BALASORE COLLEGE OF ENGINEERING AND TECHNOLOGY, SERGARH, BALASORE

Lecture Notes

On

DESIGN OF STEEL STRUCTURE

3rd Year

6th Semester

Prepared by -:

Er. JYOTISHANKAR RATH, Asst. Professor

Civil Engineering Department

Checked by

MODULE WISE DISTRIBUTION OF LOADS

Module	Chapter with title	Assigned Hour (as per BPUT)	Actual Session Needed	Range of Marksof Questions to be being asked (BPUT)
I	Limit State Design Method: design philosophy, limit state of Serviceability, riveted, bolted and pinned connections Welded connections, types of welded connection	**10**	20	35-45
II	Tension members ,types of failures, design of tension members	**6**	10	15-20
III	Compression members, design of axially loaded compression members.	**6**	10	20-30
IV	Design of beams. Types of c/s, bending shear strength, web buckling, design procedure	**8**	10	15-20
V	Plate girders- design of components eccentric and moment connections, roof truss	**6**	7	15-20
TOTAL		**36**	**57**	**100 marks**

6th Semester	RCI6C001	Design of Steel Structures	L-T-P 3-0-0	3 Credits

Module I **10 HOURS**

Introduction, advantages/disadvantages of steel, structural steel, rolled steel section, various types of loads, design philosophy.

Limit state design method, limit states of strength and serviceability, probabilistic basis for design Riveted, bolted and pinned connections,

Welded connections-assumptions, types, design of fillet welds, intermittent fillet weld, plug and slot weld, failure of welded joints, welded joints vs bolted and riveted joints

Module II **6 HOURS**

Tension members, types, net cross-sectional area, types of failure, slenderness ratio, design of tension members, gusset plate.

Module III **6HOURS**

Compression members, effective length, slenderness ratio, types of cross-section, classification of cross section,

Design of axially loaded compression members, lacing, battening, design of column bases, and foundation bolts.

Module IV **8 HOURS**

Design of beams, types of c/s, lateral stability of beams, lateral torsional buckling, bending and shear strength, web buckling and web crippling, deflection, design procedure.

Module V **6HOURS**

Plate girders- various elements and design of components Eccentric and moment connections, roof trusses

Books:

1. Design of Steel Structures- Limit State Method by N. Subramanian, Oxford University Press
2. Limit State Design of Steel structures by S.K. Duggal, Mc-Graw Hill
3. Design of steel structures by S.S.Bhavikatti, I.K. International Publishing house.
4. Design of Steel Structures by K. S. Sairam, Pearson
5. Steel Design by William T. Segui, Cengage Learning
6. Fundamentals of Structural Steel Design by M.L.Gambhir, Mc Graw Hill
7. Steel Structures-Design and Practice by N. Subramanian, Oxford University Press

Books:

Digital Learning Resources:

Course Name	Design of Steel Structure
Course Link	https://nptel.ac.in/courses/105/105/105105162/
Course Instructor	PROF. DAMODAR MAITY

Module – I Chapter – 1
Lecture – 1

INTRODUCTION TO STEEL DESIGN

Learning Objectives

1.1 Knowledge about steel
1.2 Uses of Steel as a Structural Manual
1.3 Properties of Steel (Both Physical and Mechanical)

1.1 Introduction:

The design of any structure consists of two parts viz. (i) functional design and (ii) structural design. The first part comprises planning the building to attend to its requirements taking into account ventilation, lighting, aesthetic view, etc. The structural design consists in proportioning various elements of the building such that loads acting on it are transferred safely to the ground and at the same time unnecessarily excess material is not used. For transferring the loads to the ground various materials, like asbestos sheets, tiles, bricks, cement concrete, reinforced concrete, steel, and aluminium are used. However, the main body of the present-day structures consists of R.C.C or steel. In tall structures, composite construction of steel and concrete is also commonly used

Advantages:

The advantages of steel over other materials for construction are:
1. It has high strength per unit mass.
2. It has a secure quality and high durability.
3. Speed of construction.
4. Steel structures can be strengthened at any later time, if necessary.
5. By using bolted connections, steel structures can be easily dismantled.
6. Material is reusable.

Disadvantages:

1. It is susceptible to corrosion.
2. Maintenance cost is high.
3. Steel members are costly.

1.2 Structural steel:

Steel is an alloy of iron and carbon. Apart from carbon by adding small percentage of manganese, sulphur, phosphorus, chrome nickel and copper special properties can be imparted to iron and a variety of steels can be produced. The effect of different chemical constituents on steel are generally as follows:

(i) Increased quantity of carbon and manganese imparts higher tensile strength and yields properties but lower ductility, which is more difficult to weld.
(ii) Improved sulphur and phosphorus beyond 0.06 percent communicate brittleness, affects weldability and fatigue strength.
(iii) Chrome and nickel impart corrosion resistance property to steel. It improves resistance to high temperature also.
(iv) Addition of a small quantity of copper also increases the resistance to corrosion.

By slightly varying chemical composition various types of steels are manufactured to be used as structural member, tubes, pipes, sheets, strips, reinforcements for R.C.C, rivets, bolts, nuts and for welding. Mainly structural steels are discussed and their properties presented. The structural steel is the steel used for the manufacture of rolled steel sections. These rolled steel sections are used to form steel frameworks required in the structures. Structural steel may be mainly classified as mild steel and high tensile steel. Structural steel is also known as standard quality steel. Its requirements have been specified in IS 226-1975. This steel is also available in copper bearing quality in which case it is designated as Fe 410-Cu-S, where 410 refers to ultimate tensile strength of 410 MPa (= 410 N/mm2). This is also known as grade E250 steel in which 250 refers to 250 MPa yield strength. E300 (Fe-440) and E-350 (Fe 490) steels are also manufactured. In high tensile steel mechanical properties and resistance to corrosion are enhanced by alloying with small proportions of some other alloys or increasing the carbon content. Standards of high tensile steel are covered in IS 961-1975. Weldable quality steels which are recommended by IS 2007 are designated as E410 (Fe 540), E450 (Fe 570)D and E450 (Fe 590)E. As per IS 800-2007, the structural steel used in general construction, coming under the view shall conform to IS 2062 i.e., to weldable quality steel. Structural steel other than those specified under mild steel and high tensile steel conforming to weldable quality may also be used provided that the permissible stresses and other design provisions are suitably modified and the steel is also suitable for the type of fabrication adopted. Steel (ordinary quality) that is not supported by mill test result may be permitted to be used for unimportant members, where their properties such as ductility and weldability do not affect the performance requirements of the structure as a whole. Mild steel (structural steel-standard quality) and high tensile steel of weldable quality (conforming to IS 2062) are considered for the design.

1.3 PROPERTIES OFSTRUCTURAL STEEL:

The properties of steel required for engineering design may be classified as

(i) Physical Properties

(ii) Mechanical Properties.

(i) **Physical Properties**:

Regardless of its grade physical properties of steel may be taken as given below (clause 2.2.4 of IS 800-2007):

(a) Unit mass of steel, ρ = 7850 kg /m3.

(b) Modulus of elasticity, E = 2.0 × 10 5 N/mm2.

(c) Poisson's ratio, μ = 0.3.

(d) Modulus of rigidity, G = 0.769 × 10 5 N/mm2.

(e) Coefficient of thermal expansion, αt = 12 × 10 –6 /°C.

(ii) **Mechanical Properties**: The following are the important mechanical properties in the design:

(a) Yield stress fy.

(b) The tensile or ultimate stress fu.

(c) The maximum percentage elongation on a standard gauge length and

(d) Notch toughness.

Except for notch toughness, the other properties are determined by conducting tensile tests on samples cut from the plates, sections etc. IS 800-2007 gives mechanical properties of different types of structural steel products in its Table 1.1.

INTRODUCTION TO ROLLED STEEL SECTIONS

Learning Objectives

1.4 To know about different types of steel sections

1.4 ROLLED STEEL SECTIONS:

Alike concrete, steel section of any shape and size cannot be cast on site, since steel needs very high temperature to melt it and roll into required shape. Steel sections of standard shapes, sizes and length are rolled in steel mills and marketed. User has to cut them to the required length and use required sections for the steel framework. Many steel sections are readily available in the market and are in frequent demand such steel sections are known as Regular Steel Sections. Some steel sections are not in use commonly, but the steel mills can roll them if orders are placed. Such steel sections are known as Special Sections. Various types of rolled steel sections manufactured are below:

(i) Rolled steel I-sections (Beam sections)
(ii) Rolled steel Channel sections
(iii) Rolled steel Angle sections
(iv) Rolled steel Tee sections
(v) Rolled steel Bars
(vi) Rolled steel Tubes
(vii) Rolled steel Plates
(viii) Rolled steel Flats
(ix) Rolled steel Sheets and Strips.

Steel tables give nominal dimensions, weight per metre length and geometric properties of various rolled steel sections.

1.4.1 Rolled Steel I-section:

The following five series of rolled steel I-sections are manufactured in India:

(a) Indian Standard Junior Beams – ISJB
(b) Indian Standard Light Beams – ISLB
(c) Indian Standard Medium Beams – ISMB
(d) Indian Standard Wide-flange Beams – ISWB
(e) Indian Standard Heavy Beams – ISHB.

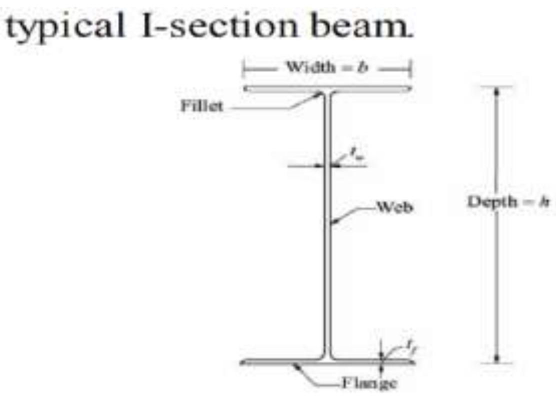
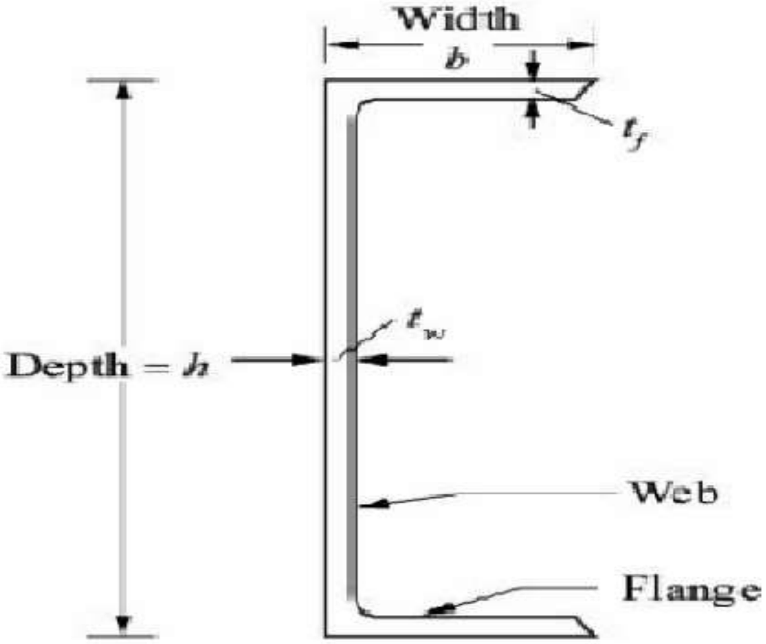


Figure 1.1 Rolled steel I-sec

ISMB 500 @ 0.852 kN/m
ISWB 600 @ 1.423 kN/m
ISWB 600 @ 1.312 kN/m
ISHB 450 @ 0.855 kN/m
ISHB 450 @ 0.907 kN/m.

1.4.2 Rolled Steel Channel Sections:

These sections are classified into the following four series:
(a) Indian Standard Junior Channel – ISJC
(b) Indian Standard Light Channel – ISLC
(c) Indian Standard Medium Weight Channel – ISMC
(d) Indian Standard Special Channel – ISSC.



•

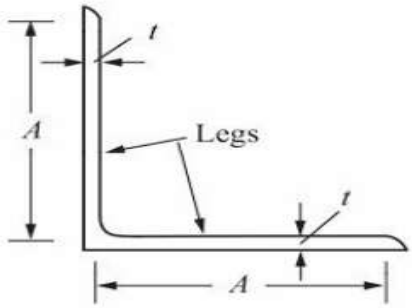
Rolled steel channel section.

Rolled steel channel sections are designated by the series to which they belong, followed by depth (in mm) and weight (in kN/m). e.g. ISMC 300 @ 0.351 kN/m.

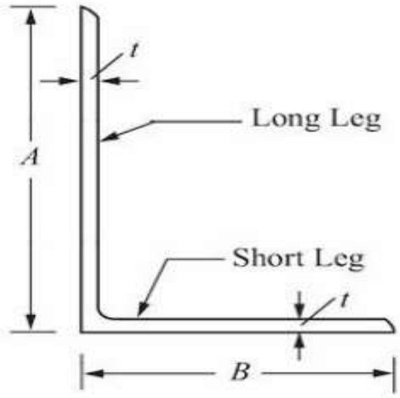
1.4.3 Rolled Steel Angle Sections:

These are classified into the following two series:
(a) Indian Standard Equal Angle – ISA
(b) Indian Standard Unequal Angle – ISA.
Figure below shows typical sections.

The thickness of legs of equal and unequal angles are the same. Rolled steel equal and unequal angles are designated by their series name ISA followed by length and thickness of legs e.g.: ISA 150 150, 12 mm thick or ISA 150 × 150 × 12 ISA 150 115, 10 mm thick or ISA 150 × 115 × 10.



(a) Rolled steel equal angle.



(b) Rolled steel unequal angle.

1.4.4 Rolled Steel Tee Sections:

Following five series of rolled steel Tee sections are available:

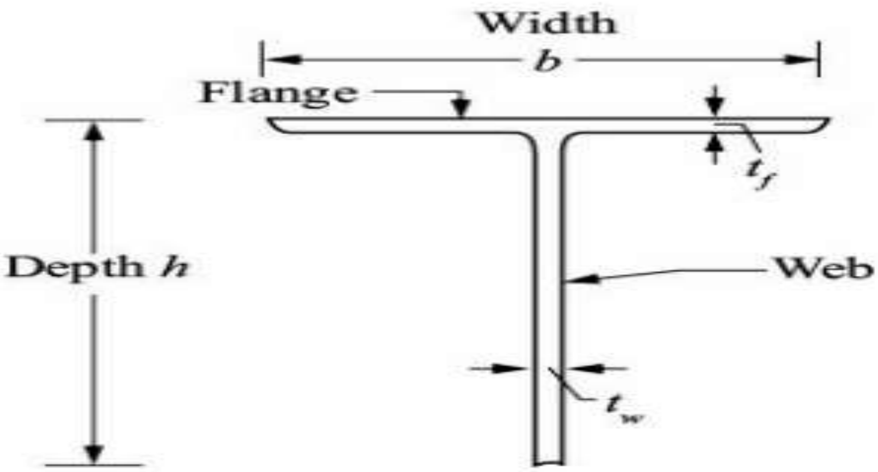
(a) Indian Standard Normal Tee bars – ISNT

(b) Indian Standard Heavy flanged Tee bars – ISHT

(c) Indian Standard Special Legged Tee bars – ISSLT

(d) Indian Standard Light Tee bars – ISLT

(e) Indian Standard Junior Tee bars – ISJT.



