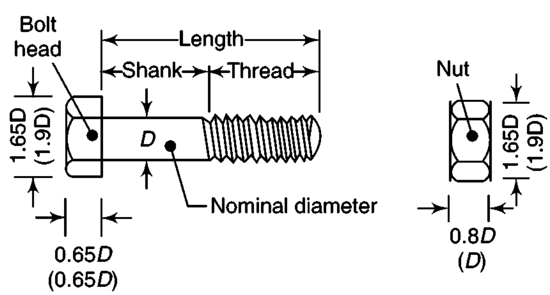


Fig. 10.4 Hexagonal head black bolt and nut. Figures in brackets are for high-strength bolts and nuts.

In steel construction, generally, bolts of property class 4.6 are used. In property class 4.6, the number 4 indicates 1/100th of the nominal ultimate tensile strength in N/mm^2 and the number 6 indicates the ratio of yield stress to ultimate stress, expressed as a percentage. Thus, the ultimate tensile strength of a class 4.6 bolt is 400 N/mm^2 and yield strength is 0.6 times 400, which is 240 N/mm.^{2} The tensile properties of commonly used fasteners are listed in Table 10.1. Due to the high percentage elongation of these bolts, they are more ductile. For bolts of property class 4.6, nuts of property class 4 are used and for bolts of property class 8.8, nuts of property class 8 or 10 are used.

Though square heads cost less, hexagonal heads give better appearance, are easier to hold by wrenches, and require less turning space. Most of the connections with black bolts are made by inserting them in clearance holes of about 1.5 mm to 2 mm more than the bolt diameter and by tightening them with nuts. They are produced in metric sizes ranging from 5–36 mm and designated as M5 to M36. In structural steelwork, M16, M20, M24, and M30 bolts are often used. The ratio of net tensile area at threads to nominal plain shank area of the bolt is 0.78 as per IS

1367 (Part 1). The other dimensions of commonly used bolts are as given in Table 10.2. These dimensions are so chosen that the bolt head does not fail unless the shank fails.

Table 10.1 Tensile properties of fasteners used in steel construction

Specification	Grade/ classification	Properties Yield stress, MPa (Min)	Ultimate tensile stress, MPa (Min)	Elongation percentage (Min)
	4.6	240	400	22
IS 1367 (Part 3)	4.8	320	420	14
(ISO 898)	5.6	300	500	20
Specifications of fasteners-threaded	5.8	400	520	10
steel for technical supply conditions	8.8 ($d < 16$ mm)	640	800	12
	10.9	940	1040	9

Table 10.2 Dimensions of hexagon head black bolts (grade 4.6) as per IS 1364 (Part 1)

Bolt size (d), mm	Head diagonal (e), mm	Head thickness (k), mm	Thread* length (b), mm	Pitch of thread, mm	Washer (IS 5370) Outer diameter, mm	Inner diameter, mm	Thickness, mm
16	26.17	10	23	2.0	30	18	3
20	32.95	13	26	2.5	37	22	3
24	39.55	15	30	3.0	44	26	4
30	50.85	19	35	3.5	56	33	4

*For length $l \le 125$ mm. For $125 < l \le 200$, b is 6 mm more and for $l > 200$, b is 19 mm more.

10.2.2 Turned Bolts (Close Tolerance Bolts)

These are similar to unfinished bolts, with the difference that the shanks of these bolts are formed from a hexagonal rod. The surface of these bolts are prepared and machined carefully to fit in the hole. Tolerances allowed are about 0.15 mm to 0.5 mm. Since the tolerance available is small, these bolts are expensive. The small tolerance necessitates the use of special methods to ensure that all the holes align correctly. These bolts (precision and semi-precision) are used when no slippage is permitted between connected parts and where accurate alignment of components is required. They are mainly used in special jobs (in some machines and where there are dynamic loads).

10.2.3 High-Strength Bolts

High-strength bolts are made from bars of medium carbon steel. The bolts of property class 8.8 and 10.9 are commonly used in steel construction. These bolts should conform to IS 3757 and their tensile properties are given in Table 10.1. As discussed in Chapter 1, their high strength is achieved through quenching and tempering process or by alloying steel. Hence, they are less ductile than black bolts. The material of the bolts do not have a well-defined yield point. Instead of using yield stress, a so-called *proof load* is used. The proof load is obtained by multiplying the tensile stress area (may be taken as the area corresponding to root diameter at the thread and is approximately equal to 0.8 times the shank area of bolt) with the proof stress. In IS 800, the proof stress is taken as 0.7 times the ultimate tensile stress of the bolt. (In other codes such as the American code, the proof stress is taken as the yield stress, established by the 0.2% offset strain.) This bolt tension $0.7f_u$ gives adequate reserve strength, should this bolt be somewhat over stressed (e.g., 3/4 turn instead of 1/2 turn in the turn-of-the-nut method). Note that grade 10.9 bolts have lower ductility than grade 8.8 bolts and the margin between the 0.2% yield strength and the ultimate strength is also lower.

The HSFG bolt, nut, and washer dimensions are shown in Table 10.3 (also see Fig. 10.4 for approximate bolt dimensions). Bolts of sizes M16, M20, M24, and M30 are commonly used in practice. These bolts are identified by the manufacturer's identification symbol and the property class identification symbol 8S, 8.8S, 10S, or 10.9S, which will be embossed on the heads of these bolts. Since, these bolts have a tensile strength much higher than the ordinary black bolts, the number of bolts required at a joint is considerably reduced. The vibration and impact resistance of the joint are also improved considerably.

Table 10.3 High-strength friction grip bolts as per IS 3757 : 1985

Diameter, *d* mm	M16	M20	M24	M30
Head diagonal, *e* mm	29.56	37.29	45.20	55.37
Head thickness, *k* mm	10	12.5	15	18.7
Nut thickness, mm	13	16	19	24
Washer outer diameter, **D* mm	30	37	44	56
Washer thickness, heavy, mm	4	4	4	5
Thread length, ***b* mm				
<100	31	36	41	49
>100	38	43	48	56

* The outside diameter of a washer is an important dimension when detailing, for example, to avoid overlapping an adjacent weld.

**The thread length depends on the length of the bolt, which is calculated as grip length plus the allowance for grip as given in Table 10.4.

The percentage elongation of 12% at failure of these bolts is less than the black bolts, but is still acceptable for design purposes. Special techniques (see Section 10.2.4) are used for tightening the nuts to induce a specified initial tension in the bolt, which causes sufficient friction between the faying faces. These bolts with induced initial tension are called *High-Strength Friction Grip (HSFG) bolts*. Due to this friction, the slip in the joint (which is associated with black bolts) is eliminated and hence the joints with HSFG bolts are called non-slip connections or *friction type connections* (as opposed to the bearing type connections of ordinary black bolts). The induced initial tension in the bolt is called the *proof-load* of the bolt and the coefficient of friction between the bolt head and the faying surfaces is called the *slip factor*. The bolts of property class 8.8 can be hot-dip galvanized {as per IS 1367 (Part 13)} whereas class 10.9 bolts should not be hot-dip galvanized since this may cause hydrogen embrittlement (IS 3757).

Table 10.4 Allowance for Bolt Length

Nominal size of the bolt	Allowance for Grip*, in mm
16	26
20	31
24	36
30	42
36	48

*The allowance includes thickness of one nut and one washer only. If additional washers are used or where threads are excluded (in bearing type joints) from shear plane, high allowance may be required.

10.2.4 Bolt Tightening Methods

When slip resistant connections are not required and when bolts are not subjected to tension, high-strength bolts are tightened to a 'snug-tight' condition to ensure that the load transmitting plies are brought into effective contact (this may be achieved as a result of a few impacts of an impact wrench or the full effort of a man using an ordinary spud wrench).

However, the reliability of HSFG bolts in a non-slip or friction type connection depends on the method of tightening of the bolt, which will ensure whether the required proof load (pre-tension) is obtained. The three methods that may be used in practice are *the turn-of-the-nut tightening* (part-turn method), *direct tension indicator tightening, and calibrated wrench tightening* (torque control method). Only turn-of-the-nut tightening method is described below. For other methods, refer Owens and Cheal (1989) and Struik et al. (1973).

Turn-of-the-nut tightening method Turn-of-the-nut tightening method, also known as part-turn method, is the simplest and most common method. Developed in the 1950s and 1960s, the specified pre-tension in the bolt is considered to be obtained by a specified rotation of the nut from the 'snug-tight' condition. In this method, after the bolts are snug-tight, permanent marks (these permanent marks may be used in a subsequent inspection) are made on bolts and nuts to identify the relative position of the bolt and nut and to control the final nut rotation. Each nut is then tightened by a specified turn of the nut from the snug-tight position depending on the length of the bolt as prescribed in IS 4000 (see Table 10.5 which also gives the minimum tension which should be available in the bolt after tightening).

Table 10.5 Minimum Bolt Tension and Nut Rotation from Snug-Tight Condition (IS 4000 : 1992).

Nominal size of bolt	Length of bolt,* mm		Minimum bolt tension in kN for bolts of property class	
	Nut rotation 1/2 turn	Nut rotation 3/4 turn	8.8	10.9
M16	≤ 120	$>120 \leq 280$	90	112.5
M20	≤ 120	$>120 \leq 240$	140	175
M24	≤ 160	$>160 \leq 350$	202	253
M30	≤ 160	$>160 \leq 350$	316	395

*Length is measured from the underside of the head to the extreme end of the shank.

Whatever be the tightening method, the installation must begin at the most rigid part of the connection and progress systematically towards the least rigid areas. Similarly, where there are more than four bolts in a group, the bolts should be tightened in a staggered manner, working from the centre of the joint outward. It has been observed that the behaviour of galvanized bolts may differ from the behaviour of normal, uncoated high-strength bolts.

Since the turn-of-the-nut method often induces a bolt tension that may exceed the elastic limit of the threaded portion, repeated tightening of high-strength bolts may be undesirable.

10.2.5 Advantages of Bolted Connections

The black bolts offer the following advantages over riveted or welded connections:

(a) Use of unskilled labour and simple tools
(b) Noiseless and quick fabrication
(c) No special equipment/process needed for installation
(d) Fast progress of work
(e) Accommodates minor discrepancies in dimensions
(f) The connection supports loads as soon as the bolts are tightened (in welds and rivets, cooling period is involved).

The main drawback of the black bolt is the slip of the joint when subjected to loading. When large forces are to be resisted, the space required for the joint is extensive. Also, precautions such as the provision of special locking devices or the use of pre-loaded high-strength bolts are required in situations involving fluctuating loads.

Though the material cost of HSFG bolts are about 50% higher than black bolts and require special workmanship for installation, they provide the following advantages.

(a) HSFG bolts do not allow any slip between the elements connected, especially in close tolerance holes (see Fig. 10.5), thus providing rigid connections.
(b) Due to the clamping action, load is transmitted by friction only and the bolts are not subjected to shear and bearing.
(c) Due to the smaller number of bolts, the gusset plate sizes are reduced.
(d) Deformation is minimized.

(e) Since HSFG bolts under working loads do not rely on resistance from bearing, holes larger than usual can be provided to ease erection and take care of lack of fit. Thus the holes may be standard, extra large, or short/long slotted. However, the type of hole will govern the strength of the connection.
(f) Noiseless fabrication, since the bolts are tightened with wrenches.
(g) The possibility of failure at the net section under the working loads is eliminated.
(h) Since the loads causing fatigue will be within proof load, the nuts are prevented from loosening and the fatigue strength of the joint will be greater and better than welded and riveted joints. Moreover, since the load is transferred by friction, there is no stress concentration in the holes.
(i) Unlike riveted joints, few persons are required for making the connections.
(j) No heating is required and no danger of tossing of bolt. Thus, the safety of the workers is enhanced.
(k) Alterations, if any (e.g. replacement of the defective bolt) are done easily than in welded or riveted connections.

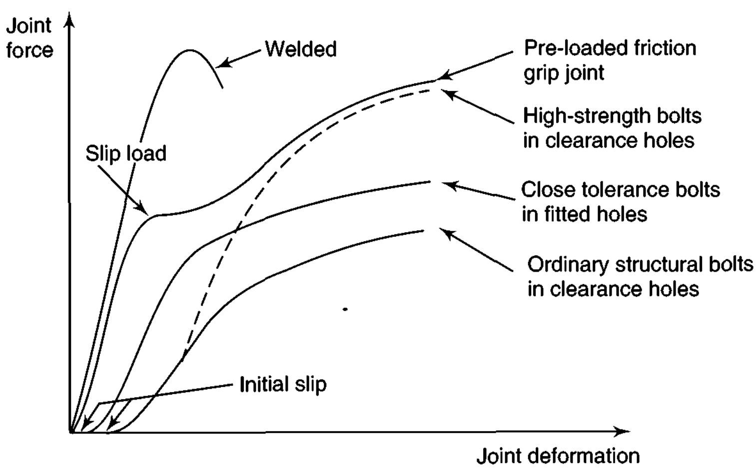


Fig. 10.5 Typical load–deformation behaviour of different types of fasteners.

However, bolting usually involves a significant fabrication effort to produce the bolt holes and associated plates or cleats. In addition, special procedures are required to ensure that the clamping actions required for pre-loaded friction-grip joints are achieved. The connections with HSFG bolts may not be as rigid as a welded connection.

10.2.6 Bolt Holes

Bolt holes are usually drilled. Punched holes (punched full size or punched undersize and reamed) are preferred by steel fabricators because it is simple and saves time and cost. However, punching can reduce ductility and toughness and may lead to brittle fracture. Hence, punched holes should not be used where plastic tensile straining can occur (Owens et al. 1981). IS 800 allows punched holes only in materials whose yield stress f_y does not exceed 360 MPa and where thickness does not exceed ($5600/f_y$) mm. It also disallows punched holes in cyclically loaded details. Holes should not be formed by gas cutting, since they affect the local properties of steel, though plasma cutting is allowed in the code for statically loaded members (clause 17.2.4.5).

Bolt holes are made larger than the bolt diameter to facilitate erection and to allow for inaccuracies in fabrication. Table 10.6 shows the standard values of holes for different bolt sizes (the clearance is 1.0 mm for bolts less than 14 mm and 2 mm for bolts between 16 mm and 24 mm and 3 mm for bolts exceeding 24 mm).

Table 10.6 Bolt diameter, pitch, and edge distances as per IS 800

Nominal diameter of bolt, d, mm	12	14	16	18	20	22	24	Above 24
Diameter of hole, d_R, mm	13.0	15.0	18.0	20.0	22.0	24.0	26.0	Bolt diameter + 3 mm
Minimum edge distance,* e_b, mm								
(a) for sheared or rough edge	22	26	30	34	37	40	44	1.7 × hole diameter
(b) for rolled, sawn, or planed edge	19	23	27	30	33	36	39	1.5 × hole diameter

*The edge distances in this table, which are for standard holes, must be increased if oversize or slotted holes are used.

Max. edge distance = $12t\varepsilon$ where $\varepsilon = (250/f_y)^{0.5}$

Pitch (min.)	2.5 × nominal diameter of bolt
Pitch (max.)	$32t$ or 300 mm
(a) parts in tension	$16t$ or 200 mm, whichever is less
(b) parts in compression	$12t$ or 200 mm, whichever is less
(c) tacking fasteners	$32t$ or 300 mm, whichever is less $16t$ or 200 mm, whichever is less for plates exposed to weather

where t is the thickness of the thinner outside plate or angle.

Oversize holes {should not exceed $1.25d$ or $(d + 8)$ mm in diameter, where d is the nominal bolt diameter in mm} and slotted holes are allowable but should not be used often. A slotted hole should not exceed the appropriate hole size in width and $1.33d$ in length, for short slotted hole and $2.5d$ in length, for long slotted hole. Slotted holes are used to accommodate movements in a structure. However, if holes are longer than $2.5d$, shear transfer in the direction of the slot is not admissible even in a friction type connection (see also Section 10.2.1 of code).

Bolt holes reduce the gross cross-sectional area of sections (plates, angles, etc.). The net value is used in the calculations, when the element is in tension (see Chapter 3). As already discussed, bolt holes produce stress concentration, but this is offset by the fact that yield at highly stressed cross section will work-harden, before fracture, resulting in the yield of adjacent cross section also. Whereas, if the member is in compression, then the gross cross section of the member is used in the calculation, because at yield the bolt hole deforms, transferring part of the load to the shank of the bolt or be resisted by bearing.

10.2.7 Spacing and Edge Distance of Bolt Holes

The centre-to-centre distance between individual fasteners in a line, in the direction of load/stress is called the *pitch*. The distance between any two consecutive fasteners in a zigzag pattern of bolts, measured parallel to the direction of load/stress is called a *staggered pitch*. A minimum spacing of 2.5 times the nominal diameter of the fastener is specified in the code to ensure that there is sufficient space to tighten the bolts, prevent overlapping of the washers, and provide adequate resistance to tear-out of the bolts. It also limits any adverse interaction between high bearing stresses due to neighbouring bolts. Similarly, the code specifies maximum pitch values, as given in Table 10.6. These values are specified to prevent buckling of plates in compression between the bolts, to ensure that the bolts act together as a group to resist the loads and to avoid corrosion by ensuring adequate bridging of the paint film between plates. The spacing between adjacent parallel line of fasteners, transverse to the direction of load/stress is called *gauge distance*. The gauge distance as specified in SP-1, published by the Bureau of Indian Standards, is given in Appendix D.

The distance from the centre of a fastener hole to the edge of an element (measured at right angles to the direction of the load) is called the *end* or *edge distance*. The edge distance should be sufficient for bearing capacity and to provide space for the bolt head, washer, and nut. Hence, minimum edge distances are specified in the code and are given in Table 10.6. The maximum edge distance to the nearest line of fasteners from an edge of any unstiffened part should not exceed $12t\varepsilon$ where $\varepsilon = (250/f_y)^{0.5}$ and t is the thickness of the thinner outer plate. (This rule is not applicable to fasteners interconnecting the components of back-to-back tension members.) In corrosive environment, the maximum edge distance should not exceed 40 mm plus $4t$.

10.3 Behaviour of Bolted Joints

Loads are transferred from one member to another by means of the connections between them. A few typical bolted connections are given in Fig. 10.6.

The possible 'limit states' or failure modes that may control the strength of a bolted connection are shown in Fig. 10.7. Thus, any joint may fail in any one of the following modes:

- Shear failure of bolt
- Shear failure of plate
- Bearing failure of bolt
- Bearing failure of plate
- Tensile failure of bolts
- Bending of bolts
- Tensile failure of plate

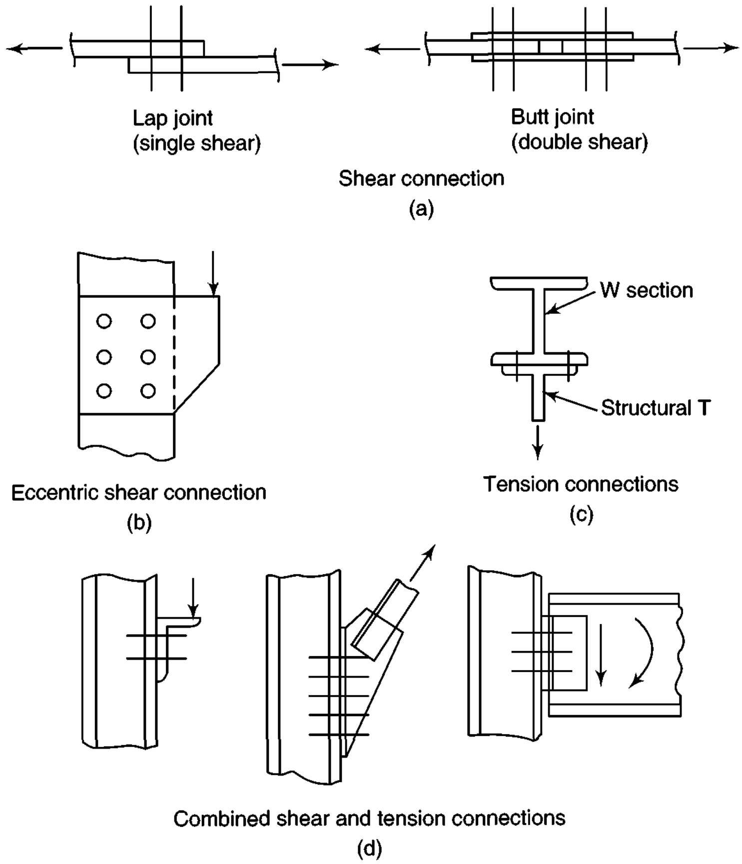


Fig. 10.6 Typical bolted connections

In bearing type connections using black bolts or high-strength bolts, as soon as the applied load overcomes the very small amount of friction at the interface, slip will occur and the force is transferred from one element to another element through bearing of bolts (see Fig. 10.5).

Once the bolts are in bearing, the connection will behave linearly, until yielding takes place at one or more of the following positions (Owens & Cheal, 1989):

- At the net section of the plate(s), under combined tension and flexure
- On the bolt shear plane(s)
- In bearing between the bolt and the side of the hole

The forces acting on the bolt are shown in Fig. 10.8(a).

The response of the connection becomes non-linear after yielding takes place, as plasticity spreads in the presence of strain hardening and failure takes place at one of the critical sections/locations listed above (see Fig. 10.7). The mode of failure and the point of initiation of yielding depends upon the proportions and relative material strength of the components.

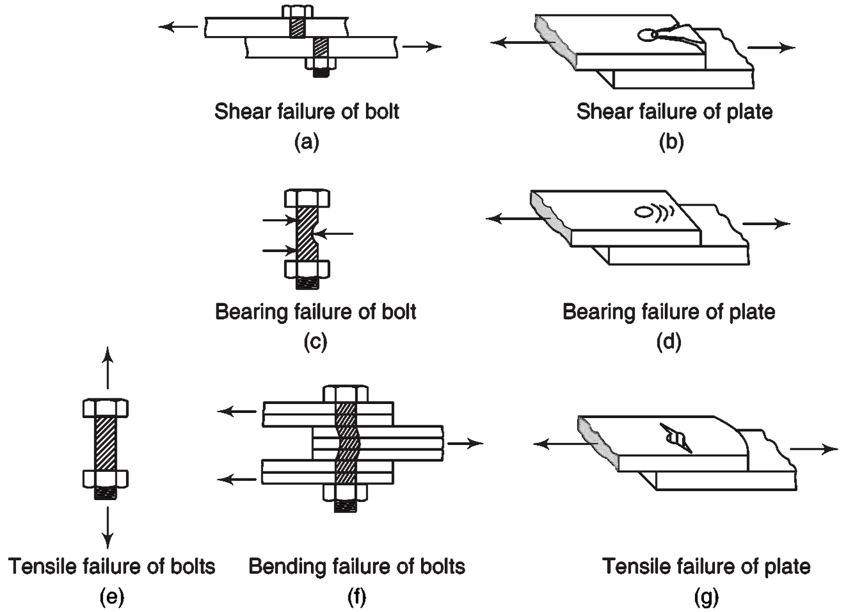


Fig. 10.7 Possible modes of failure of bolted connections.

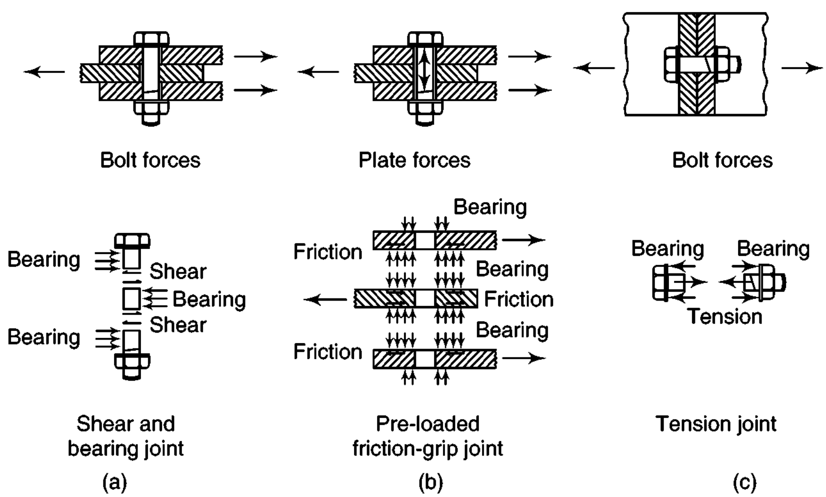


Fig. 10.8 Force transmission through bolts (Trahair et al. 2001)

In a multibolt connection, the behaviour is similar except that the more highly loaded bolt starts to yield first, and the connection will become less stiff. At a later stage, due to redistribution of forces, each bolt is loaded to its maximum capacity. However, it is generally assumed that equal size bolts share equally in transferring the external force as shown in Fig. 10.9(b), even during service loads. However, in a long bolted connection, the shear force is not evenly distributed among the bolts, and consequently the bolts at the end of a joint resist the highest amount of shear force, as shown in Fig. 10.9(c). In such joints, the end bolt forces may be so high that it may lead to a progressive joint failure called 'unbuttoning'. If the joint is short, the forces in the bolts will be redistributed by plastic action, and hence the bolts will share the shear force equally.

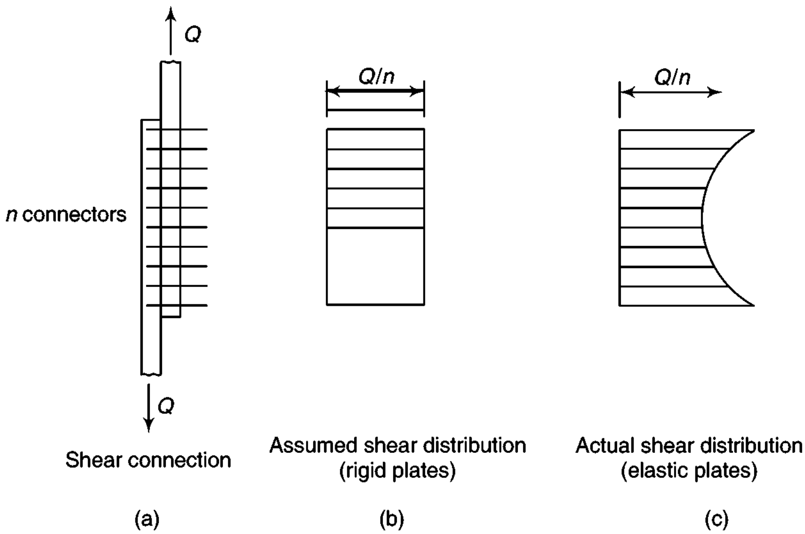


Fig. 10.9 Shear distribution in a long bolted connection

Shear and bearing connections using close tolerance bolts in fitted holes behave in a similar manner to connections with clearance holes, except that the bolt slip is considerably small (see Fig. 10.5). As mentioned earlier, close tolerance bolts are rarely used.

In the case of HSFG bolts, the slip in the bolt will not occur immediately but at a load which overcomes the frictional resistance provided by the pre-load of the bolt (see Fig. 10.5). After slip occurs, the behaviour of the bolt is similar to the normal bolts. In this case also, it is commonly assumed that equal size bolts share the loads equally in transferring the external force.

The flexibility of a connection is determined from the sum of the flexibilities of its different components (bolts, plates, and cleats used in the connection). Note that plates (gusset plates) are comparatively stiff when loaded in their own plane and are considered as rigid. However, when bent out of their plane, they are comparatively flexible. The overall behaviour of any connection should be carefully assessed by determining the force flow through the connection and by synthesizing the responses of the elements to their individual loads (Trahair et al. 2001).

10.4 Design Strength of Ordinary Black Bolts

Expressions for design strength of ordinary black bolts subjected to shear, tension, and bearing forces are given in this section. In addition, when bolts are subjected to tension, there may be additional forces due to flexibility of connections, which are called prying forces. Methods to calculate prying forces and interaction equation for bolts subjected to combined shear and tension forces are also covered. Tension capacity of plates and efficiency of joints are also discussed.

10.4.1 Bearing Bolts in Shear

The nominal capacity of a bolt in shear V_{nsb} [Figs 10.8(a) and 10.7(a)] depends on the ultimate tensile strength f_u of the bolt, the number of shear planes n ($n = n_n + n_s$), and the areas A_{sb} (nominal shank area) and A_{nb} (net tensile stress area through the threads) of the bolt in each shear plane. It is expressed in the code as

$$V_{nsb} = 0.577 f_u (n_n A_{nb} + n_s A_{sb}) \beta_{lj} \beta_{lg} \beta_{pkg} \qquad (10.1)$$

where n_n is the number of shear planes with threads intercepting the shear plane, n_s is the number of shear planes without threads intercepting the shear plane, β_{lj} is the reduction factor which allows for the overloading of end bolts that occur in long connections (see Fig. 10.9), β_{lg} is the reduction factor that allows for the effect of large grip length, and β_{pkg} is the reduction factor to account for packing plates in excess of 6 mm.

The code stipulates that the factored shear force V_{sb} should satisfy

$$V_{sb} \le 0.8 V_{nsb} \qquad (10.2)$$

When the net tensile stress area through the threads is not given, it may be taken, for ISO thread profile, as

$$A_{nb} = (\pi/4)(d - 0.9382p)^2 \qquad (10.3)$$

where d is the shank or nominal diameter of bolt in mm and p is the pitch of the thread in mm. The net tensile stress area will be approximately 78 – 80% of the gross area.

Reduction factor for long joints When the joint length l_j, of a splice or end connection, in tension or compression, exceeds $15d$ in the direction of load (the joint length is taken as the distance between the first and last rows of the bolts in a joint, measured in the direction of the load transfer), the nominal shear capacity V_{nsb} is multiplied by a reduction factor β_{lj} as shown in Eqn (10.1). This reduction factor is given by

$$\beta_{lj} = 1.075 - l_j/(200\text{ d}) \quad \text{for } 0.75 \le \beta_{lj} \le 1.0 \qquad (10.4)$$

This reduction factor should not be applied when the distribution of shear over the length of joint is uniform as in the connection of web of a section to the flanges.

Reduction factor for large grip lengths When the total thickness of connected plates or plies (grip length) l_g exceeds five times the nominal diameter of the bolts, the nominal shear capacity V_{nsb} is multiplied by a reduction factor β_{lg} as shown in Eqn (10.1). This reduction factor is given by

$$\beta_{lg} = 8d/(3d + l_g) \qquad (10.5)$$

The value of β_{lg} calculated using Eqn (10.5) should not be more than β_{lj} given in Eqn (10.4) and the grip length l_g is also restricted to $8d$ by the code.

The reason for the above reduction in strength is that as the grip length increases, the bolt is subjected to greater bending moments due to the shear forces acting on them [see Fig. 10.7(f)].

Reduction factor for packing plates Similar to the grip length, the thickness of packing plates also influence the nominal shear capacity V_{nsb}. Thus, when the packing plates are greater than 6 mm, the shear capacity is multiplied by the reduction factor β_{pkg}. This reduction factor is given by

$$\beta_{pkg} = 1 - 0.0125 t_{pkg} \tag{10.6}$$

where t_{pkg} is the thickness of the thicker packing plate in mm.

10.4.2 Bolts in Tension

The nominal capacity of a bolt in tension T_{nb} [Figs 10.8(c) and 10.7(c)] depends on the ultimate tensile strength f_{ub} of the bolt and net tensile stress area A_n (taken as the area at the bottom of the threads) of the bolt, and is given by

$$T_{nb} = 0.90 f_{ub} A_n < 1.136\, f_{yb} A_{sb} \tag{10.7}$$

where A_{sb} is the shank area of the bolt, and f_{yb} is the yield stress of the bolt. IS 800 stipulates that the factored tension force T_b should satisfy

$$T_b \leq 0.8\, T_{nb} \tag{10.8}$$

If any of the connecting plates is sufficiently flexible, then additional prying forces may be induced in the bolt (see Section 10.4.4 for the details).

10.4.3 Bolts in Bearing

When an ordinary bolt is subjected to shear forces, it comes into contact with the plates, after the slip occurs. The bearing limit state relates to deformation around a bolt hole, as shown in Fig. 10.7(d) (enclosed bearing failure for a large end distance). A shear tear-out failure (also called end bearing failure) as shown in Fig. 10.7(b) occurs when the end distance is small. Bearing failure in bolts {see Fig. 10.7(c)} is possible only by using low strength bolt with very high grade plates, which will not occur in practice.

The nominal bearing strength of the bolt V_{npb} is given by

$$V_{npb} = 2.5 k_b d t f_u \tag{10.9}$$

where f_u is the ultimate tensile stress of the plate in MPa, d is the nominal diameter of the bolt in mm, and t is the summation of the thicknesses of the connected plates experiencing bearing stress in the same direction. (If the bolts are countersunk, then t is equal to the thickness of the plate minus one half of the depth of counter sinking.)

k_b is smaller of $e/(3d_h)$, $p/(3d_h) - 0.25$, f_{ub}/f_u and 1.0 where f_{ub} is the ultimate tensile stress of the bolt, e is the end distance, p is the pitch of the fastener along bearing direction, and d_h is the diameter of bolt hole. V_{npb} should be multiplied by a factor 0.7 for over size or short slotted holes and by 0.5 for long slotted holes. The factor k_b takes care of inadequate edge distance or pitch and also prevents bearing failure of bolts. If we adopt a minimum edge distance of 1.5 × bolt hole diameter and a minimum pitch of 2.5 × diameter of bolt, k_b will be approximately 0.50.

The code stipulates that the bolt bearing on any plate subjected to a factored shear force V_{sb}, should satisfy

$$V_{sb} \leq 0.8\ V_{npb} \tag{10.10}$$

Both bolts and plates are subject to significant triaxial containment. Due to this, bearing behaviour of plates is influenced by the proximity of neighbouring holes or boundary (edge distance). Away from holes or boundaries, significant hole elongations commence at a nominal stress of $2f_u$ but failure will occur only at about $3f_u$. Though the presence of threads in the bearing zone increase the flexibility, they do not reduce the bearing strength. Similarly, bolt material often sustains bearing stresses in excess of twice the ultimate tensile . It is not generally necessary to consider bolt bearing in design (Owens & Cheal 1989). Bearing in the thinner plate will control for plate thicknesses up to about one half of the bolt diameter.

Equations (10.1), (10.7), and (10.9), which express the design shear, tensile strength, and bearing strength of a bolt, respectively, can be presented in the form of tables to avoid repeating these calculations. Tables are presented in Appendix D which will aid the designer while designing joints using ordinary bolts.

10.4.4 Prying Forces

Moment resisting beam-to-column connections often contain regions in which the bolts will be required to transfer load by direct tension, such as the upper bolts in the end plate connection as shown in Fig. 10.10. In the design of such connections, we should consider an additional force induced in the bolts as a result of so-called ‘prying action’ (Douty & McGuire 1965, Agerskov 1979, Holmes & Martin 1983, Subramanian 1984). These additional prying forces induced in the bolts are mainly due to the flexibility of connected plates (see Fig. 10.11). Thus, in a simple T-stub connection as shown in Fig. 10.11, the prying force will develop only when the ends of the flanges are in contact due to the external load, as shown in Figs 10.11(b) and (c). The plastic hinges do not always form before bolt failure. The development of prying force as the external load is raised from zero to the maximum in a T-stub connection as shown in Fig. 10.12.

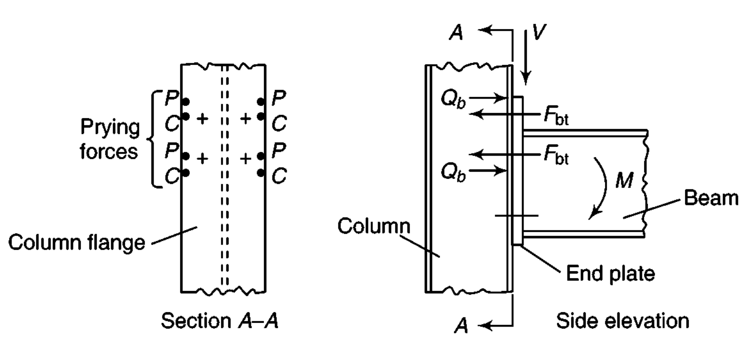


Fig. 10.10 Prying forces in a beam-to-column connection

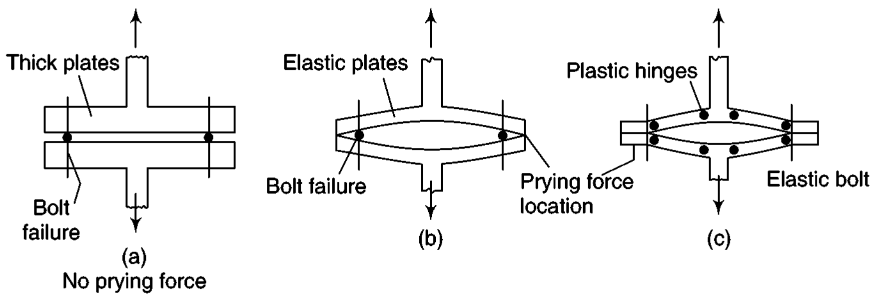


Fig. 10.11 Failure modes due to prying forces

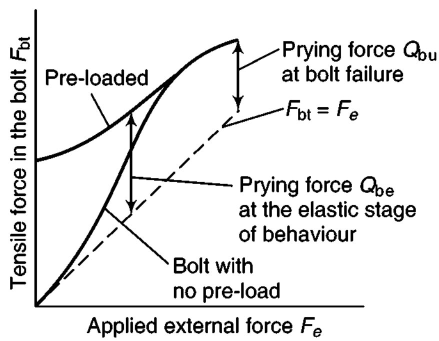


Fig. 10.12 Relationship between external force and bolt force for a T-stub

Several researchers have studied this problem and proposed equations to calculate the prying force developed in the bolt (Astanesh 1985; Kulak et al. 1987; Owens & Cheal 1989; Thornton 1985, 1992). IS 800 has adopted the equation proposed by Owens & Cheal (1989) and the additional force Q in the bolt due to prying action (see Fig. 10.13)

$$Q = \{l_v/2l_e\}\ [T_e - \beta\gamma f_o\ b_e t^4/(27 l_e l_v^2)] \tag{10.11}$$

where l_v is the distance from the bolt centre line to the toe of the fillet weld or to half the root radius of a rolled section in mm and l_e is the distance between prying force and bolt centre line in mm.

This distance is taken as the maximum of either the end distance or the value given by

$$l_e = 1.1t\sqrt{(\beta f_o/f_y)} \tag{10.12}$$

where $\beta = 2$ for non-tensioned bolt and 1 for pre-tensioned bolt, $\gamma = 1.5$, b_e is the effective width of flange per pair of bolts in mm, f_o is the proof stress in consistent units (kN or kN/mm^2), and t is the thickness of the end plate in mm.

The second term in Eqn (10.11) is usually relatively small and hence may be neglected to yield the formula

$$Q = T_e l_v/(2l_e) \tag{10.13}$$

This formula is obtained if plastic hinges are assumed at the bolt line and the root, that is, when minimum flange thickness is used in design.

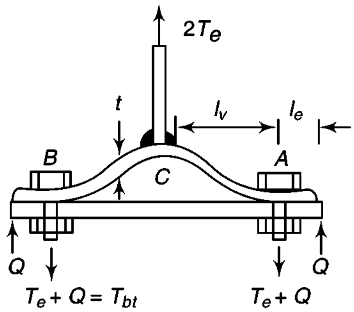


Fig. 10.13 Forces acting in the elastic stage for prying force theory

The maximum thickness of end plate to avoid yielding of the plate is obtained by equating the moment in the plate at the bolt centre line and a distance l_v from it [see Fig. 10.11(c)] to the plastic moment capacity of the plate M_p. Thus we have,

$$M_A = Ql_e \text{ and } M_c = Tl_v - Ql_e \tag{10.14}$$

$$M_A = M_c = Tl_v/2 = M_p \tag{10.15}$$

Taking $M_p = (f_y/1.10)\,(b_e t^2/4)$ (10.16)

The minimum thickness for the end plate can be obtained as

$$t_{\min} = \sqrt{4.40 M_p/(f_y b_e)} \tag{10.17}$$

The corresponding prying force can be obtained as $Q = M_p/l_e$. If the total force in the bolt $(T + Q)$ exceeds the tensile capacity of the bolt, then the thickness of the end plate has to be increased. Example 10.2 illustrates the effect of prying forces.

10.4.5 Bolts with Shear and Tension

Where bolts are subjected to both tension and shear, as in the connections shown in Fig. 10.14, then their combined effect may be conveniently assessed from a suitable interaction diagram. Tests on bolts under shear and tension showed elliptical or circular interaction curves for the ultimate strength of bolts (Chesson et al. 1965, Khalil & Ho 1979). The following equation for the circular interaction curve has been proposed in the code.

$$(V/V_{\text{sd}})^2 + (T_e/T_{\text{nd}})^2 \le 1.0 \tag{10.18}$$

where V is the applied factored shear, V_{sd} is the design shear capacity, T_e is the externally applied factored tension, and T_{nd} is the design tension capacity.

10.4.6 Efficiency of a Joint

Holes are drilled in the plates for the connection with bolts, hence the original strength of the full section is reduced. The joint which causes minimum reduction in strength is said to be more efficient. Thus, for better efficiency, a section should have the least number of holes at the critical section. The efficiency, expressed in percentage, is the ratio of the actual strength of the connection to the gross strength of the connected members. It can also be expressed as

$$\text{Efficiency} = (\text{Strength of joint per pitch length}/\text{Strength of solid plate per pitch length}) \times 100 \tag{10.19}$$

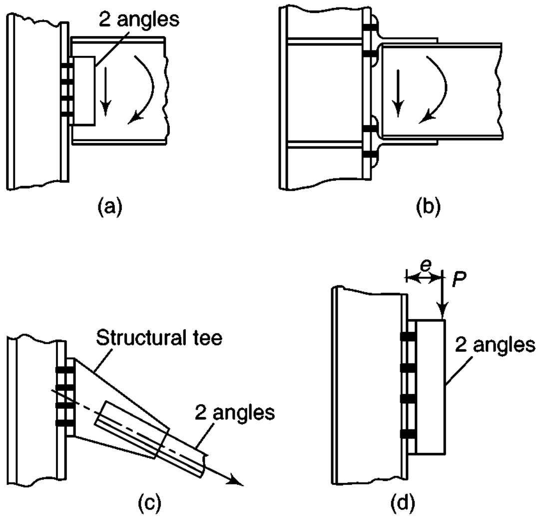


Fig. 10.14 Typical combined shear and tension connections.

10.4.7 Tension Capacity of Plate

The plate in a joint may fail in tension through the weakest section due to the holes. The holes may be arranged in the longitudinal direction of the plate, so that the number of holes is equal in all the rows across the width [see Fig. 10.15(a)], or staggered so that the number of holes across the width is reduced. In the first case, the plate will fail across the weakest section, whereas in the second case the failure is along a zigzag pattern. The tension capacity T_{nd} of the plate is expressed as

$$T_{nd} = 0.72 f_u A_n \tag{10.20a}$$

where f_u is the ultimate stress of material in MPa, and A_n is the net effective area of the plate in mm^2.

Thus, the load carrying capacity of the plate depends on the net effective area of the plate, which in turn depends on the arrangement of the holes. If the holes are not-staggered, the net area A_n can be easily computed as

$$A_n = (b - nd_h)t \tag{10.20b}$$

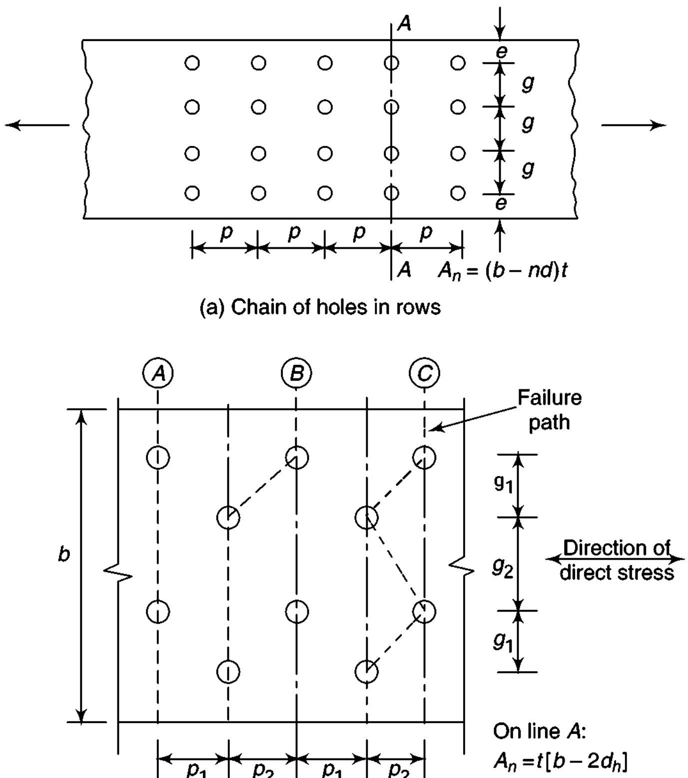
where b is the width of the plate in mm, n is the number of holes along the width b, perpendicular to the direction of load, d_h is the diameter of the hole in mm, and t is the thickness of the plate in mm.

Based on experimental evidence, a simplified empirical relationship has been proposed by Cochrane (1922) for staggered rows of holes [see Fig. 10.15 (b)] as

$$A_n = \left[b - nd_h + \sum_{i=1}^{m} p_{si}^2/(4g_i) \right] t \tag{10.20c}$$

where A_n, b, t, d_h are as defined earlier, p_s is the staggered pitch in mm, g is the gauge distance in mm, n is the number of holes in the zigzag failure path, and m is the number of staggered pitches or gauges along failure path.

All possible failure paths (straight as well as zigzag) are to be tried as shown in Fig. 10.15(b) and the corresponding net areas are to be computed as per Eqns (10.20b) and (10.20c), to find the minimum net area of the plate. If the tensile load on the plate is more than its tensile strength, the plate fails in tension through rupture.



On line A:
$A_n = t[b - 2d_h]$
On line B:
$A_n = t[b - 3d_h + 0.25p_2^2/g_1]$
On line C:
$A_n = t[b - 4d_h + 0.5p_2^2/g_1 + 0.25p_2^2/g_2]$
where d_h is the hole diameter

Fig. 10.15 Tension capacity of plates

10.5 Design Strength of High Strength Friction Grip Bolts

As we have seen already, HSFG bolts are used when forces are large, where space for the connection is limited, where erection cost can be reduced by using fewer bolts or where the structures are subjected to dynamic loads. Thus, they provide 'rigid' fatigue resistant joints.

It may be noted that HSFG bolts may be subdivided into parallel shank and waisted shank types. A parallel shank bolt, which is the most commonly used (and discussed in this section), is designed not to slip at serviceability load; but slips into bearing at ultimate load. Thus, only when the externally applied load exceeds the frictional resistance between the plates, the plates slip and the bolts bear against the bolt holes. A waisted shank bolt has higher strength and is designed not to slip both at service and ultimate load. Hence, waisted shank HSFG bolts are more rigid at ultimate load and need not be checked for bearing or long joint capacity (BS 5950, Martin & Purkiss 1992).

10.5.1 Slip Resistance

As mentioned earlier, the initial pretension in bolt develops clamping forces at the interface of elements being joined [see Fig. 10.8(b)]. The frictional resistance to slip between the plate surfaces subjected to clamping force, opposes slip due to externally applied shear.

The design slip resistance or nominal shear capacity of a bolt V_{nsf} of the parallel shank and waisted shank friction grip bolts is given by the code as

$$V_{nsf} = \mu_f n_e K_h F_o \tag{10.21}$$

where μ_f is the coefficient of friction (called as slip factor) as specified in Table 10.7 ($\mu_f \leq 0.55$); n_e is the number of effective interfaces offering frictional resistance to slip; K_h = 1.0 for fasteners in clearance holes, 0.85 for fasteners in oversized and short slotted holes and for fasteners in long slotted holes loaded perpendicular to the slot, and 0.7 for fasteners in long slotted holes loaded parallel to the slot; F_o is the minimum bolt tension (proof load) at installation and may be taken as $A_{nb} f_o$, A_{nb} is the net area of the bolt at the threads; f_o is the proof stress, taken as $0.7 f_{ub}$; and f_{ub} is the ultimate tensile stress of bolt.

Table 10.7 Typical average values for coefficient of friction (μ_f)

Treatment of surface	Coefficient of friction (μ_f)	Treatment of surface	Coefficient of friction (μ_f)
Surfaces not treated	0.20	Surfaces blasted with shot or grit and painted with ethylzinc silicate coat (thickness 60–80 μm)	0.30
Surfaces blasted with shot or grit with any loose rust removed, no pitting	0.50	Surfaces blasted with shot or grit and painted with alkalizinc silicate coat (thickness 60–80 μm)	0.30
Surfaces blasted with shot or grit and hot-dip galvanized or red lead painted surface	0.10	Surface blasted with shot or grit and spray-metallized with aluminium (thickness > 50 μm)	0.50
Surfaces blasted with shot or grit and spray-metallized with zinc (thickness 50–70 μm)	0.25	Clean mill scale	0.33

IS 800 stipulates that for a bolt subjected only to a factored design force V_{sf}, in the interface of connections at which slip cannot be tolerated, will satisfy the following

$$V_{sf} \leq V_{nsf}/\gamma_{mf} \tag{10.22}$$

where $\gamma_{mf} = 1.10$ if slip resistance is designed at service load and $\gamma_{mf} = 1.25$ if slip resistance is designed at ultimate load.

It may be noted that the resistance of a friction grip connection to slip in service is a serviceability criterion, but for ease of use it is presented in the code in a modified form, suitable for checking under factored loads.

10.5.2 Long Joints

Similar to black bolts, the design slip resistance V_{nsf} for parallel shank friction grip bolts is reduced for long joints by a factor β_{lj} given by Eqn (10.4)

It should be understood that overcoming slip does not imply that a failure mode has been reached. However, when connections are subjected to stress reversal, there is great concern regarding any slip at service load. Repeated loading may introduce fatigue concerns, if slip is occurring, especially when oversized or slotted holes are used. Generally, slip resistance governs the number of bolts used in slip-critical connections, rather than strength in bearing or shear. However, the bearing related equations for spacing of fasteners and end distance will result in smaller spacings and end distances.

10.5.3 Bearing Resistance

As a parallel shank friction grip bolt slips into bearing at ultimate limit state when subject to shear forces, the bearing stresses between the bolt and the plate need to be checked. The bolt may deform due to high local bearing stresses between the bolt and plate and the design bearing capacity of the bolt V_{npb} is obtained by using Eqn (10.9). The code stipulates that the factored shear force V_{sf} should satisfy Eqn (10.10).

Note that while checking black bolts, the ultimate tensile capacity of the bolt or the plate, whichever is smaller is used. Since the bearing strength of HSFG bolts will be greater than the plates, no check on bearing strength of bolt is necessary.

An alternative mode of failure is that of the bolt shearing through the end of the plate as shown in Fig. 10.7(b). This may be controlled by specifying the end distances and pitches. The block shear resistance of the edge distance due to bearing force should also be checked for the connection (see Section 2.5.3 and by the k_b factor of Eqn (10.9) for details).

10.5.4 Tension Resistance

The design tensile strength of parallel shank and waisted shank friction grip bolts is similar to that of black bolts (see Section 10.4.2) and is given by Eqn (10.7).

As per the code, the HSFG bolt subjected to a factored tension force T_b should satisfy Eqn (10.8).

The effect of the prying force Q has been shown in Fig. 10.12. When the external load is applied, part of the load (approximately 10%) of the load is equilibrated by the increase in bolt force. The balance of the force is equilibrated by the reduction in contact between the plates. This process continues and the contact between the plates is maintained until the contact force due to pre-tensioning is reduced to zero by the externally applied load. Normally, the design is done such that the externally applied tension does not exceed this level. After the external force exceeds this level, the behaviour of the bolt under tension is exactly similar to that of a bearing type of bolt. Eqns (10.11), (10.12), and (10.17) may be used in the calculation of HSFG bolts subjected to prying forces.

10.5.5 Combined Shear and Tension

The interaction curve suggested for combined shear and tension for HSFG bolts (for which slip in the serviceability limit state is limited) is similar to that of black bolts and is given below

$$(V_{sf}/V_{sdf})^2 + (T_f/T_{ndf})^2 \leq 1.0 \tag{10.23}$$

where V_{sf} is the applied shear at service load, V_{sdf} is the design shear strength, T_f is the externally applied tension at service load, and T_{ndf} is the design tension strength.

Since slip resistance is a service load consideration, the numerator terms of Eqn (10.23) are service loads T and V (tension and shear per bolt). It should be observed that any external tension will produce a corresponding reduction in the clamping force between the plies. Until the external load on a bolt exceeds the pre-compression force between the pieces, the tension force in the bolt will not change significantly from its initial tension (see Fig. 10.12).

The design shear, bearing, and tensile resistance of HSFG bolts can be presented in the form of a table, as shown in Appendix D, to avoid repeated calculations. Figure 10.16 shows a bracing member connected to the other members of a bridge structure using guest plates and HSFG bolts.

10.5.6 Block Shear Failure

As a result of some research work carried out in USA, it was found that angle, gusset plate, and coped beams connections may fail as a result of block shear (Kulak & Grondin 2000). Failure occurs in shear at a row of bolt holes parallel to the applied loads, accompanied by tensile rupture along a perpendicular face. This type of failure results in a block of material being torn out by the applied shear force as shown in Fig. 10.17. The block shear strength T_{db} of a connection is taken as the smaller of

$$T_{db1} = [0.525A_{vg}f_y + 0.72f_uA_{tn}] \tag{10.24a}$$

and

$$T_{db2} = [0.416f_uA_{vn} + 0.909\, f_yA_{tg}] \tag{10.24b}$$

Fig. 10.16 Example of a connection using HSFG bolts

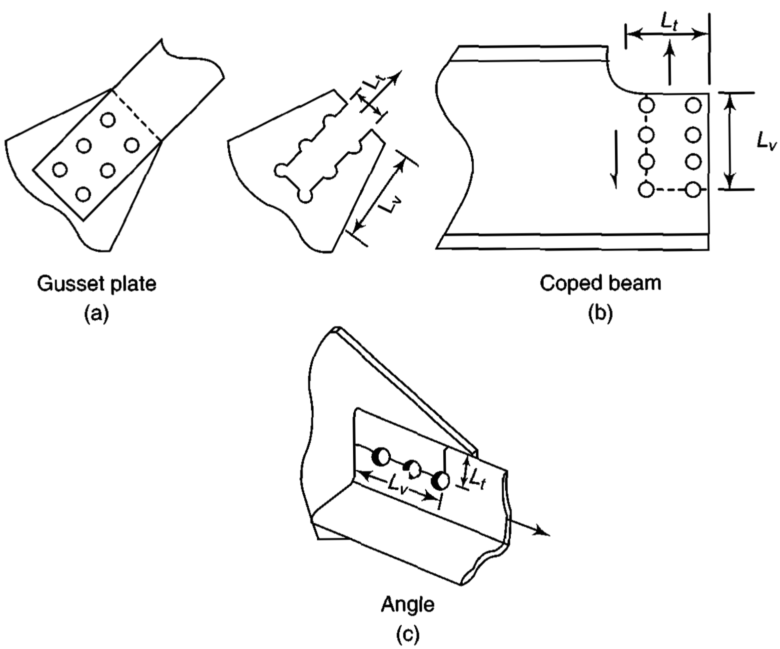


Fig. 10.17 Examples of block shear failure

where A_{vg} and A_{vn} are the minimum gross and net area, respectively, in shear along a line of transmitted force (along L_v in Fig. 10.17); A_{tg} and A_{tn} are the minimum gross and net area, respectively, in tension from the hole to the toe of the angle or next last row of bolt in gusset plates (along L_t in Fig. 10.17); and f_u and f_y are the ultimate and yield stress of the material, respectively.

10.6 Simple Connections

In many cases, a connection is required to transmit a force only and there may not be any moment acting on the group of connectors, even though the connection may be capable of transmitting some amount of moment. Such a connection is referred to as a *simple*, *force*, *pinned*, or *flexible connection.*

As already shown in Fig. 10.8, two types of load transfers occur in these connections. In the first, the force acts in the connection plane (formed by the interface between the two connected plates) and the fasteners between these plates act in shear [Fig. 10.8(a)]. In the second type, the force acts out of the plane of the connection and the fasteners act in tension as shown in Fig. 10.8(c). In practice there will always be some eccentricity and the moment due to this small eccentricity is ignored. The different types of simple connections found in steel structures may be classified as follows:

- Lap and butt joints
- Truss joint connections
- Connections at beam column junctions
  - Seat angle connection
  - Web angle connection
  - Stiffened seat angle connection
  - Header plate connection
- Tension and flange splices

Let us now discuss these connections briefly.

10.6.1 Lap and Butt Joints

Lap and butt joints are often used to connect plates or members composed of plate elements. Though lap joints are the simplest, they result in eccentricity of the applied loads. Butt joints on the other hand eliminate eccentricity at the connection.

10.6.1.1 Lap joints

When two members which are to be connected are simply overlapped and connected together by means of bolts or welds, the joint is called a lap joint [see Figs 10.18(a)–(d)]. A single bolted lap joint and a double bolted lap joint are shown in Figs 10.18(b) and 10.18(c), respectively. The drawback of such a lap joint is that the centre of gravity of load in one member and the centre of gravity of load in the second member do not coincide and hence an eccentricity as shown in Fig. 10.18(d) is created. Due to this a couple $P \times e$ is formed, which causes undesirable bending in the connection leading to failure of bolts in tension. To minimize the effect of bending in a lap joint, at least two bolts in a line should be provided. Moreover, due to the eccentricity, the stresses are distributed unevenly across the contact area between the bolts and members to be connected. Hence, the use of lap joints is not often recommended. The design of lap joint is illustrated in Examples 10.1, 10.3, 10.4, and 10.5.

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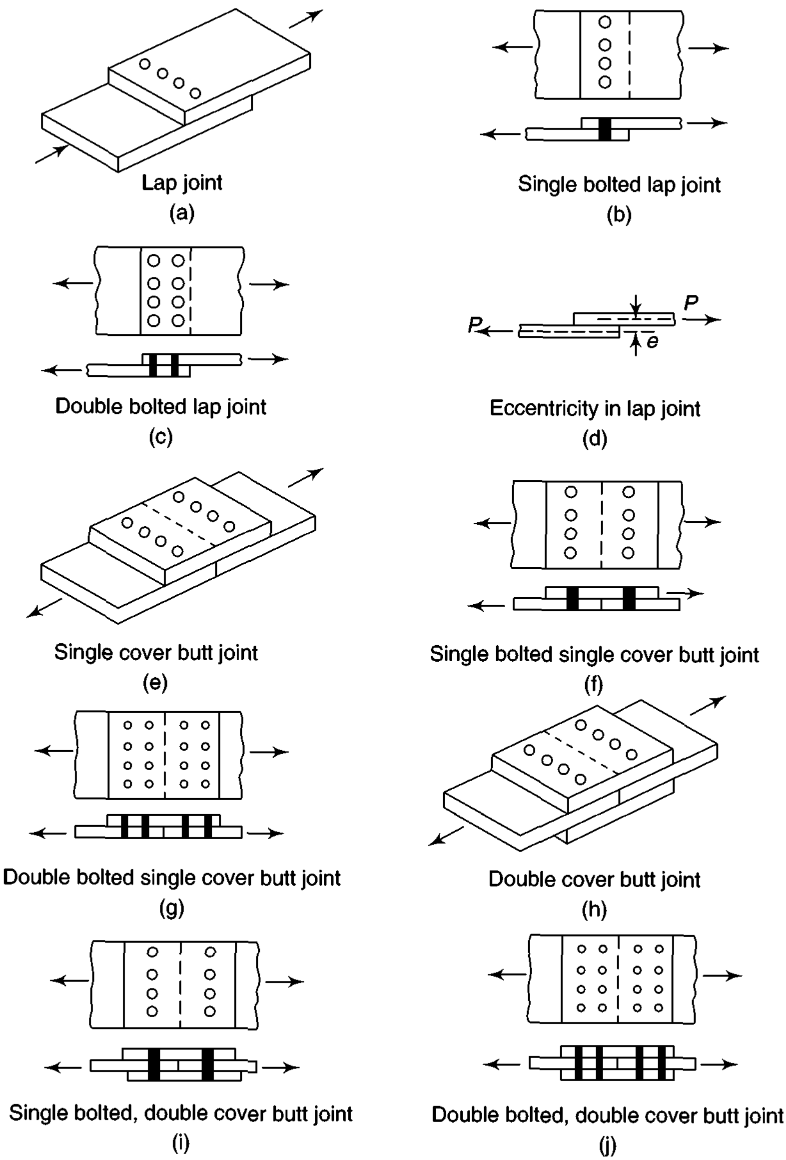


Fig. 10.18 Lap and butt joints

10.6.1.2 Butt Joints

In butt joints, the members to be connected are placed against each other and are bolted or welded together through the use of additional plates, called *cover plates*. The cover plates may be provided on either one or both sides of the connection as shown in Figs 10.18(e)–(j). If the cover plate is provided on one side of the joint only it is called as a *single cover butt joint* [see Figs 10.18(e)–(g)] and when provided on both sides of the joint, it is called as a *double cover butt joint* [see Figs

CHAPTER 11

Welded Connections

Introduction

Welding is a method of connecting two pieces of metal by heating to a plastic or fluid state (with or without pressure), so that fusion occurs. Welding is one of the oldest and reliable methods of jointing. Little progress in welding technology, as is known now, was made until 1877, though welding processes such as forge welding and brazing were known for at least 3000 years. Although there had been initial work on arc welding in the 1700s using carbon electrodes powered by batteries, development work intensified between 1880 and 1900 with the availability of electric generators to replace batteries. Professor Elihu Thompson was the first to patent the first resistance groove welding machine in 1885. Charles Coffin invented the metal arc process and patented it in USA in 1892, though Zerner introduced the carbon arc welding process in 1885. The concept of coated metal electrodes, which eliminated the problems associated with the use of bare electrodes, was introduced in 1889 by A.P. Strohmeyer. The metal arc process was first used (in 1889) in Russia by using uncoated, bare electrodes (Salmon & Johnson 1996).

Oxyacetylene welding and cutting was employed after 1903, due to the development of acetylene torches by Fouche and Picard. By the early 1900s, Lincoln Electric offered the first arc welding machine and by 1912 covered electrodes were patented. During World War I (1914–1918), welding techniques were used for repairing damaged ships. During the period 1930–1950, several improvements and techniques such as the use of granular flux to protect the weld and submerged arc welding were developed.

Today there are several welding processes available to join various metals and their alloys. The types of welds and welded joints and the advantages of using welding over bolts or rivets are also discussed. The behaviour and design of various welded connections are also outlined. A brief review of the methods of joining tubular connections is given. Several examples are given to illustrate the design procedures adopted for welded connections. This chapter concludes with the recent developments in the design of joints to resist earthquake loads.

11.1 Welding Processes

Structural welding is nearly all electric; though some gas welding may also be used [‘gas’ denotes the use of a gas (usually acetylene/oxygen mixture) to produce a very hot flame to heat the parts and the weld filler material]. However, gas is used primarily for cutting pieces to shape. It is now possible to cut metals using mechanically controlled gas cutting equipment in fabrication shop, which results in smooth cuts similar to sawed cuts. Though gas welding is simple and inexpensive, it is slow and hence it is generally used for repair and maintenance work only.

In the most common welding processes of welding structural steel, electric energy is used as the heat source. Electric welding involves passing either direct or alternating current (mostly direct current is used) through an electrode (commonly the electrode is the anode and the operation uses ‘reversed polarity’). By holding the electrode at a very short distance from the base metal, which is connected to one side of the circuit, an arc forms as the circuit is essentially ‘shorted’ [see Fig. 11.1(a)]. With this ‘shorting’ of the circuit, a very large current flow takes place, which melts the electrode’s tip (at the arc) and the base metal in the vicinity of the arc. A temperature of about 3300–5000°C is produced in the arc. The electron flow making the circuit ‘carries the molten electrode metal’ to the base metal to build up the joint. The parameters that control the quality of weld are the electrode size and the current that produces sufficient heat to melt the base metal and minimizes electrode splatter.

The different processes of arc welding that are used in structural steel applications are as follows:

- Shielded metal arc welding (SMAW)
- Submerged arc welding (SAW)
- Gas-shielded metal arc welding (GMAW)
- Flux core arc welding (FCAW)
- Electro slag welding (ESW)
- Stud welding (SW)

Details of these processes may be found in Galvery & Mavlow (2001), Jeffus (2002) and Subramaniam (2008).

11.1.1 Shielded Metal Arc Welding (SMAW)

Shielded Metal Arc Welding (SMAW) also called ‘stick’ welding is a manual process and is the most common method of welding used in structural connections owing to low capital cost and flexibility. However, for long continuous welds automatic processes are preferred due to the consistent quality. The SMAW processes require the following set up (see Fig. 11.1(a)):

(a) Constant-current (CC) welding power supply
(b) Electrode holder, lead, and its terminals
(c) Ground clamp, lead, and its terminals
(d) Welding electrodes

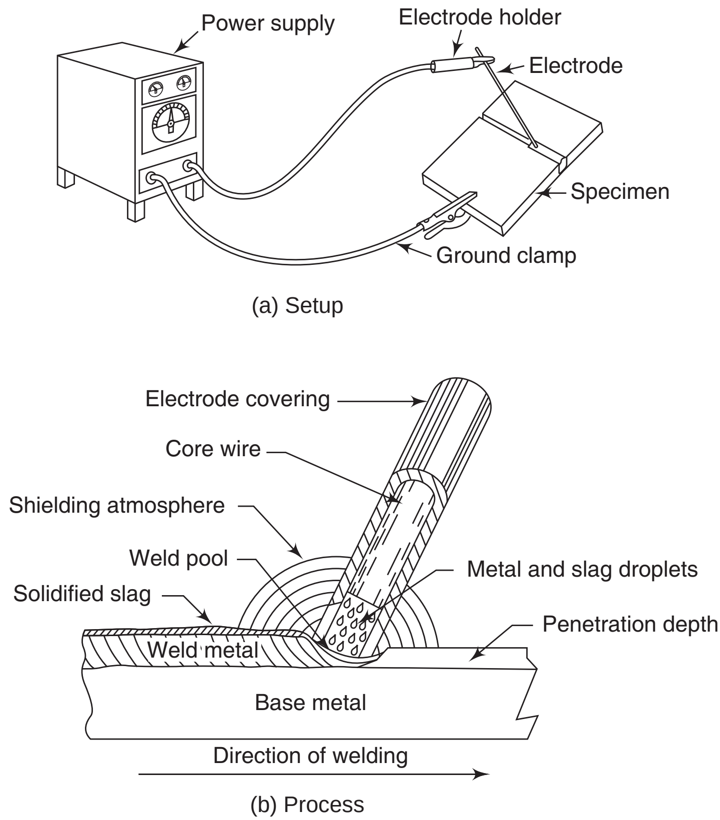


Fig. 11.1 Shielded metal arc welding

As stated earlier, the electrons flowing through the gap between the electrode and the metal produce an arc that furnishes the heat to melt both the electrode metal and the base metal. Temperatures within the arc exceed 3300°C. The arc heats both the electrode and the metal beneath it. Tiny globules of metal form at the tip of the electrode and are transferred to the molten weld pool on the base metal. As the electrode moves away from the molten pool, the molten mixture of electrode and base metal solidifies and the weld is completed (see Fig. 11.1b).

Generally the electrode is stronger than the parent metal. For example, an E41 electrode, which would be used to weld grade 410 steel, gives a weld deposit which has a maximum yield strength of 330 MPa with a tensile strength in the range of 410–510 MPa (see Table 11.1). For *manual metal arc welding (MMA),* the electrodes should comply with IS 2879, IS 1395, and IS 814.

The electrodes are available in lengths of 225–450 mm, and diameters ranging from 3.2 to 6 mm. The maximum size of weld produced in one pass is about 8 mm (Bowles 1980).

Table 11.1 Tensile properties of electrodes

Specification	Grade/Classification	Yield stress, MPa (Min)	Properties Ultimate tensile stress, MPa, (Min)	Elongation percentage (Min)
	Ex40xx	330	410–510	16
	Ex41xx	330	410–510	20
	Ex42xx	330	410–510	22
IS 814 : 2004	Ex43xx	330	410–510	24
	Ex44xx	330	410–510	24
Specification for	Ex50xx	360	510–610	16
covered electrodes	Ex51xx	360	510–610	18
for manual metal	Ex52xx	360	510–610	18
arc welding of	Ex53xx	360	510–610	20
carbon and carbon	Ex54xx	360	510–610	20
manganese steel	Ex55xx	360	510–610	20
	Ex56xx	360	510–610	20

11.1.2 Choice of the Process

One of the welding processes is selected for a particular application, based on the following parameters.

(a) *Location of the welding operation* If welding is done in a fabrication shop, SAW, GMAW, FCAW, and ESW can be used. For field applications SMAW is preferred.
(b) *Accuracy of setting up* SAW, spray transfer GMAW, and ESW require accurate set-up.
(c) *Penetration of weld* Penetration of FCAW and SAW is better than SMAW.
(d) *Volume of weld to be deposited* FCAW, GMAW, and ESW have high deposition rates.
(e) *Position of welding* SAW and ESW are not suitable for overhead positions. FCAW and GMAW can be used in all positions. SMAW is probably the best for overhead works, especially at site.
(f) *Access to joint* In easily accessible joints SAW and GMAW are used. In cramped joints SMAW is used.
(g) *Steel composition* GMAW and SAW are less likely to lead to heat-affected zone (HAZ) cracking.
(h) Thickness of connecting parts.
(i) Comparative cost.

11.2 Welding Electrodes

As stated earlier, a variety of electrodes are available so that a proper match of base metal strength and metallurgical properties to the weld metal can be chosen. Only coated electrodes are used in structural welding. Welding electrodes are classified (by the American Welding Society, in cooperation with ANSI) using the following numbering system for shielded metal arc welding (SMAW):

Exxxbc

In this numbering system, E stands for electrodes. xxx stands for two or three-digit number establishing the ultimate tensile strength of the weld metal. As per IS 814 the following values are available: 40, 41, 42, 43, 44, 50, 51, 53, 54, 55, 56 kg/cm^2. The value of b indicates the suitability of welding positions, which may be flat, horizontal, vertical, and overhead, that is, b = 1 denotes suitability for all positions, 2 denotes suitability for flat positioning of work, 4 denotes suitability for flat, horizontal, overhead, and vertical down. 'c' stands for coating and operating characteristics. The value of c equal to 5, 6, 8 indicates low hydrogen.

The various grades of electrodes as per IS 814 and their tensile properties are shown in Table 11.1.

11.3 Advantages of Welding

Welding offers the following advantages over bolting or riveting.

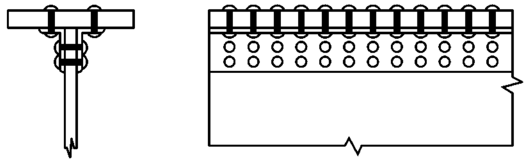
(a) Welded connections eliminate the need for making holes in the members, except for a few employed for erection purposes. Since the holes at the ends govern the design of bolted connections (edge distance), a welded connection results in a member with a smaller gross section. This has a greater influence in the case of tension members, since the calculation of net section is eliminated.

(b) Welding offers airtight and watertight jointing of plates and hence is employed in the construction of water/oil storage tanks, ships, etc.

(c) Welded joints are economical, since they enable direct transfer of stresses between the members. Moreover, the splice plates and bolt material are eliminated. The required size of gusset plates is also smaller, because of reduced connection length. Due to the elimination of operations such as drilling and punching, welding results in less fabrication costs. In addition, due to the simple design details, time is also saved in detailing, fabrication, and field erection. Welding also requires considerably less labour for executing the work. It is estimated that the total overall savings by employing welding over bolting may be up to 15%.

(d) Welded structures are more rigid (due to the direct connection of members by welding) as compared to bolted joints. In bolted joints, the cover plates, connecting angle, etc. may deflect with the member during load transfer thus making a structure flexible. Rigid structures are always more economical than flexible structures, due to the transfer of moments from one member to another.

(e) Welded connections are usually aesthetic in appearance and appear less cluttered in contrast to bolted connections. This is evident from Fig. 11.2, which shows a bolted and welded plate girder.



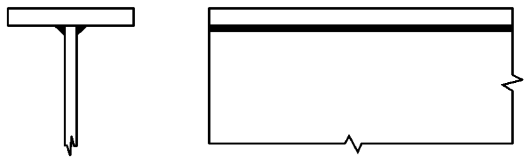


Fig. 11.2 Appearance of bolted and welded plate girders

(f) Welding offers more freedom to the designer in choosing sections. The designer is not bound by the available rolled sections, but may build up any cross section, which may be economical and advantageous. Welding has resulted in the innovation of open web joists, castellated beams, tapered beams, vierendeel trusses, composite construction, tubular trusses, and offshore platforms.

(g) Welding is practicable even for complicated shapes of joints. For example, connections with tubular sections can be made easily by welding, whereas it is difficult to make them using bolting. Tubular sections are structurally economical as compression members and their use in trusses is feasible due to welding.

(h) Alterations can be made with less expense in case of welding as compared to bolting. It is also easy to correct mistakes in fabrication during erection, whereas a mismatch of holes in a bolted connection is very difficult to correct. Also members can be shortened by cutting and rejoined by suitable welding. In the same way, members can be lengthened by splicing a piece of the same cross section.

(i) A truly continuous structure is formed by the process of fusing the members together. This gives the appearance of a one-piece integrated structure. Usually, the strength of a welded joint is as strong as or stronger than the base metal, thus there are no restrictions in the placement of joints.

(j) The efficiency of a welded joint is more than a bolted joint. In fact 100% efficiency can be obtained using welding.

(k) Due to the elimination of holes, stress concentration effect is considerably less in welded connections.

(l) The process of welding is relatively silent compared to riveting and bolting (drilling holes) and requires less safety precautions.

However, welding has the following disadvantages.

(a) Welding requires highly skilled human resources.

(b) The inspection of welded joints is difficult and expensive, whereas inspection of bolted joints is simple. Moreover, non-destructive testing is required in important structures.
(c) Members jointed by welding may distort, unless proper precautions are taken. Welded joints have large residual stresses.
(d) Costly equipment is necessary to make welded connections.
(e) Welded connections are prone to cracking under fatigue loading.
(f) Proper welding may not be done in field conditions, especially in vertical and overhead positions.
(g) The possibility of brittle fracture is more in the case of welded joints than in bolted connections.
(h) The welding performed in the field is expensive than performed in the shop.
(i) Welding at the site may not be feasible due to lack of power supply.

Several factors influence the welding cost, which include the following (Salmon & Johnson 1996):
(a) Cost of preparing the edges to be welded (in case of groove welds)
(b) Amount of weld material required
(c) Ratio of the actual arc time to overall welding time
(d) The handling required (cranes and special equipment needed during erection)
(e) General over head costs
(f) Cost of pre-heating, if any

11.4 Types and Properties of Welds

The welds may be grouped into four types as follows:
(a) Groove welds
(b) Fillet welds
(c) Slot welds
(d) Plug welds

These are shown in Fig. 11.3. Each type of weld has its own advantage and may be selected depending on the situation. It has been found that fillet welds are used extensively (about 80%) followed by groove welds (15%). Slot and plug welds are used rarely (less then 5%) in structural engineering applications. Fillet welds are suitable for lap and T-joints (see Section 11.5) and groove welds are suitable for butt, corner, and edge joints. Each of these four types of welds are discussed further in the following sections.

11.4.1 Groove Welds

Groove welds are used to connect structural members that are aligned in the same plane and often used in butt joints. Groove welds may also be used in T-connections. The grooves have a slope of 30°–60°. Edge preparation becomes necessary for plates over 10-mm thick for manual arc welding, and over 16-mm thick for automatic welding. Various types of groove welds are shown in Fig. 11.4. The

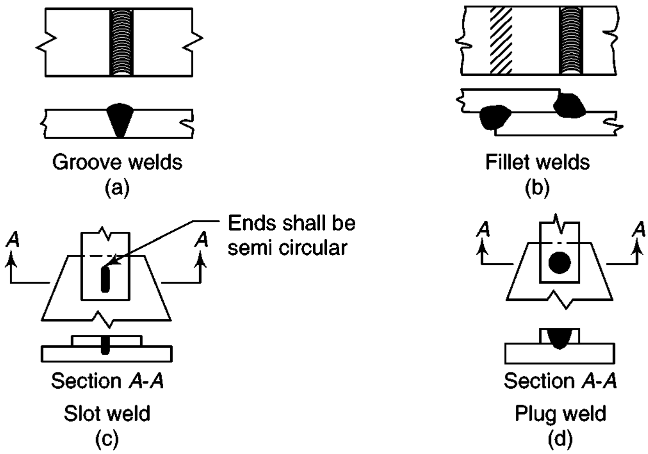


Fig. 11.3 Types of welds

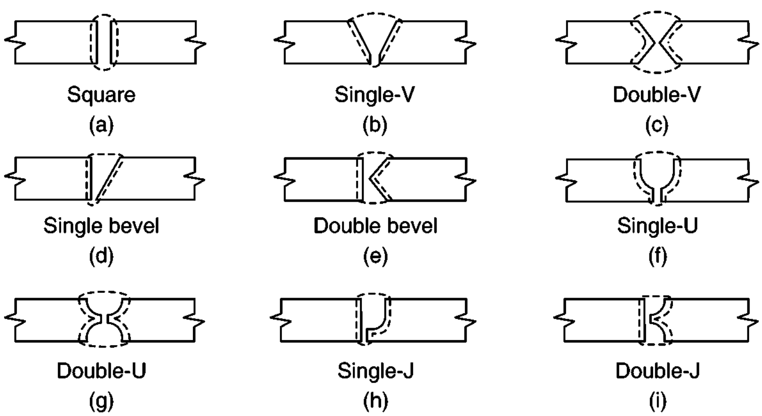


Fig. 11.4 Types of groove welds

square groove weld is used to connect plates up to 8-mm thickness. The terms that are associated with a completed groove weld are shown in Fig. 11.5. Partial penetration groove welds should not be used especially in fatigue situations.

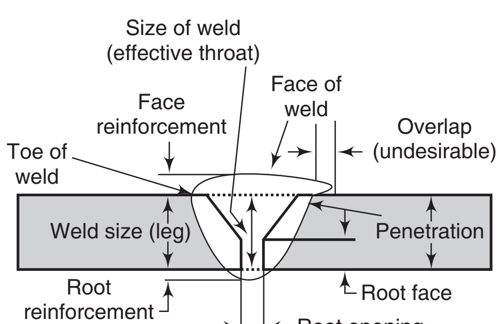


Fig. 11.5 Terms used to describe the parts of a groove weld

To ensure full penetration and a sound weld, a back-up strip is provided at the bottom of single-V/bevel/J or U grooves. Thus, the back-up strips are commonly used when all welding is done from one side or when the root opening is excessive (see Fig. 11.6). The back-up strip introduces a crevice into the weld geometry and prevents the problem of burn-through. The back-up strip can be left in place or removed after welding the pieces.

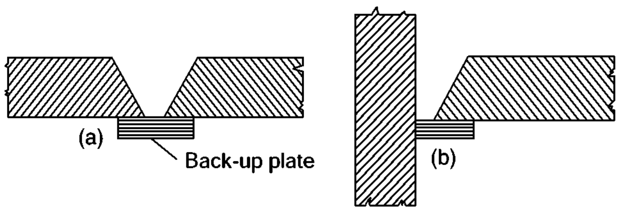


Fig. 11.6 Use of back-up plate or spacer in groove weld

For a groove weld, the root opening or gap (see Fig. 11.5), is provided for the electrode to access the base of the joint. The smaller the root opening, the greater will be the angle of the bevel (for root openings of 3 mm, 6 mm, and 9 mm, angles of 60°, 45°, and 30°, respectively, may be chosen).