Rolled steel T-section.

These rolled steel sections are designated by the series to which they belong followed by depth and weight per metre length. e.g: ISNT 60 @ 53 N/m

As per IS 808-1984, the following T-sections have also been adopted:

(a) Indian Standard Deep legged Tee bars – ISDT

(b) Indian Standard slit Medium weight Tee bars – ISMT

(c) Indian Standard slit Heavy Tee bars from I-sections – ISHT.

1.4.5 Rolled Steel Bars:

Rolled steel bars are classified into the following two series:

(a) Indian Standard Round bars – ISRO

(b) Indian Standard Square bars – ISSQ.

Rolled steel bars are designated by ISRO followed by diameter in case of round bars and ISSQ followed by side width in case of square bars e.g: ISRO 16 ISRQ 20.

1.4.6 Rolled Steel Tubes:

These sections are designated by their nominal sizes. In each size, there are three classes, namely Light, Medium and Heavy. The difference is due to the difference in their thicknesses. Hence their cross-sectional properties are also different. For example, a 40 mm tube has 3 types and their sectional properties are as given below:

1.4.7 Rolled Steel Plates:

Rolled steel plates of the following thicknesses are available: 5, 6, 8, 10, 12, 14, 16, 18, 20, 22, 25, 28, 32, 36, 40, 45, 50, 56, 63, 71, 80 mm.

They are rolled in the 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, 2500 mm.

These plates are designated by ISPL followed by length, width and thickness. e.g.: ISPL2000 × 1000 × 6.

1.4.8 Rolled Steel Strips:

Rolled steel strip is designated as ISST followed by width and thickness. These sections are available in the following width and thickness:

Width: 100, 110, 125, 140, 160, 180, 200, 220, 250, 280, 320, 355, 400, 450, 500, 560, 630, 710, 800, 900, 1000 mm.

Thickness: 0.8, 0.9, 1.0, 1.1, 1.2, 1.4, 1.6, 1.8, 2.0, 2.2, 2.5, 2.8, 3.2, 3.5, 4.0, 4.5 mm.

It may be noted that thickness of strips is less than 5 mm. Rolled steel strip is designated as ISST, followed by width and thickness

e.g.: ISST 250 × 2.5 mm.

1.4.9 Rolled Steel Flats:

Flats differ from strips in the sense that the thickness of flats is 5 mm onward and their width is limited. Flats of the following width and thickness are as

Width: 12, 16, 20, 25, 32, 40, 50, 63, 80, 100, 125, 160, 200, 250 mm.

Thickness: 5, 5.5, 6, 7, 8, 9, 10, 11, 12, 14, 16, 18, 20, 22, 25 mm.

They are designated by width followed by letters ISF and thickness

e.g.: 80 ISF 10 means, 80 mm wide Indian Standard Flat of thickness 10 mm.

TYPES OF LOADS

Learning Objectives
1.5 Knowledge about types of loads coming to the structure
1.6 Uses of IS Codes

1.5 LOADS:

Various loads expected to act on a structure may be classified as given below:
(a) Dead Loads (DL)
(b) Imposed Loads (IL)
(c) Wind Loads (WL)
(d) Earthquake Loads (EL)
(e) Erection Loads (ER)
(f) Accidental Loads (AL)
(g) Secondary Effects.

(a) Dead Loads: Dead loads include the weight of all permanent construction. In a building weight of roofs, floors, floor finishes, walls, beams, columns, footing, architectural finishing materials etc., constitute a dead load.These loads may be calculated by estimating the quantity of each material and then multiplying it by unit weight. The unit weight of various materials in a structure is given in IS code 875 (part I).

(b) Imposed Loads: The following loads are imposed loads as in IS 800-2007.
(i) Live load [IS 875 (part 2)-1987].
(ii) Crane load (IS 800-2007, clause 3.5.4).
(iii) Snow load [IS 875 (part 4)]
(iv) Dust load [probable thickness of accumulation of dust may be made]
(v)Hydrostatic and earth pressure [IS 875 (part 5)]
(vi) Impact load: (For structures supporting moving loads suitable additional allowance of load should be made by increasing imposed load.) For example

Structure	Impact Allowance, Percent, Minimum
(a) For frames supporting lifts and hoists	100
(b) For foundations, footings and piers supporting lifts and hoisting apparatus	40
(c) For supporting structures and foundations for light machinery, shafts or motor units	20
(d) For supporting structures and foundations for reciprocating machinery or power unit	50
(e) For girders supporting electric wire near head cranes	25
(f) For girders supporting hand operated cranes	10

(vii) Horizontal loads on parapets and balustrades.

Sl. No.	Usage Area	kN/m
1	Light access stairs, not more than 600 mm wide	0.25
2	Light access stairs, more than 600 mm wide	0.35
3	All other stairways (except those subject to overcrowding)	0.75
4	In place of assembly such as theatres, schools, auditoriums, stadiums and buildings likely to be overcrowded	2.25

(c)Wind Loads: The force exerted by the horizontal component of wind is to be considered in the design of buildings, towers etc. The wind force depends upon the velocity of wind, shape, size and location of building. Complete details of calculating wind load on structure is given in IS 875 (part 3).
(d) Earthquake Loads: Earthquake shocks cause movement of foundation of structures. Due to inertia additional forces develop on superstructure. The total vibration caused by earthquake may be resolved into three mutually perpendicular directions, usually taken as vertical and two horizontal directions.
(e) Erection Loads: Prefabricated or precast members are subjected to different types of supports and different types of loads during erection compared to the types of supports and types of loads after erection. It is the responsibility of engineer to see that the structure or part of the structure do not fail during erection. Many cases of such failures are reported and in all cases engineers are held responsible. During erection storage of materials, equipment and impact of hoisting equipment cause special loads. Dead load, wind load and imposed live load during erections shall be considered along with the special erection loads. Special provisions shall be made including temporary bracing to take care of all such loads during erection.
(f) Accidental Loads: IS 875 (part 5) gives certain guidelines to take care of the following accidental loads on the structures:

i. Impact and Collision
ii. Explosion and
iii. Fire

The probability of occurrence of such loads may be quite less.

1.6 LOAD COMBINATIONS

A judicious combination of the loads is necessary to ensure the required safety and economy in the design keeping in view the probability of (a) their acting together (b) their disposition in relation to other loads and severity of stresses or deformation caused by the combination of various loads. The recommended load combinations by IS 875 are as given below.

1	DL	7	DL + IL + EL
2	DL + IL	8	DL + IL + TL
3	DL + WL	9	DL + WL + TL
4	DL + EL	10	DL + EL + TL
5	DL + TL	11	DL + IL + WL + TL
6	DL + IL + WL	12	DL + IL + EL + TL

Where DL = Dead Load
IL = Imposed Load
WL = Wind Load
EL = Earthquake Load and
TL = Temperature Load.
Note: When snow load is present on roofs, replace imposed load by snow load for the purpose of above load combinations.

Module – I Chapter – 1

Lecture – 4

DESIGN PHILOSOPHY

Learning Objectives

1.7 To learn difference methods of design

1.7 **Design Philosophy**

The aim of design is to decide shape, size and connection details of the members so that the structure being designed will perform satisfactorily during its intended life. With an appropriate degree of safety the structure should

(a) Sustain all loads expected on it.

(b) Sustain deformations during and after construction.

(c) Should have adequate durability.

(d) Should have adequate resistance to misuse and fire.

(e) Structure should be stable and have alternate load paths to prevent overall collapse under accidental loading.

Analytical method of design consists in idealizing the structure, quantifying expected loads, carrying analysis to find member forces and sizing the members based on possible failure criteria. Since there are limitations in precisely modelling the structure, working condition is kept as a fraction of failure condition. The design philosophies used are listed below in the order of their evolution and they are briefly explained:

Working Stress Method (WSM)

Ultimate Load Design (ULD) and

Limit State Method (LSM).

1.7.1 WORKING STRESS METHOD: This is the oldest systematic analytical design method. Though IS 800-2007 insists for the limit state design, permits use of this method wherever LSD cannot be conveniently adopted. In this method stress strain relation is considered linear till the yield stress. To take care of uncertainties in the design, permissible stress is kept as a fraction of yield stress, the ratio of yield stress to working stress itself known as factor of safety. The members are sized so as to keep the stresses within the permissible value.

Thus for structural mild steel IS: 226 – 1975,

i.e., for E 250/Fe 410 steel,

permissible stress in direct tension = 0.6fy = 0.6 × 250 = 150 N/mm2

Permissible direct shear stress in mild steel bolts = 100 N/mm2

The following load combinations are considered and increase of permissible stress by 33% is permitted when DL, LL and WL are considered:

Stress due to DL+ LL ≤ permissible stress

Stress due to DL+ WL ≤ permissible stress

Stress due to DL+ LL+ WL ≤ 1.33 permissible stress.

The limitations of WSM: The limitations of working stress method are

1. It gives the impression that factor of safety times the working load is the failure load, which is not true. Actually it is much more, because a material can resist the load after yield appears at a fibre. In the indeterminate structures just formation of a plastic hinge is not the failure criteria, since it can resist load till some more hinges are formed resulting into collapse mechanism. Thus the redistribution of moments gives rise to the additional load carrying capacity.

2. It gives uneconomical sections.

Advantages of WSM

1. This method is simple.
2. This is reasonably reliable.
3. As the working stresses are low, the serviceability requirements are satisfied automatically.

1.7.2 ULTIMATE LOAD METHOD: The limitation of the working stress method to assess actual load-carrying capacity, made researchers develop the ultimate load method, which is also known as the load factor method (LFM). When applied to steel structure it is referred to as the plastic design method. In this method, a section is said to have formed a plastic hinge when all the fibres yield. After that, it continues to resist load which has caused plastic hinge but will not resist any more load. But structure continues to resist further load till sufficient plastic hinges are formed to develop a collapse mechanism. In this method, safety measures are introduced by suggesting a load factor, which is defined as the ratio of design load to working load. The suggested load factors as per IS 800:1984 were given in Table.

Advantages of ULD

1. Redistribution of internal forces is accounted.
2. It allows a varied selection of load factors.

Disadvantages of ULD

It does not guarantee serviceability performance.

1.7.3 LIMIT STATE METHOD: It is the comprehensive method which takes care of both strength and serviceability requirements. IS: 800 – 2007 suggests use of this method widely and restricts working stress method only wherever limit state method cannot be applied. In the limit state method the strength of structure till the formation of collapse mechanism is considered. To take care of uncertainties involved in the analysis, design and construction the code suggest increasing working loads (which are based on statistical analysis and are known as characteristic loads) by partial safety rf and terms it as design loads. There are uncertainties about uniformity of material properties and sectional dimensions. To take care of these uncertainties code recommends that design strength be reduced by partial safety factor gm. The details of these partial safety factors are given in next lecture. For example, design strength of a plate of cross section A_n in direct tension is given by

$$T_{dn} = \frac{0.9 \times A_n \times f_u}{\gamma_m}$$

where, f_u = ultimate strength of steel. Similarly design strength of a bolt of diameter ‘d’ in direct single shear is given by

$$V_{dsb} = \frac{1}{\gamma_{mb}} \times V_{nsb}$$
$$= \frac{1}{\gamma_{mb}} \times \frac{f_{ub}}{\sqrt{3}} \times \frac{\pi}{4} d^2 \times 0.78$$

where V_{nsb} = nominal shear strength of the bolt

f_{ub} = ultimate shear strength of the bolt

The factor 0.78 is used to take care of the reduction in the cross-section of the bolt due to threading. However, it is necessary to take care of serviceability requirements. Sometimes it may be necessary to revise the design for satisfying serviceability requirements. The limit state method is thoroughly dealt with onwards.

Module – I Chapter – 1

Lecture – 5

PRINCIPLES OF LIMIT STATE DESIGN

Learning Objectives

1.8 To learn details about Limit State Design

1.9 Gain knowledge about Characteristic Loads and Design loads

1.8 LIMIT STATE DESIGN

Aim of a design is to see that the structure built is safe and it serves the purpose for which it is built. A structure may become unfit for use not only when it collapses but also when it interrupts the serviceability requirements of deflections, vibrations, cracks due to fatigue, corrosion and fire. In this method of design various limiting conditions are fixed to consider a structure as fit. At any stage of its designed life (120 years for permanent structures), the structure should not exceed these limiting conditions. The design is based on probable load and probable strength of materials. These are to be selected on probabilistic approach. The safety factor for each limiting condition may vary depending upon the risk involved. It is not necessary to design every structure to withstand exceptional events like blast and earthquake. In limit state design risk based evaluation criteria is included. Thus the philosophy of limit state design method is to see that the structure remains fit for use throughout its designed life by remaining within the acceptable limit of safety and serviceability requirements based on the dangers involved.

1.8.1 DESIGN REQUIREMENTS

Steel structure designed and constructed should satisfy the requirements regarding stability, strength, serviceability, brittle fracture, fatigue, fire and durability. The structures should meet the following requirements (IS 800- 2007, clause 5.1.2):

(A) Remain fit with adequate reliability and be able to sustain all loads and other influences experienced during construction and use.

(B) Have adequate durability under normal maintenance.

(C) Do not suffer overall damage or collapse disproportionately under accidental events like explosions, vehicle impact or due to consequences of human error to an extent beyond local damage.

The disastrous damage shall be limited or avoided by appropriate choice of one or more of the following:

(a) Avoiding, eliminating or reducing exposure to hazards, which the structure is likely to sustain.

(b) Choosing structural forms, layouts and details and designing such that:

(i) the structure has low sensitivity to hazardous conditions and

(ii) the structure survives with only local damage even after serious damage to any one individual element by the hazard.

(c)Choosing suitable material, designing and detailing procedure and construction procedure as relevant to the particular structure.

The collapse is considered unequal, if more than 15 percent of the floor or roof area of 70 m2 collapses at that level and one adjoining level either above or below it, under a load equal to 1.05 or 0.9 times the dead load, 0.33 times temporary or full imposed load of permanent nature and 0.33 times wind load acting together. To avoid disproportionate collapse, the following conditions should be satisfied:

(a) The building should be effectively tied together at each principal floor level and each column should be effectively held in position. These ties may be steel members such as beams which may be designed for other purposes or the shear connectors which connect floor with beams and columns. These connections should be capable of resisting:

(i) expected tensile force subjected to a minimum of 75 KN.

(ii) one percent of the maximum axial compression in the column.

(b) All column splices should be capable of resisting a tensile force equal to the largest of a factored dead and live load from the floor above or below the splice.

(c) Lateral load system to resist horizontal loads should be distributed throughout the building in nearly orthogonal directions.

(d) Floor or roof units should be effectively anchored in the direction of their spans either to each other or directly to the support. If above provisions are not made design should be checked for disproportionate collapse.

1.8.2 LIMIT STATES:

Limit states are the states beyond which the structure no longer satisfied the specified performance requirements. The various limit states to be considered in design may be grouped into the following two major categories:

(a) Limit state of strength

(b) Limit state of serviceability.

(a) Limit state of strength: The limit states, arranged to avoid collapse of structure which may endanger the safety of life and property, are grouped under this category. The limit state of strength includes:

(i) Loss of equilibrium of whole or part of the structure.

(ii) Loss of stability of structure as a whole or part of it.

(iii) Failure by excessive deformation.

(iv) Fracture due to fatigue

(v) Brittle fracture.

(b) Limit state of serviceability: The limit state of serviceability includes:

Deformations and deflections adversely affecting the appearance or effective use of structure or causing improper functioning of equipment or services or causing damage to finishings.

Vibrations in structures or any part of its component limiting its functional effectiveness.

Repairable damage or crack due to fatigue. Corrosion. Fire.

1.9 ACTIONS (LOADS) The actions to be considered in a design are:

(1) Direct actions experienced by the structure due to self-weight and external actions.

(2) Imposed deformations such as that due to temperature and settlement. IS 800-2007, classified various actions in the following three groups:

(a) Permanent Actions (Qp): Actions due to self-weight and fixed equipment etc.

(b) Variable Actions (Qv): Actions during construction and service stage such as imposed loads, wind loads and earthquake loads etc.

(c) Accidental Actions (Qa): Actions expected due to explosions and impact of vehicles etc.

1.9.1 Characteristic Actions (Qc): The characteristic actions (Qc) are defined as the values of different actions which are not expected to be exceeded with more than 5 percent probability, during the life of the structure. One can work out these actions by statistical analysis, in all special cases, subject to minimum values specified in codes. In the absence of statistical analysis, the loads presented in IS 875 and other special codes may be considered characteristic loads.

1.9.2 Design Actions (Loads) Noting the importance of safety in civil engineering structures and the uncertainties involved in the analysis, design and construction, code specifies taking design actions as partial safety factor times the characteristic actions. The partial safety factors specified by code for limit state of strength and serviceability differ. The partial safety factors for loads are as given in Table 2.1 and design load Q_d is to be found as

$$Q_d = \sum_k \gamma_{fk} \, Q_{ck}$$

where γ_{fk} is partial safety factor for kth load.

Partial safety factors for loads, γf for limit state is given in tab-4 of IS 800-2007.

1.9.3 DESIGN STRENGTH:

In using the strength value of a material for design, the following uncertainties should be accounted:

(a) Possibility of unfavourable deviation of material strength from the characteristic value.

(b) Possibility of unfavourable variation of member sizes.

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

(d) Uncertainty in the calculation of strength of materials.

Hence IS 800-2007, recommends reduction in the strength of materials by a partial safety factor γ_m which is defined as

$$\gamma_m = \frac{S_u}{S_d} \quad \text{i.e.} \quad S_d = \frac{S_u}{\gamma_m}$$

where Su − ultimate strength

and Sd – design strength

Partial safety factors for materials are given in **Table 5 of IS 800-2007**

1.9.4 DEFLECTION LIMITS

Deflection limits are specified from the consideration that excess deformations do not cause damage to finishing. Deflections are to be checked to adverse but realistic combination of service loads and their arrangement. Elastic analysis may be used to find deflection. Design load for this purpose is the same as characteristic load (i.e. partial safety factor γf = 1.0) except when apart from DL, LL, CL and some more imposed loads are considered. The deflection limits specified by IS 800:2007 of Table 6.

1.9.5 OTHER SERVICEABILITYLIMITS:

Apart from deflection requirement, the design should also satisfy the following serviceability limits:

(a) Vibration limit

(b) Durability consideration

(c) Fire resistance.

a) Vibration Limit: Though most of the structures are designed for strength and then checked for deflection limits, some of the structures need check for vibration limits. The structures the floors of which support machineries, the flexible structures (with height to effective width ratio exceeding 5:1) etc., should be investigated for vibration under dynamic loads. In such cases, there are possibilities of resonance and fatigue failures. IS 800-2007 gives a set of guidelines to take care of vibration limits in Annex C.

b) Durability Considerations: The following factors affect the durability of a steel structure:

(i) Environment

(ii) Degree of exposure

(iii) Shape of the member and the structural detail

(iv) Protective measures

(v) Ease of maintenance

A designer should refer to the IS code provisions given in section 15 of IS 800-2007 and also to specialised literature on durability.

c) Fire Resistance: A steel structure should have sufficient fire resistance level (FRL) specified in terms of minutes depending upon the purpose for which the structure is used and the time taken to evacuate in case of fire. For detailed specifications a designer may refer section 16 of IS 800-2007 along with IS 1641, IS 1642, IS 1643 and any other specialised literature on fire resistance.

1.9.6 STABILITYCHECKS: After designing a structure for strength, it should be checked for instability due to overturning, uplift or sliding under factored loads. In checking for instability disturbing forces should be taken as design loads and stabilising forces may be taken as design loads (factored loads) with lesser factor of safety (0.9). A structure should be adequately stiff against sway and fatigue also. In the chapters to follow now onwards, design principles are made clear from the point of limit states of strength and deflections. In most of the buildings these are the predominant limit states, but in all important and special buildings, a designer has to ensure that other limit states are not exceeded.

* * *

SHORT TYPE QUESTIONS

1. Why carbon is added to iron for steel ?
2. Which material is added to steel to increase the resistance of corrosion?
3. What will be the weight density of steel
4. Which code provided physical and mechanical properties of steel?
5. f_y indicates which stress ?
6. What is the symbol of ultimate stress?
7. What do you mean by E410 (Fe 540)?
8. What is the meaning of ISA 150 150, 12 mm ?
9. Which code defines loads for steel structure?
10. What will be the Partial safety factor for loads γ_f in Limit State Method in DL+LL+CL combinations ?

Answers :

1. To improve the tensile strength and yield properties at the same time lowering the ductility.
2. Small quantity of copper, chromium and Nickel
3. 7850kg/m^3
4. IS 800 – 2007
5. Yield Stress
6. f_u
7. Yield stress of the material fy = 410 N/mm^2
   Ultimate tensile stress f_u = 540 N/mm^2
8. Indian standard Equal Angels of Legs 150mm and thickness t=12mm
9. IS 875 – 1984 (Part – I, Part – II, Part – III, Part- IV, Part – V)
10. 1.5

MEDIUM TYPE QUESTIONS

1. Explain the advantages and disadvantages of using steel structure.
2. Explain the principles of
(a) Working stress method of design
(b) Ultimate load design and
(c) Limit state design.
3. Explain how the limit state method differs from the working stress method.
4. Explain the following terms
(a) Partial Safety factor for loads
(b) Partial safety factor for material strength

DESIGN OF CONNECTIONS

Learning Objectives

2.1 *Brief explanation of Riveted connection*
2.2 Detail design of Bolted Connection

INTRODUCTION TO CONNECTIONS

As steel structures are to be formed by connecting available standard sections. There is a need for designing the following connections:

(a) different sections to form the required composite section of a member (e.g. connecting plates, angles, channels, I-sections etc.)

(b) different members at their ends (e.g. secondary beams to main beams, beams to columns, columns to footing or members of truss etc.).

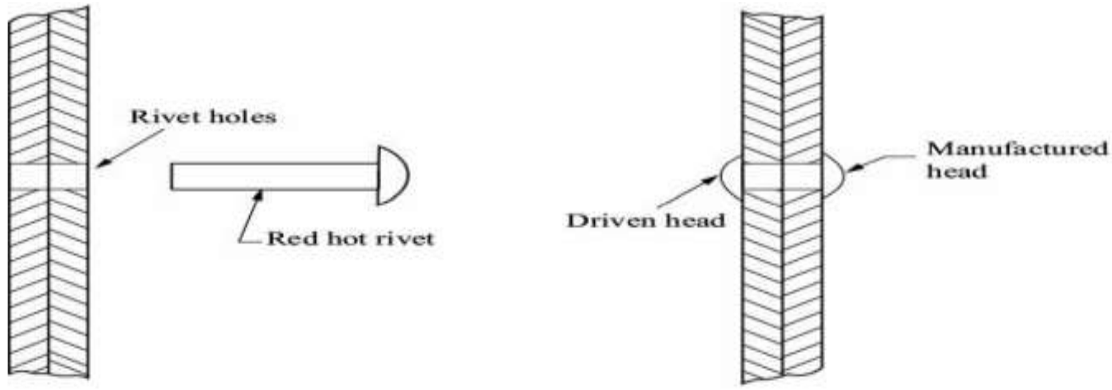
The design of connections is very important because the failure of the joint is sudden and disastrous. The following three types of connections may be made in steel structures:

(a) Riveted
(b) Bolted
(c) Welded.

Here a brief introduction is given to riveted connections and a detailed design procedure is explained for bolted connections.

2.1 RIVETED CONNECTION:

Riveting is a method of joining together pieces of metal by inserting ductile metal pins called rivets into holes of pieces to be connected and forming a head at the end of the rivet to prevent each metal piece from coming out.



The above Figure shows connecting two plates by riveting. Rivet holes are made in the structural members to be connected by punching or by drilling. The size of the rivet hole is kept slightly more (1.5 to 2 mm) than the size of the rivet. After the rivet holes in the members are matched, a red hot rivet is inserted which has a shop-made head on one side and the length of which is slightly more than the combined thicknesses of the members to be connected. Then holding a red hot rivet at the shop head end, hammering is made. It results in expansion of the rivet to completely fill up the rivet hole and also into the formation at the driven head. Desired shapes can be given to the driven head. The riveting may be in the workshops or in the field.

2.1.1 Riveting has the following disadvantages:

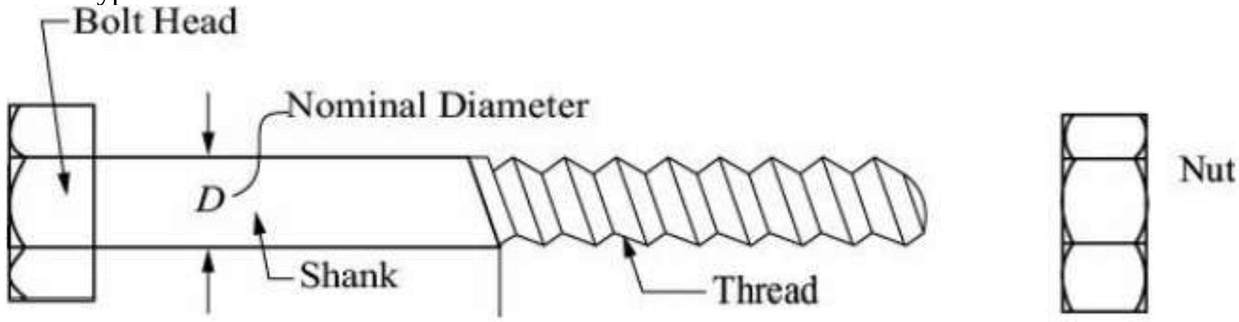
(a) It is associated with a high level of noise pollution.
(b) It needs heating the rivet to red hot.
(c) Inspection of connection is skilled work.
(d) Removing poorly installed rivets is costly.

(e) Labour cost is high.

Production of weldable quality steel and introduction of high strength friction grip bolts (HSFG) have replaced use of rivets. Design procedure for the riveted connections is the same as that for bolted connections except that the effective diameter of rivets may be taken as rivet hole diameter instead of nominal diameter of rivets. Hence riveted connection is not discussed further.

2.2 BOLTED CONNECTIONS

A bolt is a metal pin with a head formed at one end and shank threaded at the other in order to receive a nut. Bolts are used for joining together pieces of metals by inserting them through holes in the metal and tightening the nut at the threaded ends. Figure below
shows a typical bolt.



•

Bolts are classified as:

(a) Unfinished (Black) Bolts

(b) Finished (Turned) Bolts

(c) High Strength Friction Grip (HSFG) Bolts.

2.2.1 Unfinished/Black Bolts:

These bolts are made from mild steel rods with square or hexagonal head. The shank is left unfinished i.e. rough as rolled. Though the black bolts of nominal diameter (diameter of shank) of sizes 12, 16, 20, 22, 24, 27, 30 and 36 mm are available, commonly used bolt diameters are 16, 20, 24, 30 and 36 mm. These bolts are designated as M16, M20, M24, etc. IS 1364 (part 1) gives specifications for such bolts. In structural elements to be connected holes are made larger than nominal diameter of bolts. As shanks of black bolts are unfinished, the bolt may not establish contact with structural member at entire zone of contact surface. Joints remain quite loose resulting into large deflections. The yield strength of commonly used black bolts is 240 N/mm2 and ultimate strength 400 N/mm2 .

These bolts are used for light structures under static loads such as trusses, bracings and also for temporary connections required during erections.

2.2.2 Finished/Turned Bolts:

These bolts are also made from mild steel, but they are formed from hexagonal rods, which are finished by turning to a circular shape. Actual dimension of these bolts are kept 1.2 mm to 1.3 mm larger than the nominal diameter. As usual the bolt hole is kept 1.5 mm larger than the nominal diameter. Hence tolerance available for fitting is quite small. It needs special methods to align bolt holes before bolting. As connection is more tight, it results into much better bearing contact between the bolts and holes.

These bolts are used in special jobs like connecting machine parts subjected to dynamic loadings. IS 3640 covers specifications for such bolts.

2.2.3 High Strength Friction Grip (HSFG) Bolts:

The HSFG bolts are made from high strength steel rods. The surface of the shank is kept unfinished as in the case of black bolts. These bolts are tightened to a proof load using calibrated wrenches. Hence, they grip the members tightly. In addition, nuts are provided by using clamping devices. If the joint is subjected to shearing load it is primarily resisted by frictional force between the members and washers. The shank of the bolt is not subjected to any shearing. This results into no-slippage in the joint.Commonly available nominal diameter of HSFG bolts are 16, 20, 24, 30 and 36 mm.