The choice between single or double penetration depends on access on both sides, the thickness of the plate, the type of welding equipment, the position of the weld, and the means by which the distortion is controlled.

Since weld metal is expensive compared to the base metal, the groove is made of double-bevel or double-V for plates of thickness more than 12 mm, and made of double-U or double-J for plates of thickness more than 40 mm. For plates between 12–40 mm, single-J and single-U grooves may be used.

Since groove welds will transmit the full load of the members they join, they should have the same strength as the members they join. Hence, only full penetration groove welds are often used.

11.4.2 Fillet Welds

Fillet welds are most widely used due to their economy, ease of fabrication, and adoptability at site. They are approximately triangular in cross section and a few examples of application of fillet weld are shown in Fig. 11.7. Unlike groove welds, they require less precision in 'fitting up' two sections, due to the overlapping of pieces. Hence, they are adopted in field as well as shop welding. Since they do not require any edge preparation (edge conditions resulting from flame cutting or shear cutting procedures are generally adequate), they are cheaper than groove welds.

In connections, members generally intersect at right angles, but intersection angles between 60° and 120° can be used, provided the correct throat size is used in design calculations (see Section 11.9.2). Fillet welds are assumed to fail in shear.

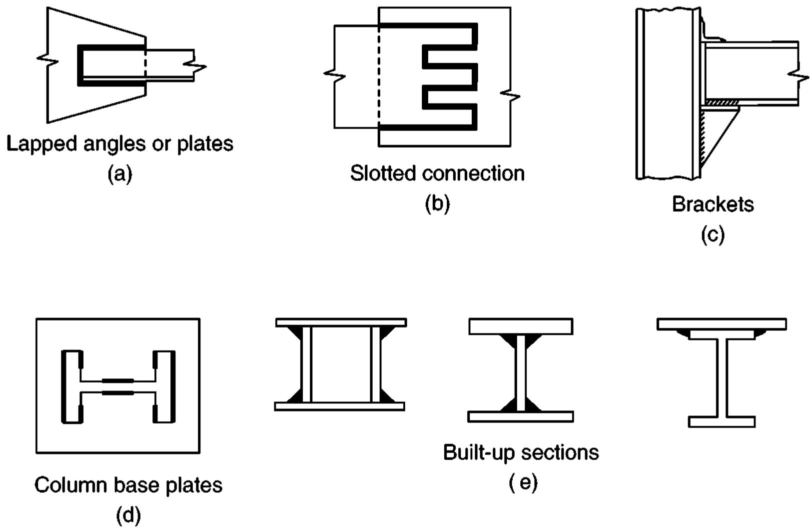


Fig. 11.7 Typical uses of fillet welds

11.4.3 Slot and Plug Welds

Slot and plug welds are not used exclusively in steel construction. When it becomes impossible to use fillet welds or when the length of the fillet weld is limited, slot and plug welds are used to supplement the fillet welds. They are also assumed to fail in shear. Thus, their design strength is similar to that of fillet welds.

11.4.4 Structure and Properties of Weld Metal

The weld metal is a mixture of parent metal and steel melted from the electrode. The solidified weld metal has properties characteristic of cast steel. Hence, it has higher yield to ultimate ratio but lower ductility compared to structural steel. When the weld pool is cooling and solidifying, the parent metal along side the joint is subjected to heating and cooling cycles and the metallurgical structure of this steel in this region will be changed. This region is called the *heat-affected zone* (HAZ).

The change in structure in HAZ should be considered in the design stage by selecting a suitable Charpy V-impact value for the (see Section 1.8.5) welding electrode (its Charpy impact value should be equal to or greater than that specified for the parent metal), corrosion resistance, etc. Pre-heating of joints will also help to reduce HAZ cracks. However, pre-heating increases the cost of welding.

Charts for finding the required pre-heat temperature are provided by Blodgett (1966). In critical cases, 'pre-heat' is maintained for a considerable period of time after welding.

11.4.5 Weld Defects

The production of sound welds is governed by the type of joint, its preparation and fit-up, the root opening, etc. In addition to this, the choice of electrode, the welding position, the welding current and voltage, the arc length, and the rate of travel also affect the quality of weld (Gaylord et al. 1992). Accessibility of the welding operation is also important, since the quality of weld is determined to a considerable extent by the positioning of the electrode. Some of the common defects in the welds are as follows:

(a) Incomplete fusion
(b) Incomplete penetration
(c) Porosity
(d) Inclusion of slag
(e) Cracks
(f) Undercutting
(g) Lamellar tearing

These defects are shown schematically in Fig. 11.8. For more details about these defects and the methods to eliminate them, refer Blodgett (1966) and Jeffus (2002).

Lamellar tearing is discussed in Section 1.8.6 also.

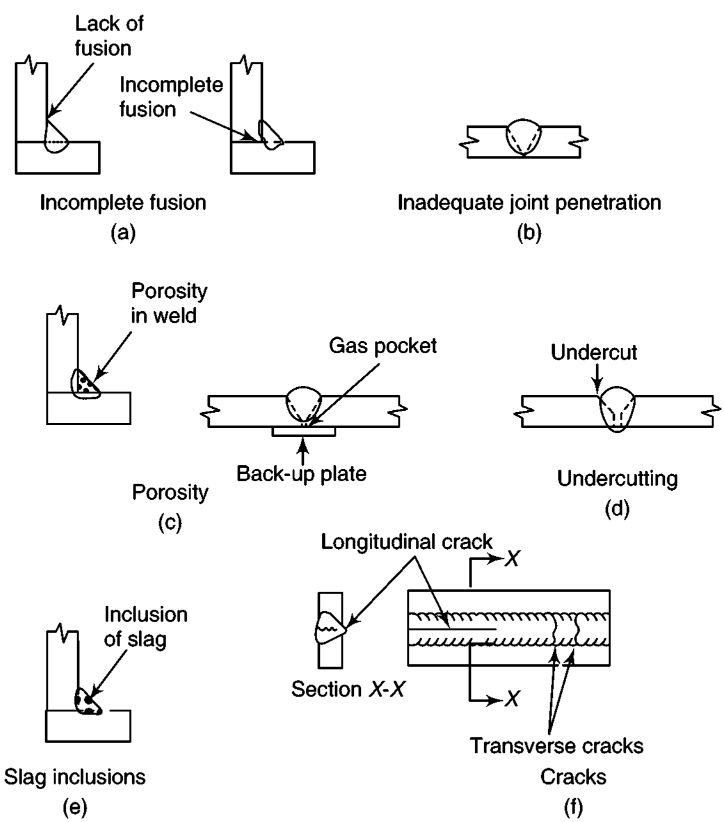


Fig. 11.8 Possible weld defects

Since a small error in a weld may lead to a catastrophic collapse, checks are to be made before, during, and after welding (Blodgett 1966).

In addition, a qualified welder, who knows the weld qualification procedures, should be employed to execute the job. Visual inspection (which is dependent on the competence of the observer) and non-destructive tests (may be employed for important structures) should be used to determine the type and distribution of weld defects (Gaylord et al. 1992). Any poor or suspicious weld should be cut and replaced. A welding gauge may be used to rapidly check the size of the fillet welds. The non-destructive tests usually employed include the following:

(a) Liquid penetrant inspection,
(b) Magnetic particle inspection,
(c) Radiographic inspection, and
(d) Ultrasonic inspection.

Failure of the King's Bridge, Australia

King's bridge in Melbourne, Australia, failed while in service on 10th July 1962 (Melbourne's winter) due to brittle fracture, when a 45-ton vehicle was passing over it. This plate girder bridge consisted of four plate girders, spanning 30 m and topped with reinforced concrete deck slab. Each plate girder's bottom flange was made of high-strength 400 × 19 mm plate, supplemented in the region of high bending moment by cover plates of size 300 × 19 mm or 360 × 12 mm. The cover plates were attached to the flange by a 5 mm fillet weld all round as shown in the figure below.

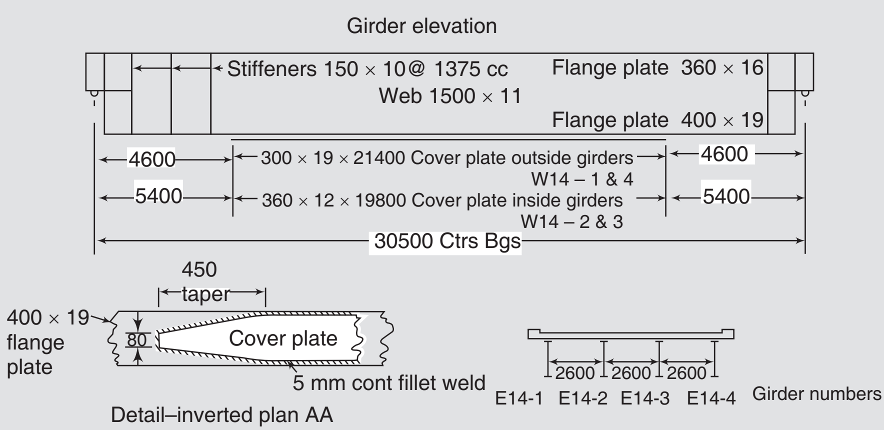


Fig. CS1 Details of plate grider and welding in King's Bridge, Melbourne, Austraila (Dowling et al. 1988)

The longitudinal welds connecting the cover plates were made before the short 80 mm transverse welds at the ends. They provided complete restraint against contraction, when the transverse welds were placed, resulting in transverse crack in flange plates. The transverse welds were made in three passes. In some instances, the cracks caused in the main flange plate by the first run were covered up by a subsequent pass. In other cases, the cracks caused by the last run were covered up with priming paint. The penetration of paint coats into the cracks showed later that the cracks passed through the full thickness of the flange even before the girders left the factory. In the span that failed, cracks existed in the main flange plate under seven of the eight transverse fillet welds. Thus, the most likely and most dangerous cracks were regularly missed by the inspectors, who however repaired several less harmful longitudinal cracks. All the seven cracks developed into complete flange failure, partly by brittle fracture and partly by fatigue, under a load that was well within the design load of the bridge.

11.5 Types of Joints

The five basic types of welded joints which can be made in four different welding positions such as flat, horizontal, vertical, and overhead are as follows (see Fig. 11.9):

(a) Butt joint
(b) Lap joint
(c) T-joint
(d) Corner joint
(e) Edge joint

These joints are discussed briefly in the following sections.

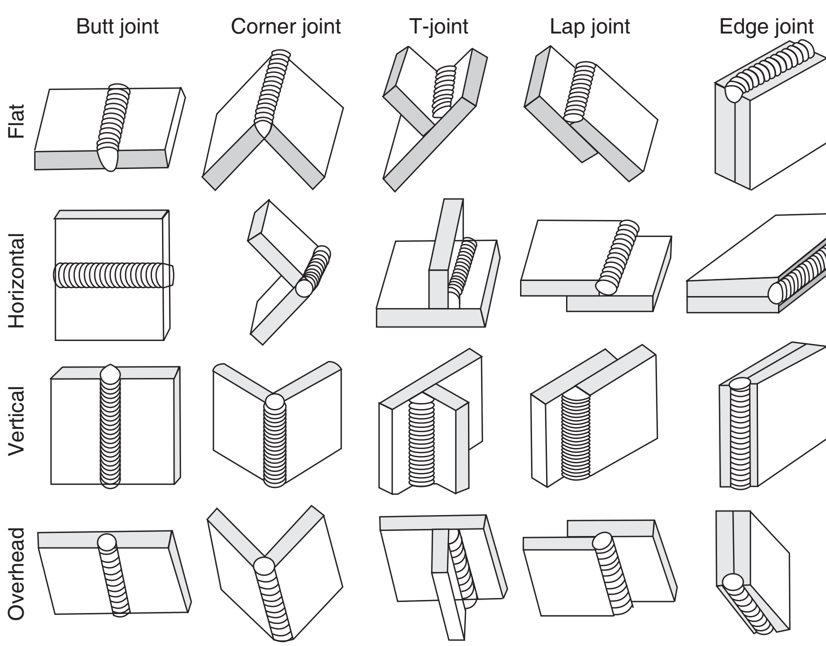


Fig. 11.9 Five basic weld joints may be made in four different welding positions

11.5.1 Butt Joints

A butt joint is used to join the ends of flat plates of nearly equal thickness. This type of joint eliminates the eccentricity developed using a lap joint (see Fig. 11.9). The butt joint obtained from a full penetration groove weld has 100% efficiency (i.e. the weld is considered as strong as the parent plate). As mentioned previously, the groove welds used in butt joints can be full penetration or partial penetration depending upon whether the penetration is complete or partial through the thickness (see Section 11.4.1 for the discussion on groove welds). Such butt joints also minimize the size of the connection and are aesthetically pleasing than lap joints. Face reinforcement (see Fig. 11.5) is the extra weld metal that makes the throat dimension greater than the thickness of the welded material. The provision of reinforcement increases the efficiency of the joint and ensures that the depth of the weld is at least equal to the thickness of the plate. Reinforcement is normally provided, since it is difficult for the welder to make the weld flush with the parent metal.

Reinforcement makes the butt joint stronger under static load and the flow of forces is generally smooth. However, when in the case of fatigue loads, stress concentration develops in the reinforcement, leading to cracking and early failure. Hence, under these circumstances, the reinforcement should be either removed by machining or kept within limits (normally within 0.75–3 mm) to avoid stress concentration. Similarly, when plates of two different thicknesses and/or widths are joined, the wider or thicker part should be reduced at the butt joint to make the width or thickness equal to the smaller part, the slope being not steeper than one in five [see Fig. 11.10(b)]. Where the reduction of the dimension of the thicker part is impracticable, and/or where dynamic/alternating forces are not involved, the weld metal shall be built up at the junction with the thicker part to dimensions at least 25% greater than those of the thinner part, or alternatively to the dimensions of the thicker member [see Fig. 11.10(c)]. Their main drawback is that the edges of the plates which are to be connected must usually be specially prepared and very carefully aligned before welding. They also result in high residual stresses.

Due to the accurate placement of parts before welding, butt joints are often made in shops, where it is possible to control the welding process. Field butt joints are rarely used.

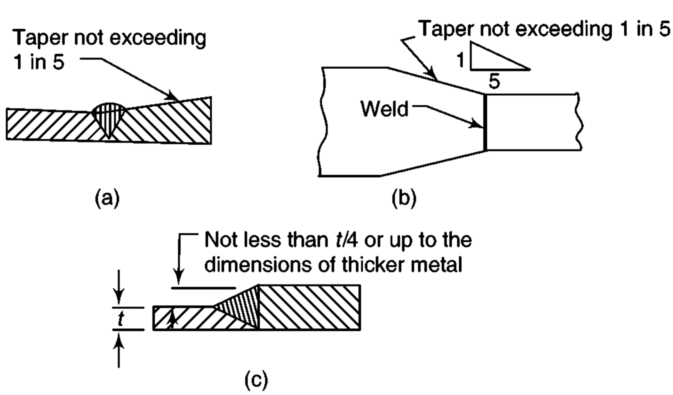


Fig.11.10 Butt welding of parts of unequal thickness and/or unequal width

11.5.2 Lap Joints

Lap joints are most commonly used because they offer ease of fitting and ease of jointing. Thus, they do not require any special preparation (even flame cut or sheared edges can be used) and can accommodate minor errors in fabrication or minor adjustment in length. Lap joints utilize *fillet welds* (see Section 11.4.2) and hence are well suited for shop as well as field welding. Some examples of lap joints are shown in Fig. 11.11. The connections using lap joints may require a small number of erection bolts, which may either be removed after welding or left in place. The additional advantage of lap joints is that plates with different thicknesses can be joined without any difficulty. Hence, it is often preferred in truss joints as shown in Fig. 11.11(c). However, the main drawback of a lap joint is that it introduces some eccentricity of loads, unless a double lap joint is used as in Fig. 11.11(e).

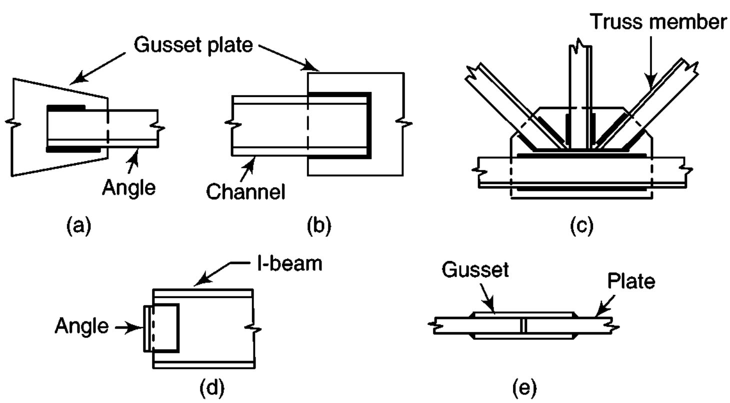


Fig. 11.11 Some examples of welded lap joints

11.5.3 Tee Joints

T-joints are often used to fabricate built-up sections such as T-shapes, I-shapes, plate girders, hangers, brackets, and stiffeners, where two plates are joined at right angles. T-joints can be made by using either fillet or groove welds. The groove weld edge shapes used on T-joints are shown in Fig. 11.12.

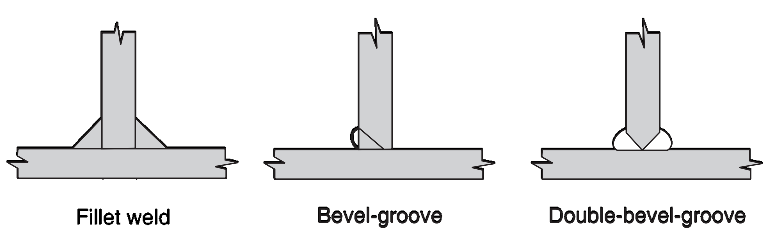


Fig. 11.12 Fillet and groove welded T-joints (Double bevel or J groove are often used with thick plates)

11.5.4 Corner Joints

Corner joints are used to form built-up rectangular box sections, which may be used as columns or beams to resist high torsional forces. Fillet weld and a few groove weld edge shapes for corner joints are shown in Fig. 11.13.

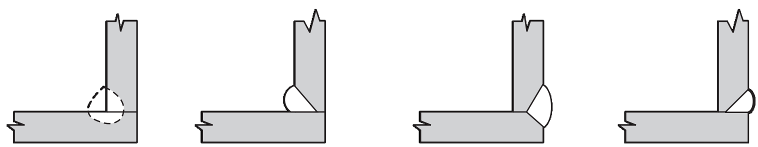


Fig. 11.13 Corner joint edge shapes

11.5.5 Edge Joints

Edge joints are not used in structural engineering applications. They are used to keep two or more plates in a given plane (see Fig. 11.9).

Since there are several variations and combinations of the five basic types of joints, the designer may choose the best joint (or combinations of the joints), which will yield an economical and efficient joint, for a particular situation.

11.6 Control of Shrinkage and Distortion

The molten weld bead that has been deposited starts to cool and while solidifying attempts to contract both along and transverse to its axis. This tendency to contract will induce tensile residual stresses and distortions (see Fig. 11.14).

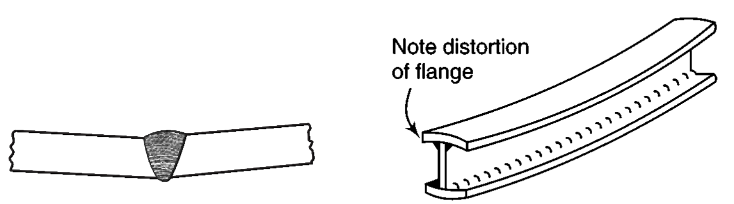


Fig. 11.14 Distortion due to welding

There are several ways to minimize these distortions and are provided by Blodgett (1966). Some of these approaches are listed here.

(a) Reduce the shrinkage forces by incorporating the following:
- Use minimum weld metal; for groove welds use the minimum root opening that is necessary; do not over weld
- Use only a few passes to complete the weld
- Use proper edge preparation and fit-up
- Use intermittent welds

- Deposit the weld metal in the direction opposite to the progress of welding the joint

(b) Allow the shrinkage to occur freely as follows.

(c) Balance shrinkage forces by incorporating the following.

- Use symmetry in welding
- Use scattered and intermittent weld segments.
- Use peening (i.e. stretching the metal by a series of blows, using a hammer).
- Use clamps, jigs, etc., to force the weld metal to stretch as it cools.

In practice, more than one method may be used at the same time for a particular situation. Minimum pre-heat and *interpass temperature* (for welds requiring more than one pass of welding operation along a joint, the interpass temperature is the temperature of the deposited weld when the next pass is about to begin) are sometimes prescribed to minimize shrinkage and ensure adequate ductility (Salmon & Johnson 1996).

11.7 Weld Symbols

The standard weld symbols used on drawings for different types of welds are shown in Fig. 11.15. Symbols save a lot of space as descriptive notes can be omitted. The location and details of the weld are shown by an arrow, a horizontal line ending with a fork (see Fig. 11.16). The 'side' below the arrow is called the arrow side and the 'side' above is called the other side. A circle at the kink indicates a weld all round and a vertical line and triangular pennant at the kink shows a field weld. The weld is denoted by symbols on both 'arrow' and 'other' side, but the weld symbol on the 'arrow side' is inverted. The surface condition is shown by a convex or horizontal line (contour symbol). The size of the welds is shown near the fork on the horizontal line. The length and pitch of the weld (for intermittent welds only) are shown after the weld symbol. The use of some of these symbols is illustrated in Fig. 11.16.

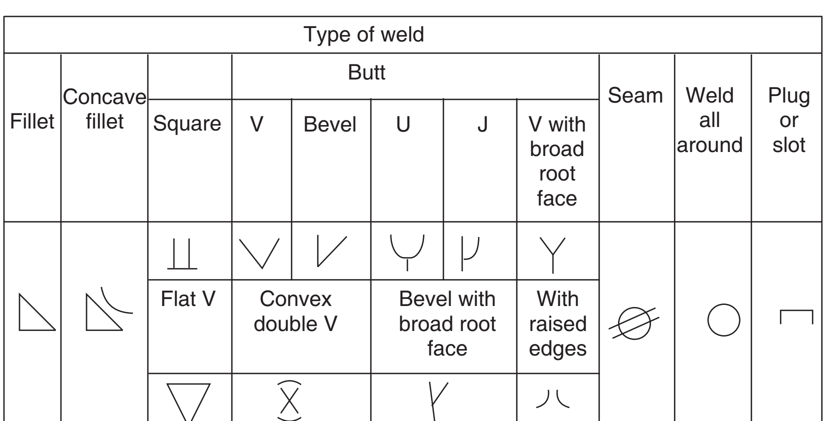


Fig. 11.15 Basic weld symbols

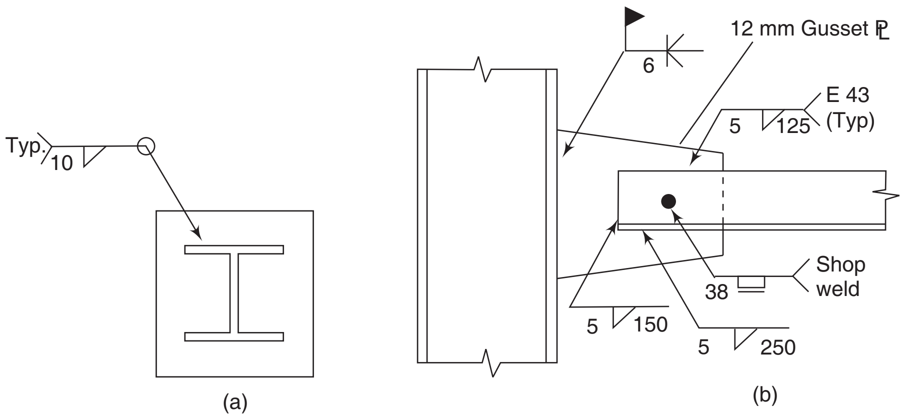


Fig. 11.16 Illusration of some of the welding symbols

11.8 Weld Specifications

Thicker plates dissipate the heat due to arc welding vertically as well as horizontally while thinner plates dissipate heat only horizontally. Thus, in thicker plates heat is removed from the welding area quickly and hence results in lack of fusion. For this reason, specifications often stipulate minimum and maximum weld sizes to achieve proper fusion of the base metal and the electrode.

11.8.1 Minimum Weld Size

To ensure fusion, minimize distortion, and to avoid the risk of cracking IS 800 : 2007 and IS 816 provide for a minimum size weld based on the thickness of the pieces being joined. The size of fillet weld should not be less than 3 mm nor more than the thickness of the thinner part joined. The minimum size of the first run or of a single run fillet weld should be as per Table 11.2. Usually the weld size closer to the minimum size is selected. Large size welds require more than one run of welding, which means that after the first run, chipping and cleaning of the weld is required to remove the slag. This will increase the cost of welding. Also note that a smaller size weld will be cheaper than a larger one for the same strength, considering the volume of welding. For example, a 300-mm-long 5-mm weld will have the same strength (198.3 kN) compared to a 150-mm-long 10-mm size weld. However, the volume of a 10-mm weld (7500 mm^3) is twice that of a 5-mm weld (3750 mm^3).

11.8.2 Maximum Fillet Weld Size Along Edges

The maximum size of fillet weld used along the edges of pieces being jointed is limited to prevent the melting of the base material at the location where the fillet would meet the corner of the plate, if the fillet were made to the full plate thickness. The maximum permitted size is as follows.

Table 11.2 Minimum size of a single run fillet weld (as per IS 800)

Thickness of thicker part		Minimum size, mm
Over, mm	Up to and including, mm	
—	10	3
10	20	5
20	32	6
32	50 (see notes below)	8 for first run, 10 for minimum size of weld

Note 1: When the minimum size is greater than the thickness of the thinner part, the minimum size should be equal to the thickness of the thinner part. Pre-heating of thicker part may be necessary.

Note 2: Where the thicker part is more than 50-mm thick, special precautions like pre-heating should be taken.

(a) Along the edge of the plate, less than 6-mm thick, the maximum size is equal to the thickness of the plate.
(b) Where the fillet weld is applied to the square edge of a plate of thickness greater than 6 mm, size of the weld should be at least 1.5 mm less than the edge thickness [see Fig. 11.17(a)]. This limit is specified such that the total strength may be developed without overstressing the adjacent metal.
(c) Where the fillet weld is applied to the rounded toe of the rolled section, the size of the weld should not exceed 3/4 of the thickness of the section at the toe [see Fig. 11.17(b)].

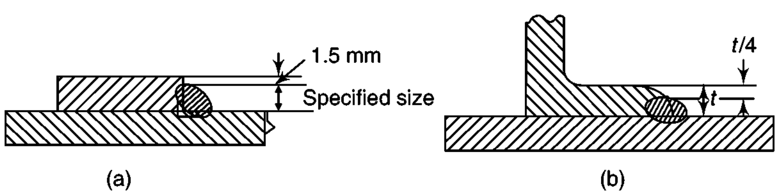


Fig. 11.17 Size of fillet welds

11.8.3 Minimum Effective Length of Fillet Weld

When placing a fillet weld, though the welder tries to build up the weld to its full dimension from the beginning, there is always a slight tapering off where the weld starts and where it ends. Therefore, a minimum length of four times the size of the weld is specified [see Fig. 11.18(b)]. If this requirement is not met, the size of the weld should be one fourth of the effective length.

For the above reasons, the effective length is taken equal to its overall length minus twice the size of weld. *End returns* as shown in Fig. 11.18(d) are made equal to twice the size of the weld to relieve the high stress concentration at the ends. Most designers neglect the end returns in the effective length calculation of the welds. End returns must be provided for welded joints that are subject to eccentricity, stress reversals, or impact loads.

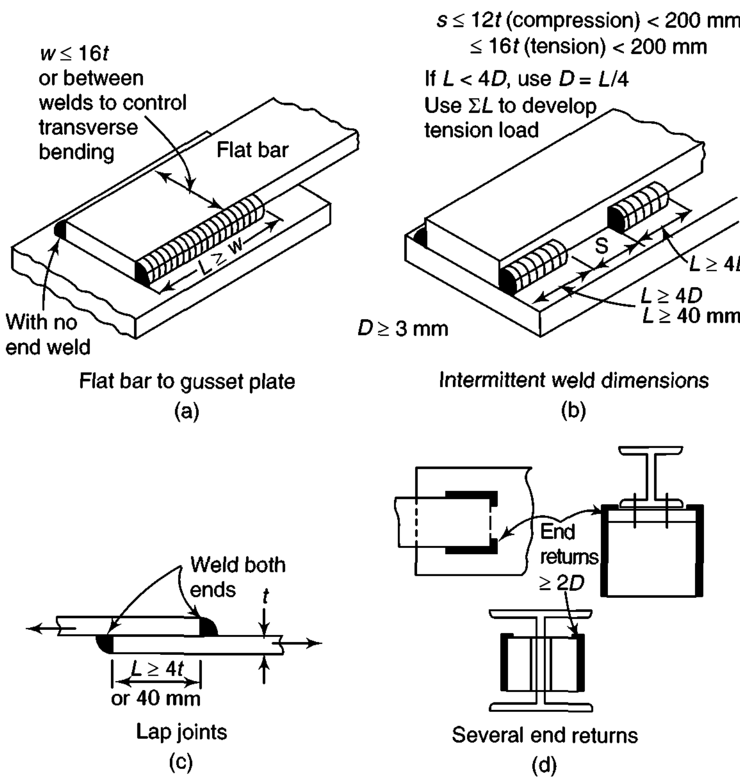


Fig. 11.18 Several welded connection specifications

In order to control the stress concentration at the edge of the plate, the length of the longitudinal (side) fillets should not be less than the width of the plate [see Fig. 11.18(a)]. The uneven stress distribution increases as the width of the plate increases. For this reason, the perpendicular distance between longitudinal fillet welds is limited to 16 times the thickness of the thinner plate jointed. If the plate is wider than this limit, slot or plug welds may be introduced, which tend to improve the distribution of stress in plate.

11.8.4 Overlap

The overlap of plates to be fillet welded in a lap joint should not be less than 4 times the thickness of the thinner part [see Fig. 11.18(c)].

11.8.5 Effective Length of Groove Welds

The effective length of groove welds in butt joints is taken as the length of continuous full size weld, but it should not be less than four times the size of the weld.

11.8.6 Effective Length of Intermittent Welds

The intermittent fillet welds should have an effective length not less than four times the weld size, with a minimum of 40 mm, as already shown in Fig. 11.18(b).

The clear spacing between the effective lengths of intermittent welds should not exceed 12 and 16 times the thickness of thinner plate jointed for compression and the tension joint respectively, and should never be more than 200 mm. The intermittent groove weld in butt joints should have an effective length of not less than four times the weld size and the longitudinal space between the effective lengths of the intermittent welds should not be more than 16 times the thickness of the thinner part joined. The IS code prohibits the use of intermittent welds in joints subjected to dynamic, repetitive, and alternate stresses.

11.8.7 Effective Area of Plug Welds

Effective area of plug welds should be taken as the nominal area of the hole in the plane of the faying surface. IS code stipulates that they should not be designed to carry any stresses.

11.9 Effective Area of Welds

The effective areas of a groove or fillet weld is the product of the effective throat dimension (t_e) multiplied by the effective length of the weld. The effective throat dimension of a groove weld or a fillet weld depends on the minimum width of expected failure plane and is explained in the next section.

11.9.1 Groove Weld

The effective throat thickness of a complete penetration groove weld is taken as the thickness of the thinner part joined [see Figs 11.19(a) and (b)]. The effective throat thickness of T- or L-joints are taken as the thickness of the abutting part. Reinforcement (see Fig. 11.5), which is provided to ensure full cross-sectional area, is not considered as part of the effective throat thickness.

The effective throat thickness of a partial penetration joint weld is taken as the minimum thickness of the weld metal common to the parts joined, excluding reinforcement (see Fig. 11.19). In unsealed single groove welds of V-, U-, J-, and bevel-types and groove welds welded from one side only, the throat thickness should be at least $7/8^{th}$ of the thickness of the thinner part joined. However, for the purpose of stress calculation, the effective throat thickness of $5/8^{th}$ thickness of the thinner member only should be used (IS 816 : 1969). The unwelded portion in incomplete penetration welds, welded from both sides, should not be greater than 0.25 times the thickness of the thinner part joined, and should be central in the depth of the weld [Fig. 11.19(d)]. In this case also, a reduced effective throat thickness of $5/8^{th}$ of the thickness of the thinner part should only be used in the calculations. Groove welds used in butt joints, where the penetration is less than those specified above, due to non-accessibility, should be considered as non-load carrying for the purposes of design calculations.

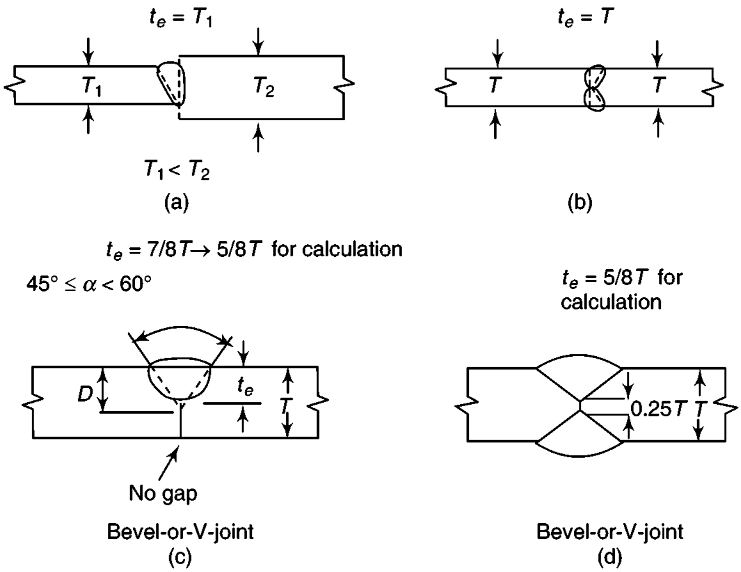


Fig. 11.19 Effective throat diamensions for groove welds

11.9.2 Fillet Weld

The effective throat dimension of a fillet weld is the shortest distance from the root of the face of the weld, as shown in Fig. 11.20. The effective throat thickness of a fillet weld should not be less than 3 mm and should not exceed 0.7a (1.0a, under special circumstances), where a is the size of the weld in mm. Thus, if the fillet weld is having unequal lengths (which is a rare situation), as shown in Fig. 11.20(b), the value of t_e should be computed from the diagrammatic shape of the weld.

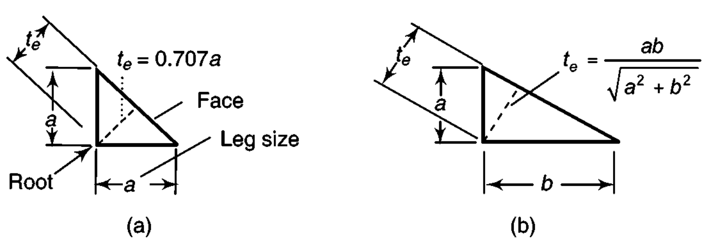


Fig. 11.20 Effective throat dimensions for fillet welds

The load–deformation relationship of fillet welds has been studied by several researchers (e.g. Butler et al. 1972; Swannel 1981; Neis 1985). Figure 11.21 shows the variations in fillet weld behaviour with the relative direction of the load vector to the weld axis (for a 8-mm fillet weld of ultimate strength 565 MPa, weld length 50 mm, plate thickness 19 mm, and ultimate strength of plate = 511 MPa). When $\theta = 0°$, the weld axis is normal to the load vector, the so-called *end fillet* (transverse fillet) situation, and the weld develops a high strength with less ductility (with deformation at rupture less than 1 mm). On the other hand, when $\theta = 90°$, the weld axis is parallel to the load vector, the *side-fillet* (longitudinal fillet) situations, and the weld shear strength is limited to about 56% of the weld metal tensile strength. However, the side fillet exhibits more ductility (rupture occurring at over 2-mm deformation). Intermediate orientations show intermediate values of both strength and ductility. Thus, the end fillet welds are 30%–40% stronger than side fillet welds. However, according to IS 800 : 2007 there is no difference between side and end fillet welds. A common design strength is specified, irrespective of loading direction.

Apart from the large difference in strength, end and side fillets also differ in both stiffness and ductility. Note that when the two types of welds are mixed, a greater share of the load is attracted to end fillets because of their high stiffness, which may result in the side fillets not developing to their full capacity. The code stipulates that the effective throat thickness in fillet welds joining faces inclined to each other should be taken as follows:

$$\text{Effective throat thickness} = K \times \text{size of weld}$$

where K is a constant depending upon the angle between fusion faces (see Fig. 11.21), as given in Table 11.3.

Table 11.3 Values of K for different angles between fusion faces (as per IS 800)

Angle between fusion faces	60°–90°	91°–100°	101°–106°	107°–113°	114°–120°
Constant K	0.70	0.65	0.60	0.55	0.50

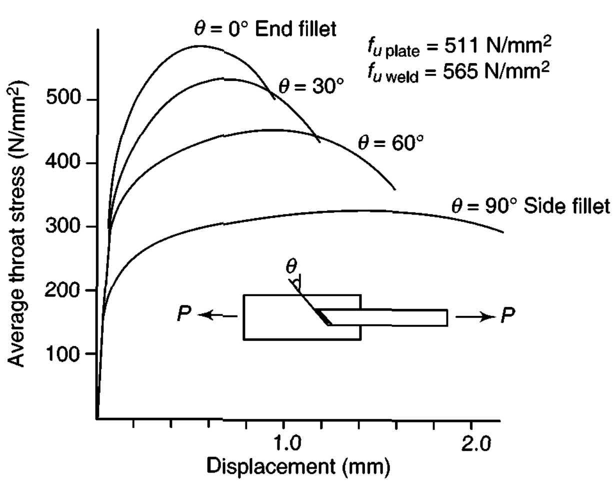


Fig. 11.21 Load-deformation curves for an 8 mm leg fillet weld at varying angles θ to load vector

LIVE GRAPH
Click here to view

11.9.4 Long Joints

If the maximum length of side weld exceeds $150t_e$, where t_e is the throat size of weld, a reduction factor as per clause 10.5.7.3 of the code has to be applied to the calculated strength.

11.10 Design of Welds

The following assumptions are usually made in the analysis of welded joints.

(a) The welds connecting the various parts are homogenous, isotropic, and elastic.
(b) The parts connected by the welds are rigid and their deformation is, therefore, neglected.
(c) Only stresses due to external forces are considered. The effects of residual stresses, stress concentrations, and the shape of the weld are neglected.

11.10.1 Groove Welds

As per IS 800 : 2007, the groove welds in butt joints will 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.

(a) For tension or compression normal to effective area and tension and compression parallel to the axis of the weld

$$T_{dw} = f_y L_w t_e / \gamma_{mw} \tag{11.1}$$

where T_{dw} is the design strength of the weld in tension, f_y is the smaller of ultimate stress of the weld and the parent metal in MPa, t_e is the effective throat thickness of the weld in mm, L_w is the effective length of the weld in mm, and γ_{mw} is the partial safety factor taken as 1.25 for shop welding and as 1.5 for site welding.

(b) For shear on effective area

$$V_{dw} = L_w t_e f_{yw} / (\sqrt{3} \times \gamma_{mw}) \tag{11.2}$$

where V_{dw} is the design strength of the weld in shear. Other quantities have been defined already.

As stated earlier, in the case of complete penetration groove weld in butt joints, design calculations are not required as the weld strength of the joint is equal to or even greater than the strength of the member connected. In the case of incomplete penetration groove weld in butt joints, the effective throat thickness is computed and the required effective length is determined and checked whether the strength of the weld is equal to or greater than the strength of the member connected or the applied external force.

11.10.2 Fillet Welds

The actual distribution of stress in a fillet weld is very complex. A rigorous analysis of weld behaviour has not been possible so far. Multi-axial stress state, variation in yield stress, residual stresses, and strain hardening effects are some of the factors, which complicate the analysis. In many cases, it is possible to use the simplified approach of average stresses in the weld throat.