Hence such bolts can be used to connect members subjected to dynamic loads also. The successful introduction of HSFG bolt resulted into replacement of rivets. IS 3747 specifies various dimensions for such bolts and for their washers and nuts.

CLASSIFICATION OF BOLTS BASED ON TYPE OF LOAD TRANSFER

Learning Objectives

2.3 *Classification of Bolts*

2.4 Terminology for Bolted Connection

2.3 CLASSIFICATION OF BOLTS BASED ON TYPE OF LOAD TRANSFER:

On the basis of load transfer in the connection bolts may be classified as

(a) Bearing Type

(b) Friction Grip Type.

Unfinished (black) bolts and finished (turned) bolts belong to bearing type since they transfer shear force from one member to other member by bearing, whereas HSFG bolts belong to friction grip type since they transfer shear by friction.

Advantages of HSFG Bolts Over Bearing Type Bolts:

HSFG bolts have the following advantages over unfinished or finished bolts:

1. Joints are rigid i.e., no slip takes place in the joint.
2. As load transfer is mainly by friction, the bolts are not subjected to shearing and bearing stresses.
3. High static strength due to high frictional resistance.
4. High fatigue strength since nuts are prevented from loosening and stress concentrations avoided due to friction grip.
5. Smaller number of bolts result into smaller sizes of gusset plates.

Disadvantages of HSFG Bolts:

The following are the disadvantages of HSFG bolts over bearing type bolts:

1. Material cost is high.
2. The special attention is to be given to workmanship especially to give them right amount of tension.

ADVANTAGES AND DISADVANTAGES OF BOLTED CONNECTIONS:

The following are the **advantages** of bolted connections over riveted or welded connections:

1. Making joints is noiseless.
2. Do not need skilled labour.
3. Needs less labour.
4. Connections can be made quickly.
5. Structure can be put to use immediately.
6. Accommodates minor discrepancies in dimensions.
7. Alterations, if any, can be done easily.
8. Working area required in the field is less.

The **disadvantages** of unfinished (black) bolt connections are listed here.

1. Tensile strength is reduced considerably due to stress concentrations and reduction of area at the root of the threads.
2. Rigidity of joints is reduced due to loose fit, resulting into excessive deflections.
3. Due to vibrations nuts are likely to loosen, endangering the safety of the structures.

However it may be noted that most of these disadvantages are overcome by using HSFG bolts.

2.4 TERMINOLOGY:

The following terms used in the bolted connections are defined below:

1**. Pitch** of the bolts (p): It is the centre to centre spacing of the bolts in a row, measured along the direction of load. It is shown as ‘p’ in figure below.

2. **Gauge** Distance (g): It is the distance between the two consecutive bolts of adjacent rows and is measured at right angles to the direction of load.

3. **Edge** Distance (e): It is the distance of centre of bolt hole from the adjacent edge of plate (Ref. Fig. 3.3).

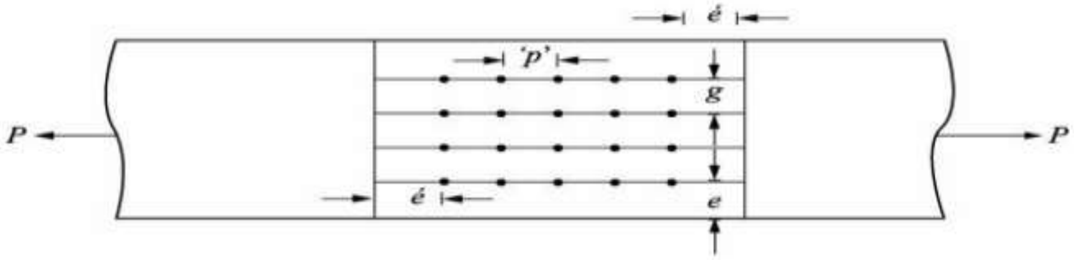
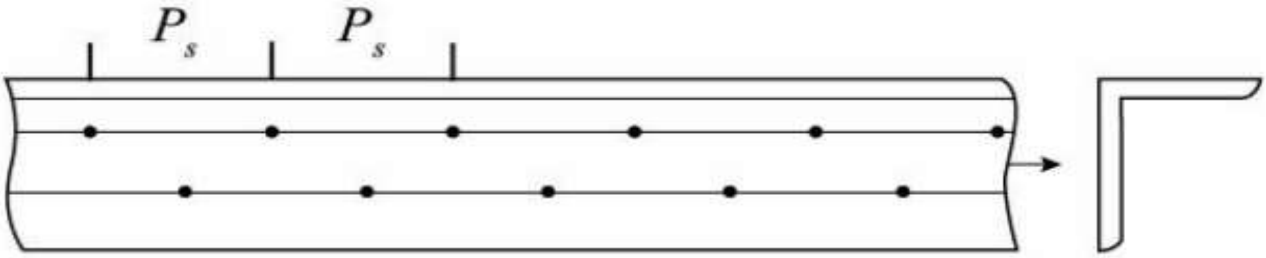


Figure of Pitch, gauge distance and edge distance.

4. **End** Distance (e′): It is the distance of the nearest bolt hole from the end of the plate

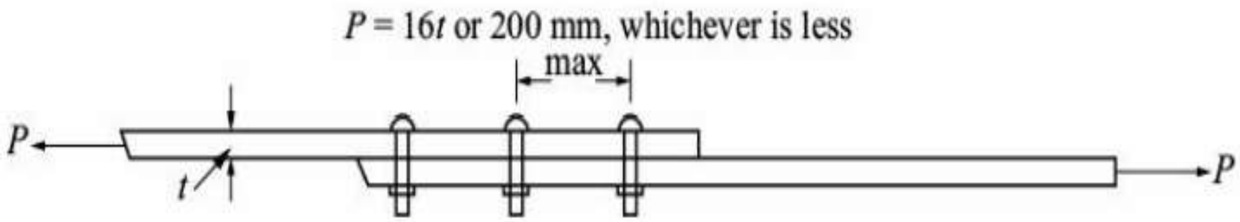
5. **Staggered** Distance: It is the centre to centre distance of staggered bolts measured obliquely on the member as shown in Fig. below.



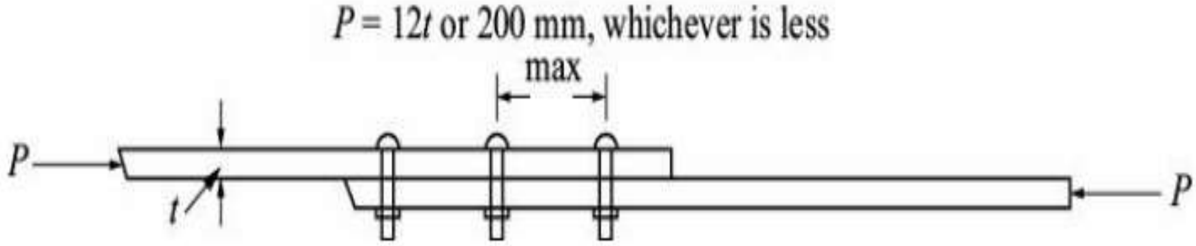
IS 800-2007 **SPECIFICATIONS FOR SPACING AND EDGE DISTANCES OFBOLT HOLES**:

1. Pitch 'p'shall not be less than 2.5d, where 'd ' is the nominal diameter of bolt.
2. Pitch 'p'shall not be more than

   (a) 16t or 200 mm, whichever is less, in case of tension members as in figure below.

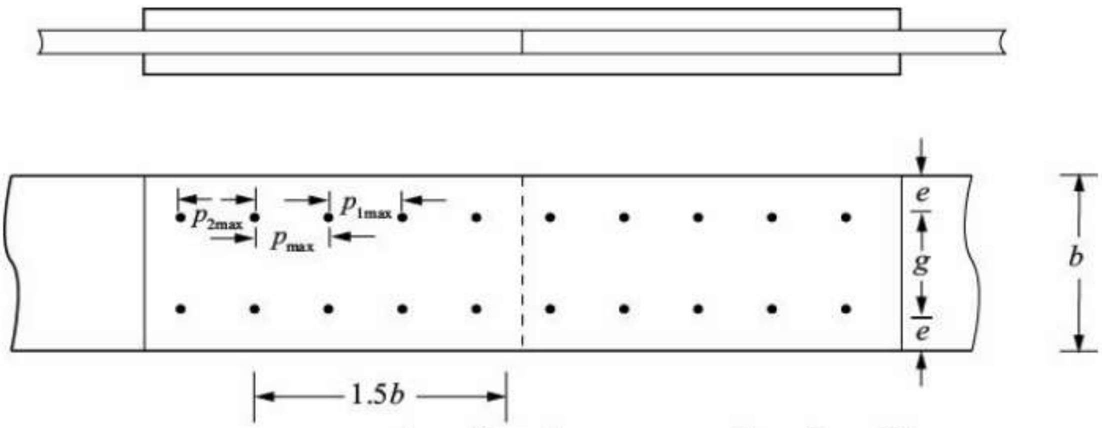


(b) 12t or 200 mm, whichever is less, in case of compression members where t is the thickness of thinnest member as below.



(c) In case of staggered pitch, pitch may be increased by 50 percent of values specified above provided gauge distance is less than 75 mm.

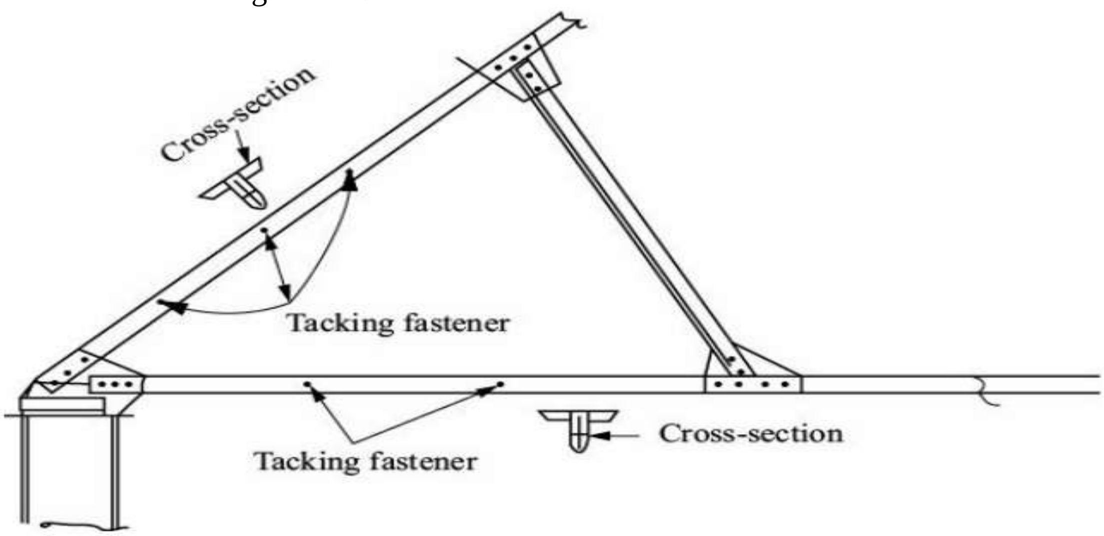
3. In case of butt joints maximum pitch is to be restricted to 4.5d for a distance of 1.5 times the width of plate from the butting surface as below.



p_{1max} as per clause 3c and $p_{2\,max}$ as per clause 2a or 2b.

4. The gauge length 'g' should not be more than 100 + 4t or 200 mm whichever is less.
5. Minimum edge distance shall not be
   (a) Less than 1.7 × hole diameter in case of sheared or hand flame cut edges
   (b) Less than 1.5 × hole diameter in case of rolled, machine flame cut, sawn and planed edges.
6. Maximum edge distanc (e) should not exceed
   (a) 12t ∈, where and t is the thickness of thinner outer plate
   (b) 40 + 4t, where t is the thickness of thinner connected plate, if exposed to corrosive influences.
7. Apart from the required bolt from the consideration of design forces, additional bolts called tacking fasteners should be provided as specified below. If value of gauge length exceeds after providing design fasteners at maximum edge distances tacking rivets should be provided
   (i) At 32 t or 300 mm, whichever is less, if plates are not exposed to weather
   (ii) At 16 t or 200 mm, whichever is less, if plates are exposed to weather.
8. In case of a member made up of two flats, or angles or tees or channels, tacking rivets are to be provided along the length to connect its components as specified below:
   (a) Not exceeding 1000 mm, if it is tension member
   (b) Not exceeding 600 mm, if it is compression member

This situation is shown in Fig. below.



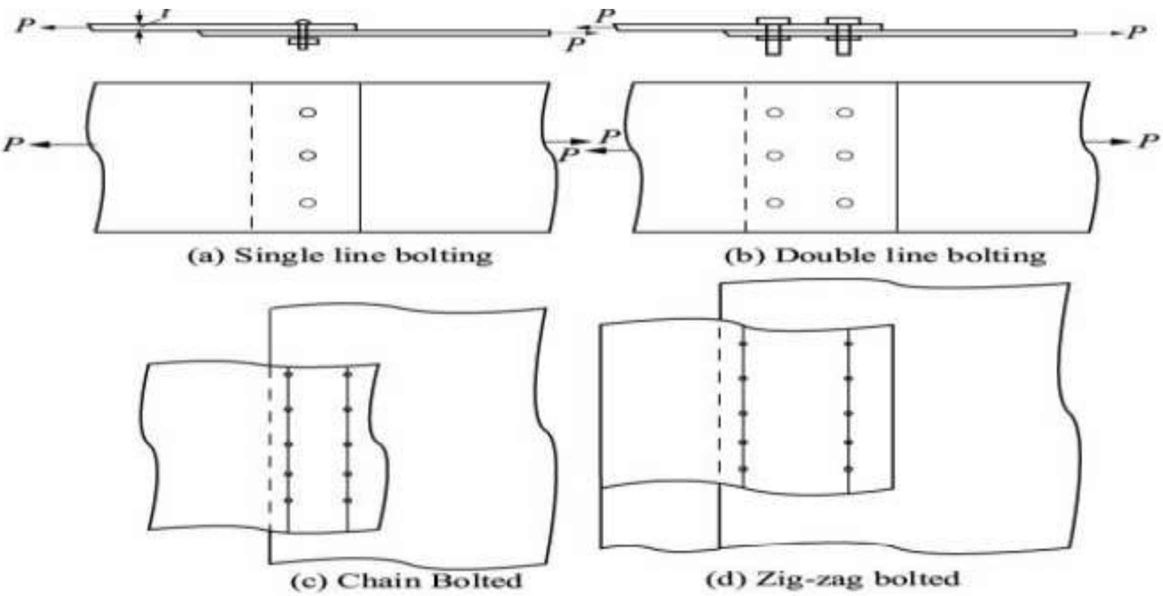
Module – I Chapter – 2
Lecture – 8

TYPES OF BOLTED CONNECTIONS

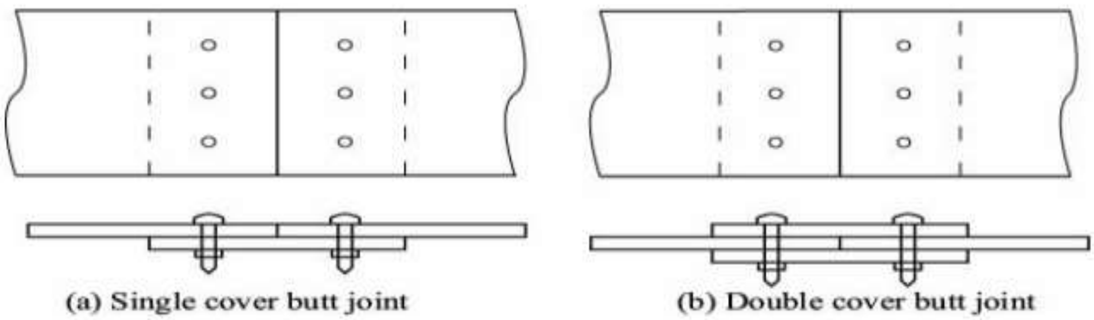
Learning Objectives
2.5 *To know different types of Bolted Connections*
2.6 Learn Types of Actions on Fasteners
2.7 Design of Bearing Bolts

2.5 TYPES OF BOLTED CONNECTIONS:

Types of joints may be grouped into the following two:
(a) Lap joint
(b) Butt joint
(a) Lap Joint:
It is the simplest type of joints. In this the plates to be connected overlap one another. Figure below shows a typical lap joints



Butt Joint:
In this type of connections, the two main plates abut against each other and the connection is made by providing a single cover plate connected to main plate or by double cover plates, one on either side connected to the main plates (Ref. below fig)



2.6 TYPES OF ACTIONS ON FASTENERS:

Depending upon the types of connections and loads, bolts are subjected to the following types of actions:

(a) Only one plane subjected to shear (single shear)

(b) Two planes subjected to shear (double shear)

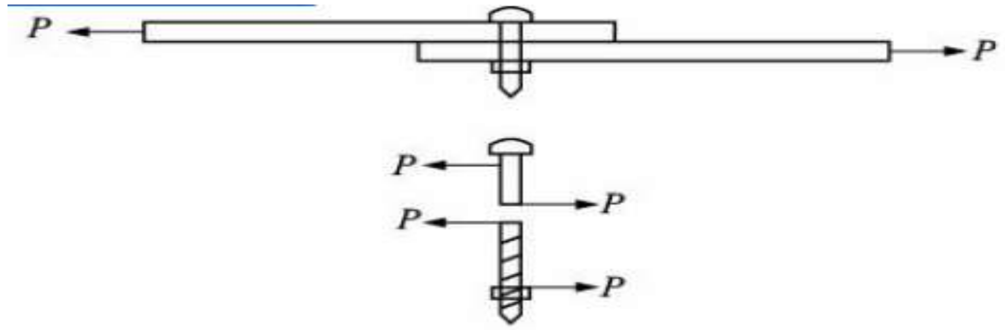
(c) Pure tension

(d) Pure moment

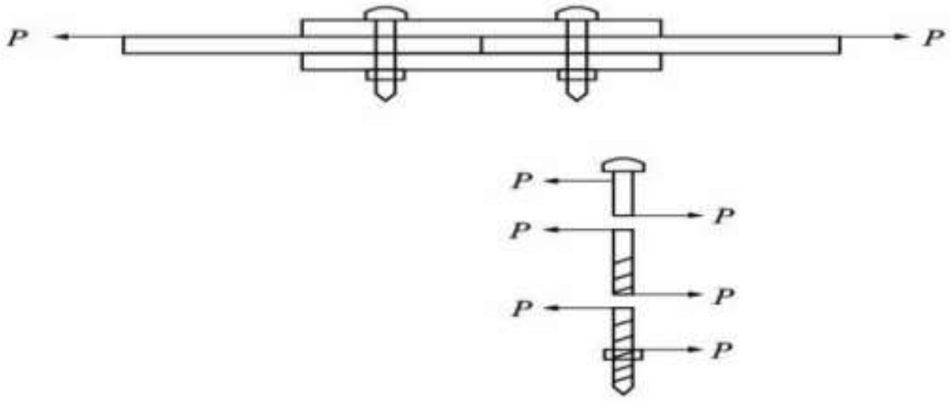
(e) Shear and moments in the plane of connection

(f) Shear and tension

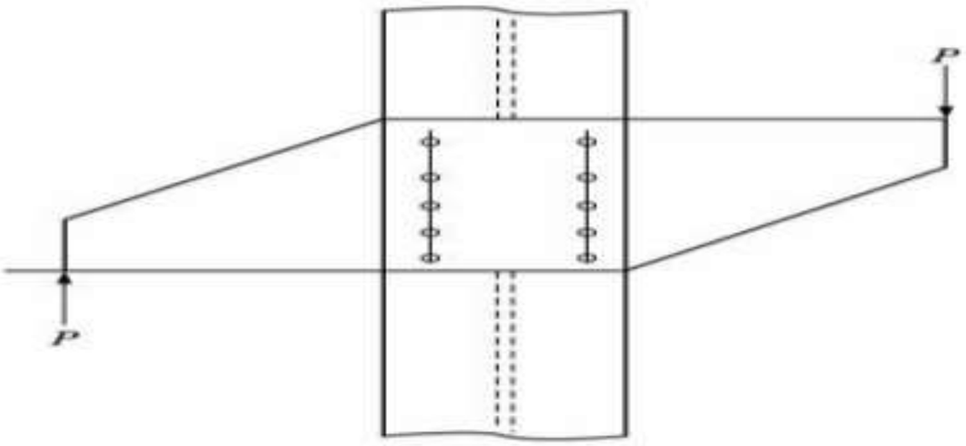
These cases are shown below.



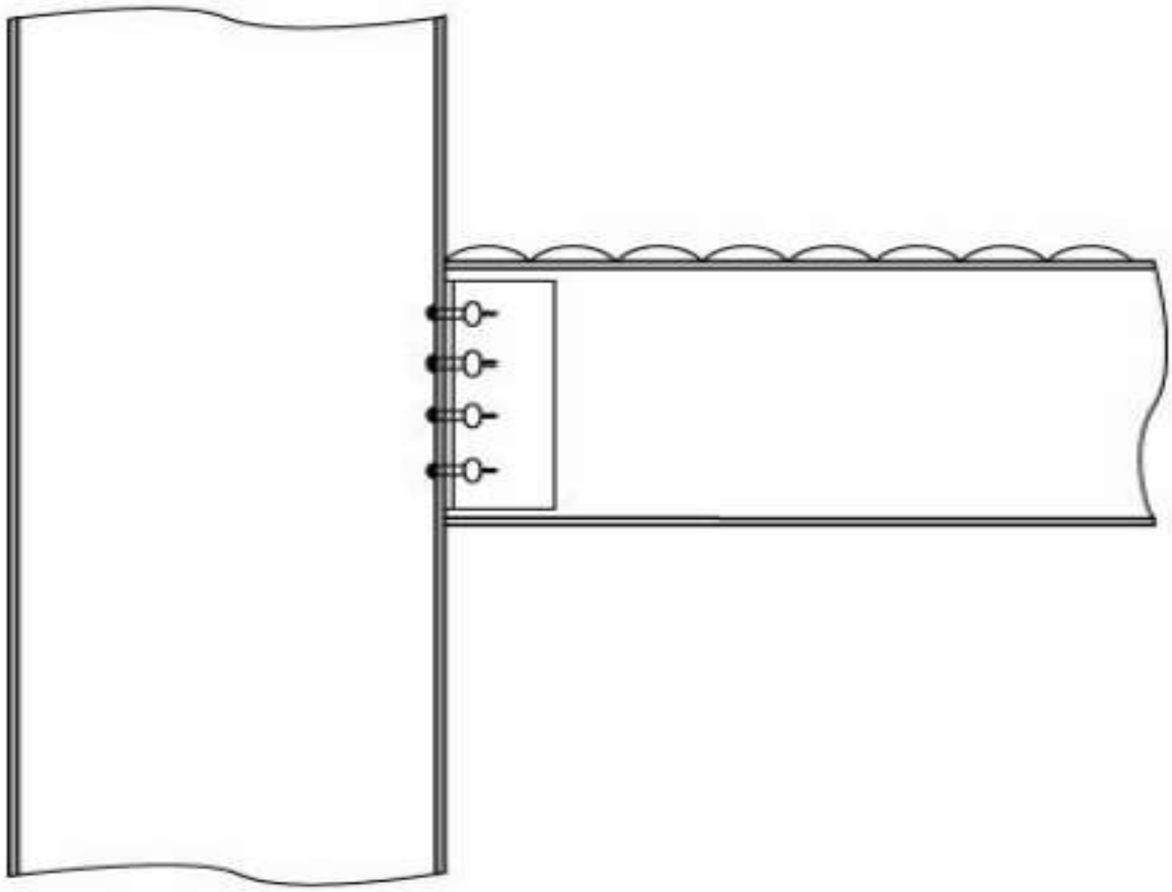
(a) Shear plane on thread



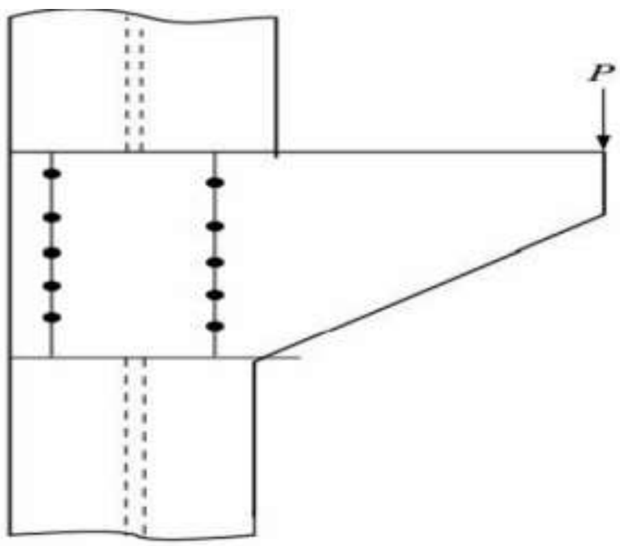
(b) Two planes subject to shear



(d) Bolts resisting pure moment



(e) Bolts subject to shear and tension



(f) Joint subject to shear and moment in its plane

DESIGN OF BEARING BOLTS

Learning Objectives
2.7 Design of Bearing Bolts
2.8 Design of Tensile Strength of Plates

2.7 <u>ASSUMPTIONS IN DESIGN OF BEARING BOLTS:</u>

The following assumptions are made in the design of bearing (finished or unfinished) bolted connections:
1. The friction between the plates is negligible
2. The shear is uniform over the cross-section of the bolt
3. The distribution of stress on the plates between the bolt holes is uniform
4. Bolts in a group subjected to direct loads share the load equally
5. Bending stresses developed in the bolts is neglected.

Assumption 1 is not correct because friction exists between the plates as they are held tightly by bolts. But this assumption results on safer side in the design.

Actual stress distribution in the plate is not uniform in working conditions. Stresses are very high near bolt holes. But with increase in load the fibres near the hole start yielding and hence stresses at other parts start increasing. At failure, the stress distribution is uniform and the ultimate load carrying capacity is given by the net area times the yield stress.

The fourth assumption is questionable. The bolts far away from centre of gravity of bolt groups are subject to more loads. In the ultimate stage all rivets have to fail, till then redistribution of load will be taking place. Hence the assumption is not completely wrong.

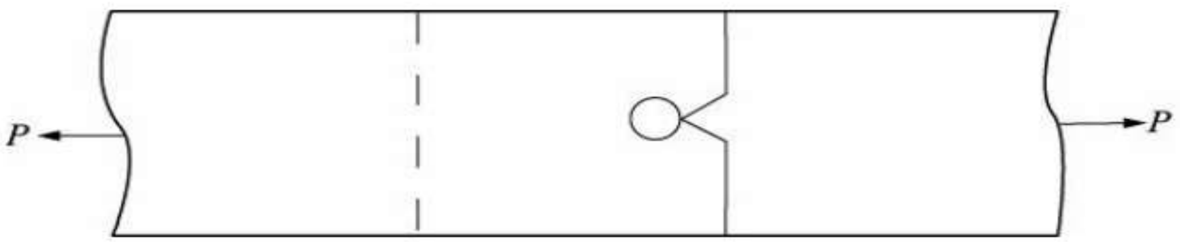
IS 800-2007 permits this assumption for short joints (distance between first and the last bolt in the direction of load being less than ($5 \times d$). For long a reduction factor has been recommended for finding the strength of joint.

2.7.1 PRINCIPLES OBSERVED IN THE DESIGN:

The following principles are observed in the design of connections:
1. The centre of gravity of bolts should coincide with the centre of gravity of the connected members.
2. The length of connection should be kept as small as possible.

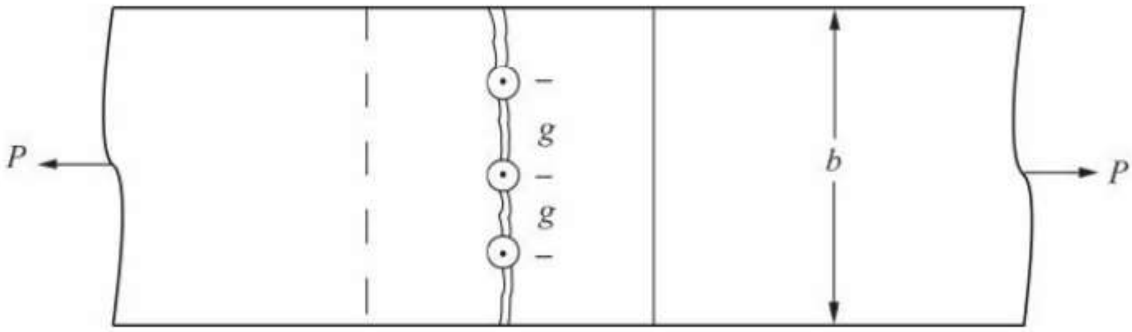
2.8 DESIGN TENSILE STRENGTH OF PLATES IN A JOINT:

Plates in a joint made with bearing bolts may fail under tensile force due to any one of the following: Bursting or Shearing of the edge (Fig. 3.12).



1. Crushing of Plates.

2. Rupture of Plates.



3.Critical section.

The bursting or shearing and crushing failures are avoided if the minimum edge/end distances as per IS 800-2007 recommendations are provided.