In the code, the design strength of fillet weld, f_{wd}, is given by

$$f_{wd} = f_u / (\sqrt{3}\gamma_{mw}) \tag{11.3}$$

where f_u is the smaller of the ultimate stress of the weld and parent metal, and γ_{mw} is the partial safety factor which equals 1.25 or 1.5 depending on whether the weld is made at a shop or at the site, respectively. (Note that the weld metal always has a higher strength; hence we should use the parent metal strength only in the equation as per IS code.)
Hence as per IS 800 : 2007 the design strength is given by

$$P_{dw} = L_w t_e f_u/(\sqrt{3}\gamma_{mw}) \tag{11.4a}$$

or

$$P_{dw} = L_w K s f_u/(\sqrt{3}\gamma_{mw}) \tag{11.4b}$$

where P_{dw} is the design strength of the fillet weld and s is the size of the weld. Other terms have been defined earlier.

Tables have been prepared to simplify the calculation while using Eqn (11.4) and are presented in Appendix D.

11.10.2.1 Design Procedure

The design procedure is as follows.

1. Assume the size of the weld based on the thickness of the members to be joined.
2. By equating the design strength of the weld to the external factored load, the effective length of the weld to be provided is calculated. The length may be provided either as longitudinal fillet welds (parallel to the load axis) or as transverse fillet welds (perpendicular to the load axis) along with longitudinal fillet welds. It is a common practice to treat both the welds as if they are stressed equally. If the length exceeds $150t_e$, reduce design capacity by a factor β_{lw} as per clause 10.5.7.3 of the code.
3. If only the longitudinal fillet weld is provided, a check is made to see if the length of each longitudinal fillet weld is more than the perpendicular distance between them.
4. End returns of length equal to twice the size of the weld are provided at each end of the longitudinal fillet weld.

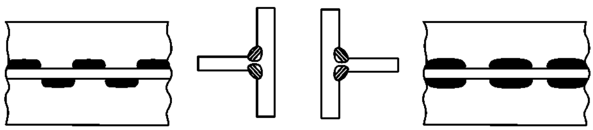
When subjected to combined tensile and shear stress, the equivalent stress, f_e, should satisfy

$$f_e = \sqrt{(f_a^2 + 3q^2)} \le f_u/\left(\sqrt{3}\,\gamma_{mw}\right) \tag{11.5}$$

where f_a = normal stress due to axial force or bending moment, and q = shear stress due to shear force or tension.

11.10.3 Intermittent Fillet Welds

Intermittent fillet welds are provided to transfer calculated stress across a joint, when the strength required is less than that developed by a continuous fillet weld of the smallest practical size. Such intermittent welds are often found in the connection of stiffeners to the web of plate girders. In such situations, first the fillet weld length required is computed as a continuous fillet weld. A chain of intermittent fillet welds of total length equal to the computed length, is provided as shown in Fig. 11.22. Intermittent fillet welds shown in Fig. 11.22(a) are structurally better than those shown in Fig. 11.22(b), since they reduce the distortion due to the balancing nature of the welds.



(a) (b)

Fig. 11.22 Intermittent fillet weld

In the design of intermittent welds, the following procedure is adopted (IS 816 : 1969).

1. Assume the size of weld and compute the total length of required intermittent weld.
2. The minimum effective length (four times the size of weld or 40 mm) and clear spacing (12t for compressions and 16t for tension and should not be less than 200 mm, where t is the thickness of the thinner plate joined) clauses of IS codes should be followed.
3. At the ends, the longitudinal intermittent fillet weld should be of length not less than the width of the member, otherwise transverse welds should be provided. If transverse welds are provided along with longitudinal intermittent welds, the total weld length at the ends should not be less than twice the width of the member.

11.11 Simple Joints

In this section we will discuss the design of some simple welded joints such as truss member connections, angle seat connections, web angle and end seat connections, and end plate connections.

(a) (b) (c)

Some simple welded connections: (a) column splice with CJP grove weld, (b) welded double angle connection, (c) unstiffened seat connection © American Institute of Steel Construction, Inc., Reprinted with permission. All rights reserved.)

11.11.1 Design of Fillet Welds for Truss Members

In the design of welds connecting tension or compression members, the welds should be at least as strong as the members they connect and the connection should not result in significant eccentricity of loading. Truss members often consist of single or double angles, and occasionally T-shapes and channels. Consider the angle tension member shown in Fig. 11.23, with two longitudinal welds (on the two sides parallel to the axis of the load) and a transverse weld (perpendicular to the axis of the load). The axial force T in the member will act along the centroid of the member. The force T has to be resisted by the forces P_1, P_2, and P_3 developed by the weld lines. The forces P_1 and P_2 are assumed to act at the edges of the angle and the force P_3 at the centroid of the weld length, located at $d/2$. Taking moments about point A located at the bottom edge of the member and considering clockwise moments as positive, we get

$$\Sigma M_A = -P_1 d - P_2 d/2 + Ty = 0 \tag{11.6}$$

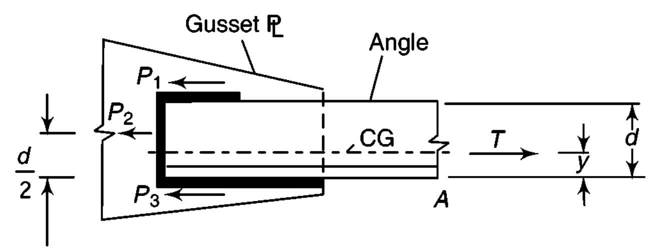


Fig. 11.23 Balancing the welds on a tension member connection.

Hence,

$$P_1 = Ty/d - P_2/2 \tag{11.7}$$

The force P_2 is equal to the resistance R_w of the weld per mm multiplied by the length L_w of the weld.

$$P_2 = R_w L_{w2} \tag{11.8}$$

Considering the horizontal equilibrium, we get

$$\Sigma F_H = T - P_1 - P_2 - P_3 = 0 \tag{11.9}$$

Solving Eqns (11.7) and (11.9) simultaneously, we get

$$P_3 = T(1 - y/d) - P_2/2 \tag{11.10}$$

Designing the connection shown in Fig. 11.23, to eliminate the eccentricity caused by the unsymmetrical welds is called *balancing the weld.* The procedure adopted for balancing the weld is as follows.

1. After selecting the proper weld size and electrode, compute the force resisted by the end weld P_2 (if any) using Eqn (11.8).
2. Compute P_1 using Eqn (11.7).
3. Compute P_3 using Eqn (11.10) or

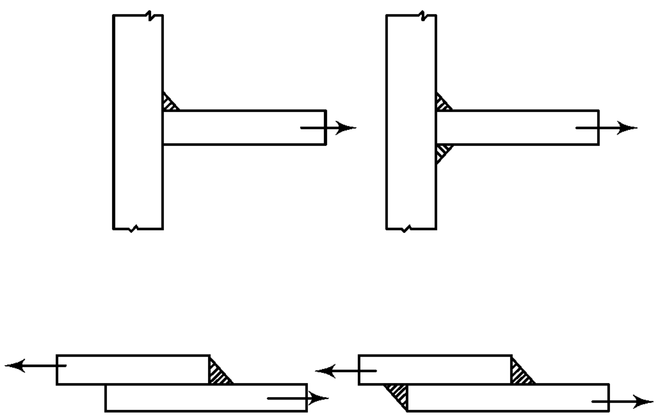
4. Compute the lengths L_{w1} and L_{w3} on the basis of

$$L_{w1} = P_1/R_w \text{ and } L_{w3} = P_3/R_w \tag{11.11}$$

Alternatively, the total length required to resist the load, L_w may be calculated. The length of end weld may then be subtracted from the total and the remaining length is allocated to P_1 and P_2 in inverse proportion to the distances from the centre of gravity.

Single sided welds in tension

Examples of unsatisfactory and satisfactory welds for tension connections are given in Fig. 11.24. In the examples shown in Fig. 11.24(a), the eccentricity between the line of action of the load and the throat centroid creates a moment on the weld throat. Hence, this should be avoided in practice. In the symmetric arrangements shown in Fig. 11.24(b), though there is a small variation in stress across the weld throats, with little ductility, this variation is redistributed, resulting in uniform stress fields on the weld throats.



(a) (b)

Fig. 11.24 (a) Unsatisfactory and (b) satisfactory welds for tension connections

11.11.2 Angle Seat Connections

As discussed in Section 10.6.3, a beam may be supported on a seat, either unstiffened or stiffened. In this section, the unstiffened seat connection as shown in Fig. 11.25 is discussed, where an angle is designed to carry the entire reaction. This type of connection uses a top clip angle, whose intended function is to provide lateral support to the compression flange. The seated connection is designed to transfer only the vertical reaction and should not give significant restraining moment at the end of the beam. Hence, the seat and top angle are selected in such a way that they are relatively flexible.

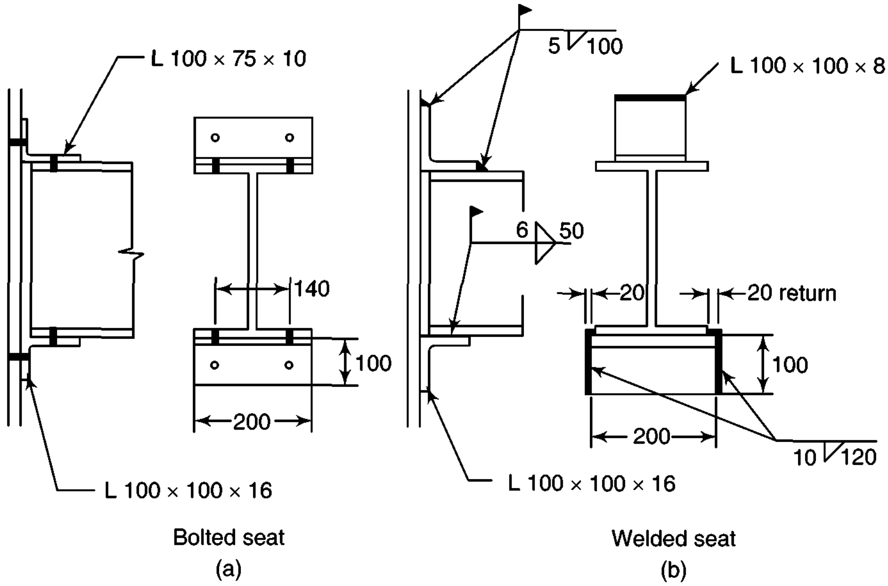


Fig. 11.25 Welded seat angle connection (for comparison bolted seat angle connection is also shown; The angle sizes and weld sizes and weld sizes may change depending on load)

For welded seat, since the weld along the end holds the angle tight against the column, the critical section is the same (whether or not the beam is attached to the seat) as that for the previous case of bolted beam, connected to seat (see Fig. 10.23).

The design of the unstiffened angle seat involves the following steps (see Section 10.6.4 and Fig. 10.23).

1. Selection of seat angle having a length equal to width of the beam.
2. Length of the outstanding leg of the seat angle is calculated on the basis of web crippling of the beam.

$$b = R/[t_w(f_{yw}/\gamma_{m0})] \tag{11.12}$$

   where R is the reaction of the beam, t_w is the thickness of the web of the beam, f_{yw} is the yield strength of the web, and γ_{m0} is the partial safety factor for material = 1.10.
3. Determine the length of the bearing on cleat

$$b_1 = b - (t_f + r_b) \tag{11.13}$$

   where t_f is the thickness of the flange of the beam and r_b is the root radius of the beam flange.
4. Determine the distance from the end of the bearing on cleat to the root of the angle

$$b_2 = b + g - (t_a + r_a) \tag{11.14}$$

where g = erection clearance + tolerance, t_a is the thickness of the angle, and r_a is the root radius of the angle.

5. The bending moment at the critical section may be calculated by assuming that the reaction from beam is uniformly distributed over bearing length b_2

$$M_u = R \times (b_2/b_1) \times b_2/2 \tag{11.15}$$

By equating it to the strength of solid rectangular section (angle leg), bent about its weak axis, the thickness of the seat angle may be determined.

6. Determine the required weld size.
   (a) Without taking eccentricity, the length of the weld on each side can be found out by using

$$L_w = R/(2 \times R_w) \tag{11.16}$$

   where R_w is the strength of weld per mm.

   (b) If the eccentricity is considered, the resultant force in the weld due to shear and bending (Salmon & Johnson 1996) is given by

$$R_{\text{res}} = [R/(2L_w^2)]\sqrt{[L_w^2 + 20.25(b_2/2)^2]} \tag{11.17}$$

11.11.3 Web Angle and End Plate Connections

The field-welded shear connection using web angles is shown in Fig. 11.26. The intension of such a connection is that the angles are as flexible as possible so that the beams are capable of rotating at the ends. They are assumed to provide simply supported end condition and designed to transmit shear only. The pair of angles are shop welded to the beam and field welded or connected to the column by means of HSFG bolts at site. The angles (called clip angles) project out of the beam web by a distance of about 12 mm (called set back), so that the beam can be fitted with acceptable tolerances. When beams intersect and have the same depth, the flanges are coped (cut away) as shown in Fig. 11.26(c), resulting in some loss of shear strength. The coping of beams will result in block shear failure and are susceptible to local web buckling (Gupta 1984; Yura et al. 1982).

Erection bolts are used to erect these beams and then the angles are welded at site. These bolts are often provided at the bottom of the angle. Generally 100-mm size legs are used for connecting the beam and the leg size at the column or girder side is kept a little longer. The length of the angle is kept equal to the distance between fillets of the beam, so that sufficient length is available for welding. Usually a weld size 2–3 mm smaller than the web angle thickness is chosen.

The connection, though assumed to transfer only shear forces, due to the eccentricity of connection, is also subjected to a bending moment (Blodgett 1966). Due to the rotation effect, the field welds cause web angles to press against the beam web at the top and tear apart from the bottom, thus indicating horizontal shear in the fillet weld. It is assumed that the neutral axis is at a distance of $L/6$ from the top of the angle. The horizontal shear is taken as zero at this point and maximum at the bottom of the angle [see Fig. 11.27(b)]. Neglecting the effects of the returns at top, the horizontal component R_x can be obtained from moment equilibrium (Blodgett 1966).

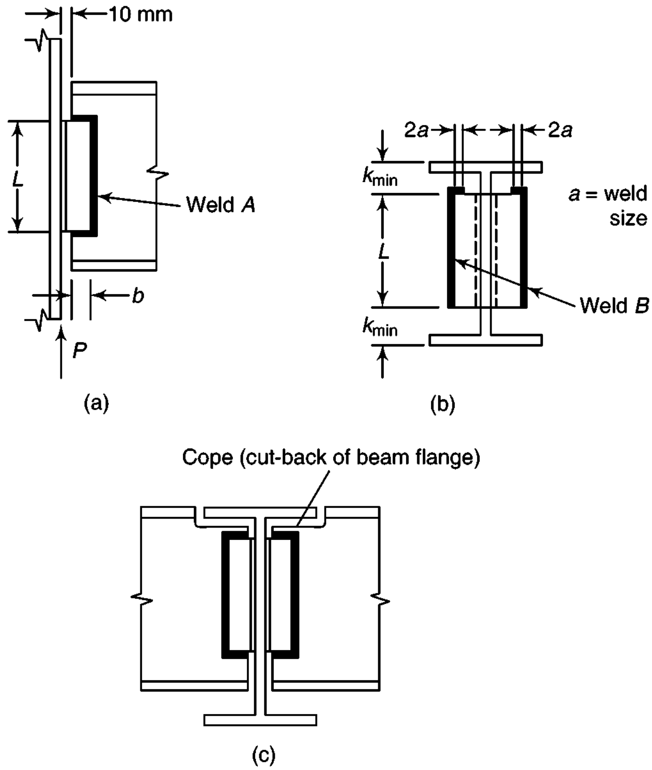


Fig. 11.26 Simple shear double-anlge connections

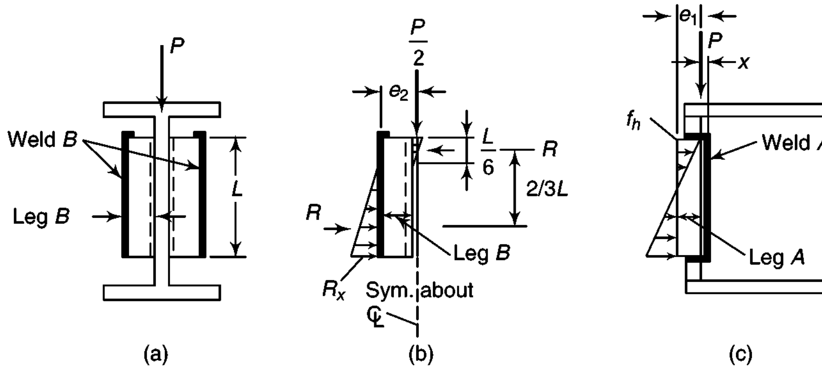


Fig. 11.27 Eccentric shear in the plane of the web as per Blodgett (1966)

Applied moment from load = Resisting moment of weld

$$(P/2)e_2 = (2/3)RL \tag{11.18}$$

where L is the length of weld

Hence,

$$R = 0.75Pe_2/L$$

From force triangle, we get

$$R = 0.5R_x \times (5/6)L$$

From these two equations, we get

$$R_x = 9Pe_2/(5L^2) \tag{11.19}$$

Vertical force on weld

$$R_v = P/(2L) \tag{11.20}$$

Resultant force on weld

$$R_{\text{res}} = \sqrt{\{[9Pe_2/(5L^2)]^2 + [P/(2L)]^2\}} \tag{11.21}$$

or

$$R_{\text{res}} = [P/(2L^2)]\sqrt{(L^2 + 12.96e_2^2)} \text{ N/mm} \tag{11.22}$$

The above equation neglects eccentricity e_1, which tends to cause tension at the top of the weld lines [see Fig. 11.27(c)]. The flexural tension component R_x at the top of the weld B is

$$R_x = My/I = Pe_1(L/2)/[2L^3/12] = 3Pe_1/L^2 \tag{11.23}$$

Thus,

$$R_{\text{res}} = \sqrt{[(P/2L)^2 + (3Pe_1/L^2)^2]} \tag{11.24}$$

or

$$R_{\text{res}} = P/(2L^2)\sqrt{(L^2 + 36e_1^2)} \text{ N/mm} \tag{11.25}$$

Note that the above equations disregard the weld returns, which have the greatest effect if L is short. Considering the returns to be equal to $L/12$, Salmon and Johnson (1996), derived the following equation

$$R_{\text{res}} = P/(2L^2)\sqrt{(L^2 + 20.25e_1^2)} \text{ N/mm} \tag{11.26}$$

11.11.4 End Plate Connections

End plate shear connection; the plate in this connection is welded to the web only and bolted to the other member at site (© American Institute of Steel Construction, Inc., Reprinted with permission. All rights reserved.)

End plate connection has been discussed in Section 10.7.2. The end plates are shop welded to the beam and connected to column flanges/web by means of HSFG bolts. The flange of the beam can be groove welded or fillet welded to the plate. The web will usually be fillet welded. A conservative approach to end plate connection design is to use the prying action concept discussed in Section 10.4.4. The region near the tension flange of the beam is designed similar to that of a split-beam T-connection. This fastener group is designed for shear and tension, including the effect of prying action. The welds are designed for the resultant force using the elastic vector analysis as below.

$$\text{Resultant force} = \sqrt{(P/A)^2 + (My/I)^2} \leq \text{design strength of the weld} \tag{11.27}$$

where P/A is the vertical stress due to shear force and My/I is the tension component (horizontal) due to the bending moment.

11.12 Moment Resistant Connections

When beams are connected to columns through brackets, depending on the way in which they are connected, the welds may be subjected to either twisting moment or bending moment, in addition to shear forces. Welded stiffened seat connections are also subjected to bending moment and shear forces. These connections are termed as moment resistant connections and are discussed in this section.

11.12.1 Eccentrically Loaded Connections

Loads acting eccentrically from the centroid of a weld line or weld group may cause either a twisting moment or a bending moment on the weld, depending upon the location of the welds, in addition to the direct shear forces.

Eccentric load causing twisting moment Since no initial tension is involved with welded connections, the eccentricity of loading, even though small, has to be considered. Also, there are situations where the loading of fillet welds is neither parallel not transverse to the axis of fillet welds, as shown in Fig. 11.28. Analysis of such eccentric loading is complicated since the load–deformation behaviour is a function of the angle θ between the direction of applied load and axis of the fillet weld (see Fig. 11.21).

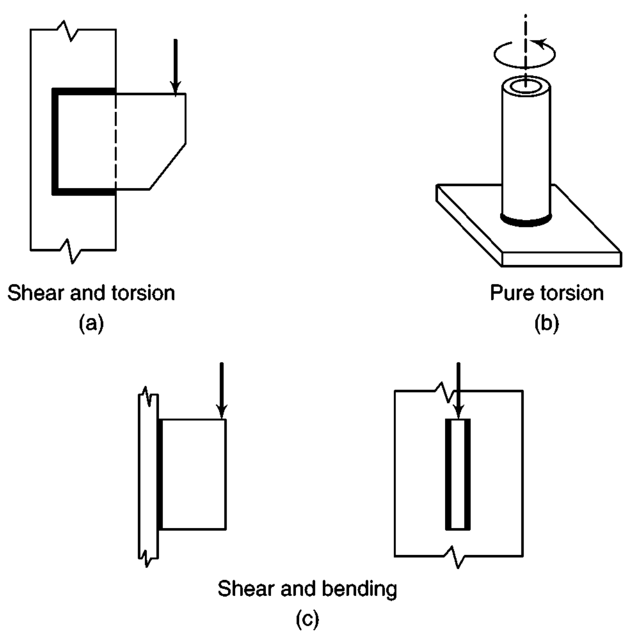


Fig. 11.28 Types of eccentric loading.

As discussed in Section 10.7.1, the strength of an eccentrically loaded fillet weld can also be determined by locating an instantaneous centre of rotation, using the load–deformation relationship of the fillet weld.

Here the more conservative traditional elastic vector analysis which is easier than the strength method is described. The following assumptions are made in the elastic method.

(a) Each segment of weld (of the same size) resists the concentrically applied load with an equal force.
(b) The rotation caused by the torsional moment is assumed to occur about the centroid of weld configuration.
(c) The load on a weld segment caused by the torsional moment is assumed to be proportional to the distance from the centroid of the weld configuration.
(d) The components of the forces caused by the direct load and by torsion are combined vectorially to obtain a resultant force.

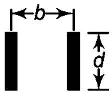
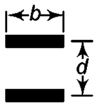
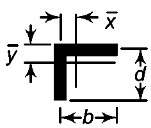
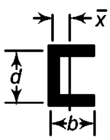
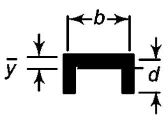
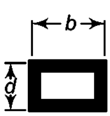
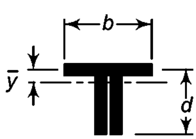
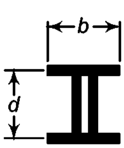
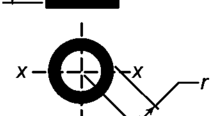
The steps involved in checking the adequacy of the weld are as follows.

1. The centroid of the weld line is calculated. The twisting moment and the forces at the centroid are determined (see Fig. 11.29)
$$T = P_x e_x + P_y e_y \tag{11.28}$$
where P_x and P_y are the x and y components of the eccentric load and e_x and e_y are the eccentricities of P_x and P_y with respect to the centroid of weld line.
2. The critical weld points are located.
3. The force components due to twisting moment and maximum shear force for critical weld point are determined as
$$F_x^T = Ty/I_p \text{ and } F_y^T = Tx/I_p \tag{11.29}$$
where x and y are the coordinates of critical weld point and I_p is the polar moment of inertia of the weld line about the centroid (see Table 11.4) and
$$F_x^P = P_x/L_w \text{ and } F_y^P = P_y/L_w \tag{11.30}$$
where L_w is the total length of the weld.
4. The resultant shear force is calculated as
$$F_R = [(F_x^P + F_x^T)^2 + (F_y^P + F_y^T)^2]^{0.5} \tag{11.31}$$
5. The maximum shear force should be less than the capacity of weld
$$F_R < R_w \text{ (weld strength)} \tag{11.32}$$

11.12.2 Eccentric Load Causing Bending Moment

When the applied load is eccentric to the plane of the weld configuration, as shown in Fig. 11.30, the strength method of analysis (by locating the instantaneous center of rotation) can be used (Dawe & Kulak 1974). However, we will consider only the elastic (vector) analysis which is conservative and relatively easy to use for loading resulting in shear and tension.

Table 11.4 Properties of welds treated as lines

Section b = width; $d = L_w$ = depth	Section modulus $I_x/\bar{y}$	Polar moment of interia, I_p about centre of gravity
1. 	$Z = \dfrac{d^2}{6}$	$I_p = \dfrac{d^3}{12}$
2. 	$Z = \dfrac{d^2}{3}$	$I_p = \dfrac{d(3b^2 + d^2)}{6}$
3. 	$Z = bd$	$I_p = \dfrac{b(3d^2 + b^2)}{6}$
4.  $\bar{y} = \dfrac{d^2}{2(b+d)}$ $\bar{x} = \dfrac{b^2}{2(b+d)}$	$Z = \dfrac{4bd + d^2}{6}$	$I_p = \dfrac{(b+d)^4 - 6b^2d^2}{12(b+d)}$
5.  $\bar{x} = \dfrac{b^2}{2b+d}$	$Z = bd + \dfrac{d^2}{6}$	$I_p = \dfrac{8b^3 + 6bd^2 + d^3}{12} - \dfrac{b^4}{2b+d}$
6.  $\bar{y} = \dfrac{d^2}{b+2d}$	$Z = \dfrac{2bd + d^2}{3}$	$I_p = \dfrac{b^3 + 6b^2d + 8d^3}{12} - \dfrac{d^4}{2d+b}$
7. 	$Z = bd + \dfrac{d^2}{3}$	$I_p = \dfrac{(b+d)^3}{6}$
8.  $\bar{y} = \dfrac{d^2}{b+2d}$	$Z = \dfrac{2bd + d^2}{3}$	$I_p = \dfrac{b^3 + 8d^3}{12} - \dfrac{d^4}{b+2d}$
9. 	$Z = bd + \dfrac{d^2}{3}$	$I_p = \dfrac{b^3 + 3bd^2 + d^3}{6}$
10. 	$Z = \pi r^2$	$I_p = 2\pi r^3$

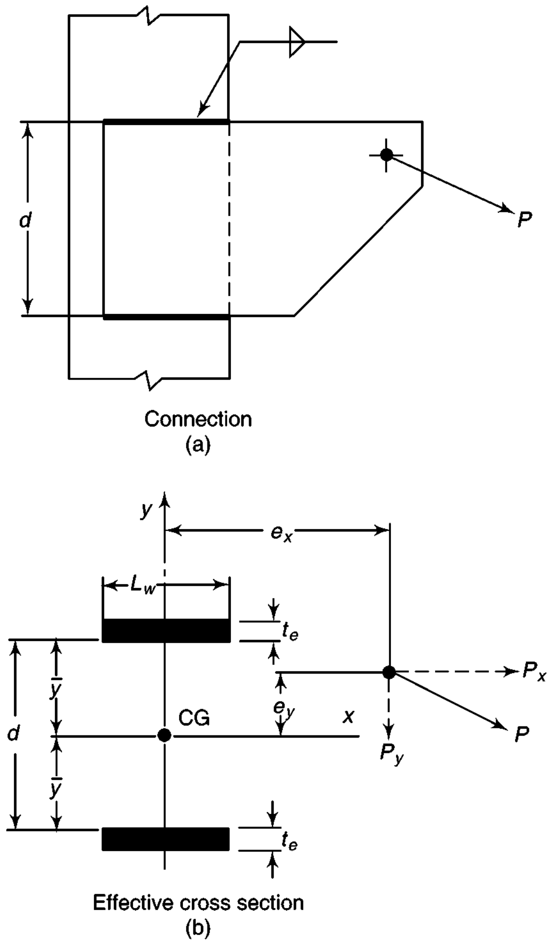


Fig. 11.29 Eccentric bracket connection

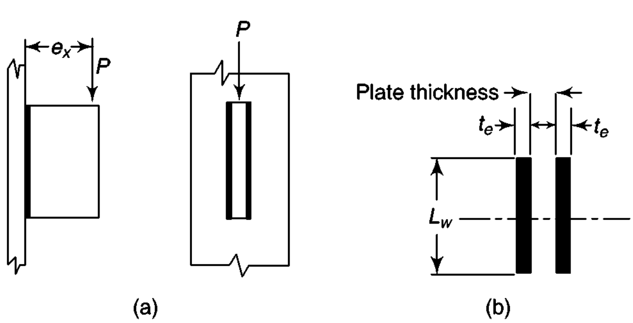


Fig. 11.30 Loads applied eccentric to the plane of weld

The effect of eccentric load on the weld group about its centroid is equivalent to a bending moment and direct force at the centroid. The bending moments (Pe) cause bending tensile and compressive stresses (i.e. normal stresses) at the throat section of the fillet weld, while the direct forces cause direct or shear stresses. Either a groove weld or a fillet weld can be used in such connections. Thus, the welds must carry the loads in the same manner as the members being connected carry them. The stresses are shown in Fig. 11.31. Thus, the direct stresses in the weld = load/effective area of weld

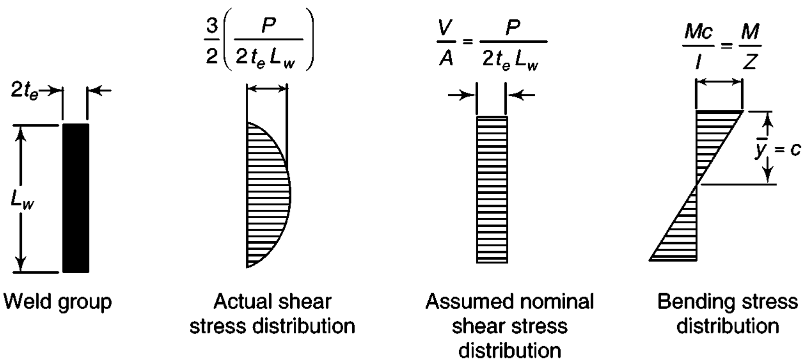


Fig. 11.31 Stresses in weld subject to eccentric load causing bending moment

(a) In the case of a fillet weld

$$\tau_{vf, cal} = P/(2L_w t_e) \quad (11.33)$$

(b) For groove welds

$$\tau_{vf, cal} = P/dt \quad (11.34)$$

where t is the thickness of plate.

The bending stress in the weld = moment/section modules

1. For fillet weld the failure is due to critical stress in the throat of the weld (at 45° to the weld leg length)
   Hence,
   $$f_b = M/Z = [Pe(L_w/2)]/[2 \times (L_w^3 t_e/12)] = 3Pe/(t_e L_w^2)$$
   The throat stress is treated as shear since a 45° line of failure is assumed. This shear stress is assumed linearly varying from zero at mid depth to the maximum value at the extreme fibres. Hence,
   $$\tau_{vf1, cal} = 3Pe/(t_e L_w^2) \quad (11.35)$$
2. For groove welds
   $$f_b = 6Pe/(tL_w^2) \quad (11.36)$$
   The combined stress in the fillet weld is given by the following equation
   $$f_e = \sqrt{(\tau_{vf, cal}^2 + \tau_{vf1, cal}^2)} < \text{weld strength} = f_u/(\sqrt{3}\gamma_{mw}) \quad (11.37)$$
   The combined bending and shear stress in the groove weld is checked by using the interaction formula (clause 10.5.10.1.1 of code)

$$f_e = \sqrt{(f_{b,\text{cal}}^2 + 3\tau^2)} \tag{11.38}$$

and f_e should not exceed the values allowed for the parent metal. The code also states that the check for the combination of stresses need not be done:

(a) for side fillet welds joining cover plates and flange plates and

(b) for fillet welds, where sum of normal and shear stresses does not exceed

$$f_{\text{wd}} = f_a/(\sqrt{3}\gamma_{\text{mw}})$$

Similarly a check for the combination of stresses in groove welds need not be done if:

- groove welds are axially loaded and
- in single and double bevel welds, where the sum of normal and shear stresses does not exceed the design normal stress and the shear stress does not exceed 50% of the design shear stress.

Note that the locations of maximum bending and shearing stresses are not the same (see Fig. 11.31). Hence, if the welds are used as shown in Fig. 11.32, it can be safely assumed that the web welds would carry the entire shear force and the flange welds would carry the entire bending moment.

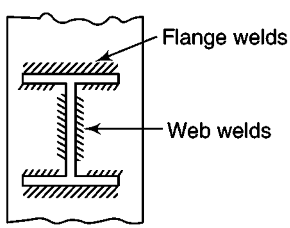


Fig. 11.32 Welding of brackets for carrying shear and moment.

11.12.2.1 Design

The design of brackets subject to combined bending and shear is done using the following steps.

1. Assume the size of the weld and compute the throat thickness, design strength, and capacity of weld (R_{nw}).
2. Calculate the depth of bracket (length of weld) using the following appropriate equations.
   (a) In the case of groove welds
   $$L_w = [6M/(tf_b)]^{1/2} \tag{11.39}$$
   Where $f_b = f_y/\gamma_{\text{m0}}$ with $\gamma_{\text{m0}} = 1.10$
   (b) In the case of fillet welds
   $$L_w = [6M/\{2t_eR_{\text{nw}}\ (\text{est.})\}]^{1/2} \tag{11.40}$$

A reduced value of R_{nw} is used to account for the direct shear effect also.

3. The direct shear stress is computed using Eqn (11.33) or (11.34), as appropriate.

4. The stress due to bending moment is computed from Eqn (11.35) or (11.36), as appropriate.
5. The equivalent stress is computed from Eqn (11.37) or (11.38), as appropriate.
6. If the equivalent stress exceeds the weld strength (fillet welds) or the design stress of the parent metal (groove weld), the length of the bracket (weld length) may be increased and the process repeated till the checks are satisfied.

11.12.3 Stiffened Beam Seat Connection

A welded stiffened seat connection for a beam is much simpler than the one with a bolted connection. It consists of two plates forming a T or a split I-section used as a seat (see Fig. 11.33). The thickness of the stem of the T should not be less than the web thickness of the beam it supports. Similarly, the thickness of seat plate should not be less than the thickness of the flange of the beam. The length of bearing is governed by the strength as well as by the web crippling requirement of the beam (as in the case of unstiffened seat angle). The depth of the stem should be short enough to avoid local buckling and it depends upon the length of the vertical weld required. The under side of the flange of the T is also welded to increase the torsional stiffness of the connection (see Fig. 11.34). The seat plate is kept wider than the flange of the beam by at least twice the size of the weld on each side of beam flange to facilitate welding. As in the unstiffened beam seat connection, a cleat angle of nominal size is welded to the top of beam in the shop and to the column at the field to provide lateral support to the beam's top flange.

Fig. 11.33 Welded stiffened seat connection using the split I-section in a car parking structure in Bethesda, USA. (The bolts are erection bolts.)

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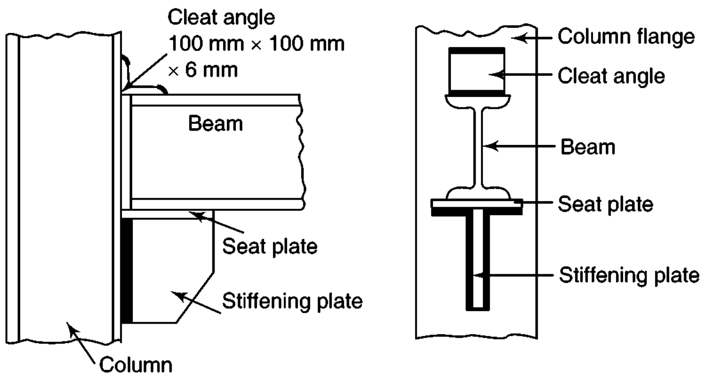


Fig. 11.34 Welded stiffened seat connection

There are two basic types of loading used on stiffened seats. The common one is shown in Fig. 11.34, where a beam web is placed directly in line with the stiffener. The other type which occurs when supporting gantry girders, is shown in Fig. 11.35, in this case the beam is oriented in such a way that the plane of the web is at 90° to the plane of the stiffener. This stiffener behaves similar to an unstiffened element under uniform compression, and local buckling may be prevented by satisfying the limiting width to thickness ratios given in Table 2 of the code.

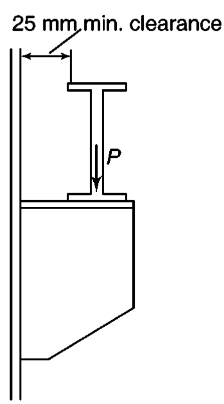


Fig. 11.35 Bracket supporting concentrated load

11.12.3.1 Design

The design of the stiffened welded seat connection is similar to that of the unstiffened welded seat connection. The steps to be followed are as follows.

1. The width of the seat angle is calculated.
2. The thickness of the seat plate is chosen as equal to the thickness of the flange plate.
3. The thickness of stiffening plate is chosen as equal to the thickness of the web of the beam.
4. The eccentricity of the load and bending moment due to it are calculated.

CHAPTER 12

Design of Industrial Buildings

Introduction

High-rise steel buildings account for a very small percentage of the total number of structures that are built around the world. The majority of steel structures being built are low-rise buildings, which are generally of one storey only. Industrial buildings, a subset of low-rise buildings are normally used for steel plants, automobile industries, utility and process industries, thermal power stations, warehouses, assembly plants, storage, garages, small scale industries, etc. These buildings require large column free areas. Hence interior columns, walls, and partitions are often eliminated or kept to a minimum. Most of these buildings may require adequate head room for the use of an over head travelling crane.

The structural engineer has to consider the following points during the planning and design of industrial buildings (Fisher 1984):

(a) Selection of roofing and wall material
(b) Selection of bay width
(c) Selection of structural framing system
(d) Roof trusses
(e) Purlins, girts, and sag rods
(f) Bracing systems to resist lateral loads
(g) Gantry girders, columns, base plates, and foundations

Out of the listed points, gantry girders which support cranes have been discussed in Chapter 8 and columns and base plates have been discussed in Chapters 5 and 9. Foundations are made with reinforced concrete and are outside the scope of this book. Hence, this chapter focusses on the rest of points in the sections to follow.

12.1 Selection of Roofing and Wall Material

The type of roof deck, type of purlin used, purlin spacing, deflections of secondary structural members, roof pitch, and drainage requirements are all determined by the choice of roofing. The roof weight also affects the gravity load design of the roof system and in the case of seismic calculations, the lateral load design.

Similar considerations apply to the cladding/wall systems. In selecting the cladding/wall system, the designer should consider the following areas: (a) cost, (b) interior surface requirements, (c) aesthetic appearance (including colour), (d) acoustics and dust control, (e) maintenance, (f) ease and speed of erection, (g) insulating properties, and (h) fire resistance.

Note that *cladding* carries only its own weight and the weight of the loads imposed by wind. In the case of roofs, the sheeting supports insulation and water proofing in addition to self weight and weight of loads due to wind and/or snow. Hence, it is often termed as *roof decking*. The cladding/wall system will have an impact on the design of girts, wall bracing, eave members, and foundation.

In India, corrugated galvanized iron (GI) sheets are usually adopted as coverings for roofs and sides of industrial buildings. Now light-gauge cold-formed ribbed steel or aluminium decking (manufactured by cold drawing flat steel or aluminium strips through dies to produce the required section) is also available. Sometimes asbestos cement (AC) sheets are also provided as roof coverings owing to their superior insulating properties. Their insulating properties may be enhanced by painting them white on the top surface. These three types of sheets are discussed briefly in the following section.

12.1.1 Steel or Aluminium Decking/Cladding

The modern built-up roof system consists of three basic components: steel/aluminium deck, thermal insulation, and membrane. The structural deck transmits gravity, wind, and earthquake forces to the roof framing. Thermal insulation is used for reducing heating and cooling costs, increasing thermal comfort, and preventing condensation on interior building surfaces. The membrane is the water-proofing component of the roof systems. On sloping roofs, the insulation consists of the insulation board or glass wool. On flat roofs, insulation board, felt, and bitumen are laid over the steel decking as shown in Fig. 12.1.

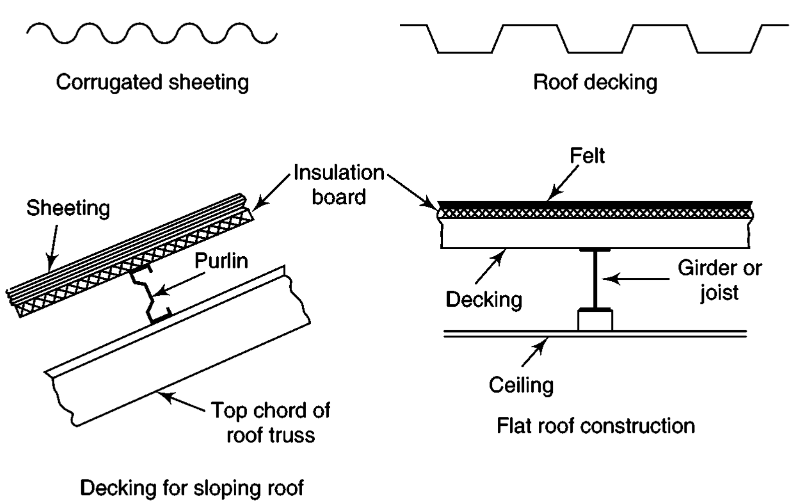


Fig. 12.1 Roof materials and constructions

The steel decking has a ribbed cross section, with ribs generally spaced at 150 mm (centre to centre) and 37.5 mm or 50 mm deep (see Fig. 12.2). The sloped-side ribs measure about 25 mm wide at the top for a narrow rib deck, 44 mm for an intermediate rib deck, and 62.5 mm for a wide rib deck. Wide rib decking is more popular, which can be used with 25-mm-thick insulation boards. Thinner insulation boards may require narrow deck rib opening. Wide rib decking also has higher section properties than other patterns, and hence can be used to span greater distances. These steel decks may be anchored to supporting flexural members by puddle welds by a welder, (power-activated and pneumatically driven fasteners, and self-drilling screws can also be used), as soon as the deck is placed properly on the rafters or top chord of the roof truss (Vinnakota 2006).

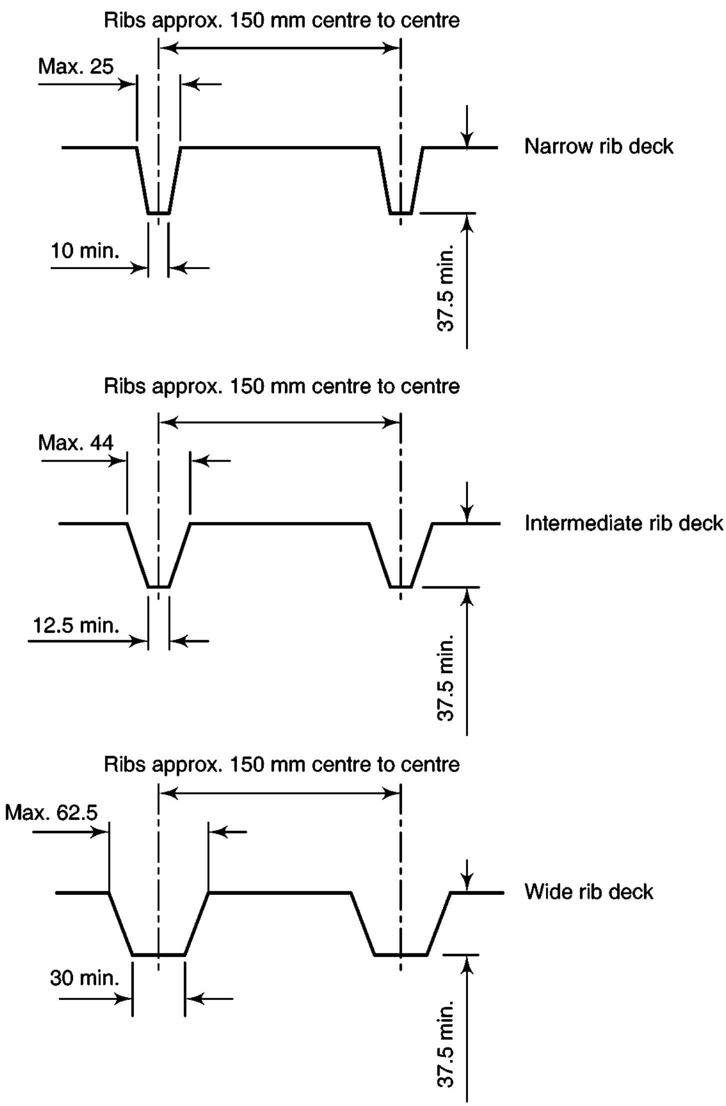


Fig. 12.2 Typical profiles of roof deck

Steel decks are available in different thicknesses, depths, rib spacing, widths, and lengths. They are available with or without stiffening elements, with or without acoustical material, as in cellular and non-cellular forms. The cellular decks can be used to provide electrical, telephone, and cable wiring and also serve as ducts for air distribution. They are also available with different coatings and in different colours. They are easy to maintain, durable, and aesthetically pleasing.

When properly anchored to supporting members, steel/aluminium decks provide lateral stability to the top flange of the structural member. They also resist the uplift forces due to wind during the construction stage. Steel decks may be considered as a simply supported or continuous depending on the purlin and joist spacing. Aluminium sheets also offer excellent corrosion resistance. But they expand approximately twice as much as steel and are easily damaged in hailstorms. Moreover, aluminium sheeting should be separated not only from steel purlins but also from any non-aluminium roof-top framing and conduits, in order to avoid bi-metallic corrosion. The fasteners connecting aluminium sheets to steel purlins should be made of stainless steel. The aluminium alloy panels should be at least 0.8 mm thick and at least 1 mm thick for longer spans.

The load carrying capacity of the deck is influenced by the depth of the cross section, the span length, the thickness of metal, and whether it is simply supported or continuous. The manufacturers provide load tables, which can be used to select the deck for the required span. The weight of roofing varies from 0.3 kN/m^2 to 1.0 kN/m^2, including the weight of joists, and 0.05 kN/m^2 to 0.1 kN/m^2, excluding the weight of joists.

Metal roofing can also be classified by the method of attachment to supports. *Through-fastened roofs* are attached directly to purlins, usually by self-tapping screws, self-drilling, or lock rivets. *Standing-seam roofs*, on the other hand, are connected indirectly by concealed clips formed into the seams. Standing-seam roofing is often used in USA and was introduced by Butler Manufacturing Company in 1969.

More details about steel decking are provided by Petersen (1990), Schittich (2001), and Newman (2004).

12.1.2 Galvanized Iron (GI) Sheets

Most common sizes of corrugated GI sheets are as follows:

(a) 8 corrugations (75 mm wide and 19 mm deep) per sheet
(b) 10 or 11 corrugations (75 mm wide and 19 mm deep) per sheet

The available sizes of sheets are as follows:

(a) Length—1.8, 2.2, 2.5, 2.8, and 3 m
(b) Width—0.75 m and 0.9 m
(c) Thickness—0.63, 0.8, 1.0, 1.25, and 1.6 mm

The weights of the sheets vary from 50–156 N/m^2. When the sheets are installed, side laps and end laps should be provided to make the joint water proof. The following overlaps are normally used:

(a) For roof: Side overlap—1½ to 2 corrugations
End overlap—150 mm

(b) For side cladding: Side overlap—1 corrugation
End overlap—100 mm

The sheets are fastened to purlins or side girts by 8-mm-diameter J- or L-type hook bolts with GI nuts along with GI and bituminous felt washers at a maximum pitch of 350 mm. Where laps do not occur over supports, 6-mm diameter bolts at a maximum pitch of 250 mm for roofs and 300–450 mm for sides are used.

Spacing of purlins and girts, which support the sheeting is governed by the length of the sheet, thickness of the sheet, and applied loading. The approximate section modulus of the corrugated GI sheeting may be taken as

$$Z = (4/15)bdt \tag{12.1}$$

where b is the curvilinear width (equal to $1.13 \times$ covering width), d is the depth of the corrugation, and t is the thickness of the sheet.

Based on the above formula, the maximum purlin spacing is 1.8 m for a 2000-mm long and 750-mm wide sheet.

12.1.3 Asbestos Cement Sheets

Asbestos cement sheets may be used to cover the roof as an alternative to corrugated steel sheets. (These sheets are banned in many countries due to the risk of lung cancer caused by inhaling the fibres, while working with these sheets.) AC sheets are manufactured in two shapes, corrugated and Trafford, and are available in lengths of 1.75, 2.0, 2.5, and 3 m. They are manufactured in thicknesses of 6 mm or 7 mm. The maximum permissible spacing of purlins is as follows:

(a) for 6-mm sheet—1.4 m
(b) for 7-mm sheet—1.6 m

For side cladding, the spacing may be increased by 300 mm.

A side overlap of one corrugation is normally given. The end lap should not be less than 150 mm for slopes less than 18° and for flatter slopes this overlap may be increased. For side covering, an overlap of 100 mm is sufficient.

The weight of asbestos sheets varies from 160 N/m^2 to 170 N/m^2. The load per square metre of the sheet on the slope may be increased by 30% to get the load per square metre of the plan area, to account for the larger area on the slope and additional material in the side and end lapping. The sheets are fastened to purlins or girts by using 8-mm-diameter hook bolts at a maximum spacing of 350 mm.

In addition to steel, aluminium, GI and AC sheets, stainless steel and ferrocement roofing sheets can also be used. Ferrocement sheets can be produced in different shapes and sizes. Ferrocement sheets withstand heavy rainfall, cyclone, fire, and termite attack and are as durable as reinforced concrete. The fabrication does not involve any heavy machinery and the cost of ferrocement sheets is approximately 30% cheaper than conventional GI or corrugated AC roof sheeting (Mathews & Rao 1979).

Roof-top Equipment Roof-top mounted or suspended HVAC (Heating, Ventilation, and Air Conditioning) equipment may include anything from small fans and unit heaters to large air-conditioning units. They may be supported by a continuous curb or an elevated steel frame on legs. A properly designed and installed curb with sheet flashing may be less prone to leakage than discrete penetrations at frame legs (Newman 2004).

12.2 Selection of Bay Width

A bay is defined as the space between two adjacent bents (see Fig. 1.26). The roof truss along with the columns constitutes a bent. The space between two rows of columns of an industrial building is called an aisle or span. An industrial building may have a single span or multiple spans. Figure 12.3 shows industrial buildings with single, double, and multiple spans.

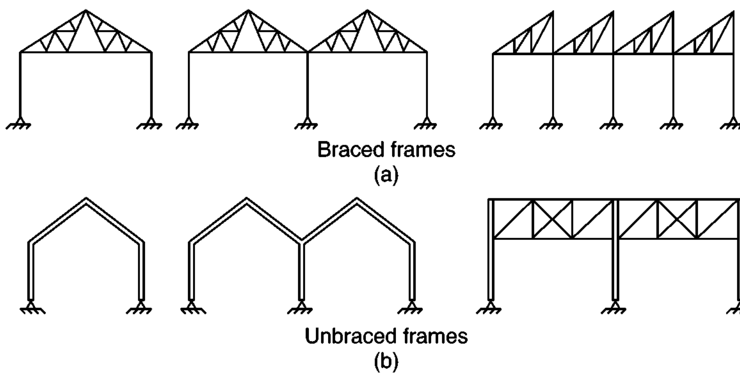


Fig. 12.3 Industrial buildings with single, double, and multiple spans

In most cases, the bay width may be dictated by owner requirements. Gravity loads generally control the bay size.

For crane buildings (for light and medium cranes), bays of approximately 4–8 m may be economical because of the cost of the crane gantry girders. Large bays may increase the cost of the tension flange bracing of the gantry girders. Though the bay widths in the range of 4–8 m provide economy, truss spans may range from 10–25 m or more.

12.3 Structural Framing

For the purpose of structural analysis and design, industrial buildings are classified as (see Fig. 12.3):

- Braced frames
- Unbraced frames

In braced buildings, the trusses rest on columns with hinge type of connections and the stability is provided by bracings in the three mutually perpendicular planes. These bracings are identified as follows:

(a) Bracings in the vertical plane in the end bays in the longitudinal direction [see Fig. 12.4(a)]

(b) Bracings in the horizontal plane at bottom chord level of the roof truss [see Fig. 12.4(c)]

(c) Bracings in the plane of upper chords of the roof truss [see Figs 12.4(a) and (b)]
(d) Bracings in the vertical plane in the end cross sections usually at the gable ends [see Figs 12.4(a) and (c)]

The function of a bracing is to transfer horizontal loads from the frames (such as due to wind or earthquake or horizontal surge due to acceleration and breaking of travelling cranes) to the foundation. The longitudinal bracing on each longitudinal end provides stability in the longitudinal direction. The gable bracings provide stability in the lateral direction. The tie bracings at the bottom chord level transfer lateral loads (due to wind or earthquake) of trusses to the end gable bracings.
Similarly stability in the horizontal plane is provided by

- a rafter bracing in the end bays, which provide stability to trusses in their planes or
- a bracing system [see Fig. 12.4(c)] at the level of bottom chords of trusses, which provide stability to the bottom chords of the trusses.

Purlins act as lateral bracings to the compression chords of the roof trusses, which increase the design strength of the compression chords. The lateral ties provide similar functions to the bottom chord members when they are subjected to compression due to reversal of loading (see Section 12.5.8 also). X bracings (as shown in Fig. 12.4) are the commonly used bracing systems. K-type bracing systems may also be used. If the building is lengthy, bracings in the end bays alone may not be sufficient. In these cases, every fourth or fifth bay is braced and the roof upper chord bracings are also provided in these bays.

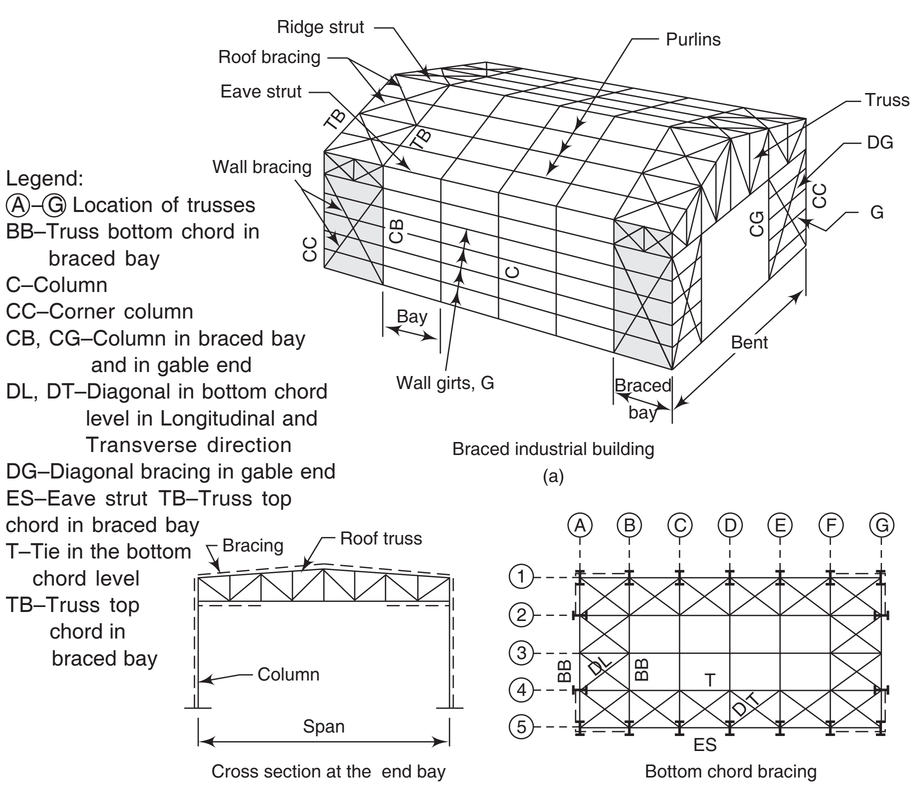


Fig. 12.4 Structural framing for an industrial building

Braced frames are efficient in resisting the loads and do not sway. However, the braces introduce obstructions in some bays and may cause higher forces or uplift forces in some places. Wide flange columns are often used for exterior columns of braced frames. (For interior columns of braced frames with height less than 7 m, Square Hollow Section (SHS) columns may yield most economical solution because of their high radius of gyration about both axes.)

12.3.1 Unbraced Frames

Unbraced frames in the form of *portal frames* is the most common form of construction for industrial buildings, distinguished by its simplicity, clean lines, and economy. The frames can provide large column free areas, offering maximum adaptability of the space inside the building. Such large span buildings require less foundation, and eliminate internal columns, valley gutters, and internal drainage. Portal frame buildings offer many advantages such as more effective use of steel than in simple beams, easy extension at any time in the future, and ability to support heavy concentrated loads. The disadvantages include relatively high material unit cost and susceptibility to differential settlement and temperature stresses. In addition, these frames produce horizontal reaction on the foundation, which may be resisted by providing a long tie beam or by designing the foundation for this horizontal reaction.

Basically, a portal frame is a rigid jointed plane frame made from hot-rolled or cold-rolled sections, supporting the roofing and side cladding via hot-rolled or cold-formed purlins and sheeting rails (see Fig. 12.5). The typical span of portal frames is in the range of 30–40 m, though they have been used in 15–80 m, spans. The bay spacing of portal frame may vary from 4.5 to 10 m (typical bay spacing is 6 m). The eave height in a normal industrial building is about 4.5 m to 6.0 m (which corresponds to the maximum height of one level of sprinklers for fire protection). Recent portal frames have a roof slope between 6° and 12°, mainly chosen because of the smaller volume of air involved in heating the building. But in such cases, frame horizontal deflections must be carefully checked and proper foundations should be provided to take care of the large horizontal thrust.

Although the steel weight in braced frame buildings is often less than that for a comparable portal frame building, the overall cost is generally higher because of the greater amount of labour involved in fabrication. The portal frame systems, as shown in Fig. 12.5, are often designed, prefabricated, supplied, and erected at site by firms in USA and are called *pre-engineered buildings* or *metal building systems.*

Complete information on the elastic and plastic design method of portal frames is provided by King (2005).

12.4 Purlins, Girts, and Eave Strut

Secondary structural members such as *purlins* and *girts* span the distance between the primary building structures (portal frames or truss-column system). They support the roof and wall covering and distribute the external load to the main

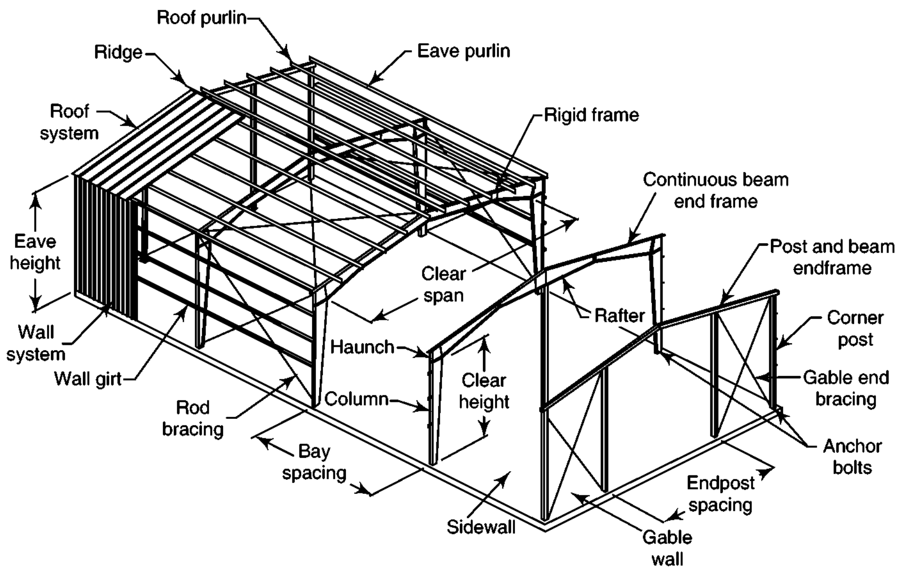


Fig. 12.5 Typical portal frame construction

frames or trusses (see Fig. 12.4). They also serve as the flange bracing for the rafters or columns and may function as a part of the building's lateral load resisting system. Purlin is a part of the roof bracing system and girts form a part of the wall bracing system of the building. The behaviour and design of purlins has been discussed in Section 6.11 and the design of sag rods in Section 3.9. Examples 6.10 and 3.10 illustrate the various steps involved in the design of purlins and sag rods, respectively. When sag rods are used for bracing the purlins top flange, it is advantageous to locate the sag rods 50–75 mm below the top of the compression flange. The weight of purlins in the total weight of the steel structure could vary from 10% to 25%. The weight of purlins may be equal to or even greater than the weight of the trusses. Hence they have to be designed properly. Usual members adopted for purlins include channels, angles, tubes, and cold-formed C- and Z-sections. When cold-formed sections are used, they should be properly protected with anti-corrosive treatment, since their thickness ranges from 1.6 mm to 4 mm only. Cold-formed C-, Z-, or sigma-purlins may be economical than hot-rolled purlins for spans of 5 m to 8 m. Angles and channel purlins without sag bars may be economical up to 5 m and tubes up to 6 m.

The main function of *girts* is to transfer wind loads from wall materials to the primary frame. Girts are positioned horizontally (see Fig. 12.4) to span between the columns. When the space between primary columns is more than 9 m, *wind columns* may be provided to reduce the girt span. Wind columns are essentially intermediate vertical girts spanning from the foundation to the eave. Since typical eave strut may not be capable of resisting the lateral reaction imposed by the wind column, a system of diagonal braces should be provided to transfer the lateral reaction to the adjacent primary framing columns. Similar to purlin spacing, girt spacing is governed by the load—resisting properties of wall panels.

The third type of secondary structural members, after purlins and girts, is the *eave strut.* This member is located at the intersection of the roof and the exterior wall (see Fig. 12.4) and hence acts as both the first purlin and the last (highest) girt. The building's eave height is measured to the top of this member.

The eave strut is a relatively strong member and its functions are as follows:

- It serves as a stiff binder beam.
- Cladding is often hung from the eave strut; hence the total load of cladding including side girts should be carried by this beam.
- In braced buildings, the wind bracing along the eave strut acts as a truss in the plan view [see Fig. 12.4(c)]. As already discussed, this truss transfers the horizontal loads on the roof and cladding to the gable end bracings. Therefore, the eave strut acts as a compression chord of the wind bracing truss.
- Eave strut also supports drain gutters and other secondary elements.

Since a relatively stiff section is required, the eave girder is often composed of a built-up two channels face-to-face.

12.5 Plane Trusses

A structure that is composed of a number of line members pin-connected at the ends to form a triangulated framework is called a *truss*. If all the members lie in a plane, the structure is a *planar truss.* In a truss, the members are so arranged that all the loads and reactions occur only at the joints (intersection points of the members). The centroidal axis of each member is straight, coincides with the line connecting the joint centres at each end of the member, and lies in a plane that also contains the lines of action of all the loads and reactions. The primary principle underlying the use of the truss as a load-carrying structure is that arranging elements into a triangular configuration results in a stable shape. Any deformations that occur in this stable structure are relatively minor and are associated with small changes in member length caused by the forces in the members by the external loads. Similarly, the angle formed between any two members remains relatively unchanged under load.

Trusses were found to have been constructed as early as 500 B.C. when Romans built a bridge using a form of timber truss across the Danube river. Though the potential of trusses were known and used in a few large public buildings in USA and Italy, the bridge builders of the early nineteenth century were responsible for the systematic use of the truss systems.

In simple roof systems the three-dimensional framework can be subdivided into planar components for analysis as planar trusses, purlins, etc., without seriously compromising the accuracy of the results. The external loads (which are applied at the joints) produce only tensile or compressive forces in the individual members of the truss. For common trusses with vertically acting loads, compressive forces are usually developed in the top chord members and tensile forces in the bottom chord members. Though the forces in the web members of a truss may be either tension or compression, there is often an alternating pattern of tensile and compressive forces present. Note that when the external loads reverse in direction (e.g., as in the case of wind loads) the top chords will be in tension and bottom chords will be in compression. Hence, it is often necessary to design the various members of a truss both for tension and compression and select the member size based on the critical force.

It is extremely important to note that when the loads are applied directly onto truss members themselves (as in the case of intermediate purlins), bending stresses will also develop in those members in addition to the basic tensile or compressive stresses. This results in complicated design procedures (they should be designed as per the provisions of beam-columns discussed in Chapter 9) and the overall efficiency of the truss is reduced (see Section 12.5.6 also).

12.5.1 Analysis of Trusses

The first step in the analysis of a truss is always to determine whether the truss under consideration is a stable configuration of members. There exists a relation between m, the number of members, j, the number of joints, and r, the reaction components. Thus, the expression

$$m = 2j - r \tag{12.2}$$

must be satisfied, if the truss is a internal statically determinate structure. The least number of reaction components required for external stability is r (equal to 3 for plane trusses). If m exceeds $(2j - r)$, then the excess members are called redundant members and the truss is said to be statically indeterminate. If fewer members, than those given by the expressions in Eqn (12.2), are present, then the truss will be unstable.

For a determinate truss, when purlins are located at the nodal points, the member forces can be found by employing the laws of statics to assure internal equilibrium of the truss. The process requires the repeated use of free-body diagrams, from which individual member forces are determined. *The method of joints* is a technique of truss analysis in which the member forces are determined by the sequential isolation of joints—the unknown member forces at one end of the truss are solved, which are then used to determine the member forces at subsequent joints. The other method is known as the *method of sections* in which the equilibrium of a part of the truss is considered, and the member forces are determined by using the three equations of equilibrium. $\Sigma F_x = 0$, $\Sigma F_y = 0$, and $\Sigma M = 0$. The forces in triangulated trusses may also be found by graphical means using force diagrams. The details of the method of joints and method of sections are provided by Thandavamoorthy (2005).

In statically indeterminate trusses (which have more number of members than a determinate truss), though the principles of statics are still valid, they can not be analysed using the method of joints or the method of sections. It is because we may have more number of unknowns than the equations of equilibrium. For analysing these indeterminate trusses, matrix methods of structural analysis are used (Livesley 1975). In this method of structural analysis, a set of simultaneous equations that describe the load-deformation characteristics of the structure under consideration is formed. These equations are solved using matrix algebra to obtain deformations or forces. From the deformations of joints, the member forces may be determined. Matrix algebra is ideally suited for setting up and solving equations in the computer. Two methods of matrix structural analysis are available. The flexibility or force method assumes the forces as unknowns and the displacement or stiffness method assumes the deformations as unknowns. Several commercial computer programs are available for the analysis of indeterminate trusses, which are based mainly on the stiffness method of structural analysis (e.g., SAP 2000 by Computers & Structures, Inc., California, STAAD Pro 2004 by Research Engineers International).

The analysis to find the forces in a multi-member truss by hand calculation can be tedious and time-consuming. Hence in practice, computer programs are often used to analyse determinate or indeterminate trusses. Most of these programs require the user to specify member sizes, so that the analysis can be performed. While forces in determinate structures do not depend upon the 'initial' member sizes given by the user, forces in indeterminate structures do depend upon initial member sizes. Hence it may be necessary to repeat the analysis two or three times, such that the 'initial' member sizes and the designed member sizes are the same. The member sizes given in Tables 12.1 and 12.2 may be used for giving the 'initial' member size input for the trusses shown in Fig. 12.6. These sizes are applicable to trusses with purlins placed at truss joints, having a minimum number of longitudinal ties, as shown in Fig. 12.6. These sizes are derived based on (a) the rise to span ratio being greater than 1/6 and (b) wind permeability of the building being ≤ 20%.

Table 12.1 Initial member size for roof trusses (angle sections) with f_y = 250 MPa

	Member Notations (see Fig. 12.6)				
Span (m)	a	b	c, d	e	f
		Truss spacing < 5 m			
10.0	1-65 × 6*	1-50 × 6	1-50 × 6	1-45 × 6	1-50 × 6
16.0	2-50 × 6	2-50 × 6	2-50 × 6	1-45 × 6	1-60 × 6
21.0	2-60 × 6**	2-50 × 6	2-50 × 6	1-50 × 6	1-65 × 6
26.0	2-65 × 6	2-60 × 6	2-60 × 6	1-60 × 6	1-65 × 6
		(Longitudinal ties: ISA 75 × 6)			
		5 m < Truss spacing < 6.5 m			
10.0	1-75 × 6	1-60 × 6	1-60 × 6	1-45 × 6	1-60 × 6
16.0	2-50 × 6	2-50 × 6	2-50 × 6	1-50 × 6	1-65 × 6
21.0	2-65 × 6	2-60 × 6	2-60 × 6	1-50 × 6	1-75 × 6
26.0	2-75 × 6	2-65 × 6	2-65 × 6	1-65 × 6	1-75 × 6
		(Longitudinal ties: ISA 90 × 6)			

* 1-65 × 6 means single angle ISA 65 × 65 × 6

** 2-60 × 6 means double angle ISA 60 × 60 × 6

12.5.2 Types of Trusses and Truss Configurations

Prior to analysing and selecting members for a roof truss, three engineering decisions are to be made: (a) the form of the chords must be determined, that is, whether they should be flat or sloping and whether they should be straight or curved; (b) the pattern of internal triangulation; (c) whether the trusses are simply supported or continuous. Important dimensional variables include the spans and depths of trusses, lengths of specific truss members (especially compression members), spacing of trusses, and transverse purlin spacing (this, in turn, dictates the way loads are applied on to the trusses and frequently, the placement of nodes within a truss).

Table 12.2 Initial member sizes for roof trusses with f_y = 220 MPa (Tubular members)

Span (m)	Member Notations (see Fig. 12.6) a	b	c, d	e	f
		Truss spacing-5 m			
10.5	60.3 × 3.25*	48.3 × 3.25	48.3 × 3.25	42.4 × 3.25	42.5 × 3.25
16.0	76.1 × 3.25	48.3 × 3.25	60.3 × 3.65	42.4 × 3.25	42.5 × 3.25
21.0	88.9 × 4.05	60.3 × 3.25	76.1 × 3.25	42.4 × 3.25	48.3 × 3.25
26.0	101.6 × 4.85	76.1 × 3.25	88.9 × 4.05	48.3 × 3.25	60.3 × 3.25
		(Longitudinal tie = ISO 60.3 × 3.65)			
		5 m < truss spacing < 6.5 m			
10.5	76.1 × 3.25	48.3 × 3.25	60.3 × 3.65	42.4 × 3.25	42.4 × 3.25
16.0	88.9 × 4.05	60.3 × 3.25	76.1 × 3.65	42.4 × 3.25	48.3 × 3.25
21.0	101.6 × 4.85	76.1 × 3.25	88.9 × 4.05	48.3 × 3.25	48.3 × 3.25
26.0	114.3 × 4.50	76.1 × 3.65	88.9 × 4.85	48.3 × 3.25	60.3 × 3.25
		(Longitudinal ties = ISO 76.1 × 3.25)			

* 60.3 × 3.25 means tube having 60.3 mm outer diameter and 3.25 mm thickness.

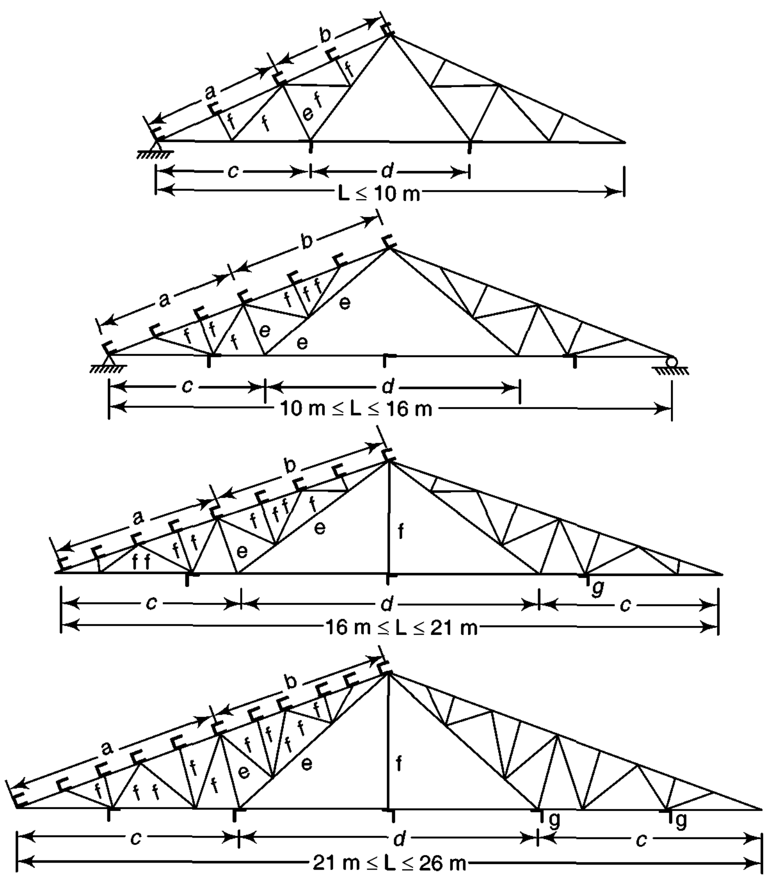


Fig. 12.6 Member notation: guidelines to member design (Refer Tables 12.1 and 12.2)

A variety of truss types have been used successfully and some common truss types are shown in Fig. 12.7. The designations *Pratt*, *Howe*, and *Warren* were originally used with parallel chord trusses [see Figs 12.7(e), (f), (g)], but now they are used more to distinguish between web systems in either flat or sloped chorded trusses. Pratt, Howe, and Warren were nineteenth century bridge designers who developed and popularized these forms. In Pratt truss the diagonals, which are longer and more heavily loaded than the adjacent verticals, are in tension under gravity loading; whereas in the comparable Howe truss they are in compression [for the loading shown in Fig. 12.7(f)]. However, the wind uplift may cause reversal of stresses in the members and nullify this benefit. Hence Pratt type trusses are more desirable than the Howe type trusses. Note that the diagonals having a slope of 40°–50° with the horizontal have been found to be the most efficient. Also the reversal of the direction of diagonals at mid-span in such trusses is characteristic of design for symmetrical loading. In a Warren truss, approximately half the diagonals are in compression. All except the end verticals are secondary members, and hence may be eliminated, without affecting the overall stability.

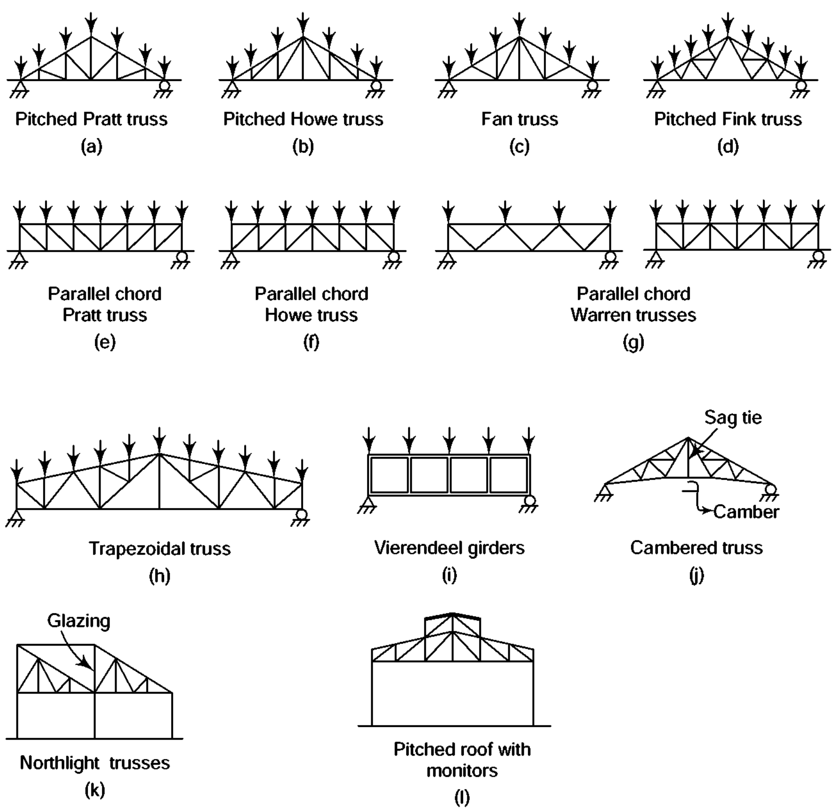


Fig. 12.7 Common types of trusses

The advantage of parallel chord trusses is that they use webs of the same lengths and thus reduce fabrication costs for very long spans. The economical span to depth ratio of parallel chord trusses is in the range of 12 m to 24 m.

For very long span pitched roof, some depth of truss is provided at the ends, resulting in trapezoidal configuration for the trusses [see Fig. 12.7(h)]. Using this configuration results in the reduction of axial forces in the chord members adjacent to the supports. The secondary bending effects in those members are also reduced. The vertical members at supports with length about 1/10th of the truss height at mid-span are found to reduce the forces in the members adjacent to supports considerably (Sree Ramachandra Murthy et al. 2004). For very long spans (greater than 30 m), it may be economical to have trapezoidal trusses with sloping bottom and top chords. Such a configuration will reduce the length of web members and will result in uniform force in the chord members over the entire span. The slope of the bottom chord equal to about half the slope of the top chord is found to be more efficient.

The pitched Fink truss [Fig. 12.7(d)] usually proves to be economical for small spans (< 9 m), since the web members in such trusses are arranged in a fashion to obtain shorter members. As already mentioned, Pratt, Howe, and Warren trusses need not have the top chord parallel to the bottom one. Such an arrangement is used to provide a slope for drainage [Fig. 12.7(a) and 12.7(b)]. Pratt trusses with four or six panels are used for spans varying between 6 m to 15 m. The compound Fink truss shown in Fig. 12.7(d) may be used for a longer span. The simple fan truss [Fig. 12.7(c)] may be used to span 12 m and a compound fan truss can span up to 24 m. Fan trusses are often used when the rafter members of the roof trusses are to be subdivided into odd number of panels. A combination of fink and fan can also be used.

When the chords are parallel and diagonals are removed, as shown in Fig. 12.7(i), they are called as *Vierendeel girders*. In a Vierendeel girder, the loading is carried by a combination of pure flexure and flexure due to shear induced by the relative deformation between the ends of the top and bottom chord members, similar to that found in castellated beams. Though vierendeel girders are usually fabricated using I-sections, their load carrying capacity may be enhanced by using rolled hollow sections with butt or fillet welded connections. They may be analysed using elastic analysis and the moment capacities of the members checked using effective length factors for frames not braced against side sway (Martin & Purkiss 1992).

The lower chord of the trusses may be left straight as shown in Figs 12.7(a) and (d) or may be cambered, that is, fabricated with slight upward curve in the bottom chord member as shown in Fig. 12.7(j). The camber may be approximately in the range of 0.5 m to 1.0 m. Cambering is done for the sake of appearance so that in a lengthy room, a series of trusses, one behind the other, may not appear to sag. Cambering results in additional fabrication cost and cambered trusses involve careful assembly at site. A sag tie, as shown in Fig. 12.7(j), may be used to reduce the moment due to self weight in the long middle tie member and to reduce the resulting deflection of this member. The sag tie may also be used to carry the load due to the weight of the ceiling hung from the bottom chord, if any.

In roof trusses, the drainage, lighting, and ventilation requirements are the most important elements in establishing the upper chord slope, but occasionally, structural or aesthetic reasons may also control.

In single storey industrial buildings of the type shown in Figs 12.7(k) and (l), drainage is provided toward the eaves and valleys where longitudinal gutters and downspouts are used to carry off water. Artificial lighting is supplemented by windows in the sides of the monitors or the steeper slopes of north light or saw tooth roofs. Portions of the windows can be opened for natural ventilation.