If the minimum distances are ensured in a joint, the design tensile strength of plate in the joint is the strength of the thinnest member This strength is given by

$$T_{dn} = \frac{0.9\, A_n\, f_u}{\gamma_{ml}}$$

Where

γ ml = partial safety factor for failure at ultimate stress = 1.25

fu = ultimate stress of the material

An = net effective area of the plate at critical section,

which is given by

$$A_n = \left[b - n\, d_0 + \sum \frac{p_{si}^2}{4 g_i} \right] t$$

where b = width of plate

t = thickness of thinner plate in joint

d_0 = diameter of the bolt hole (2 mm in addition to the diameter of the hole, in case of directly punched holes)
g = gauge lengths between the bolt holes (Ref. Fig. 3)
p_{si} = staggered pitch length between lines of bolt holes
n = number of bolt holes in the critical section
i = subscript for summation of all inclined legs
It may be noted that, if there is no staggering, $p_{si} = 0$
and hence, $A_n = (b - nd_0)t$,
which is the critical section shown in Fig. 3 above.

DESIGN STRENGTH OF BEARING BOLTS

Learning Objectives

2.9 Design Strength of Bearing Bolts

2.10 Reduction factor and shear capacity of Bolts

2.9 DESIGN STRENGTH OF BEARING BOLTS:

The design strength of bearing bolts under shear is the least of the following:

(a) Shear capacity (strength)

(b) Bearing capacity (strength)

(a) Shear Capacity (Strength) of Bearing Bolts in a Joint

Design strength of the bolt V_{dsb} is

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$$

where V_{nsb} , nominal shear capacity of bolt and γ_{mb} = partial safety factor of material of bolt.

In the above expression V_{nsb} is given by

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}} \left(n_n A_{nb} + n_s A_{sb} \right)$$

where, f_{ub} = ultimate tensile strength of the bolt

n_n = number of shear planes with threads intercepting the shear plane

n_s = number of shear planes without threads intercepting the shear plane

A_{sb} = nominal shank area of the bolt, and

A_{nb} = net shear area of the bolt at threads, may be taken as the area corresponding to root diameter of the thread.

$$A_{nb} = \frac{\pi}{4}(d - 0.9382p)^2$$

where p is pitch of thread

$$\approx 0.78 \frac{\pi}{4} d^2$$

for ISO threads.

For example (a), per bolt, n_n = 1 and ns = 0 and in

(b), per bolt, n_n = 1 and ns = 1.

2.1.1 Reduction Factors for Shear Capacity of Bolts

The code suggests the use of reduction factors for shear capacity in the following situations:

If the joint is too long

If the grip length is large

(iii) If the packing plates of thickness more than 6 mm are used.

2.1.2 Reduction Factor for Long Joints (βij)

If the distance between the first and the last bolt in the joint (l j) measured in the direction of load exceeds 15_d,

the shear capacity V_{db} shall be reduced by the factor β_{lj} given by

$$\beta_{lj} = 1.075 - 0.005 \frac{l_j}{d}$$

subjected to the limits $0.75 \leq \beta_{lj} \leq 1.0$, where d is nominal diameter of bolt.

2.1.3 Reduction Factor if Grip Length is Large (βlg)

If the total thickness of the connected plates exceed 5 times the diameter d of bolts, the design shear capacity V_{db} , shall be reduced by

$$\beta_{lg} = \frac{8d}{3d + l_g}$$

subject to conditions maximum value = β_{lj} ,
where l_g = grip length = total thickness of the connected plates.
In no case l_g be greater than 8_d.

2.1.4 Reduction Factor if Packing Plates are Used (βpk)

If packing plates of thickness more than 6 mm are used in the joint, then shear capacity is to be reduced by a factor

$\beta_{pk} = 1 - 0.0125\ t_{pk}$

where t_{pk} = thickness of the thicker packing in mm.

Thus bearing capacity of the bolts in shear is

$$\frac{f_u}{\sqrt{3}}\left(n_n\, A_{nb} + n_s\, A_{sb}\right)\beta_{lj}\, \beta_{lg}\, \beta_{pk}$$

2.1.5 Bearing Capacity of Bolts (V_{dpb})

IS 800-2007 suggests the following procedure to find bearing strength of bolts:

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$$

where V_{dpb} = design bearing strength
V_{npb} = nominal bearing strength and
γ_{mb} = partial safety factor of material = 1.25
(Nominal bearing strength may be found from the following relation:
$V_{npb} = 2.5\ K_b$ dt fu where Kb is smaller of

$$\frac{e}{3d_0},\ \frac{p}{3d_0} - 0.25,\ \frac{f_{ub}}{f_u},\ 1.0$$

in which e, p = end and pitch distances.
d_0 = diameter of hole.
f_{ub} , f_u = ultimate tensile stress of the bolt and plate.
d = nominal diameter of the bolt.
t = summation of the thickness of the connected plates experiencing bearing stress in the same direction. If bolts are counter sunk, it is to be reduced by the half depth of counter sinking.

DESIGN PROCEDURE

Learning Objectives

2.11 Design Procedure of bearing type bolts

2.11.1 DESIGN PROCEDURE WITH BEARING TYPE BOLTS SUBJECT TO SHEARING FORCES:

Determine the design (factored) action acting on the joint.
Then select connection with suitable diameter of the bolts.
Determine the strength of connection and ensure that design strength is not less than the design action.
The following information is useful in the design of joint:

1. **Diameter of bolt hole**:

Nominal size of bolts (d) in mm	12 14 16 20 22 24 30 36
Diameter of bolt hole (d0) in mm	13 15 18 22 24 26 33 39
Outer diameter of washers in mm	− − 30 37 − 44 56 60

2. **Area of bolt at root (Anb)**:

Anb ≈ 0.78 Asb
where Asb = area of bolt at shank

$$= \frac{\pi}{4} d^2$$

3. **Properties of materials of bolts:**

Commonly used bolts have the following material properties (IS 1367):

Grade 4.6 f_{yb} = 240 MPa
f_{ub} = 400 MPa
Grade 4.8 f_{yb} = 320 MPa
f_{ub} = 420 MPa
Grade 5.6 f_{yb} = 300 MPa
f_{ub} = 500 MPa
Grade 5.8 f_{yb} = 400 MPa
f_{ub} = 520 MPa

4. **Properties of rolled steel sections:**

These values have been shown in Appendix

2.11.2 EFFICIENCYOFA JOINT

It is defined as the ratio of strength of joint and strength of solid plate in tension. It is usually expressed in percentage.
Thus,

$$\text{efficiency } \eta = \frac{\text{Strength of joint}}{\text{Strength of solid plate}} \times 100$$

Strength of solid plate is less in yielding compared to tearing of solid plate.
For example, consider Fe 415 plates.
f_y = 250 N/mm2
f_u = 410 N/mm2
γ_{mo} =1.1
γ_{ml} =1.25
∴ Design strength of solid plate per unit area

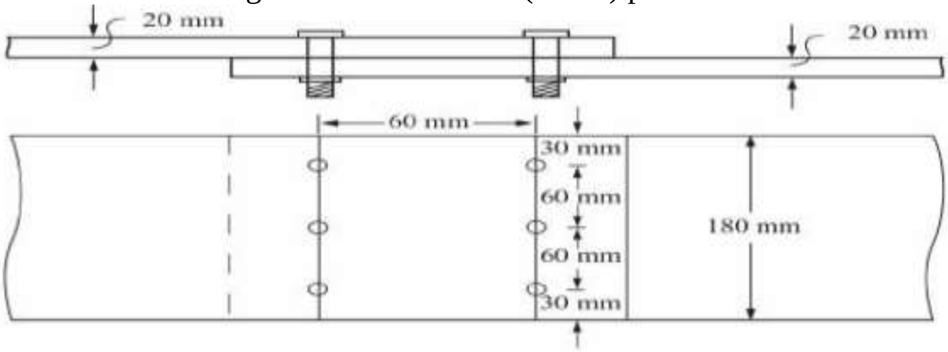
in yielding is

$$\frac{250}{1.1} \times 1 = 227.27 \text{ N/mm}^2$$

(b) in rupture is

$$\frac{0.9 \times 410}{1.25} \times 1 = 295.2 \text{ N/mm}^2$$

Hence strength of solid plate is governed by its strength in yielding.
Strength of joint is the smaller of strength in shear and strength in bearing.
Problem 1 .Find the efficiency of the lap joint shown in Fig. below.
Given: M20 bolts of grade 4.6 and Fe 410 (E 250) plates are used.



Solution:

For M20 bolts of grade 4.6,
diameter of bolt, $d = 20$ mm
diameter of bolt hole, $d_0 = 22$ mm
Ultimate strength $f_{ub} = 400$ MPa
Partial safety factor, $\gamma_{mb} = 1.25$
For Fe 410 (E 250) plates,
Ultimate stress, $f_u = 410$ MPa

Partial safety factor, $\gamma_{ml} = 1.25$

Strength of plates in the joint:
Thickness of thinner plate, $t = 20$ mm

Width of plate $b = 180$ mm

There is no staggering $p_{si} = 0$

Number of bolt holes in the weakest section = 3

∴ Net area at weakest section

$A_u = [b - nd_0 + 0]\, t$

$= [180 - 3 \times 22] \times 20 = 2280 \text{ mm}^2$

Design strength of plates in the joint

$$T_{dn} = \frac{0.9 f_u A_n}{ml} = \frac{0.9 \times 410 \times 2280}{1.25}$$

$= 673056$ N $= 673.056$ kN.

Strength of Bolts:

Total number of bolts = 6

(i) *Design Strength in Shear:*

Number of shear planes at thread $n_n = 1$ per bolt.

Number of shear planes at shank $n_s = 0$ per bolt.

∴ Total $n_n = 1 \times 6 = 6$ and total $n_s = 0$.

$$A_{nb} = 0.78 \times \frac{\pi}{4} d^2 = 0.78 \times \frac{\pi}{4} \times 20^2 = 245 \text{ mm}^2$$

There are no reduction factors i.e. $\beta_{lj} = \beta_{lg} = \beta_{pk} = 1$

∴ Nominal shear strength,

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})$$
$$= \frac{400}{\sqrt{3}} (6 \times 245 + 0) = 339482 \text{ N} = 339.482 \text{ kN}$$

∴ Design strength in shear,

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{339.482}{1.25} = 271.586 \text{ kN}$$

(ii) *Design Strength in Bearing:*

Nominal strength

$$V_{npb} = 2.5\, K_b dt\, f_u$$

where K_b = least of the following:

(a) $\frac{e}{3d_0} = \frac{30}{3 \times 22} = 0.4545$

(c) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.9756$

(d) 1.0.

Note: Edge distance provided is less. Hence it is critical in this case.

$K_b = 0.4545$

∴ $V_{npb} = 2.5 \times 0.4545 \times 20 \times 20 \times 410 = 186345$ N per bolt

Design strength = $\frac{V_{npb}}{\gamma_{mb}} = \frac{186345}{1.25} = 149076$ N

∴ Design strength of joint = 6 × 149076 = 894456.8 N

$= 894.456$ kN

$\therefore$ Design strength of bolts in joint = 271.586 kN < T_{dn}.

$\therefore$ Strength of joint = 271.586 kN.

Efficiency of Joint:

Area of solid plate = $180 \times 20 = 3600$ mm^2.

$\therefore$ Design strength of solid plate

$$= \frac{f_y}{\gamma_{mb}} \times A_g = \frac{250}{1.1} \times 3600 = 818181.8\text{ N} = 818.182\text{ kN}$$

$\therefore$ Efficiency of the joint $= \frac{\text{Strength of joint}}{\text{Strength of solid plate}} \times 100$ percent

$$= \frac{271.586}{818.182} \times 100 = 33.19\%$$

NUMERICALS

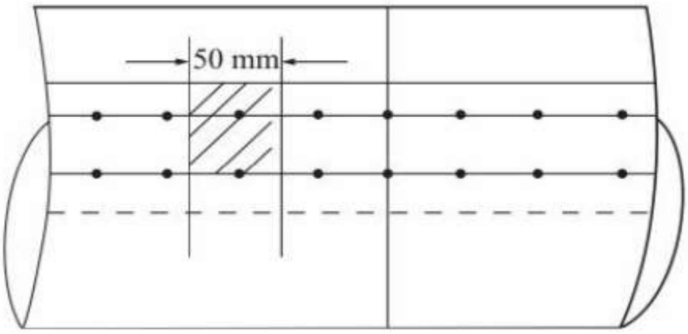
Learning Objectives

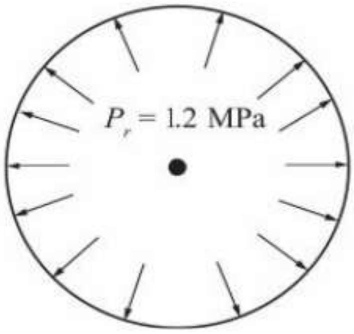
2.12 Numericals on double bolted lap joints.

Problem 2

A boiler shell is made up of 14 mm thick Fe:415 plates. If the joint is double bolted lap joint with M16 bolts of grade 4.6 at distances of 50 mm, determine the design strength of the joint per pitch width. Is it a safe design if the internal diameter of bolt is 1 m and steam pressure is 1.2 MPa?

Solution:





Strength of plate per 50 mm width:

Diameter of bolts $d = 16$ mm.

$\therefore$ Diameter of bolt hole $d_0 = 18$ mm.

Strength of plate per 50 mm width:

$t = 14$ mm, $b = p = 50$ mm, $f_u = 410$ MPa

No. of bolts in double bolted joint per 50 mm width $n = 1$

$\therefore A_n = (50 - 1 \times 18) \times 14 = 448$ mm^2

$\therefore$ Design strength of plate per 50 mm width.

$$T_{dn} = \frac{0.9 \times 410 \times 448}{1.25} = 132250 \text{ N} = 132.250 \text{ kN}$$

Strength of bolts per 50 mm width:

Since it is lap joint, shear planes at shanks = 0. As there are two bolts per pitch width considered, $n_n = 2$

Area of bolt at roots = $0.78 \times \frac{\pi}{4}(16)^2$

= 156.83 mm^2

$\therefore$ Ultimate strength $V_{nsb} = \frac{400}{\sqrt{3}}(0 + 2 \times 156.83)$

= 72436 N

$\therefore$ Design strength, $V_{dsb} = \frac{V_{nsb}}{mb} = \frac{72436}{1.25} = 57949 \text{ N} = 57.949 \text{ kN}$

Design strength in bearing:

Ultimate strength in bearing per bolt:

K_b is least of the following:

(a) $\frac{e}{3d_0}$, since 'e' is not given, assume that sufficient edge distance is provided and hence it will not decide K_b.

(c) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.9756$

(d) 1.0.