For large column free areas, *lattice girders* are often used (see Fig. 12.8). Because of their greater depth, they usually provide greater stiffness against deflection. When lattice girders are employed as shown in Fig. 12.8, they normally span the width of the building. The saw-tooth, umbrella, and butterfly roofs span the length of the building and are supported by these lattice girders at frequent intervals. The lattice girders are parallel chord trusses as shown in Figs 12.7(e) to (g). The external loads applied through the roofing sheets and purlins are transferred to the lattice girders through the saw-tooth, umbrella, or butterfly trusses, which transfer the load to the end columns. Individual panel lengths are selected as per the spacing of the saw-tooth, umbrella, or butterfly trusses.

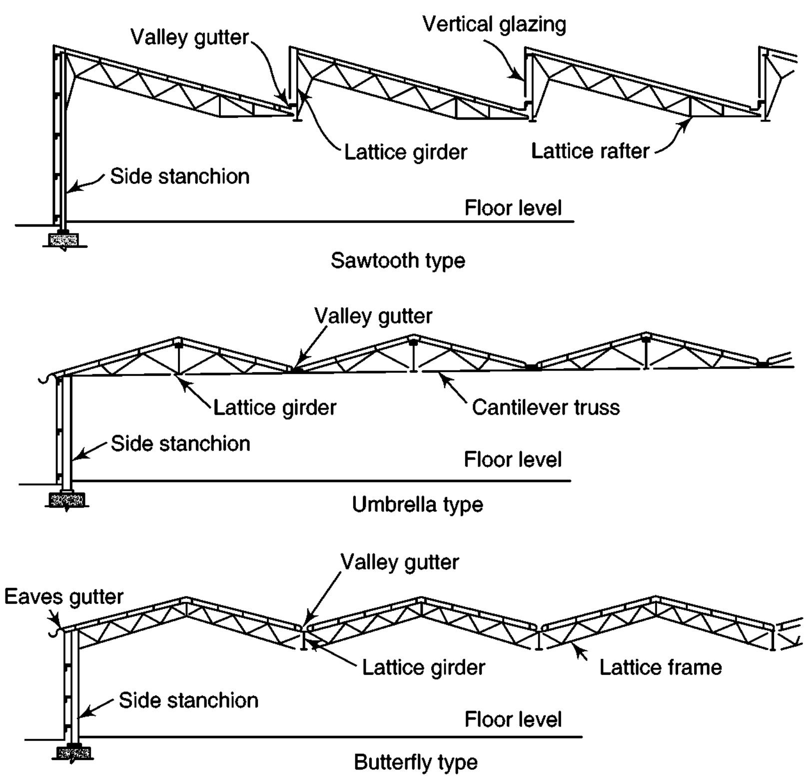


Fig. 12.8 Sawtooth, umbrella, and butterfly roofs

12.5.3 Pitches of Trusses

As seen in Fig. 12.7, most of the trusses are pitched. This is mainly done to drain off rain water on the sheeted slopes. In addition to providing the slope, the joints in the sheetings should be effectively sealed with mastic or washers. The pitch of a truss is defined as the ratio of the height of the truss to its span. The pitches usually provided for various types of roof coverings are given in Table 12.3.

Table 12.3 Pitch for roof trusses

Roof covering	Pitch
Corrugated GI sheets	1/3 to 1/6
Corrugated AC sheets	1/6 to 1/12
Lapped shingles (e.g. wood, asphalt, clay, and tile)	1/24 to 1/12
Flat roof and trapezoidal trusses	1/48 to 1/12

A pitch of 1/4 is found economical in cases where the roof has to carry snow loads in addition to wind loads. Where snow loads do not occur, lower pitches up to 1/6 are suitable. Lower pitches are advantageous since the wind pressure on the roof is reduced.

12.5.4 Spacing of Trusses

The spacing of trusses is mostly determined by the spacing of supporting columns, which in turn is determined by the functional requirements. Where there are no functional requirements, the spacing should be such that the cost of the roof is minimized. The larger the spacing, the smaller the cost of trusses, but larger is the cost of purlins and vice-versa. Roof coverings also cost more, if the spacing of the trusses is large.

Let us derive an approximate formula for arriving at the minimum cost, by considering the following variables.

S is the spacing of the trusses, C_t is the cost of trusses/unit area, C_p is the cost of purlins/unit area, C_r is the cost of roof coverings/unit area, and C is the overall cost of the roof system/unit area.

Since the cost of the truss is inversely proportional to the spacing of truss,

$$C_t = k_1/S$$

where k_1 is a constant. Similarly, the cost of purlins is directly proportional to the square of spacing of trusses. Thus,

$$C_p = k_2S^2$$

The cost of roof coverings is directly proportional to the spacing of trusses. Thus, we have

$$C_r = k_3S$$

Total cost $C = C_t + C_p + C_r$

$$= (k_1/S) + k_2S^2 + k_3S$$

For the overall cost is to be minimum, $\mathrm{d}C/\mathrm{d}S$ should be zero. Thus,

$$-(k_1/S^2) + 2k_2S + k_3 = 0$$

or $$(-k_1/S) + 2k_2S^2 + k_3S = 0$$

or $$-C_t + 2C_p + C_r = 0$$

Thus, we get $C_t = C_r + 2C_p$ (12.3)

Equation (12.3) shows that an economic system is obtained when the cost of trusses is equal to the cost of roof covering plus twice the cost of purlins. It has been found that the economic range of spacing is 1/5 to 1/3 of span. For lighter load, say, carrying no snow or superimposed load except wind, the larger spacing may be more economical. Spacing of 3–4.5 m for spans up to 15 m and 4.5–6 m for spans of 15–30 m may result in economy.

12.5.5 Spacing of Purlins

The spacing of purlins depends largely on the maximum safe span of the roof covering and glazing sheets. Hence, they should be less than or equal to their safe spans when they are directly placed on purlins. Thus for corrugated GI sheets, the purlin spacing may vary from 1.5 to 1.75 m, and for corrugated AC sheets, it is limited to 1.4 m, for 6-mm thick sheets, and 1.6 m, for 7-mm thick sheets. For larger spans, if the configuration of the truss is such that it is not feasible to place purlins at the nodes of upper chords, the purlins are placed between the nodes, thus introducing bending moments in the upper chords, in addition to the compressive force due to truss action (see Fig. 12.9). Hence in this case, the weight of the truss may be increased by about 10–15%. Therefore, it is preferable to place purlins at the nodal point of the truss, so that the upper chord members are subjected to only direct compression.

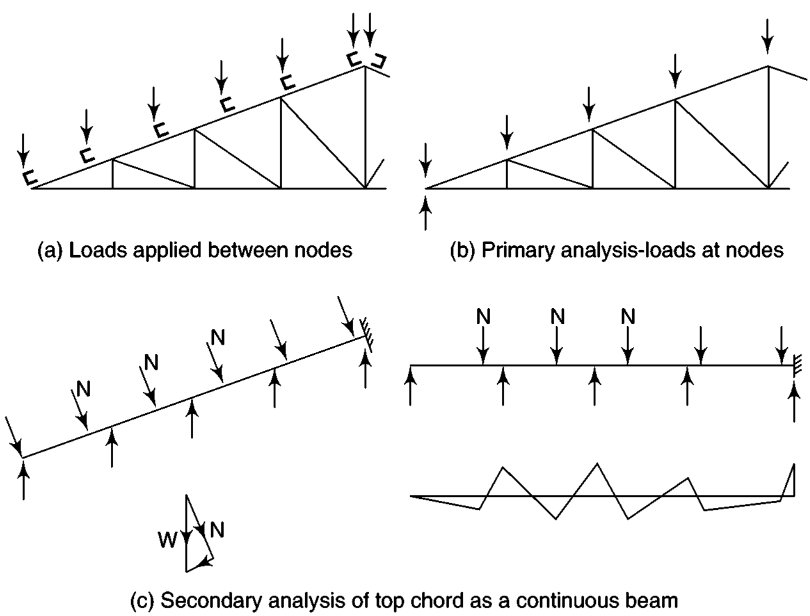


Fig. 12.9 Loads applied between nodes of truss

As discussed in Chapter 2, wind loading is not uniform over the roof; for example, the loading is much higher along the roof's perimeter and sometimes along the ridge. Instead of using structural roofing panels of heavier gauges, in the areas of higher localized loads, it is better to space the purlins closer.

12.5.6 Loads on Trusses

The main loads on trusses are dead, imposed, and wind loads. The dead load is due to sheeting or decking and their fixtures, insulation, felt, false ceiling (if provided), weight of purlins, and self weight. This load may range from 0.3 to 1.0 kN/m^2. Also the truss may be used for supporting some pipe line, fan, lighting fixtures, etc. Hence to take into account this probability, it may be worthwhile considering an occasional load of about 5 to 10 kN distributed at the lower panel points of the truss.

The weights of the purlins are known in advance as they are designed prior to the trusses. Since the weight of the truss is small compared to the total dead and imposed loads, considerable error in the assumed weight of the truss will not have a great impact on the stresses in the various members. For live load up to 2 kN/m^2, the following formula may be used to get an approximate estimate of the weight of the trusses:

$$w = 20 + 6.6L \tag{12.4}$$

where w is the weight of the truss in N/m^2 and L is the span of the truss in m. For welded trusses, the self weight of the truss is given by

$$w = 53.7 + 0.53A \tag{12.5}$$

where A is the area of one bay.

For live loads greater than 2kN/m^2, the value of w may be multiplied by the ratio of actual live load in kN/m^2/2. The dead weight of the truss, inclusive of lateral bracing, may also be assumed to be equal to about 10% of the load it supports. Long span trusses are likely to be heavier. The weight of bracings may be assumed to be 12–15 N/m^2 of the plan area. The weight of the truss should be computed after it has been designed to make sure that it is within 10% of the assumed weight.

The imposed load on roofs will be as per IS 875 (Part 2). The snow loads may be computed as per IS 875 (Part 4). The wind loads should be calculated as per IS 875 (Part 3). Wind loads are important in the design of light roofs where the suction can cause reversal of load in truss members. For example, a light angle member is satisfactory when used as a tie but may buckle when the reversal of load makes it to act as a strut.

Since earthquake load on a building depends on the mass of the building, earthquake loads calculated as per IS 1893 (Part 1), 2002, usually do not govern the design of light industrial buildings. Thus wind loads usually govern the design of normal trussed roofs.

12.5.7 Load Combination for Design

As mentioned earlier, the earthquake loads are not critical in the design of industrial building, since the weight of the roof is not considerable. Hence, the following combinations of loads are considered when there is no crane load:

1. Dead load + imposed load (live load)
2. Dead load + snow load
3. Dead load + wind load (wind direction being normal to the ridge or parallel to ridge whichever is severe)
4. Dead load + imposed load + wind load (which may not be critical in most of the cases)

The third combination should be considered with internal positive air pressure and internal suction air pressure separately to determine the worst combination of wind load. When crane load is present, the load combinations as given in IS 875 (Part 2) should be considered. The load combinations mentioned in this section should be considered along with appropriate partial load factors.

All the loads are assumed to act as concentrated loads at points where purlins are located on the upper chord. The weight of the truss is included in the purlin point loads.

12.5.8 Design of Truss Members

The members of the trusses are made of either rolled steel sections or built-up sections depending upon the span length and intensity of loading. Rolled steel single or double angles, T-sections, hollow circular, square, or rectangular sections are used in the roof trusses of industrial buildings [see Fig. 12.10(a)]. In long-span roof trusses and short span bridges, heavier rolled steel sections, such as channels and I-sections are used [see Fig. 12.10(b)]. Built-up I-sections, channels, angles, and plates are used in the case of long-span bridge trusses [Fig. 12.10(c)]. Access to the surface of the members for inspection, cleaning, and repainting during service are important considerations while using built-up sections. Hence in highly corrosive environments, fully closed welded box-sections or hollow sections are used, with their ends fully sealed to reduce the maintenance cost and improve the durability of the trusses.

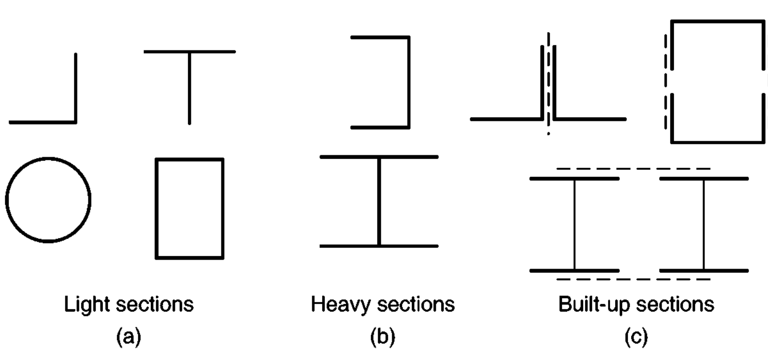


Fig. 12.10 Cross section of truss members

The various steps involved in the design of truss members are as follows:

1. Depending upon the span, required lighting, and available roofing material, the type of truss is selected and a line diagram of the truss is sketched.

2. Various loads acting over the truss are calculated using IS 875 (Parts 1-5).
3. The purlins are designed and the loads acting on the truss at the purlin points are computed.
4. The roof truss is analysed for the various load combinations using the graphical method or the method of sections or joints or by a computer program and the forces acting on the members for various combinations are tabulated.
5. Each member may experience a maximum compressive or tensile force (called the design force) under a particular combination of loads. Note that a member which is under tension in one loading combination may be subjected to reversal of stresses under some other loading combination. Hence, the members have to be designed for both maximum compression and maximum tension and the size for the critical force has to be adopted. The design for compression is done as per Sections 5.8.4, 5.9.1, 5.10, and 5.12. The principal rafter is designed as a continuous strut and the other compression members are designed as discontinuous struts. The limiting slenderness ratios are discussed in Section 5.10.1. The effective length of compression members is taken as per Section 5.8.4.1. Similarly, the design for tension is done as per Section 3.7.2.
6. When purlins are placed at intermediate points, i.e., between the nodes of the top chord, the top chord will be subjected to bending moment in addition to axial compression. Since the rafter is a continuous member, the bending moments may be computed by any suitable method (say, moment distribution method or computer program). Then the member is designed for combined bending and axial compression as per Section 9.5.1.
7. The members meeting at a joint are so proportioned that their centroidal axes intersect at the same point, in order to avoid eccentricity. Then the joints of the trusses are designed either as bolted (see Section 10.6.2) or as welded joints (see Section 11.11.1). If the joint is constructed with eccentricity, then the members and fasteners must be designed to resist the moment that arises. The moment at the joint is divided between the members in proportion of their stiffness.
8. The maximum deflection of the truss may be computed by using either strain energy method or matrix stiffness analysis program. A computer analysis gives the value of deflection as part of the output. This deflection should be less than that specified in Table 6 of the code.
9. The detailed drawings and fabrication drawings are prepared and the material-take-off is worked out.
10. The lateral bracing members are then designed. When a cross braced wind girder is used, as shown in Fig. 12.11(a), it is necessary to use a computer analysis program, since the truss will be redundant. However, it is usual to neglect the compression diagonal and assume that the panel's shear is taken by the tension diagonals, as shown in Fig. 12.11(b). This idealization is useful to make the wind girder determinate and obtain the forces in various members by using method of sections or method of joints.

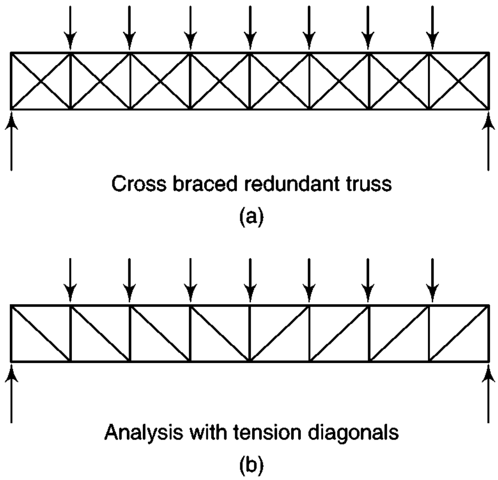


Fig. 12.11 Cross-braced lattice wind girder

The design as per the procedure described here may result in very small angles that are sufficient to resist the forces in the various members of the truss. However, the members should be fairly stiff to avoid damage during loading, transport, off-loading, and erection. Since rafter is the primary compression member, a double angle (equal or unequal) is often preferred. Similarly, the main ties may be subjected to compression during handling or due to wind suction. Moreover, these ties often have a long unsupported length and hence double angle sections are used for these main ties also. All other web members can be designed as single angle members.

From practice, the following minimum sections are recommended for use in compound fink roof trusses.

Rafters—2 ISA 75 × 50 × 6
Main ties—2 ISA 75 × 50 × 6
Centre tie—2 ISA 65 × 45 × 6
Main sling—2 ISA 65 × 45 × 6
Main strut—ISA 65 × 45 × 6
All other members—ISA 50 × 50 × 6

The width of the members should be kept as minimum as possible, since wide members have greater secondary stresses.

While trusses are stiff in their plane, they are very weak out-of-plane. Consider the planar truss as shown in Fig. 12.12(a), in which, the top chord is braced at each panel point. The top chord when subjected to in-plane loading may buckle in the horizontal *xz* plane or the vertical *xy* plane. Unless we provide members having equal moment of inertia about both axes (e.g., square or round members including hollow sections), we have to calculate the buckling strength in the horizontal and vertical plane and adopt the least strength. When transverse members are provided, as shown in Fig. 12.12(b), it is still possible for the top chords to buckle in the horizontal plane. The effective lengths of the top chord members, as far as their resistance to buckling in the horizontal plane is concerned, is $2L$ and not just L, where L is the distance between the nodal points. Note that the members in the vertical plane do nothing to prevent this type of buckling in *xz* plane. Hence in order to make the buckling load in both the horizontal and vertical plane equal, we may have to provide members which are stiffer in the horizontal plane (such as rectangular sections, double angles, or H-sections) as shown in Fig. 12.12(b). When such lateral bracings in the form of purlins are not provided, the entire top chord of the truss may buckle laterally.

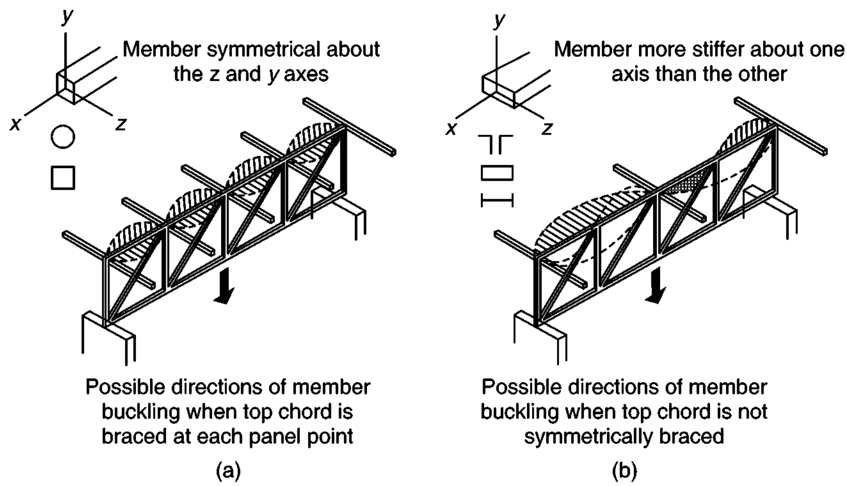


Fig. 12.12 Lateral buckling of truss members: use of transverse members for bracing

The above discussions are valid for the bottom chord member of the truss also, when it is subjected to compression due to reversal of stresses owing to wind suction. Hence lateral (longitudinal) ties are often provided at regular intervals in the bottom chords also (Fisher 1983). Depending upon the *L/r* ratio of the top and bottom chord members in the horizontal and vertical planes, it may be advantageous to adopt unequal angles. The longitudinal ties are under tension in most load situations but may be subjected to compression under wind loading condition depending upon the bracing orientation. There must be at least two longitudinal ties to form a truss action under wind load condition. The longitudinal ties may also be used to support false ceiling. It is desirable to restrict the slenderness ratio of such ties to 250, to avoid sagging.

12.5.9 Connections

Members of trusses can be jointed by riveting, bolting, or welding. As explained in Chapter 10, rivets have become obsolete and for important structures high-strength friction grip (HSFG) bolts and welds are often preferred. Trusses having short span are usually fabricated in shops using welding and transported to site as one unit. Longer span trusses are prefabricated in segments by welding in shop and assembled at site by bolting or welding. For example, fink or compound fink trusses are fabricated as two halves in the workshop. The two halves are transported and assembled at site, where the centre tie is also fitted up (see detail 1 and 3 of

Fig. 12.13). In such situations, the main slings will be subjected to severe handling stresses and hence are made of double angle sections.

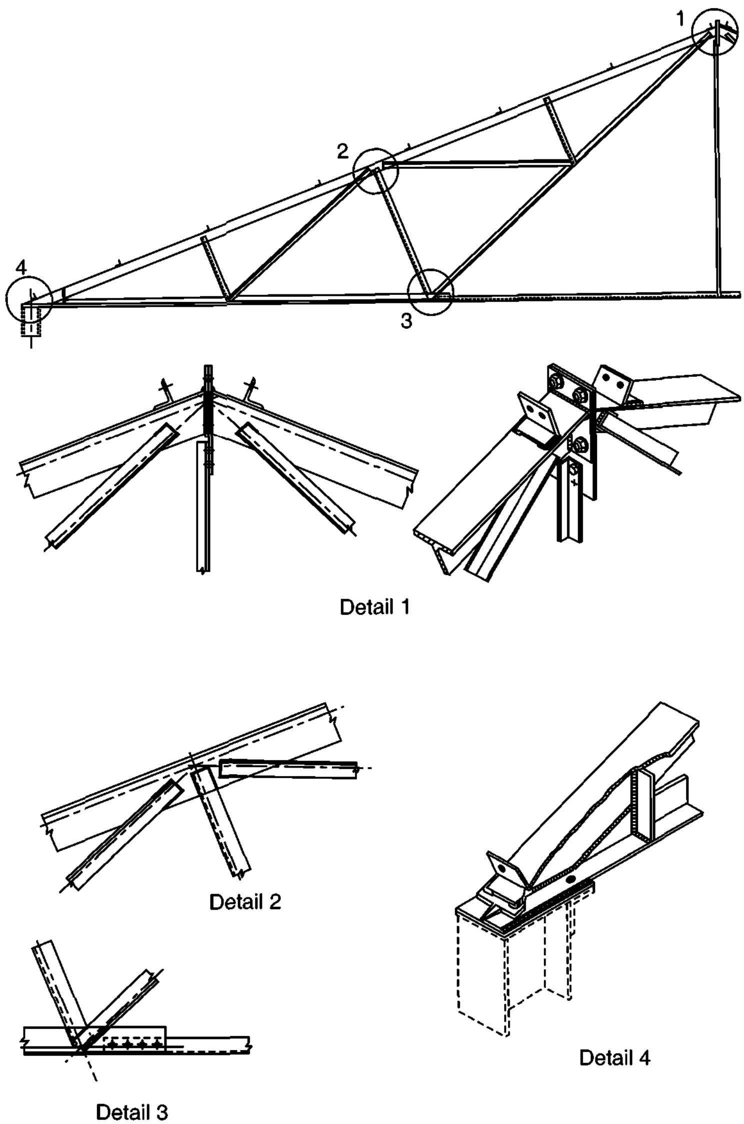


Fig. 12.13 Connection details of welded fink roof truss

If the rafter and tie members are made of T-sections, angle diagonals can be directly welded or bolted to the web of the T-sections. Often, it may not be possible to confine the connection within the width of the member due to inadequate space to accommodate the joint length. In such cases, gusset plates are used. The size, shape, and thickness of gusset plates depend upon the size of the members being joined, number and size of bolt or length of the required weld, and the force to be transmitted (see Section 10.6.2 for more discussion on gusset plate design). The connections should be so arranged that the centroidal axes of members meeting at the connection meet at a point. Example 10.12 shows the calculations required for the design of a bolted truss joint. Figure 12.14 shows typical bolted joints in trusses and lattice girders. Examples 11.8 and 11.9 showed the calculations required for the design of welded joints in trusses. The choice between welded and bolted connections depends on the equipment available with the fabricator and availability of electricity at site (if site welding is preferred). However, when large number of trusses are made, welded joints are economical. Morever, welded joints give better appearance and are easy to maintain. Standard joints should be used with as much repetitions of member shapes and sizes, end preparation, and fabrication operations as possible. This can be easily achieved with parallel chord lattice girders.

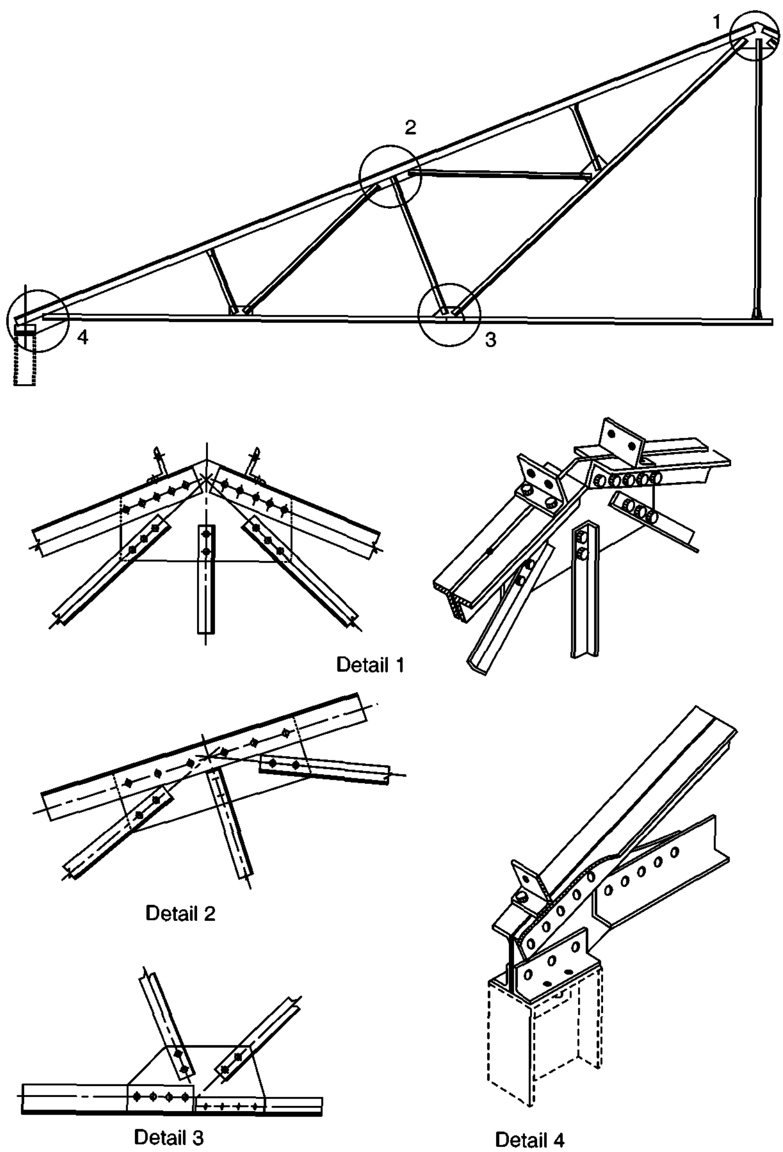


Fig. 12.14 Connections of bolted fink roof trusses

12.6 End Bearings

When the roof truss is supported on steel columns, suitable connections should be provided to transfer the reaction from the truss to the column. One end of the truss may be fixed to the column and the other end should be allowed to slide to account for the expansion of the truss. Slotted holes may be provided in the base angles (angles connecting the truss to the bearing plates at the top of the columns) so as to permit expansion of the truss due to differences in temperature. If concrete or masonry columns are used to support the roof truss, suitable bearing plates have to be used to distribute the load on these supporting members so that the pressures on masonry or concrete are less than their allowable values. Anchor bolts have to be provided at each end to prevent uplifting of the truss. When the truss is supported at both the ends by hinges, the horizontal load on the truss has to be shared by the columns.

Examples

Example 12.1 *Design a roof truss, rafter bracing, purlin, tie runner, side runner, and eave girder for an industrial building located at Guwahati with a span of 20 m and a length of 50 m. The roofing is galvanized iron sheeting. Basic wind speed is 50 m/s and the terrain is an open industrial area. Building is class B building with a clear height of 8 m at the eaves.*

Solution

1. *Structural Model*

A trapezoidal truss is adopted with a roof slope of 1 to 5 and end depth of 1 m. For this span range, the trapezoidal trusses would be normally efficient and economical. Approximate span to depth ratio is about L/8 to L/12. Adopt a depth of 3 m at mid-span.

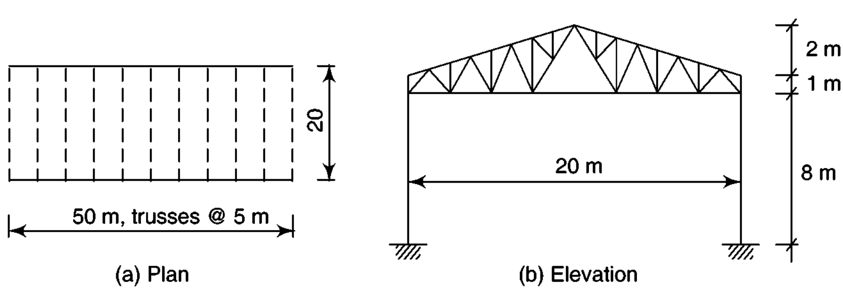
Span/depth ratio = 20/3.0 = 6.67

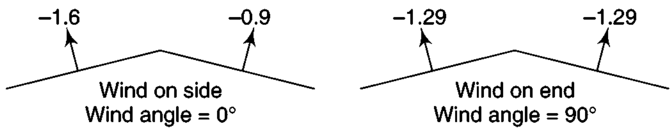
Hence the span/depth ratio is fine.

Truss spacing may be in the range of 1/4th to 1/5th of the span length. Hence adopt a spacing of 20/4 = 5 m. Then,

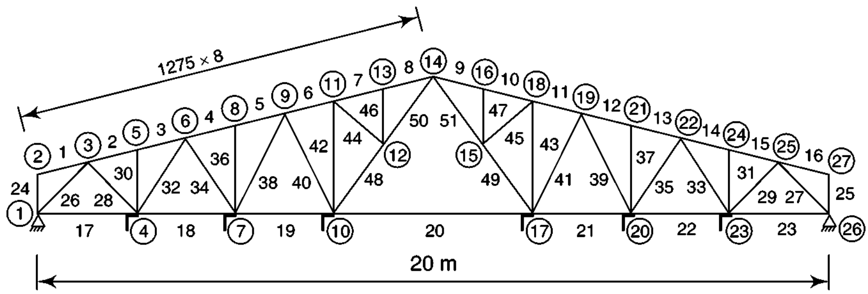
Number of bays = 50/5 = 10

The plan of the building and the elevation of the truss are shown in Figs 12.15(a) and (b).

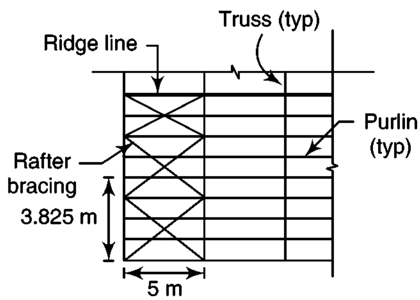




(c) Wind pressure

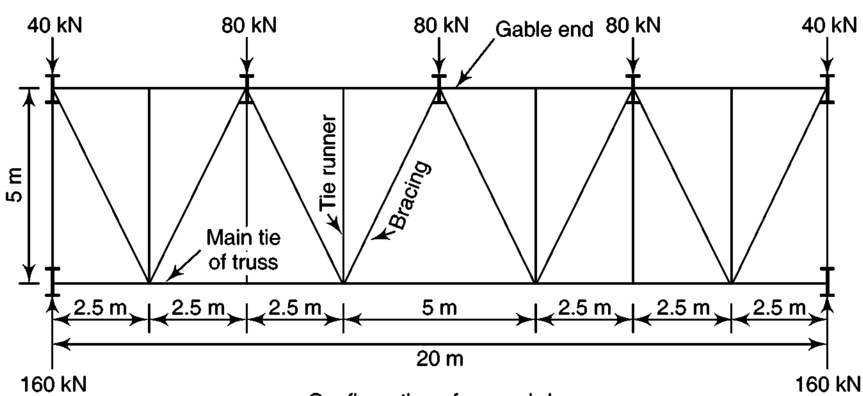


(d) Configuration of truss with member numbers and joint numbers adopted for the analysis



Layout of rafter bracing in the edge bay

(e)



Configuration of eave girder

(f)

Fig. 12.15

2. *Loading*

Calculation for dead load:

GI sheeting = 0.085 kN/m^2
Fixings = 0.025 kN/m^2
Services = 0.100 kN/m^2
Total load = 0.210 kN/m^2

For 5 m bays,

Roof dead load = $0.21 \times 20 \times 5 = 21$ kN
Weight of purlin (assuming 70 N/m^2) = $0.07 \times 5 \times 20 = 7$ kN
Self-weight of one truss* = $0.1067 \times 5 \times 20 = 10.67$ kN
Total dead load = 38.67 kN

* For welded sheet roof trusses, the self-weight is given approximately by

$$w = 53.7 + 0.53\,A = 53.7 + 0.53 \times 5 \times 20 = 0.1067 \text{ kN/m}^2$$

Calculation for nodal dead loads:

Since the truss has 16 internal nodes at the top chord [see Fig.12.15(b)],

Intermediate nodal dead load (W_1) = 38.67/16 = 2.42 kN
Dead load at end nodes ($W_1/2$) = 2.42/2 = 1.21 kN

(All these loads act vertically downwards at the nodes.)

Wind load as per IS 875 (Part 3)-1987

Basic wind speed in Guwahati = 50 m/s

Wind load F on a roof truss by static wind method is given by (clause 6.2.1 of IS 875) as follows:

$$F = (C_{pe} - C_{pi}) \times A \times P_d$$

where C_{pe} and C_{pi} are the force coefficients for the exterior and interior of the building.

Value of C_{pi}:

Assume wall openings between 5%–20% of wall area (clause 6.2.3.2 of IS 875), we have

$$C_{pi} = \pm\, 0.5$$

Value of C_{pe}:

Roof angle = $\alpha = \tan^{-1}(1/5) = 11.3°$
Height of the building to eaves $h = 8$ m
Short dimension of the building in plan $w = 20$ m
Building height to width ratio is given by

$$\frac{h}{w} = \frac{8}{20} = 0.4 < 0.5$$

Wind angle – 0° [Table 5 of IS 875 (Part 3)]

For 10° in windward side, $C_{pe} = -1.2$ and for leeward side $C_{pe} = -0.4$
For 20° in windward side and leeward side $C_{pe} = -0.4$
Roof angle $\alpha = 11.3°$

Then by interpolation we get

$$C_{pe} = -1.1 \text{ for windward and } -0.4 \text{ for leeward}$$

Wind angle – 90° [Table 5 of IS 875 (Part 3)]

For 10° in windward and leeward, $C_{pe} = -0.8$

For 20° in windward and leeward, $C_{pe} = -0.7$

For 11.3°, $C_{pe} = -0.79$ for windward and leeward

Risk coefficient, $k_1 = 1.0$, assuming that the industrial building is under general category and its probable life is 50 years.

Terrain, height and structure size factor, k_2:

Roof elevation: 8–11 m

Considering category 1 (exposed open terrain) and class B structure (length between 20–50 m) from Table 2 of IS 875 (Part 3)-1987, for 11 m, $k_2 = 1.038$

Assume topography factor $k_3 = 1.0$ (because of flat land)

Wind pressure calculation

Total height of the building = 11 m

Basic wind speed $V_b = 50$ m/s

Design wind speed

$$V_z = k_1 \times k_2 \times k_3 \times V_b$$

$$k_1 = 1.0;\ k_2 = 1.038;\ k_3 = 1.0;$$

$$V_z = 1.038 \times 1 \times 1 \times 50 = 51.9 \text{ m/s}$$

Design wind pressure $p_d = 0.6v_z^2 = 0.6 \times (51.9)^2$

$$= 1616.17 \text{ N/m}^2$$

$$= 1.616 \text{ kN/m}^2$$

Wind load on roof truss

Wind angle	Pressure coefficient			$(C_{pe} \pm C_{pi})$		$A \times P_d$	Wind load, F (kN)	
	C_{pe}		C_{pi}	Windward	Leeward	(kN)	Windward	Leeward
	Windward	Leeward						
0°	–1.10	–0.4	–0.5	–1.6	–0.9	10.3	–16.48	–9.27
			0.5	–0.6	0.1	10.3	–6.18	1.03
90°	–0.79	–0.79	–0.5	–1.29	–1.29	10.3	–13.29	–13.29
			0.5	–0.29	–0.29	10.3	–2.987	–2.987

The critical wind pressure is shown in Fig.12.15(c).

3. *Design of Purlin*

Span of purlin = 5 m

Spacing of purlin = 1.275 m

$\theta = 11.3°$

Load calculations:

Live load = 0.75 – (11.3 – 10)0.02 = 0.724 kN/m^2 > 0.4 kN/m^2

Dead load = 0.21 kN/m^2

Wind pressure = 1.616 × 1.6 = 2.586 kN/m^2

Load combinations:

1. DL + LL = 0.21 + 0.724 = 0.934 kN/m^2
2. DL + WL

   Normal to slope = –2.586 + 0.21cos11.3 = –2.38 kN/m^2

   Parallel to slope = 0.21 sin 11.3 = 0.041 kN/m^2

(a) Load combination 1: DL + LL

$$w_z = (0.934 \times \cos 11.3) \times 1.275 = 1.168 \text{ kN /m}$$
$$w_y = (0.934 \times \sin 11.3) \times 1.275 = 0.233 \text{ kN/m}$$

where w_z is the load normal to z-axis, w_y is the load normal to y-axis, and 1.275 is the spacing of the purlin. Due to continuity of purlins, factored bending moments and shear force are as follows:

$$M_z = 1.5 \times 1.168 \times 5^2/10 = 4.38 \text{ kN m}$$
$$M_y = 1.5 \times 0.233 \times 5^2/10 = 0.874 \text{ kN m}$$
$$SF_z = 1.5 \times 1.168 \times 5/2 = 4.38 \text{ kN}$$

Try MC100 for which the properties are as follows:

$$D = 100 \text{ mm}; \; b_f = 50 \text{ mm}; \; t_w = 5 \text{ mm}; \; t_f = 7.7 \text{ mm}$$
$$I_{zz} = 192 \times 10^4 \text{ mm}^4$$
$$Z_{ez} = 37.3 \times 10^3 \text{ mm}^3, \; Z_{ey} = 7.71 \times 10^3 \text{ mm}^3$$
$$Z_{pz} = 43.83 \times 10^3 \text{ mm}^3, \; Z_{py} = 16.238 \times 10^3 \text{ mm}^3$$

Section classification:

$$b/t_f = 50/7.7 = 6.49 < 9.4$$
$$d/t_w = (100 - 2 \times 7.7)/5.0 = 16.92 < 42$$

Hence the section is plastic.

Check for shear capacity

As per clause 8.4 of IS 800,

$$A_v = (100 \times 5.0) = 500 \text{ mm}^2$$

$$\frac{A_v f_{yw}}{\sqrt{3}\gamma_{m0}} = \frac{500 \times 250}{\sqrt{3} \times 1.10 \times 10^3} = 65.6 \text{ kN} > 4.38 \text{ kN}$$

Hence shear capacity is very large compared to the shear force.