$\therefore K_b = 0.6759.$

Ultimate bearing strength of each bolt

$= 2.5\ K_b\ dt\ f_u$

$= 2.5 \times 0.6759 \times 16 \times 14 \times 410 = 155187$ N $= 155.187$ kN

As there are two bolts, design strength of bolts in bearing $= 2 \times 155.187 = 310.374$ kN $> V_{dsb}$

$\therefore$ Design strength of bolts = 57.949 kN.

Strength of joint per 50 mm width is lesser of design strength of bolts (57.949 kN) and strength of plate (132.5 kN).

$\therefore$ Design strength of joint = 57.949 kN per 50 mm width.

To check the safety of joint:

Action of applied force is a hoop strers $= \frac{P_r D}{2t}$

where P_r is applied pressure and D is diameter of boiler.

$$= \frac{1.2 \times 1000}{2 \times 14} = 42.857 \text{ N/mm}^2$$

$\therefore$ Force per 50 mm length $= 42.857 \times 14 \times 50$

$= 30{,}000$ N

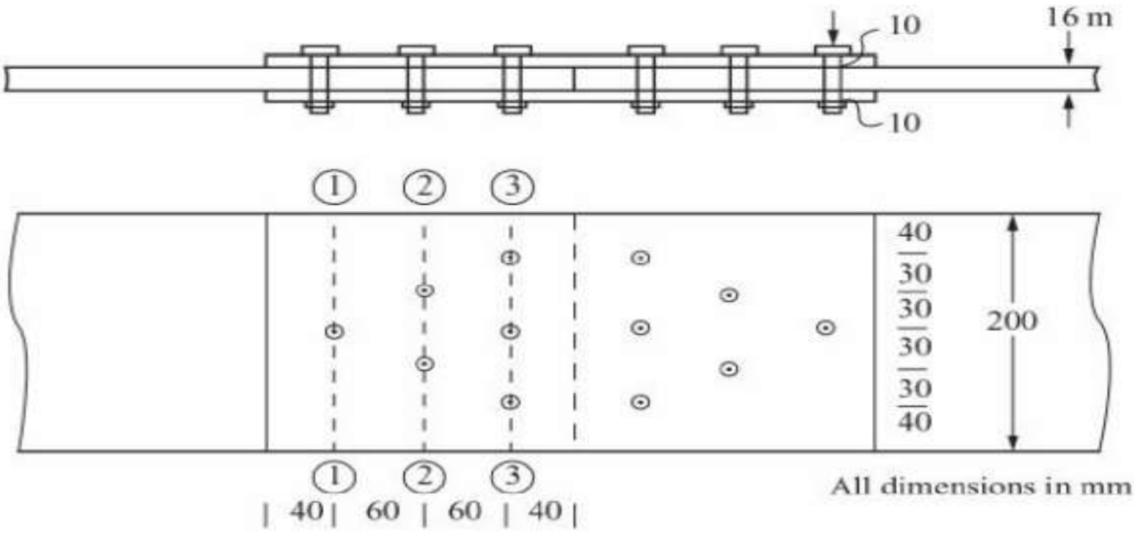
$= 30$ kN

$\therefore$ Factored design action $= 1.5 \times 30 = 45$ kN

Design strength = 57.949 kN > Design action.

Hence the design is safe. **Answer**

Problem 3
Find the maximum force which can be transferred through the double covered butt joint shown in Fig. below. Find the efficiency of the joint also. Given M20 bolts of grade 4.6 and Fe 410 steel plates are used.



Solution:

For M20 bolts of Grade 4.16,
$d = 20$ mm $d_0 = 22$ mm $f_{ub} = 400$ N/mm^2.
For grade Fe 410 plates, $f_u = 410$ N/mm^2.
∴ Nominal strength of one bolt in shear (double shear)

$$= \frac{f_{ub}}{\sqrt{3}}\left(1 \times \frac{\pi}{4} d^2 + 0.78 \times \frac{\pi}{4} d^2\right)$$
$$= \frac{400}{\sqrt{3}}(1.78) \times \frac{\pi}{4} 20^2$$

= 129143 N
∴ Design strength of one bolt in double shear

$$= \frac{129143}{1.25} = 103314 \text{ N}$$

Design strength of joint in double shear
= 6 × 103314 = 619886 N = 619.886 kN
Strength of bolts in bearing:
K_b is the least of the following:

$\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0$

∴ For bolts at section (3) − (3), it is least of

$\frac{40}{3\times22}, \frac{60}{3\times22} - 0.25, \frac{400}{410}, 1.0$

i.e., $K_b = 0.6061$

∴ Nominal strength of six bolts in bearing

$= 6 \times 2.5 \times 0.6061 \times 20 \times 16 \times 410$

$= 1192805$ N

∴ Design strength in bearing $= \frac{1192805}{1.25}$

= 954244 N
= 954.244 kN

∴ Strength of bolts in the joint = 619.886 kN and strength of each bolt = 103314 N

Strength of plate:

It is to be checked along all the three sections.

Now, t = 16 mm (least of the thicknesses of cover plates and main plate)

$f_u = 410$ N/mm^2

(a) At section (1) − (1)

$$T_{dn_1} = \frac{0.9\, f_u\, A_n}{1.25} = \frac{0.9 \times 410\,(200-22) \times 16}{1.25}$$

= 840133 N

(b) At section (2) − (2)

When this section fails, bolt in section (1) − (1) also has to fail. In other words bold at section (1) – (1) has already transferred a force of 103314 N to cover plates. Hence strength of plate at section (2) − (2)

$$T_{dn_2} = \frac{0.9 \times 410\,(200 - 2\times22) \times 16}{1.25} + 103314$$

= 840133 N

At section (3) − (3)

T_{dn3} = Plate strength + strength of 3 bolts

$$= \frac{0.9 \times 410\,(200 - 3 \times 22) \times 16}{1.25} + 3 \times 103314$$

$= 942851$ N

$\therefore$ Strength of plate in the joint = 840133 N

= 840.133 kN

$\therefore$ Strength of joint = 619.886 kN

$\therefore$ Maximum design force that can be transferred safely = 619.886 kN.

$\therefore$ Permissible force at working condition = $\frac{619.886}{1.5}$ = 413.257 kN **Answer**

Design strength of solid plate = $\frac{250 \times 200 \times 16}{1.1}$ = 727272 N

$\therefore$ Efficiency of the joint = $\frac{619.886}{727.272} \times 100 = 85.23\%$ **Answer**

TENSION CAPACITYOFBOLTS:

According to IS 800-2007, clause 10.3.5, nominal tension capacity of bolt Tnb is given by,

$$T_{nb} = 0.9\, f_{ub}\, A_n \leq f_{yb}\, A_{sb}\, \frac{\gamma_{mb}}{\gamma_{mo}}$$

and design capacity Tdb is given by

$$T_{db} = \frac{T_{nb}}{\gamma_{mb}}$$

$$\therefore \quad T_{db} = \frac{0.9\, f_{ub}\, A_n}{\gamma_{mb}} \leq \frac{f_{yb}\, A_{sb}}{\gamma_{mo}}$$

where fub = ultimate tensile stress of bolt
fyb = yield stress of the bolt
An = net area of the rest of bolt and
Asb = shank area of the bolt.
For ordinary (bearing-bolt) bolt of grade 4.6,
fub = 400 N/mm2
fyb = 240 N/mm2

$$A_n = 0.78\, \frac{\pi}{4} d^2 \qquad A_{sb} = \frac{\pi}{4} d^2$$

gmb = 1.25 gmo = 1.1

Hence,

$$T_{db} = \frac{0.9 \times 400 \times 0.78 \left(\frac{\pi}{4}\right) d^2}{1.25} \leq \frac{240 \times \left(\frac{\pi}{4}\right) d^2}{1.1}$$

= 176.432 d 2 ≤ 171.360 d 2

∴ Tdb = 171.360 d 2

Thus yield stress criteria governs the tension capacity i.e.

$$T_{db} = \frac{240 \times \frac{\pi}{4} d^2}{1.1}$$

If Tb is factored tensile force, the design criteria is
Tb ≤ Tdb .

Module – I Chapter – 2

Lecture – 13

COMBINED SHEAR AND TENSION

Learning Objectives

2.13 Application of combined forces (Shear and Tension)

2.14 *Application of Shear Capacity HSFG Bolts*

2.13.1 DESIGN CRITERIA FOR BOLT SUBJECTED TO COMBINED SHEAR AND TENSION:

According to IS 800-2007, clause 10.3.6, a bolt required to resist both design shear force Vsb and design tensile force Tb at the same time shall satisfy

$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1.0$$

where V_{db} − design shear strength and T_{db} − design tensile capacity of bolts.

2.14.1 SHEAR CAPACITY OF HSFG BOLTS:

As stated in IS code, these are the bolts made of high tensile steel which are pretensioned and then provided with nuts. The nuts are clamped also. Hence resistance to shear force is mainly by friction. There are two types of HSFG bolts. They are parallel shank and waisted shank type. Parallel shank type HSFG bolts are designed for no-slip at serviceability loads. Hence they slip at higher loads and slip into bearing at ultimate load. Such bolts should be checked for their bearing strength at ultimate load. Waisted shank HSFG bolts are designed for no slip even at ultimate load and hence there is no need to check for their bearing strength. IS 800-2007 (clause 10.4) recommends use of the following expression for finding nominal shear capacity of HSFG (parallel shank or waisted shank) bolts:

V_{nsf} = mf ne K_h F_0

where, mf = coefficient of friction (called slip factor) as specified in Table 20 of IS 800-2007.

ne = number of effective interfaces offering frictional resistance to the slip.

[Note: ne = 1 for lap joints and 2 for double cover butt joints]

K_h = 1.0 for fasteners in clearance holes

= 0.85 for fasteners in oversized and short slotted holes

and for long slotted holes loaded perpendicular to the slot.

= 0.70 for fasteners in long slotted holes loaded parallel to the slot.

F_0 = Minimum bolt tension at installation and may be taken as A_{nb} f_0

A_{nb} = net area of the bolt at threads $\left[= 0.78 \frac{\pi}{4} d^2 \right]$

F_0 = proof stress = 0.70 f_{ub}

The slip resistance should be taken as

$$V_{sf} = \frac{V_{nsf}}{\gamma_{mf}}$$

where g_{mf} = 1.10,

if the slip resistance is designed at service load (parallel shank HSFG) = 1.25, if the slip resistance is designed at ultimate load (waisted shank HSFG).

It may be noted that the reduction factors for bearing bolts hold good for HSFG bolts also.

For commonly used HSFG bolts (grade 8.8), yield stress fyb = 640 N/mm2 and ultimate stress fub = 800 N/mm2 .

NUMERICALS

Learning Objectives

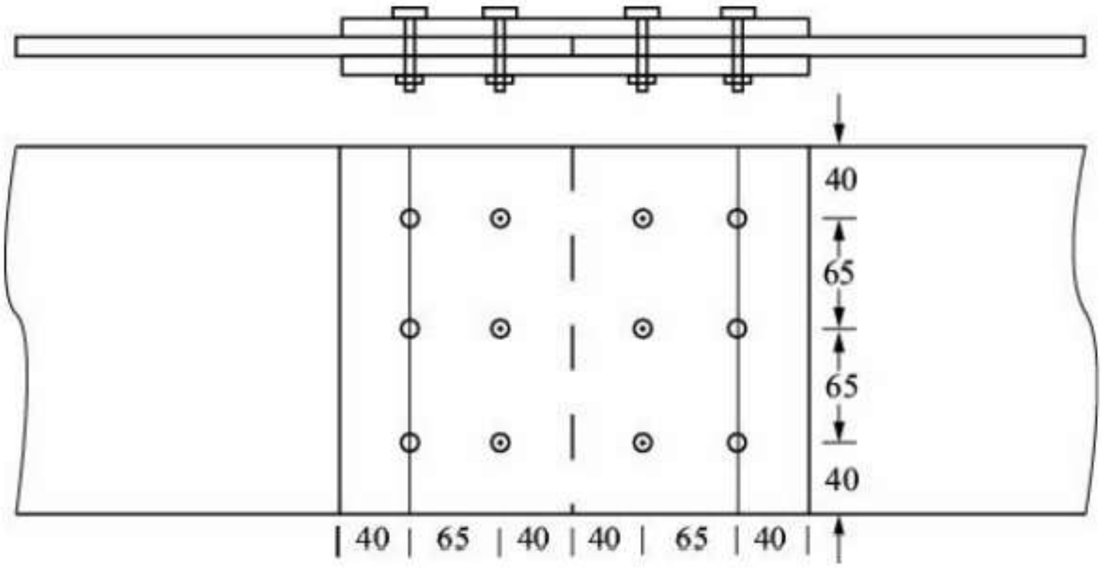
2.15 Numerical on slip resistance.

Problem .3

Determine the shear capacity of bolts used in connecting two plates as shown in Fig below if

(i) Slip resistance is designated at service load

(ii) Slip resistance is designated at ultimate load



Given:

(1) HSFG bolts of grade 8.8 are used.

(2) Fasteners are in clearance holes.

(3) Coefficient of friction = 0.3.

Solution:

For HSFG bolts of grade 8.8,

f_{ub} = 800 N/mm^2

For fasteners in clearance hole K_h = 1.0

Coefficient of friction mf = 0.3 (given)

∴ Nominal shear capacity of a bolt $V_{nsf} = m_f\, n_c\, K_h\, F_0$

where $F_0 = 0.7\, f_{ub}\, A_{nb}$

$$= 0.7 \times 800 \times 0.78 \times \frac{\pi}{4} \times 20^2$$

= 137225 N

n_e = 2, since it is double cover butt joint

∴ V_{nsf} = 0.3 × 2 × 1.0 × 137225 = 82335 N
Design capacity of one bolt, if slip resistance is designated at service load

$$= \frac{82335}{1.1} = 74850 \text{ N}$$

∴ Design capacity of joint = 6 × 74850, since 6 bolts are used
= 449099 N
= 449.099 kN
Design capacity of one bolt, if the slip resistance is designated at ultimate load

$$= \frac{82335}{1.25} = 65868 \text{ N.}$$

∴ Design capacity of joint = 6 × 65868
= 395208 N
= 395.208 kN
In case (i), bearing strength at ultimate load should be checked. If it is low that will be the governing factor.

2.15.1 **TENSION RESISTANCE OF HSFG BOLTS**:

The expression for nominal tension strength of HSFG bolts is also same as that for bearing bolts i.e,

$$T_{nf} = 0.9 f_{ub} A_n \leq f_{yb} A_{sb} \frac{\gamma_{mb}}{\gamma_{mo}}$$

and hence

$$T_{df} = \frac{0.9 f_{ub} A_n}{\gamma_{mb}} \leq \frac{f_{yb} A_{sb}}{\gamma_{mo}}$$

where An = net tensile area as specified in various parts of IS 1367,
it may be taken as the area at the root of the thread

$$\approx 0.78 \frac{d^2}{4}$$

A_{sb} = shank area.
g_{mb} = 1.25,
g_{mo} = 1.1 fub for bolts of grade 8.8 is 800 MPa
and f_{yb} = 640 MPa.