Check for moment capacity

$$M_{dz} = \frac{{}_b Z_{pz} f_y}{\gamma_{m0}} = \frac{1 \quad 43.83 \quad 250 \quad 10^3}{1.10 \times 10^6} = 9.96 \text{ kNm}$$

The above value should be less than

$$\frac{1.2 \times 37.3 \times 250 \times 10^3}{1.10 \times 10^6} = 10.17 \text{ kN m}$$

Hence $M_{dz} = 9.96$ kN m $> M_z = 4.38$ kN m

Hence the assumed section is safe.

$$M_{dy} = \frac{1 \times 16.238 \times 250 \times 10^3}{1.10 \times 10^6} = 3.69 \text{ kN m}$$

The above value should be less than

$$\frac{1.2 \times 7.71 \times 250 \times 10^3}{1.1} = 2.10 \text{ kN m}$$

Hence $M_{dy} = 3.69$ kN m < $M_y = 0.874$ kN m
Hence the section is satisfactory.
Check for biaxial bending

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

Thus, $\frac{4.38}{9.96} \quad \frac{0.874}{3.69} = 0.68 < 1.0$

Check for deflection
Calculation for deflection is based on the serviceability condition, i.e., with unfactored imposed loads.

$$W = 1.168 \times 5 = 5.84 \text{ kN}$$

$$\delta = \frac{5WL^3}{384EI_z}$$

$$= \frac{5 \times 5.84 \times 1000 \times 5000^3}{384 \times 2 \times 10^5 \times 192 \times 10^4}$$

$$= 24.75 \text{ mm}$$

As per IS 800, Table 6, deflection limit is $\frac{L}{150} = 33.33$ mm > 24.75 mm

Hence the deflection is within allowable limits.
(b) Load combination 2: DL + WL

$$w_z = 2.38 \times 1.275 = 3.035 \text{ kN/m}$$

$$w_y = 0.041 \times 1.275 = 0.052 \text{ kN/m}$$

Factored bending moments in this case are

$$M_z = 1.5 \times 3.035 \times 5^2/10 = 11.38 \text{ kN m} > M_{dz} = 9.96 \text{ kN m}$$

$$M_y = 1.5 \times 0.052 \times 5^2/10 = 0.195 \text{ kN m} < M_{dy} = 3.69 \text{ kN m}$$

Hence, the section is not safe. Let us adopt MC125, which has an

$$I_{zz} = 425 \times 10^4 \text{ mm}^2,$$

$$Z_{pz} = 77.88 \times 10^3 \text{ mm}^3 \text{ and } Z_{py} = 29.46 \times 10^3 \text{ mm}^3$$

$$M_{dz} = 1 \times 77.88 \times 250 \times 10^{-3}/1.1 = 17.7 \text{ kNm}$$

$$M_{dy} = 1 \times 29.46 \times 250 \times 10^{-3}/1.1 = 6.69 \text{ kNm}$$

Thus, the check for biaxial bending is

$$\frac{11.38}{17.7} + \frac{0.195}{6.69} = 0.67 < 1.0$$

Hence the section is safe.
Check for deflection

$$\delta = \frac{5 \times (3.035 \times 5) \times 1000 \times 5000^3}{384 \times 2.0 \times 10^5 \times 425 \times 10^4} = 29.06 < 33.33 \text{ mm}$$

Note that the purlins at the edges and ridge of the building will be subjected to a local pressure of $1.4p \pm 0.5p$ [as per Table 5 of IS 875 (Part 3)-1987] instead of $1.6p$ taken in the preceding calculations. Hence, the purlins at the edges or at the ridge of the building have to be checked for this local pressure or closer spacing of purlins may be adopted at these locations.

4. *Truss Analysis and Design*

Tributary area for each node of the truss:

Length of each panel along sloping roof

$$= \frac{1.25}{\cos 11.3°} = 1.275 \text{ m } < 1.4 \text{ m}$$

Spacing of trusses = 5 m

Tributary area for each node of the truss = 5 × 1.275 = 6.375 m^2

Imposed load calculations:

From IS 875 (Part 2)-1987,

Live load = 0.75 kN/m^2

Reduction due to slope (see Table 2.3 and footnote 3)

$= (0.75 - 0.02 \times 1.3)2/3 = 0.483$ kN/m^2

Load at intermediate nodes $W_2 = 0.483 \times 5 \times 1.25$

$= 3.02$ kN

Load at end nodes $W_2/2 = 1.51$ kN

(All these loads act vertically downwards.)

Maximum $C_{pe} \pm C_{pi}$ (critical wind loads to be considered for analysis):

Wind Angle	Windward side (W_3) Intermediate nodes W_3	End and apex nodes $W_3/2$	Leeward (W_4) Intermediate nodes W_4	End and apex nodes $W_4/2$
0°	–16.48	–8.24	–9.27	–4.64
90°	–13.29	–6.645	–13.29	–6.645

*Loads in kN

All these loads act perpendicular to the top chord member of the truss.

Forces in the members The truss has been modelled as a pin jointed plane truss as shown in Fig. 12.15(d) and analysed using the software PLTRUSS developed by the author. The analysis results are tabulated as follows [see truss configuration shown in Fig. 12.15(d) for member numbers]

Load factors and combinations (Table 4 of IS 800):

For dead + imposed load

1.5 × DL + 1.5 × LL

For dead + wind load

1.5 × DL + 1.5 × WL

Dead + imposed + wind loading case will not be critical as wind loads act in opposite direction to dead and imposed loads.

Table 12.4 Member forces under factored loads in kN

Member number	Dead load + Live load	Dead load + Wind load (0°)	Dead load + Wind load (90°)
1.	0	2.472	1.985
2.	–97.086	212.066	193.914
3.	–97.086	217.01	197.898
4.	–124.83	269.25	253.304
5.	–124.83	274.20	257.29
6.	–124.83	263.50	258.09
7.	–128.99	279.45	270.58
8.	–128.99	284.39	274.57
9.	–128.99	244.35	274.57
10.	–128.99	241.57	270.57
11.	–124.83	233.41	258.08
12.	–124.83	221.17	257.29
13.	–124.83	218.39	253.30
14.	–97.086	163.98	197.9
15.	–97.086	161.20	193.91
16.	0	1.39	1.99
17.	61.20	–141.17	–118.91
18.	113.66	–251.86	–219.05
19.	124.67	–261.51	–237.61
20.	108.801	–202.49	–201.68
21.	124.67	–212.69	–237.61
22.	113.66	–186.67	–219.06
23.	61.20	–97.66	–118.91
24.	–4.08	10.79	8.34
25.	–4.08	5.27	8.34
26.	–86.55	185.54	168.17
27.	–86.55	138.114	168.17
28.	48.084	–98.26	–92.45
29.	48.084	–79.65	–92.45
30.	–8.16	21.58	16.69
31.	–8.16	10.55	16.69
32.	–31.76	58.86	59.82
33.	–31.76	56.25	59.82
34.	15.04	–20.94	–26.92
35.	15.04	–30.85	–26.92
36.	–8.16	21.58	16.69
37.	–8.16	10.55	16.69
38.	–4.67	5.20	5.97
39.	–4.67	16.65	5.97
40.	–4.67	26.38	12.41

(contd)

(*contd*)

41.	–4.67	–2.46	12.41
42.	–12.24	32.38	25.03
43.	–12.24	15.82	25.03
44.	5.225	–13.82	–10.69
45.	5.225	–6.75	–10.69
46.	8.16	21.58	16.69
47.	8.16	10.55	16.69
48.	21.245	–72.17	–46.70
49.	21.245	–17.793	–46.70
50.	27.62	–89.024	–59.74
51.	27.62	–26.03	–59.74

Truss Reactions (kN)

Joint	Case 1(DL + LL)		Case 2(DL + WL(0))		Case 3(DL + WL(90))	
number	*X*	*Y*	*X*	*Y*	*X*	*Y*
1	0	43.52	11.31	–94.62	0	–84.84
26	0	43.52	0	–68.61	0	–84.84

5. *Design of Top Chord Member (Member No. 8)*

Factored compressive force = 128.99 kN

Factored tensile force = 284.39 kN

Trying two ISA 75 × 75 × 6 mm @ 0.136 kN/m

Sectional properties:

Area of cross section $A = 2 \times 866 = 1732 \text{ mm}^2$

Radius of gyration $r_{zz} = 23$ mm

Assuming 8-mm thick gusset plate,

$$I_y = 2[45.7 \times 10^4 + 866\,(4 + 20.6)^2] = 196.21 \times 10^4 \text{ mm}^4$$

$$r_y = \sqrt{(196.21 \times 10^4/1732)} = 33.66 \text{ mm}$$

Section classification:

$$\varepsilon = (250/f_y)^{0.5} = (250/250)^{1/2} = 1.0$$

$$b/t = 75/6 = 12.5 < 15.7$$

∴ the section is semi-compact.

As no member in the section is slender, the full section is effective and there is no need to adopt reduction factor.

Maximum unrestrained length = L = 1275 mm

$KL = 0.85 \times \text{L} = 0.85 \times 12.75 = 1083.75$ mm

Note The effective length of top chord member may be taken as 0.7–1.0 times the distance between centres of connections as per clause 7.2.4 of IS 800. We have assumed the effective length factor as 0.85.

$$\lambda_y = 1083.75/23 = 47.12 < 180$$

Hence λ_y is within the allowable limits. From Table 9c of the code for $KL/r = 47.12$ and $f_y = 250$ MPa,

$$f_{cd} = 187.32 \text{ N/mm}^2$$

$$\text{Axial capacity} = 187.32 \times 1732/1000 = 324.4 \text{ kN} > 128.99 \text{ kN}$$

Hence, section is safe against axial compression.

$$\text{Axial tension capacity of the section} = 1732 \times 250/1.10 = 393.64 \text{ kN} > 284.39 \text{ kN}$$

Hence, section is safe in tension.

Note Though a smaller section may be chosen, this section is adopted to take care of handling stresses.

6. *Design of Bottom Chord Member (Member No. 20)*

Factored compressive force = 202.49 kN

Factored tensile force = 108.801 kN

Try two L 100 × 100 × 8 @ 0.242 kN/m.

Sectional properties:

Area of cross section A = 2 × 1540 = 3080 mm^2

Radius of gyration $r_z = 30.7$ mm

Assuming a 10-mm thick gusset plate,

$$I_y = 2[145 \times 10^4 + 1540\,(5 + 27.6)^2] = 6.173 \times 10^6 \text{ mm}^4$$

$$r_y = \sqrt{(6.173 \times 10^6/3080)} = 44.77 \text{ mm}$$

Section classification:

$$b/t = 90/6 = 15 < 15.7\varepsilon$$

the section is semi-compact.

$$\text{Axial tension capacity of the selected section} = 3080 \times (250/1.10) \times 10^{-3} = 700 \text{ kN} \gg 108.801 \text{ kN}$$

Hence, section is safe in tension. Providing longitudinal tie runner at every bottom node of the truss,

Maximum unrestrained length = L = 5000 mm

$r_y = 44.71$ mm

$\lambda_y = 5000 \times 0.85/44.71 = 95$

From Table 9c of IS 800 for $\lambda_y = 95$ and $f_y = 250$ MPa,

$$f_{cd} = 114 \text{ N/mm}^2$$

$$\text{Axial capacity} = 114 \times 3080/1000 = 351.12 \text{ kN} > 202.49 \text{ kN}$$

Hence, section is safe against axial compression also.

Note It may be economical to adopt unequal angles for the top and bottom chord members. However, many unequal angle sections are not readily available in the market. Hence, equal angle sections have been used in the truss in this example.

7. *Design of Web Member (Member No. 28)*

Maximum compressive force = 98.26 kN

Maximum tensile force = 185.54 kN

Try ISA 90 × 90 × 6.0 with

$$A = 1050 \text{ mm}^2;\ r_{zz} = 27.7 \text{ mm};\ r_{vv} = 17.5 \text{ mm}$$

Section classification:

$$b/t = 90/6 = 15 < 15.7\varepsilon$$

Hence, the section is not slender.

Length of member = 1768 mm

The angle will be connected through one leg to the gusset. A minimum of two bolts will be provided at the ends to connect the angle. Assuming fixed condition (Table 12 of the code) and using the Table given in Appendix D for the capacity of eccentrically connected angles, we get

for 1.5 m length, capacity = 101 kN

for 2 m length, capacity = 89 kN

Hence for 1.768 m, capacity = 94.56 kN ≈ 98.26 kN

Tensile capacity of the section = (250/1.1) × 1050/1000

= 238.64 kN > 185.54 kN

Hence ISA 90 × 90 × 6.0 is adequate for the web member.

Note Web members away from the support have less axial force. However, their length will be more. If desired, these may be designed and a smaller section be adopted.

8. *Check for Deflection*

Maximum deflection from computer output = 20.07 mm

Allowable deflection as per Table 6 of IS 800 : 2007

= span/240 = 20000/240 = 83.33 mm > 20.07 mm

Hence the deflection is within allowable limits.

9. *Design of Rafter Bracing Members*

Considering the layout of the rafter bracing as shown in Fig. 12.15(e),

Design wind pressure = 1.616 kN/m^2

Maximum force coefficient = –1.6

Factored wind load on rafter bracing

= 1.5 × 1.616 × 1.6 × 3.825 × 5/2 × sec 11.3°

= 37.8 kN

$$\text{Length of bracing} = \sqrt{(3825^2 + 5000^2)} = 6295.29 \text{ mm}$$

Try 90 × 90 × 6, $A = 1050$ mm^2, $r_{min} = 17.5$ mm, and

$$L/r = 6295.29/17.5 = 359.7 < 400 \text{ (Table 3 of IS 800 : 2007)}$$

In the X bracing system, as shown in Fig. 12.15(e), the compression bracing will buckle and only the tension bracing will be effective. Also, the bracing members are usually bolted to the trusses at site.

Axial tensile capacity:

Design strength of member due to yielding of gross section

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

$$= 1050 \times (250/1.1)/1000$$

$$= 238.64 \text{ kN} > 37.8 \text{ kN}$$

Design strength due to rupture of critical section $T_{dn} = \alpha A_n f_y / \gamma_{m1}$

$\alpha = 0.6$ (Assuming two bolts of 16 mm diameter at the ends)

$A_n = 1050 - 18 \times 6 = 942\ \text{mm}^2$

$T_{dn} = 0.6 \times 942 \times (410/1.25) \times 1000 = 185\ \text{kN} > 37.8\ \text{kN}$

Hence L $90 \times 90 \times 6$ is safe. The member has been found to be safe for block shear failure.

Note The forces in the bracing members are often small and rarely govern the design; but their slenderness limitations decide the size because of their long length.

10. *Design of Tie Runner*

Portion of wind load from gable end along the ridge will be transferred as axial load to tie runners provided along the length of building at tie level. Assume three intermediate gable end columns at a spacing of 5 m.

Wind load on cladding:

$L/w = 50/20 = 2.5 < 4$ and $h/w = 11/20 = 0.55 > 1/2$

From Table 4 of IS 875 (Part 3)-1987, external wind pressure = +0.7, with internal pressure of ±0.5, maximum pressure = $1.2p$

Factored wind load on intermediate runner = $1.5 \times 1.616 \times 1.2 \times 5 \times (8/2 + 3/2)$

= 80 kN (Tension)

Min r required = $5000 \times 0.85/250 = 17$ (maximum allowable $KL/r = 250$)

Try L $90 \times 90 \times 6$, with $A = 1050\ \text{mm}^2$, $r_{min} = 17.5$ mm,

Design strength due to yielding of gross section

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

$= 1050 \times (250/1.1)/1000 = 238.64\ \text{kN} > 80\ \text{kN}$

The design strength due to rupture of critical section (see the design of rafter bracing)

= 185 kN > 80 kN

Hence L $90 \times 90 \times 6$ is adequate.

11. *Design of Side Runner*

Assuming that the side sheetings are provided at a spacing of 1.25 m,

Span of side runner = 5 m

Calculation of loads

(a) Vertical loads

Self weight of side runner (MC100) = 9.56 kg/m

Weight of side sheeting = 5 kg/m^2

Thus weight of GI sheeting = $5 \times 1.25 = 6.25$ kg/m

Total w_y = 15.81 kg/m

= 0.16 kN/m

(b) Wind loads

Maximum wind force co-efficient = 1.2

Wind load UDL = $1.2 \times 1.616 \times 1.25$

= 2.424 kN/m

Factored loading

$$w_y = 1.5 \times 0.16 = 0.24 \text{ kN/m}$$

$$w_z = 1.5 \times 2.424 = 3.64 \text{ kN/m}$$

Try MC100, which has the following properties:

$D = 100$ mm; $b_f = 50$ mm; $t_w = 5.0$ mm; $t_f = 7.7$ mm; $I_{zz} = 192 \times 10^4$ mm^4;
$Z_{ez} = 37.3 \times 10^3$ mm^3; $Z_{pz} = 43.83 \times 10^3$ mm^3; $Z_{ey} = 7.71 \times 10^3$ mm^3
$Z_{py} = 16.238 \times 10^3$ mm^3

Assuming continuity of side runner,

$$BM_z = wl^2/8 = 3.64 \times 5^2/10 = 9.1 \text{ kN m}$$

$$BM_y = 0.24 \times 5^2/10 = 0.6 \text{ kN m}$$

Shear capacity:

From the design of purlin, shear capacity of the section = 65.6 kN

$$V = 3.64 \times 5/2 = 9.1 \text{ kN} < 65.6 \text{ kN}$$

Hence safe against shear.

Check moment capacity

From the calculations of purlin design

$$M_{dz} = 9.96 \text{ kN m} > 9.1 \text{ kN m}$$

$$M_{dy} = 3.69 \text{ kN m} > 0.6 \text{ kN m}$$

Check for biaxial bending

$$\frac{9.1}{9.96} + \frac{0.6}{3.69} = 1.08 > 1.0$$

Hence, the section may be revised to ISMC 125. The deflection has been calculated (similar to the purlin) and found to be within the codal limits. Note that for both the purlin and side runner, biaxial bending has been considered as per the code. If the sheeting is assumed to resist the in-plane loads, a smaller size of purlin and side runner may be sufficient to resist the bending moment in the *z*-direction.

12. *Design of Eave Girder*

Eave girders are provided at tie level and along both longitudinal and transverse directions to carry the wind forces at the tie level (see Fig. 12.4). Here the design of an eave girder along transverse direction is only considered. The design of an eave girder along the longitudinal direction is similar to the one in the transverse direction. The eave girder is in the form of a truss by connecting the bottom chord members of the truss to the gable columns at one end and main truss bottom points at the other end. The configuration of the eave girder is given in Fig. 12.15(f). In this example we are assuming a truss at both the ends of the building.

Wind loads on eave girders:

Design wind pressure = 1.616 kN/m^2

Maximum force coefficient = 1.2

Eaves girder bracings are connected between gable columns which are assumed to be spaced at 5 m centre to centre and truss bottom joints.

$$\text{Wind load on internal joint} = [1.616 \times 1.2 \times 5 \times (8/2 + 3/2)] \times 1.5 = 80 \text{ kN}$$

Wind load on end joints = 40 kN

Reaction = $(3 \times 80 + 2 \times 40)/2 = 160$ kN

Using the method of joints, the maximum force in the bracing at the end of the eave girder $F_{br} = (160 - 40)/\cos 26.565 = \pm 134.2$ kN

Members of the girder are subjected to reversal of stresses and hence they have been checked for compression as well as tension.

Length of the bracing = $\sqrt{2.5^2 + 5^2} = 5.59$ m

Required $r_{min} = 5590/350 = 15.97$ mm

Try section L 130 × 130 × 10 with the following properties:

$A = 2410$ mm^2; $r_{min} = 25.7$ mm

Section classification:

$\varepsilon = (250/f_y)^{0.5} = 1.0$

Flange:

$B/T = 130/10 = 13 < 15.7\varepsilon$

∴ It is semi compact.

From the table of capacity of eccentrically connected angle with fixed end conditions (see Appendix D),

Capacity for a length of 5590 mm = 148.1 kN > 134.2 kN

Hence the section is safe.

Axial tensile capacity (due to yielding of gross section)

$= 2410 \times 250/(1.1 \times 1000)$

$= 547$ kN > 134.2 kN

Design strength due to rupture of critical section.

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

$= 0.6 \times (2410 - 18 \times 10) \times 410 / (1.25 \times 1000)$

$= 438.2$ kN > 134.2 kN

The section has to be checked for block shear failure also. Note that the tension capacity is very high compared to the capacity in compression. It is due to the long length of the member. In order to reduce the member size, X bracing could be adopted and the member may be designed to be effective only in tension.

Note that the top and bottom chords of the eave girder are the bottom chord members of the main truss and carry a force of ±160 kN. The bottom chord members have been designed for a compressive force of 202.49 kN and a tensile force of 108.8 kN (tensile capacity 700 kN). Hence, the sections are safe under gable wind loads. Similarly, the vertical members are tie runners (with a maximum force of 80 kN), which have been designed already.

Slotted holes may be provided in the base angles to permit expansion of the truss. For a change in temperature of 34°C (see IS 875–Part 5), the maximum change in length will be $0.000012 \times 34 \times 20{,}000 = 8.16$ mm. The slot for a 20-mm bolt may be 22 mm wide and 30 mm long.

Summary

Structural steel is often the material of choice for the construction of single storey industrial buildings, which constitute the major percentage of the total number of steel structures built around the world. The planning and design of these buildings require the knowledge of several items such as site condition, plant layout and work flow, availability of new materials and waste disposal facilities, HVAC equipment, crane types and capacity, future expansion plans, and budget and project schedule. In particular, the structural engineer should select items such as roofing and walling material, bay width, structural framing system, and type and shape of trusses.

A number of factors have to be considered while selecting the roofing (decking) or wall (cladding) material. Steel, aluminium, galvanized iron, asbestos, stainless steel, and ferrocement sheets can be used as cladding or decking material. Metal roofing can be classified by the method of attachment to supports. Through-fastened roofs are directly attached to purlins and hence provide lateral stability to the purlins. However, standing- seam roofing (which is used extensively in USA) is connected indirectly by concealed clips formed in to the seam and requires a separate system of purlin bracings. Some details about the various types of sheeting are provided. Some guidelines for fixing the bay width of industrial buildings are also given.

Depending on the structural framing system adopted, industrial buildings may be classified as braced frames and unbraced frames. In braced frames, trusses rest on columns with hinge type connections and stability is provided by bracings in three mutually perpendicular directions.

Since the weight of purlins may be equal to the weight of trusses, they should be properly designed. Channels, angles, tubes, cold-formed C-, Z- or sigma sections are employed as purlins. The functions of girts are similar to purlins except that they are used in the walls. The eave strut is located at the intersection of the roof and exterior wall and has to be designed carefully.

A triangulated framework of pin-ended members is called a truss. In most situations, the loads are applied at the nodal points of trusses by purlins. When the purlins are placed in between the nodal points, the top chord members have to be designed for the secondary bending moments. With the availability of digital computers and software packages, the trusses are often analysed using these software packages. However, hand methods will be quite useful to check the results of the computer output (especially the errors made in the input data to the programs). The software packages require the member sizes of the truss to be given as input. Hence, some guidelines to assume the initial member sizes are given.

The various types of trusses and their configurations are described. It is to be noted that these configurations may change from project to project and is often selected based on aesthetics, economy, and performance. Some guidelines are also provided to select the pitch, and spacing of trusses. The loads and load combination to be considered are briefly discussed and the various steps involved

in the design of truss members are given. Sometimes it may be necessary to adopt a bigger section than those indicated by the actual design to take care of transportation and handling stresses. A brief discussion on the connections has been included. A few sketches showing the connection details of welded and bolted trusses are provided. The design of various members of trusses is illustrated through examples.

It has to be noted that with the knowledge of the behaviour and design of plates, beams, columns, tension members, compression members, and beam-columns, different types of structures (e.g., towers, multi-storey buildings, water tanks, bridges, chimneys, etc.) can be designed, using the appropriate code of practice.

Exercises

1. Design a roof truss for a railway platform of size 30 × 12 m situated in Chennai and as shown in Fig. 12.16. Assume asbestos cement sheetings.

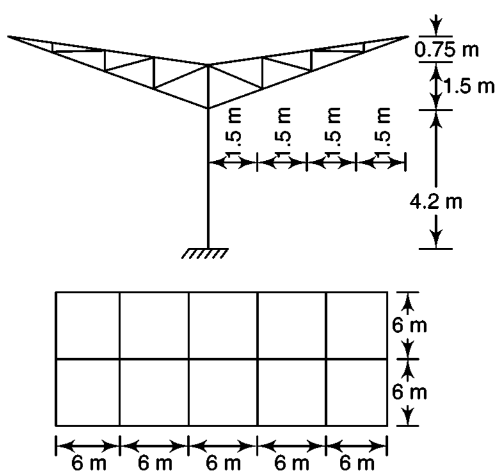


Fig. 12.16

2. An industrial building is shown in Fig. 12.17. The frames are at 5 m centres and the length of the building is 40 m. The purlin spacing of the roof is as shown in Fig. 12.17(b). The building is situated in Delhi. Assume live and wind loads as per IS 875 (part 2 and part 3) and the roof is covered with GI sheeting. Design the roof truss using angle members and gusseted joints. The truss is to be fabricated using welded joints in two parts for transport and assembled at site using bolted joints at A, B, and C as shown in Fig. 12.17.

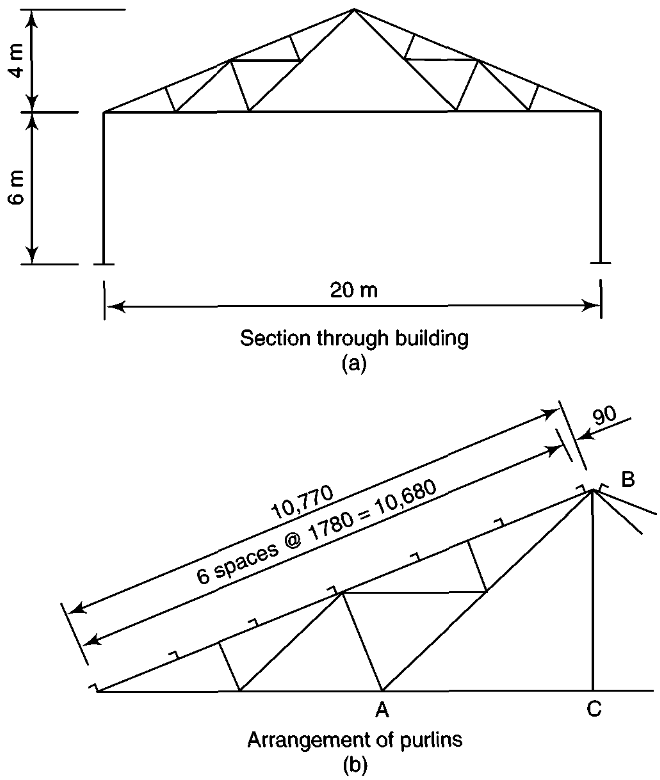


Fig. 12.17

3. Design the members of the truss of the previous exercise using tubular members.
4. A flat roof building of 18 m span has 1.5-m deep trusses at 5 m centres. The total dead load is 0.7 kN/m^2 and the imposed load is 0.75 kN/m^2. Design the truss using angle sections with welded internal joints and bolted field splices.

Review Questions

1. List the items that are to be considered while planning and designing an industrial building.
2. List the items to be considered while selecting a cladding/decking system.
3. Name some of the cladding/decking materials that are used in practice.
4. What are the purposes of structural decking?
5. Under what condition will the decking provide lateral stability to the top flange of purlins?
6. What are the advantages and drawbacks of the following:
   (a) Aluminium decking
   (b) GI sheeting
   (c) Asbestos sheeting
   (d) Ferrocement sheeting
7. Why is it necessary to design cladding and fixtures for higher pressure coefficient than that used for the design of structural frameworks?

8. What are the different types of bracings used in a braced building?
9. What is the function of a bracing?
10. State the difference between a purlin and a girt.
11. What are the sections that are normally used as purlins or girts?
12. What are wind columns?
13. What are the functions of an eave strut?
14. How can one determine whether a given truss forms a stable configuration?
15. Why is it necessary to design truss members for both compression and tension forces?
16. Distinguish between determinate and indeterminate trusses?
17. When are bending moments to be considered in the design of the top chord of trusses?
18. Sketch the different truss configurations that are often used in practice.
19. Why are Pratt trusses more advantageous compared to Howe trusses?
20. What are the advantages of parallel chord trusses?
21. What are the requirements that are considered while fixing the upper chord slope of trusses?
22. State the advantage of north light roof trusses over other forms of trusses.
23. What is the economic range of spacing of a truss?
24. How is the spacing of purlins fixed?
25. What are the load combinations that are usually considered for truss analysis?
26. List the various steps involved in the design of truss members.
27. Why are the minimum sections recommended and adopted for truss members, even though a lighter section may be indicated by the design?
28. Describe the behaviour of top and bottom chord members of a truss when lateral purlins/ties are not provided at each node.
29. When are gusset plates used in a truss having T-section for rafters and bottom tie members?
30. Why is it necessary to provide connections that will allow movement in the supports of trusses?

APPENDIX A

Properties of Structural Steel Sections

The structural designer has choice of a variety of sections, which are available in the market. This appendix provides properties of structural steel sections often used in practice. For more complete details of I-sections, channels, equal and unequal angles, and T-sections refer to IS: 808-1989. Note that IS: 808 does not give values of the plastic section modulus. Hence these values for I-sections and channels have been provided based on IS: 800. Note that there are some small differences in the values given by IS: 808 and IS: 800. Only the values given by IS: 808 have been used in this book. However, these differences in values will not affect the design much. Also included in this appendix are the wide flange sections, which have been introduced recently (more information on these sections may be obtained from M/s. Jindal Vijayanagar Steel Limited). For the properties of castellated beams, circular tubes, square and rectangular hollow sections, and cold formed lipped channel and zed sections, refer Appendix A of Subramanian 2008.

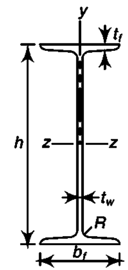
Indian Standard Rolled Steel Plates

Steel plates are available in the following widths and thicknesses.

Widths: 160, 180, 200, 220, 250, 280, 320, 355, 400, 450, 500, 560, 630, 710, 800, 900, 1000, 1100, 1250, 1400, 1600, 1800, 2000, 2200, and 2500

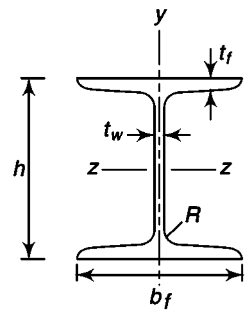
Thickness: 5.0, 5.5, 6.0, 7.0, 8, 9, 10, 11, 12, 14, 16, 18, 20, 22, 25, 28, 32, 36, 40, 45, 50, 56, 63, 71, and 80

Table A.1 Sectional properties for beams



Designation	Mass N/m	Sectional dimensions: Area (mm^2)	h (mm)	R (mm)	b_f (mm)	t_w (mm)	t_f (mm)	Sectional properties: I_z (cm^4)	I_y (cm^4)	r_z (mm)	r_y (mm)	Z_z (cm^3)	Z_y (cm^3)	Plastic modulus Z_{pz}(cm^3)	Shape factor
MB 100	89	1140	100	9	50	4.7	7	183	12.9	40	10.5	36.6	5.16	41.24	1.1268
MB 125	133	1700	125	9	70	5	8	445	38.5	51.6	15.1	71.2	11	81.85	1.1399
MB 150	150	1910	150	9	75	8	8	718	46.8	61.3	15.7	95.7	12.5	110.48	1.1401
MB 175	196	2500	175	10	85	5.8	9	1260	76.7	71.3	17.6	144	18	166.08	1.1422
MB 200	242	3080	200	11	100	5.7	10	2120	137	82.9	21.1	212	27.4	253.86	1.1358
MB 225	311	3970	225	12	110	6.5	11.8	3440	218	93.1	23.4	306	39.7	348.27	1.1385
MB 250	373	4750	250	13	125	6.9	12.5	5130	335	104	26.5	410	53.5	465.71	1.1345
MB 300	460	5860	300	14	140	7.7	13.1	8990	486	124	28.6	599	69.5	651.74	1.1362
MB 350	524	6670	350	14	140	8.1	14.2	13600	538	143	28.4	779	76.8	889.57	1.1421
MB 400	615	7840	400	14	140	8.9	16	20500	622	162	28.2	1020	88.9	1176.18	1.1498
MB 450	724	9220	450	15	150	9.4	17.4	30400	834	182	30.1	1350	111	1533.36	1.15
MB 500	869	11100	500	17	180	10.2	17.2	45200	1370	202	35.2	1810	152	2074.67	1.1471
MB 550	1040	13200	550	18	190	11.2	19.3	64900	1830	222	37.3	2360	193	2711.98	1.1492
MB 600	1230	15600	600	20	210	12	20.3	91800	2650	242	41.2	3060	252	3510.63	1.1471

Table A.2 Sectional properties of columns and heavy weight beams



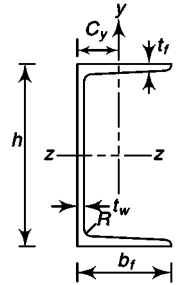
		Sectional dimensions						*Sectional properties*							
Designation	*Mass (N/m)*	*Area (mm^2)*	*h (mm)*	*R (mm)*	*b_f (mm)*	*t_w (mm)*	*t_f (mm)*	*I_z (cm^4)*	*I_y (cm^4)*	*r_z (mm)*	*r_y (mm)*	*Z_z (cm^3)*	*Z_y (cm^3)*	*Plastic modulus, Z_{pz} (cm^3)*	*Shape factor*
Column sections															
SC 100	200	2550	100	12	100	6	10	436	136	41.3	23.1	87.2	27.2	99.60	1.1422
SC 120	262	3340	120	12	120	6.5	11	842	255	50.2	27.6	140	42.6	159.49	1.1392
SC 140	333	4240	140	12	140	7	12	1470	438	58.9	32.1	211	62.5	238.59	1.1307
SC 150	371	4740	152	11.7	152	7.9	11.9	1970	700	64.5	38.4	259	91.9	285.87	1.1038
SC 160	419	5340	160	15	160	8	13	2420	695	67.4	36.1	303	86.8	341.67	1.1276
SC 180	505	6440	180	15	180	8.5	14	3740	1060	76.2	40.5	415	117	467.42	1.1263
SC 200	603	7680	200	18	200	9	15	5530	1530	84.8	44.6	553	153	620.03	1.1212
SC 220	704	8980	220	18	220	9.5	16	7880	2160	93.5	49	716	196	802.02	1.1201
SC 250	856	10900	250	23	250	10	17	12500	3260	107	54.6	997	260	1106.89	1.1102

(*contd*)

Table A.2 (*contd*)

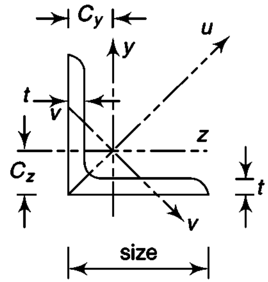
Heavy weight beams/columns															
HB 150	271	3450	150	8	150	5.4	9	1460	432	65	35.4	194	57.6	213.87	1.1024
HB 200	373	4750	200	9	200	6.1	9	3600	967	87.1	45.1	361	96.7	394.31	1.0923
HB 225	431	5490	225	10	225	6.5	9.1	5300	1350	98	49.6	469	120	511.55	1.0907
HB 250	510	6500	250	10	250	6.9	9.7	7740	1960	109	54.9	619	156	674.46	1.0896
HB 300	588	7480	300	11	250	7.6	10.6	12600	2200	130	54.1	836	175	914.60	1.0940
HB 350	674	8590	350	12	250	8.3	11.6	19200	2450	149	53.4	1090	196	1202.97	1.1036
HB 400	774	9870	400	14	250	9.1	12.7	28100	2730	169	52.6	1400	218	1548.92	1.1064
HB 450	872	11100	450	15	250	9.8	13.7	39200	3000	188	51.8	1740	239	1931.87	1.1103

Table A.3 Sectional properties of channel sections



Designation	Mass (N/m)	Sectional dimensions: Area (mm^2)	h (mm)	R (mm)	b_f (mm)	t_w (mm)	t_f (mm)	Sectional properties: C_y (mm)	I_z (cm^4)	I_y (cm^4)	r_z (mm)	r_y (mm)	Z_z (cm^3)	Z_y (cm^3)	Plastic modulus, Z_{pz} (cm^3)	Shape factor
MC 75	71.4	910	75	8.5	40	4.8	7.5	13.2	78.5	12.9	29.4	11.9	20.9	4.81	24.57	1.1756
MC 100	95.6	1220	100	9	50	5	7.7	15.4	192	26.7	39.7	14.8	37.3	7.71	44.48	1.1584
MC 125	131	1670	125	9.5	65	5.3	8.2	19.5	425	61.1	50.5	19.1	68.1	13.4	77.88	1.1436
MC 150	168	2130	150	10	75	5.7	9	22	788	103	60.8	22	105	19.5	120.00	1.1429
MC 175	196	2490	175	10.5	75	6	10.2	21.9	1240	122	70.4	22.1	141	23	161.92	1.1484
MC 200	223	2850	200	11	75	6.2	11.4	22	1830	141	80.2	22.2	181	26.4	209.92	1.1598
MC 225	261	3330	225	12	80	6.5	12.4	23.1	2710	188	90.2	23.7	241	33	276.03	1.1453
MC 250	306	3900	250	12	80	7.2	14.1	23	3880	211	99.2	23.7	307	38.5	354.65	1.1552
MC 300	363	4630	300	13	90	7.8	13.6	23.5	6420	313	118	26	428	47.1	495.67	1.1581
MC 350	427	5440	350	14	100	8.3	13.5	24.4	10000	434	136	28.2	576	57.3	670.76	1.1645
MC 400	501	6380	400	15	100	8.8	15.3	24.2	15200	508	154	28.2	760	67	888.79	1.1695

Table A.4 Sectional properties of equal leg angles



Designation	Mass (N/m)	Area (mm^2)	Sectional dimensions C_z (mm)	C_y (mm)	I_z (cm^4)	Sectional properties I_y (cm^4)	r_z (mm)	r_y (mm)	$r_{u(max)}$ (mm)	$r_{v(min)}$ (mm)	Z_z (cm^3)	Z_y (cm^3)	Z_{pz} (cm^3)
L20 20 × 3	9	112	5.9	5.9	0.4	0.4	5.8	5.8	7.3	3.7	0.3	0.3	0.52
× 4	11	145	6.3	6.3	0.5	0.5	5.8	5.8	7.2	3.7	0.4	0.4	0.67
L25 25 × 3	11	141	7.1	7.1	0.8	0.8	7.3	7.3	9.3	4.7	0.4	0.4	0.84
× 4	14	184	7.5	7.5	1	1	7.3	7.3	9.1	4.7	0.6	0.6	1.08
× 5	18	225	7.9	7.9	1.2	1.2	7.2	7.2	9.1	4.7	0.7	0.7	1.31
L30 30 × 3	14	173	8.3	8.3	1.4	1.4	8.9	8.9	11.3	5.7	0.6	0.6	1.23
× 4	18	226	8.7	8.7	1.8	1.8	8.9	8.9	11.2	5.7	0.8	0.8	1.59
× 5	22	277	9.2	9.2	2.1	2.1	8.8	8.8	11.1	5.7	1.0	1.0	1.93
L35 35 × 3	16	203	9.5	9.5	2.3	2.3	10.5	10.5	13.3	6.7	0.9	0.9	1.69
× 4	21	266	10.0	10.0	2.9	2.9	10.5	10.5	13.2	6.7	1.2	1.2	2.20
× 5	26	327	10.4	10.4	3.5	3.5	10.4	10.4	13.1	6.7	1.4	1.4	2.68
× 6	30	386	10.8	10.8	4.1	4.1	10.3	10.3	12.9	6.7	1.7	1.7	3.14

(*contd*)

Table A.4 *(contd)*

L40 40 × 3	18	234	10.8	10.8	3.4	3.4	12.1	12.1	15.4	7.7	1.2	1.2	2.23
× 4	24	307	11.2	11.2	4.5	4.5	12.1	12.1	15.3	7.7	1.6	1.6	2.91
× 5	30	378	11.6	11.6	5.4	5.4	12.0	12.0	15.1	7.7	1.9	1.9	3.56
× 6	35	447	12.0	12.0	6.3	6.3	11.9	11.9	15.0	7.7	2.3	2.3	4.18
L45 45 × 3	21	264	12.0	12.0	5	5	13.8	13.8	17.4	8.7	1.5	1.5	2.85
× 4	27	347	12.5	12.5	6.5	6.5	13.7	13.7	17.3	8.7	2	2	3.72
× 5	34	428	12.9	12.9	7.9	7.9	13.6	13.6	17.2	8.7	2.5	2.5	4.56
× 6	40	507	13.3	13.3	9.2	9.2	13.5	13.5	17.0	8.7	2.9	2.9	5.37
L50 50 × 3	23	295	13.2	13.2	6.9	6.9	15.3	15.3	19.4	9.7	1.9	1.9	3.54
× 4	30	388	13.7	13.7	9.1	9.1	15.3	15.3	19.3	9.7	2.5	2.5	4.63
× 5	38	479	14.1	14.1	11	11	15.2	15.2	19.2	9.7	3.1	3.1	5.68
× 6	45	568	14.5	14.5	12.9	12.9	15.1	15.1	19.0	9.6	3.6	3.6	6.70
L55 55 × 5	41	527	15.3	15.3	14.7	14.7	16.7	16.7	21.1	10.6	3.7	3.7	6.93
× 6	49	626	15.7	15.7	17.3	17.3	16.6	16.6	21.0	10.6	4.4	4.4	8.19
× 8	64	818	16.5	16.5	22	22	16.4	16.4	20.7	10.6	5.7	5.7	10.58
× 10	79	1000	17.2	17.2	26.3	26.3	16.2	16.2	20.3	10.6	7	7	12.83
L60 60 × 5	45	575	16.5	16.5	19.2	19.2	18.2	18.2	23.1	11.6	4.4	4.4	8.31
× 6	54	684	16.9	16.9	22.6	22.6	18.2	18.2	22.9	11.5	5.2	5.2	9.82
× 8	70	896	17.7	17.7	29	29	18.0	18.0	22.7	11.5	6.8	6.8	12.72
× 10	86	1100	18.5	18.5	34.8	34.8	17.8	17.8	22.3	11.5	8.4	8.4	15.46
L65 65 × 5	49	625	17.7	17.7	24.7	24.7	19.9	19.9	25.1	12.6	5.2	5.2	9.81
× 6	58	744	18.1	18.1	29.1	29.1	19.8	19.8	25.0	12.6	6.2	6.2	11.61
× 8	77	976	18.9	18.9	37.4	37.4	19.6	19.6	24.7	12.5	8.1	8.1	15.06
× 10	94	1200	19.7	19.7	45	45	19.4	19.4	24.4	12.5	9.9	9.9	18.34

(contd)

Table A.4 *(contd)*

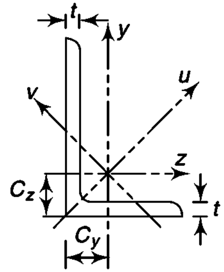
L70 70 × 5	53	677	18.9	18.9	31.1	31.1	21.5	21.5	27.1	13.6	6.1	6.1	11.44
× 6	63	806	19.4	19.4	36.8	36.8	21.4	21.4	27.0	13.6	7.3	7.3	13.54
× 8	83	1060	20.2	20.2	47.4	47.4	21.2	21.2	26.7	13.5	9.5	9.5	17.60
× 10	102	1300	21.0	21.0	57.2	57.2	21.0	21.0	26.4	13.5	11.7	11.7	21.46
L75 75 × 5	57	727	20.2	20.2	38.7	38.7	23.1	23.1	29.2	14.6	7.1	7.1	13.19
× 6	68	866	20.6	20.6	45.7	45.7	23.0	23.0	29.1	14.6	8.4	8.4	15.63
× 8	89	1140	21.4	21.4	59	59	22.8	22.8	28.8	14.5	11	11	20.34
× 10	110	1400	22.2	22.2	71.4	71.4	22.6	22.6	28.4	14.5	13.5	13.5	24.84
L80 80 × 6	73	929	21.8	21.8	56	56	24.6	24.6	31.1	15.6	9.6	9.6	17.86
× 8	96	1220	22.7	22.7	72.5	72.5	24.4	24.4	30.8	15.5	12.6	12.6	23.28
× 10	118	1500	23.4	23.4	87.7	87.7	24.1	24.1	30.4	15.5	15.5	15.5	28.47
× 12	140	1780	24.2	24.2	102	102	23.9	23.9	30.1	15.4	18.3	18.3	33.44
L90 90 × 6	82	1050	24.2	24.2	80.1	80.1	27.7	27.7	35.0	17.5	12.2	12.2	22.78
× 8	108	1380	25.1	25.1	104	104	27.5	27.5	34.7	17.5	16	16	29.76
× 10	134	1700	25.9	25.9	127	127	27.3	27.3	34.4	17.4	19.8	19.8	36.47
× 12	158	2020	26.6	26.6	148	148	27.1	27.1	34.1	17.4	23.3	23.3	42.93
L100 100 × 6	92	1170	26.7	26.7	111	111	30.9	30.9	39.1	19.5	15.2	15.2	28.30
× 8	121	1540	27.6	27.6	145	145	30.7	30.7	38.8	19.5	20	20	37.05
× 10	149	1900	28.4	28.4	177	177	30.5	30.5	38.5	19.4	24.7	24.7	45.48
× 12	177	2260	29.2	29.2	207	207	30.3	30.3	38.2	19.4	29.2	29.2	53.61
L110 110 × 8	134	1710	30.0	30.0	197	197	34.0	34.0	42.8	21.8	24.6	24.6	45.13
× 10	166	2110	30.9	30.9	240	240	33.7	33.7	42.5	21.6	30.4	30.4	55.48
× 12	197	2510	31.7	31.7	281	281	33.5	33.5	42.2	21.5	35.9	35.9	65.50
× 16	257	3280	33.2	33.2	357	357	33.0	33.0	41.5	21.4	46.5	46.5	84.62

(contd)

(*contd*)

L130 130 × 8	159	2030	35.0	35.0	331	331	40.4	40.4	51.0	25.9	34.9	34.9	63.69
× 10	197	2510	35.9	35.9	405	405	40.2	40.2	50.7	25.7	43.1	43.1	78.48
× 12	235	2990	36.7	36.7	476	476	39.9	39.9	50.3	25.6	51	51	92.86
× 16	307	3920	38.2	38.2	609	609	39.4	39.4	49.7	25.4	66.3	66.3	120.48
L150 150 × 10	229	2920	40.8	40.8	634	634	46.6	46.6	58.7	29.8	58	58	105.48
× 12	273	3480	41.6	41.6	746	746	46.3	46.3	58.4	29.7	68.8	68.8	125.03
× 16	358	4560	43.1	43.1	959	959	45.8	45.8	57.7	29.4	89.7	89.7	162.74
× 20	441	5620	44.6	44.6	1160	1160	45.3	45.3	57.1	29.3	110	110	198.73
L200 200 × 12	369	4690	53.9	53.9	1830	1830	62.4	62.4	78.7	39.9	125	125	226.44
× 16	485	6180	55.6	55.6	2370	2370	61.9	61.9	78.0	39.6	164	164	296.37
× 20	600	7640	57.1	57.1	2880	2880	61.4	61.4	77.3	39.3	201	201	363.80
× 25	739	9410	59.0	59.0	3470	3470	60.7	60.7	76.1	39.1	246	246	444.82

Table A.5 Sectional properties of unequal leg angles



Designation	Mass	Sectional dimensions			Sectional properties								
	(N/m)	Area (mm^2)	C_z (mm)	C_y (mm)	I_z (cm^4)	I_y (cm^4)	r_z (mm)	r_y (mm)	$r_{u(max)}$ (mm)	$r_{v(min)}$ (mm)	Z_z (cm^3)	Z_y (cm^3)	Z_{pz} (cm^3)
L30 20 × 3	11	141	9.8	4.9	1.2	0.4	9.2	5.4	9.9	4.1	0.6	0.3	1.15
× 4	14	184	10.2	5.3	1.5	0.5	9.2	5.4	9.8	4.1	0.8	0.4	1.48
× 5	18	225	10.6	5.7	1.9	0.6	9.1	5.3	9.7	4.1	1	0.4	1.78
L40 25 × 3	15	188	13	5.7	3	0.9	12.5	6.8	13.3	5.2	1.1	0.5	2.06
× 4	19	246	13.5	6.2	3.8	1.1	12.5	6.8	13.2	5.2	1.4	0.6	2.67
× 5	24	302	13.9	6.6	4.6	1.4	12.4	6.7	13.1	5.2	1.8	0.7	3.25
× 6	28	356	14.3	6.9	5.4	1.6	12.3	6.6	12.9	5.2	2.1	0.9	3.80
L45 30 × 3	17	218	14.2	6.9	4.4	1.5	14.2	8.4	15.2	6.3	1.4	0.7	2.67
× 4	22	286	14.7	7.3	5.7	2	14.1	8.4	15.1	6.3	1.9	0.9	3.48
× 5	28	352	15.1	7.7	6.9	2.4	14	8.3	15	6.3	2.3	1.1	4.25
× 6	33	416	15.5	8.1	8	2.8	13.9	8.2	14.9	6.3	2.7	1.3	4.98
L50 30 × 3	18	234	16.3	6.6	5.9	1.6	15.9	8.3	16.7	6.5	1.7	0.7	3.23
× 4	24	307	16.8	7	7.7	2.1	15.8	8.2	16.6	6.3	2.3	0.9	4.22

(*contd*)

Table A.5 *(contd)*

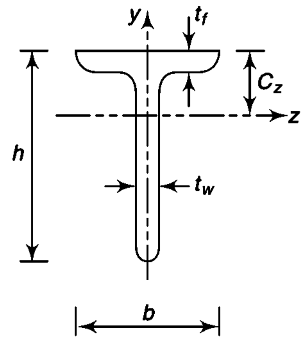
× 5	30	378	17.2	7.4	9.3	2.5	15.7	8.1	16.5	6.3	2.8	1.1	5.16
× 6	35	447	17.6	7.8	10.9	2.9	15.6	8	16.4	6.3	3.4	1.3	6.05
L60 40 × 5	37	476	19.5	9.6	16.9	6	18.9	11.2	20.2	8.5	4.2	2	7.78
× 6	44	565	19.9	10	19.9	7	18.8	11.1	20.1	8.5	5	2.3	9.17
× 8	58	737	20.7	10.8	25.4	8.8	18.6	11	19.8	8.4	6.5	3	11.81
L65 45 × 5	41	526	20.7	10.8	22.1	8.6	20.5	12.8	22.2	9.6	5	2.5	9.28
× 6	49	625	21.1	11.2	26	10.1	20.4	12.7	22.1	9.5	5.9	3	10.96
× 8	64	817	21.9	12	33.2	12.8	20.2	12.5	21.8	9.5	7.7	3.9	14.15
L70 45 × 5	43	552	22.7	10.4	27.2	8.8	22.2	12.6	23.6	9.6	5.7	2.5	10.63
× 6	52	656	23.2	10.9	32	10.3	22.1	12.5	23.5	9.6	6.8	3	12.56
× 8	67	858	24	11.6	41	13.1	21.9	12.4	23.2	9.5	8.9	3.9	16.24
× 10	83	1050	24.8	12.4	49.3	15.6	21.6	12.2	22.9	9.5	10.9	4.8	19.69
L75 50 × 5	47	602	23.9	11.6	34.1	12.2	23.8	14.2	25.6	10.7	6.7	3.2	12.38
× 6	56	716	24.4	12	40.3	14.3	23.7	14.1	25.5	10.7	8	3.8	14.64
× 8	74	938	25.2	12.8	51.8	18.3	28.5	14	25.2	10.6	10.4	4.9	18.98
× 10	90	1150	26	13.6	62.2	21.8	23.3	13.8	24.9	10.6	12.7	6	23.06
L80 50 × 5	49	627	26	11.2	40.6	12.3	25.5	14	27	10.7	7.5	3.2	13.91
× 6	59	746	26.4	11.6	48	14.4	25.4	13.9	26.9	10.7	9	3.8	16.46
× 8	77	978	27.3	12.4	61.9	18.5	25.2	13.7	26.6	10.6	11.7	4.9	21.37
× 10	94	1200	28.1	13.2	74.7	22.1	24.9	13.6	26.3	10.6	14.4	6	26.00
L90 60 × 6	68	865	28.7	13.9	70.6	25.2	28.6	17.1	30.7	12.8	11.5	5.5	21.38
× 8	89	1140	29.6	14.8	91.5	32.4	28.4	16.9	30.4	12.8	15.1	7.2	27.85
× 10	110	1400	30.4	15.5	111	39.1	28.1	16.7	30.1	12.7	18.6	8.8	34.00
× 12	130	1660	31.2	16.3	129	45.2	27.9	16.5	29.8	12.7	22	10.3	39.85
L100 65 × 6	75	955	31.9	14.7	96.7	32.4	31.8	18.4	34	13.9	14.2	6.4	26.42
× 8	99	1260	32.8	15.5	126	41.9	31.6	18.3	33.8	13.9	18.7	8.5	34.48
× 10	122	1550	33.7	16.3	153	50.7	31.4	18.1	33.5	13.8	23.1	10.4	42.19

(contd)

Table A.5 (*contd*)

L100 75 × 6	80	1010	30.1	17.8	101	48.7	31.5	21.9	35	15.9	14.4	8.5	27.32
× 8	105	1340	31	18.7	132	63.3	31.4	21.8	34.8	15.9	19.1	11.2	35.68
× 10	130	1650	31.9	19.5	160	76.9	31.2	21.6	34.5	15.8	23.6	13	43.69
× 12	154	1950	32.7	20.3	188	89.5	31	21.4	34.2	15.8	27.9	16.3	51.36
L125 75 × 6	92	1170	40.5	15.9	188	51.6	40.1	21	42.3	16.2	22.2	8.7	40.93
× 8	121	1540	41.5	16.8	246	67.2	40	20.9	42.1	16.1	29.4	11.5	53.63
× 10	149	1900	42.4	17.6	300	81.6	39.7	20.7	41.8	16.1	36.5	14.2	65.88
L125 95 × 6	101	1290	37.2	22.4	205	103	39.9	28.3	44.3	20.7	23.4	14.3	43.33
× 8	134	1700	38	23.2	268	135	39.7	28.1	44.1	20.5	30.9	18.8	56.83
× 10	165	2110	38.9	24	328	164	39.5	27.9	43.8	20.4	38.1	23.1	69.88
× 12	197	2500	39.7	24.8	385	192	39.2	27.7	43.5	20.3	45.1	27.3	82.48
L150 75 × 8	137	1750	52.4	15.4	410	71.1	48.8	20.2	49.9	16.2	42	11.9	74.08
× 10	170	2160	53.3	16.2	502	86.3	48.2	20	49.6	16.1	51.9	14.7	91.19
× 12	202	2570	54.2	17	590	100	47.9	19.8	49.3	16	61.6	17.3	107.76
L150 115 × 8	163	2070	44.8	27.6	474	244	47.8	34.3	53.3	25	45.1	28	82.88
× 10	201	2570	45.7	28.4	582	299	47.6	34.1	53.1	24.8	55.8	34.5	102.19
× 12	240	3050	46.5	29.2	685	351	47.4	33.9	52.8	24.7	66.2	40.8	120.96
× 16	314	4000	48.1	30.7	878	447	46.9	33.4	52.1	24.4	86.2	53	177.24
L200 100 × 10	229	2920	69.8	20.3	1230	215	64.8	27.1	66.8	21.7	94.3	26.9	165.25
× 12	273	3480	70.7	21.1	1450	251	64.6	26.9	66.5	21.6	112	31.9	196.03
× 16	358	4570	72.3	22.7	1870	320	64	26.6	65.9	21.3	147	41.3	255.42
L200 150 × 10	269	3430	60.2	35.5	1410	689	64.1	44.8	71	32.8	101	60.2	184.00

Table A.6 Sectional properties of rolled steel tee sections



Designation	*Weight (N/m)*	*Sectional area (mm^2)*	*Depth of section h (mm)*	*Width of flange b (mm)*	*Thickness of flange t_f (mm)*	*Thickness of web t_w (mm)*	*Centre of gravity C_z (mm)*	*Moment of inertia* I_z (cm^4)	I_y (cm^4)	*Radius of gyration* r_z (mm)	r_y (mm)	*Moduli of section* Z_z (cm^3)	Z_y (cm^3)
ISNT 20	9	113	20	20	3	3	6	0.4	0.2	5.9	3.9	0.3	0.2
ISNT 30	14	175	30	30	3	3	8.3	1.4	0.6	8.9	5.7	0.6	0.4
ISNT 40	35	448	40	40	6	6	12	6.3	3	11.8	8.2	2.2	1.5
ISNT 50	45	570	50	50	6	6	14.4	12.7	5.9	15	10.2	3.6	2.4
ISNT 60	54	690	60	60	6	6	16.7	22.5	10.1	18.1	12.1	5.2	3.4
ISNT 80	96	1225	80	80	8	8	22.3	71.2	32.3	24.1	16.2	12.3	8.1
ISNT 100	150	1910	100	100	10	10	27.9	173.8	79.9	30.2	20.5	24.1	16
ISNT 150	228	2908	150	150	10	10	39.5	608.8	257.5	45.6	30.3	54.6	35.7
ISHT 75	153	1949	75	150	9	8.4	16.2	96.2	230.2	22.2	34.4	16.4	30.1
ISHT 100	200	2547	100	200	9	7.8	19.1	193.8	497.3	27.6	44.2	24	49.3

(*contd*)

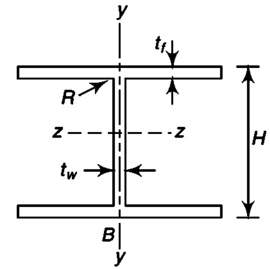
Table A.6 *(contd)*

ISHT 125	274	3485	125	250	9.7	8.8	23.7	415.4	1005.8	34.5	53.7	41	79.9
ISHT 150	294	3742	150	250	10.6	7.6	26.6	573.7	1096.8	39.2	54.1	46.5	87.7
ISST 100	81	1037	100	50	10	5.8	30.3	99	9.6	30.9	9.6	14.2	3.8
ISST 150	157	1996	150	75	11.6	8	37.5	450.2	37	47.5	13.6	43.9	9.9
ISST 200	284	3622	200	165	12.5	8	47.8	1267.8	358.2	59.2	31.5	83.3	43.4
ISST 250	375	4775	250	180	12.1	9.2	64	2774.4	532	76.2	33.4	149.2	59.1
ISLT 50	40	511	50	50	6.4	4	11.9	9.9	6.4	13.9	11.2	2.6	2.5
ISLT 75	71	904	75	80	6.8	4.8	17.2	41.9	27.6	21.5	17.5	7.2	6.9
ISLT 100	127	1616	100	100	10.8	5.7	21.3	116.6	75	26.9	21.5	14.8	15
ISJT 75	35	450	75	50	4.6	3	20	24.8	4.6	23.5	10.1	4.5	1.8
ISJT 87.5	40	514	87.5	50	4.8	3.2	25	39	4.8	27.5	9.7	6.2	1.9
ISJT 100	50	632	100	60	5	3.4	28.1	63.5	8.6	31.7	11.7	8.8	2.9
ISJT 112.5	64	814	112.5	80	5	3.7	30.1	101.6	20.2	35.3	15.8	12.3	5.1

Table A.7 Sectional properties of parallel flange beams and columns

IPE - European I-beams
HE - European wide flange beams
W - American wide flange beams
UC - British universal columns
HD - Wide flange columns

H – Depth
B - Flange width
t_w - Web thickness
t_f - Flange thickness
R - Fillet radius



Designation	*Mass (N/m)*	*Sectional area (mm²)*	*Main dimensions (mm)*					*Moment of inertia (cm⁴)*		*Radius of gyration (mm)*		*Modulus of section (cm³)*	
			H	*B*	t_w	t_f	*R*	I_z	I_y	r_z	r_y	Z_z	Z_y
(1) Nominal size 200 mm													
IPE 200	224	2848	200	100	5.6	8.5	12	1943	142.4	82.6	22.4	194.3	28.47
HE 200 A	423	5383	190	200	6.5	10	18	3692	1336	82.8	49.8	388.6	133.6
HE 200 B	613	7808	200	200	9	15	18	5696	2003	85.4	50.7	569.6	200.3
HE 200 M	1030	13130	220	206	15	25	18	10640	3651	90.0	52.7	967.4	354.5
W 200 × 135 × 26.6	266	3400	207	133	5.8	8.4	10	2587	329.8	87.2	31.1	250	49.6
W 200 × 135 × 31.3	313	3992	210	134	6.4	10.2	10	3139	409.6	88.7	32	298.9	61.13
W 200 × 165 × 35.9	359	4575	201	165	6.2	10.2	10	3438	764.3	86.7	40.9	342.1	92.64
W 200 × 165 × 41.7	417	5317	205	166	7.2	11.8	10	4088	900.5	87.7	41.2	398.8	108.5
IPE 220	262	3337	220	110	5.9	9.2	12	2772	204.9	91.1	24.8	252	37.25
HE 220 A	505	6434	210	220	7	11	18	5410	1955	91.7	55.1	515.2	177.7
HE 220 B	715	9104	220	220	9.5	16	18	8091	2843	94.3	55.9	735.5	258.5
HE 220 M	1170	14940	240	226	15.5	26	18	14600	5012	98.9	57.9	1217	443.5

(*contd*)

Table A.7 *(contd)*

Column Sections													
UC 200 × 203 × 46	461	5873	203.2	203.6	7.2	11	10.2	4568	1548	88.2	51.3	449.6	152.1
UC 200 × 203 × 52	520	6628	206.2	204.3	7.9	12.5	10.2	5259	1778	89.1	51.8	510.1	174
UC 200 × 203 × 60	600	7637	209.6	205.8	9.4	14.2	10.2	6125	2065	89.6	52	584.4	200.6
UC 200 × 203 × 71	710	9043	215.8	206.4	10	17.3	10.2	7618	2537	91.8	53	706	245.9
UC 200 × 203 × 86	860	10960	222.2	209.1	12.7	20.5	10.2	9449	3127	92.8	53.4	850.5	299.1
UC 200 × 200 × 100	1000	12670	229	210	14.5	27.3	10	11000	3660	93.2	53.7	961	349
(2) Nominal size 250 mm													
IPE 240	307	3912	240	120	6.2	9.8	15	3892	283.6	99.7	26.9	324.3	47.27
HE 240 A	603	7684	230	240	7.5	12	21	7763	2769	100.5	60	675.1	230.7
HE 240 B	832	10600	240	240	10	17	21	11260	3923	103.1	60.8	938.3	326.9
HE 240 M	1570	19960	270	248	18	32	21	24290	8153	110.3	63.9	1799	657.5
W 250 × 145 × 32.7	327	4175	258	146	6.1	9.1	13	4895	472.6	108.3	33.6	379.4	64.74
W 250 × 145 × 38.5	385	4929	262	147	6.6	11.2	13	6014	593.7	110.5	34.7	459.1	80.77
W 250 × 145 × 44.8	448	5732	266	148	7.6	13	13	7118	703.5	111.4	35	535.2	95.06
W250 × 200 × 49.1	491	6254	247	202	7.4	11	13	7070	1510	106	49	572	150
W250 × 200 × 58	580	7426	252	203	8	13.5	13	8740	1880	108	50.3	694	185
W250 × 200 × 67	670	8559	257	204	8.9	15.7	13	10400	2220	110	50.9	809	218
HD 260 × 68.2	682	8682	250	260	7.5	12.5	24	68.2	28.76	9198	4302	10450	3668
HD 260 × 93	930	11840	260	260	10	17.5	24	93	37.59	12830	6022	14920	5135
HD 260 × 114	1140	14570	268	262	12.5	21.5	24	114	46.08	16000	7525	18910	6456
HD 260 × 142	1420	18030	278	265	15.5	26.5	24	142	56.65	20150	9505	24330	8236
HD 260 × 172	1720	21960	290	268	18	32.5	24	172	66.89	25240	11920	31310	10450
HE 280 A	764	9726	270	280	8	13	24	13670	4763	118.6	70	1013	340.2

(contd)

Table A.7 *(contd)*

HE 280 B	1030	13140	280	280	10.5	18	24	19270	6595	121.1	70.9	1376	471
HE 280 M	1890	24020	310	288	18.5	33	24	39550	13160	128.3	74	2551	914.1
Column Sections													
W 250 × 250 × 73	730	9299	253	254	8.6	14.2	13	11290	3880	110.2	64.6	892.1	305.5
W 250 × 250 × 80	800	10210	256	255	9.4	15.6	13	12570	4314	111	65	982.4	338.3
W 250 × 250 × 89	890	11410	260	256	10.7	17.3	13	14260	4841	111.8	65.1	1097	378.2
W 250 × 250 × 101	1010	12900	264	257	11.9	19.6	13	16380	5549	112.7	65.6	1241	431.9
W 250 × 250 × 115	1150	14620	269	259	13.5	22.1	13	18940	6405	113.8	66.2	1408	494.6
W 250 × 250 × 131	1310	16700	275	261	15.4	25.1	13	22150	7446	115.2	66.8	1611	570.6
W 250 × 250 × 149	1490	18970	282	263	17.3	28.4	13	25940	8622	116.9	67.4	1840	655.7
W 250 × 250 × 167	1670	21320	289	265	19.2	31.8	13	30020	9879	118.7	68.1	2078	745.6
(3) *Nominal size 300 mm*													
IPE 300	422	5381	300	150	7.1	10.7	15	8356	603.8	124.6	33.5	557.1	80.5
HE 300 A	880	11250	290	300	8.5	14	27	18260	6310	127.4	74.9	1260	420.6
HE 300 B	1170	14910	300	300	11	19	27	25170	8563	129.9	75.8	1678	570.9
HE 300 M	2380	30310	340	310	21	39	27	59200	19400	139.8	80	3482	1252
W 310 × 100 × 23.8	238	3038	305	101	5.6	6.7	8	4280	115.6	118.7	19.5	280.7	22.89
W 310 × 100 × 28.3	283	3609	309	102	6	8.9	8	5431	158.1	122.7	20.9	351.5	30.99
W 310 × 100 × 32.7	327	4181	313	102	6.6	10.8	8	6507	191.9	124.7	21.4	415.8	37.62
W 310 × 165 × 38.7	387	4953	310	165	5.8	9.7	8	8527	726.8	131.2	38.3	550.1	88.1
W 310 × 165 × 44.5	445	5691	313	166	6.6	11.2	8	9934	854.7	132.1	38.8	634.8	103
W 310 × 165 × 52	520	6678	317	167	7.6	13.2	8	11851	1026	133.2	39.2	747.7	122.9
W 310 × 200 × 60	600	7588	303	203	7.5	13.1	15	12900	1830	130	49.1	851	180
W 310 × 200 × 67	670	8503	306	204	8.5	14.6	15	14500	2070	131	49.3	948	203

(contd)

Table A.7 *(contd)*

W 310 × 200 × 74	740	9484	310	205	9.4	16.3	15	16500	2340	132	49.7	1060	228
W 310 × 250 × 79	790	10046	306	254	8.8	14.6	15	17700	3990	133	63	1160	314
W 310 × 250 × 86	860	10998	310	254	9.1	16.3	15	19800	4450	134	63.6	1280	350
Column Sections													
W 310 × 310 × 97	970	12330	308	305	9.9	15.4	15	22240	7286	134.3	76.9	1444	477.8
W 310 × 310 × 107	1070	13620	311	306	10.9	17	15	24790	8123	134.9	77.2	1594	530.9
W 310 × 310 × 117	1170	14970	314	307	11.9	18.7	15	27510	9024	135.6	77.6	1753	587.9
W 310 × 310 × 129	1290	16510	318	308	13.1	20.6	15	30770	10040	136.5	78	1935	651.9
W 310 × 310 × 143	1430	18230	323	309	14	22.9	15	34760	11270	138.1	78.6	2153	729.4
W 310 × 310 × 158	1580	20050	327	310	15.5	25.1	15	38630	12470	138.8	78.9	2363	804.8
W 310 × 310 × 179	1790	22770	333	313	18	28.1	15	44530	14380	139.9	79.5	2675	918.7
W 310 × 310 × 202	2020	25800	341	315	20.1	31.8	15	51982	16588	141.9	80.2	3049	1053
W 310 × 310 × 226	2260	28880	348	317	22.1	35.6	15	59560	18930	143.6	81	3423	1194
HE 320 A	976	12440	310	300	9	15.5	27	22930	6985	135.8	74.9	1479	465.7
HE 320 B	1270	16130	320	300	11.5	20.5	27	30820	9239	138.2	75.7	1926	615.9
HE 320 M	2450	31200	359	309	21	40	27	68130	19710	147.8	79.5	3796	1276
(4) Nominal size 350 mm													
HE 340 A	1050	13350	330	300	9.5	16.5	27	27690	7436	144	74.6	1678	495.7
HE 340 B	1340	17090	340	300	12	21.5	27	36660	9690	146.5	75.3	2156	646
HE 340 M	2480	31580	377	309	21	40	27	76370	19710	155.5	79	4052	1276
IPE 360	571	7273	360	170	8	12.7	18	16270	1043	149.5	37.9	903.6	122.8
HE 360 A	1120	14280	350	300	10	17.5	27	33090	7887	152.2	74.3	1891	525.8
HE 360 B	1420	18060	360	300	12.5	22.5	27	43190	10140	154.6	74.9	2400	676.1
HE 360 M	2500	31880	395	308	21	40	27	84870	19520	163.2	78.3	4297	1268

(contd)

Table A.7 (*contd*)

Column Sections														
W 360 × 370 × 134	1340	17060	356	369	11.2	18	20	41510	15080	156	94	2332	817.3
W 360 × 370 × 147	1470	18790	360	370	12.3	19.8	20	46290	16720	157	94.3	2572	903.9
W 360 × 370 × 162	1620	20630	364	371	13.3	21.8	20	51540	18560	158.1	94.9	2832	1001
W 360 × 370 × 179	1790	22830	368	373	15	23.9	20	57440	20680	158.6	95.2	3122	1109
W 360 × 370 × 196	1960	25030	372	374	16.4	26.2	20	63630	22860	159.4	95.6	3421	1222
W 360 × 410 × 216	2160	27550	375	394	17.3	27.7	20	71140	28250	160.7	101.3	3794	1434
W 360 × 410 × 237	2370	30090	380	395	18.9	30.2	20	78780	31040	161.8	101.6	4146	1572
W 360 × 410 × 262	2620	33460	387	398	21.1	33.3	20	89410	35020	163.5	102.3	4620	1760
W 360 × 410 × 287	2870	36630	393	399	22.6	36.6	20	99710	38780	165	102.9	5074	1944
W 360 × 410 × 314	3140	39920	399	401	24.9	39.6	20	110200	42600	166.2	103.3	5525	2125
W 360 × 410 × 347	3470	44200	407	404	27.2	43.7	20	124900	48090	168.1	104.3	6140	2380
(5) *Nominal size 400 mm*													
IPE 400	663	8446	400	180	8.6	13.5	21	23130	1318	165.5	39.5	1156	146.4
HE 400 A	1250	15900	390	300	11	19	27	45070	8564	168.4	73.4	2311	570.9
HE 400 B	1550	19780	400	300	13.5	24	27	57680	10820	170.8	74	2884	721.3
HE 400 M	2560	32580	432	307	21	40	27	104100	19340	178.8	77	4820	1260
W 410 × 140 × 38.8	388	4970	399	140	6.4	8.8	11	12620	403.5	159.3	28.5	632.6	57.65
W 410 × 140 × 46.1	461	5880	403	140	7	11.2	11	15550	513.6	162.6	29.5	771.9	73.37
W 410 × 180 × 53	530	6800	403	177	7.5	10.9	11	18600	1009	165.4	38.5	922.9	114
W 410 × 180 × 60	600	7580	407	178	7.7	12.8	11	21570	1205	168.7	39.9	1060	135.4
W 410 × 180 × 67	670	8580	410	179	8.8	14.4	11	24530	1379	169.1	40.1	1196	154.1
W 410 × 180 × 75	750	9520	413	180	9.7	16	11	27460	1559	169.8	40.5	1330	173.2
W 410 × 180 × 85	850	10830	417	181	10.9	18.2	11	31530	1803	170.6	40.8	1512	199.3

(*contd*)

Table A.7 *(contd)*

W 410 × 260 × 100	1000	12700	415	260	10	16.9	11	39800	4950	177	62.4	1920	381
W 410 × 260 × 114	1140	14600	420	261	11.6	19.3	11	46200	5720	178	62.6	2200	438
W 410 × 260 × 132	1320	16840	425	263	13.3	22.2	11	53900	6740	179	63.3	2540	513
W 410 × 260 × 149	1490	19030	431	265	14.9	25	11	61900	7770	180	63.9	2870	586
(6) Nominal size 450 mm													
IPE 450	776	9882	450	190	9.4	14.6	21	33740	1676	184.8	41.2	1500	176.4
HE 450 A	1400	17800	440	300	11.5	21	27	63720	9465	189.2	72.9	2896	631
HE 450 B	1710	21800	450	300	14	26	27	79890	11720	191.4	73.3	3551	781.4
HE 450 M	2630	33540	478	307	21	40	27	131500	19340	198	75.9	5501	1260
W 460 × 150 × 52	520	6620	450	152	7.6	10.8	11	21200	634	178.9	30.9	942	83.43
W 460 × 150 × 60	600	7580	455	153	8	13.3	10	25480	796.1	183.3	32.4	1120	104.1
W 460 × 150 × 68	680	8730	459	154	9.1	15.4	10	29680	940.5	184.4	32.8	1293	122.1
W 460 × 190 × 74	740	9460	457	190	9	14.5	10	33260	1661	187.5	41.9	1456	174.8
W 460 × 190 × 82	820	10440	460	191	9.9	16	10	37000	1862	188.3	42.2	1608	195
W 460 × 190 × 89	890	11390	463	192	10.5	17.7	10	40960	2093	189.6	42.9	1769	218
W 460 × 190 × 97	970	12350	466	193	11.4	19	10	44680	2282	190.2	43.1	1917	237.8
W 460 × 190 × 106	1060	13460	469	194	12.6	20.6	10	48790	2515	190.4	43.2	2081	259.2
Column Sections													
W 460 × 280 × 113	1130	14400	463	280	10.8	17.3	18	55600	6335	196.5	66.3	2402	452.5
W 460 × 280 × 128	1280	16360	467	282	12.2	19.6	18	63690	7333	197.3	67	2728	520.1
W 460 × 280 × 144	1440	18410	472	283	13.6	22.1	18	72600	8358	198.6	67.4	3076	590.7
W 460 × 280 × 158	1580	20080	476	284	15	23.9	18	79620	9137	199.1	67.5	3346	643.5
W 460 × 280 × 177	1770	22600	482	286	16.6	26.9	18	91040	10510	200.7	68.2	3777	734.7
W 460 × 280 × 193	1930	24820	489	283	17	30.5	18	103000	11500	204	68.1	4210	813

(contd)

Table A.7 *(contd)*

W 460 × 280 × 213	2130	27290	495	285	18.5	33.5	18	115000	13000	205	69	4650	912
W 460 × 280 × 235	2350	30100	501	287	20.6	36.6	18	128000	14500	206	69.4	5110	1010
(7) Nominal size 500 mm													
IPE 500	907	11550	500	200	10.2	16	21	48200	2142	204.3	43.1	1928	214.2
HE 500 A	1550	19750	490	300	12	23	27	86970	10370	209.8	72.4	3550	691.1
HE 500 B	1870	23860	500	300	14.5	28	27	107200	12620	211.9	72.7	4287	841.6
HE 500 M	2700	34430	524	306	21	40	27	161900	19150	216.9	74.6	6180	1252
W 530 × 210 × 92	920	11760	533	209	10.2	15.6	14	55240	2379	216.7	45	2073	227.7
W 530 × 210 × 101	1010	12940	537	210	10.9	17.4	14	61760	2692	218.5	45.6	2300	256.4
W 530 × 210 × 109	1090	13870	539	211	11.6	18.8	14	66730	2951	219.3	46.1	2476	279.7
W 530 × 210 × 123	1230	15690	544	212	13.1	21.2	14	76100	3377	220.2	46.4	2798	318.6
W 530 × 210 × 138	1380	17640	549	214	14.7	23.6	14	86160	3870	221	46.8	3139	361.7
W 530 × 310 × 150	1500	19220	543	312	12.7	20.3	14	101000	10300	229	73.2	3720	660
W 530 × 310 × 165	1650	21090	546	313	14	22.2	14	111000	11400	229	73.5	4070	728
W 530 × 310 × 182	1820	23170	551	315	15.2	24.4	14	124000	12700	231	74	4500	806
W 530 × 310 × 196	1960	25060	554	316	16.5	26.3	14	134000	13900	231	74.5	4840	880
W 530 × 310 × 213	2130	27920	560	318	18.3	29.2	14	151000	15700	233	75	5390	987
W 530 × 310 × 248	2480	31440	571	315	19	34.5	14	178000	18000	238	75.7	6230	1140
(8) Nominal size 550 mm													
IPE 550	1060	13440	550	210	11.1	17.2	24	67120	2668	223.5	44.5	2441	254.1
HE 550 A	1660	21180	540	300	12.5	24	27	111900	10820	229.9	71.5	4146	721.3
HE 550 B	1990	25410	550	300	15	29	27	136700	13080	232	71.7	4971	871.8
HE 550 M	2780	35440	572	306	21	40	27	198000	19160	236.4	73.5	6923	1252

(contd)

Table A.7 *(contd)*

(9) Nominal size 600 mm													
IPE 600	1220	15600	600	220	12	19	24	92080	3387	243	46.6	3069	307.9
HE 600 A	1780	22650	590	300	13	25	27	141200	11270	249.7	70.5	4787	751.4
HE 600 B	2120	27000	600	300	15.5	30	27	171000	13530	251.7	70.8	5701	902
HE 600 M	2850	36370	620	305	21	40	27	237400	18980	255.5	72.2	7660	1244
W 610 × 230 × 101	1010	12980	603	228	10.5	14.9	14	76470	2950	242.7	47.7	2536	258.8
W 610 × 230 × 113	1130	14440	608	228	11.2	17.3	14	87570	3425	246.2	48.7	2881	300.5
W 610 × 230 × 125	1250	15960	612	229	11.9	19.6	14	98650	3932	248.6	49.6	3224	343.4
W 610 × 230 × 140	1400	17850	617	230	13.1	22.2	14	111990	4514	250.5	50.3	3630	392.5
W 610 × 325 × 155	1550	19730	611	324	12.7	19	14	129000	10780	255.7	73.9	4222	666
W 610 × 325 × 174	1740	22200	616	325	14	21.6	14	147200	12370	257.4	74.6	4778	761
W 610 × 325 × 195	1950	24930	622	327	15.4	24.4	14	167900	14240	259.5	75.6	5398	871
W 610 × 325 × 217	2170	27760	628	328	16.5	27.7	14	190800	16310	262.1	76.7	6076	995
W 610 × 325 × 241	2410	30340	635	329	17.1	31	14	214200	18430	265.7	77.9	6746	1120
W 610 × 325 × 262	2620	33270	641	327	19	34	14	235990	19850	266.3	77.2	7363	1214
W 610 × 325 × 285	2850	36360	647	329	20.6	37.1	14	260700	22060	267.8	77.9	8059	1341
W 610 × 325 × 341	3410	43370	661	333	24.4	43.9	14	318300	27090	270.9	79	9630	1627
W 610 × 320 × 372	3720	47630	669	335	26.4	48	20	355000	30200	273	79.6	10600	1800
(10) Nominal size 650 mm													
HE 650 A	1900	24160	640	300	13.5	26	27	175200	11720	269.3	69.7	5474	781.6
HE 650 B	2250	28630	650	300	16	31	27	210600	13980	271.2	69.9	6480	932.3
HE 650 M	2930	37370	668	305	21	40	27	281700	18980	274.5	71.3	8433	1245
(11) Nominal size 700mm													
HE 700 A	2040	26050	690	300	14.5	27	27	215300	12180	287.5	68.4	6241	811.9
HE 700 B	2410	30640	700	300	17	32	27	256900	14440	289.6	68.7	7340	962.7
HE 700 M	3010	38300	716	304	21	40	27	329300	18800	293.2	70.1	9198	1237

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