Short types Questions

1. Why riveted connections are not used now a days ?
2. Which diameter of bolt is known as nominal diameter ?
3. What does M16 signify?
4. Which code give the specifications for Unfinished / Black Bolts ?
5. Thread Diameter is more than or less than shank diameter of bolt ?
6. State the type of bolts on the basis of Load transfer.
7. What do you mean by pitch of the bolts?
8. What is gauge distance ?
9. What should be gauge length?
10. What will be the design strength of bearing bolts under shear?

11. What are types of bolted connection?
12. What do you mean by Grade 4.6 bolt?
13. What is the efficiency of the joint?

Answers :

1. Production of weldable quality steel and introduction of high strength friction grip bolts (HSFG) has replaced uses of rivet.
2. Shank diameter
3. M stands for diameter of shank and 16 is the dimensions of diameter in mm
4. IS 1364 (Part-1)
5. Less than shank diameter
6. a) Bearing type b) Friction grip type
7. It is the center to center placing of the bolts in a row measured along the direction of load.
8. It is the distance between two consecutive bolts of adjacent rows and is measured at right angels to the direction of the loads.
9. The gauge length g should not be more than 100 + 4t or 200mm whichever is less
10. The design strength of bearing bolts under shear will be least of the following
    a) Shear capacity b) Bearing capacity
11. The type of bolted connections are – a) Lap joint b) Butt joint
12. f_{yb} = 240MPa , f_{ub} = 400 MPa
13. Efficiency of the joint = $\text{efficiency } \eta = \dfrac{\text{Strength of joint}}{\text{Strength of solid plate}} \times 100$

Medium types of Questions

1. Write short notes on
   (a) Riveted connection. (b) HSFG bolts.
2. Distinguish between (a) Black bolts and turned bolts. (b) Bearing bolts and friction grip bolts.
3. Discuss the advantages and disadvantages of (a) Riveted connection and bolted connection. (b) Bearing bolts and HSFG bolts. (c) Black bolts and turned (finished) bolts.
4. Explain the following terms: (a) Pitch of Bolts (b) Gauge Distance (c) Edge Distance (d) Staggered Distance (e) Tacking Bolts.
5. List the assumptions made in the design of bearing bolts.

Long type question

1. Two plates 16 mm are to be joined using M20 bolts of grade 4.6 in (a) Lap joint. (b) Butt joint using 10 mm cover plates.

Module – I Chapter – 3

Lecture – 15

WELDED CONNECTIONS

Learning Objectives

3.1 Welded Connections

3.2 Different types of welded joints

3.1.1 WELDED CONNECTIONS

Welding consists of joining two pieces of metal by establishing a metallurgical bond between them. The elements to be connected are brought closer and the metal is melted by means of electric arc or oxyacetylene flame along with weld rod which adds metal to the joint. After cooling the bond is established between the two elements.

ADVANTAGES AND DISADVANTAGES OFWELDED CONNECTIONS:

The following are the advantages of welded connections:

1. Due to the absence of gusset plates, connecting angles etc., welded structures are lighter.
2. The absence of making holes for fasteners, makes welding process quicker.
3. Welding is more adaptable than bolting or riveting.
For example, even circular tubes can be easily connected by welding.
4. It is possible to achieve 100 percent efficiency in the joint whereas in bolted connection it can reach a maximum of 70–80 percent.
5. Noise produced in welding process is relatively less.
6. Welded connections have good aesthetic appearance.
7. Welded connection is airtight and watertight. Hence there is less danger of corrosion of steel structures and welded connection is
preferred for making water tanks.
8. Welded joints are rigid.
9. There is no problem of mismatching of holes in welded connections whereas in bolted connections mismatching of bolt holes
creates considerable problem.
10. Alterations in connections can be easily made in the design of welded connections.

The following are the disadvantages of welded connection:

1. Due to uneven heating and cooling, members are likely to distort in the process of welding.
2. There is a greater possibility of brittle fracture in welding.
3. A welded joint fails earlier than a bolted joint, if the structure is under fatigue stresses.
4. The inspection of welded joints is difficult and expensive. It needs non-destructive testing.
5. Highly skilled person is required for welding.
6. Proper welding in field conditions is difficult.
7. Welded joints are over rigid.

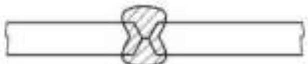
3.2.1 TYPES OF WELDED JOINTS:

There are three types of welded joints:

1. Butt weld
2. Fillet weld
3. Slot weld and Plug weld.

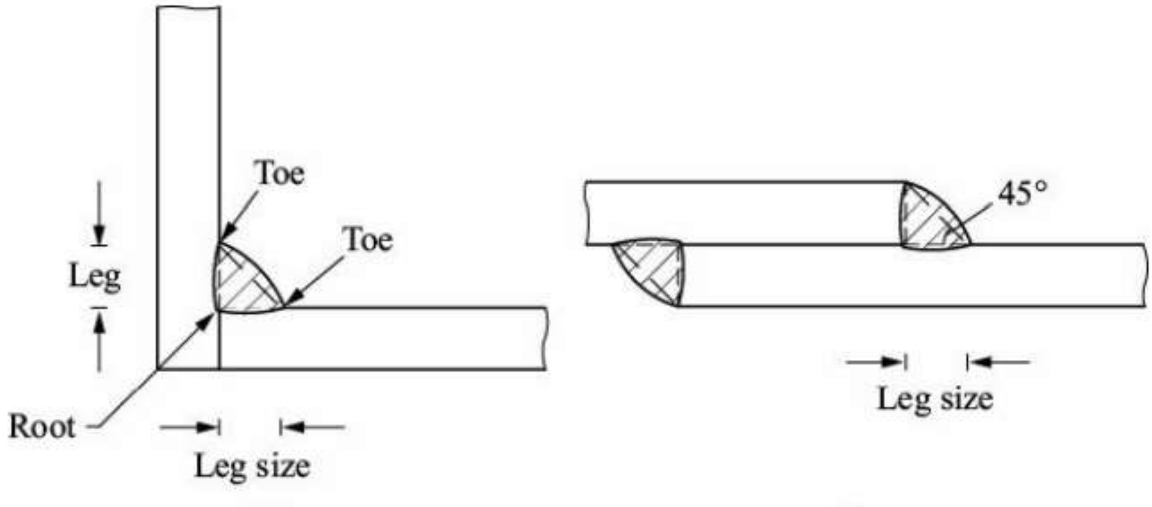
3.2.2 Butt Weld

Butt weld is also known as groove weld. Depending upon the shape of the groove made for welding butt welds are classified as shown in
Table below.

Sl. No.	Type of Butt Weld	Sketch
(a)	Square butt weld, on one side	
(b)	Square butt weld, both sides	
(c)	Single V butt joint	
(d)	Double V-butt joint	
(e)	Single U butt joint	
(f)	Single J-butt joint	
(g)	Single bevel butt joint	

3.2.3 **Fillet Weld**

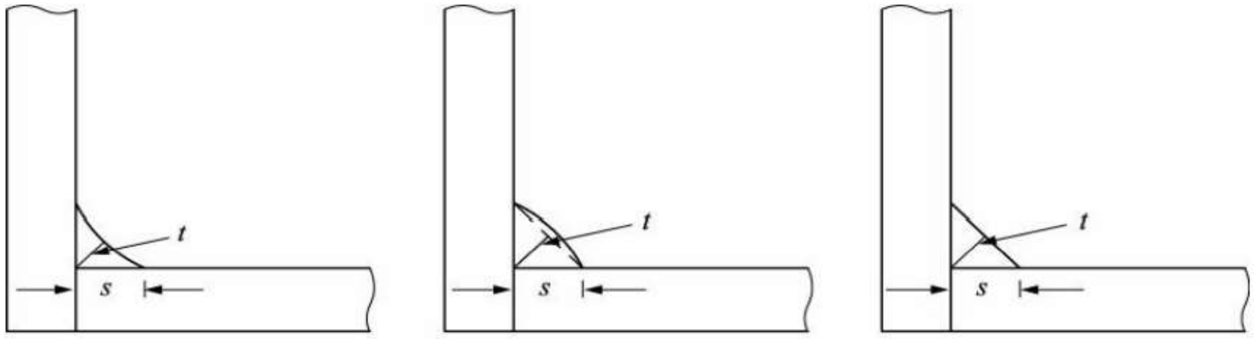
Fillet weld is a weld of approximately triangular cross-section joining two surfaces approximately at right angles to each other
in lap joint, tee joint or corner joint. Figure below shows typical fillet welds.



(a) (b)

When the cross-section of fillet weld is isoceles triangle with face at 45º, it is known as a standard fillet weld. In special
circumstances 60º and 30º angles are also used.

A fillet weld is known as concave fillet weld, convex fillet weld or as mitre fillet weld depending upon the shape of weld face,
(Referring Figure below.)



(a) Concave (b) Convex (c) Mitre

s - size *t* - throat thickness

3.2.4 Slot Weld and Plug Weld

Figure below shows a typical slot weld in which a plate with circular hole is kept with another plate to be joined and then fillet welding is made along the periphery of the hole.

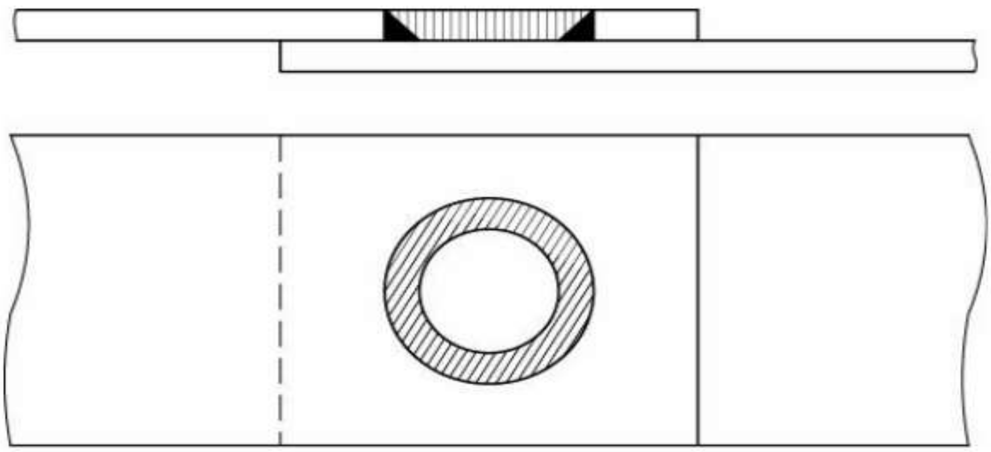
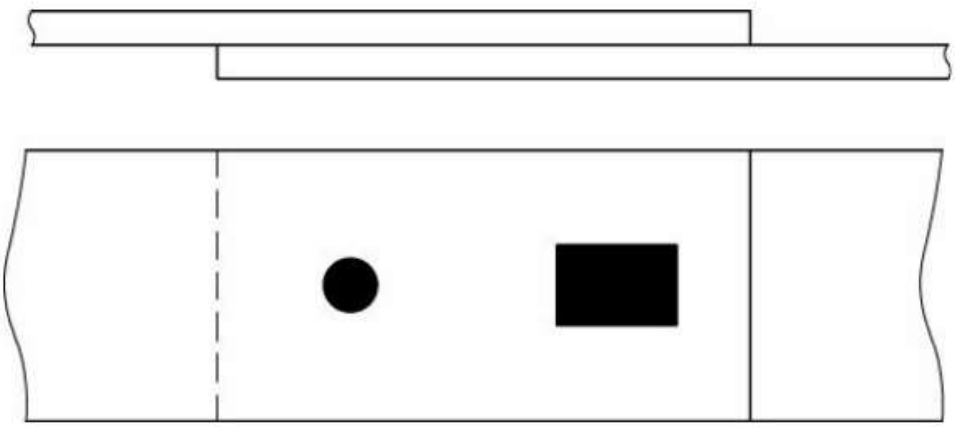


Figure below shows typical plug welds in which small holes are made in one plate and is kept over another plate to be connected and then the entire hole is filled with filler material.



IMPORTANT SPECIFICATIONS FOR WELDING:

Learning Objectives
3.3 Specifications for welding as per IS code.

3.3.1 IMPORTANT SPECIFICATIONS FOR WELDING:

Butt Weld:

The size of butt weld shall be specified by the effective throat thickness. In case of a complete penetration butt weld it shall be taken as thickness of the thinner part joined. Double U, double V, double J and double bevel butt welds may be generally regarded as complete penetration butt welds.

The effective throat thickness in case of incomplete penetration butt weld shall be taken as the minimum thickness of the weld metal common to the parts joined, excluding reinforcement. In the absence of actual data it may be taken as 5/8th of thickness of thinner material (IS 800-1969).

2. The effective length of butt weld shall be taken as the length of full size weld.
3. The minimum length of butt weld shall be four times the size of the weld.
4. If intermittent butt welding is used, it shall have an effective length of not less than four times the weld size and space between the two welds shall not be more than 16 times the thickness of the thinner part joined.

Fillet Weld:

1. Size of fillet weld:

(a) The size of normal fillet weld shall be taken as the minimum weld leg size.
(b) For deep penetration welds with penetration not less than 2.4 mm, size of weld is minimum leg size + 2.4 mm.
(c) For fillet welds made by semi automatic or automatic processes with deep penetration more than 2.4 mm, if purchaser and contractor agree,
s = minimum leg size + actual penetration

Minimum size of fillet weld specified is 3 mm. To avoid the risk of cracking in the absence of preheating the minimum size specified are

For less than 10 mm plate 3 mm
For 10 to 20 mm plate 5 mm
For 20 to 32 mm plate 6 mm
For 32 to 50 mm plate 8 mm

Effective throat thickness: It shall not be less than 3 mm and shall not generally exceed 0.7t (or t under special circumstances), where t is the thickness of the thinner plate of the elements being welded. If the faces of plates being welded are inclined to each other, the effective throat thickness shall be taken as K times the fillet size where K is as given in table below:

Angle between fusion faces	60º–90º	91º–100º	101º–106º	107º–113º	114º–120º
Constant K	0.70	0.65	0.60	0.55	0.5

Effective length: The effective length of the weld is the length of the weld for which specified size and throat thickness exist. In drawings only effective length is shown. While welding length made is equal to effective length plus twice the size of the weld. Effective length should not be less than four times the size of the weld.

Lap joint: The minimum lap should be four times the thickness of thinner part joined or 40 mm whichever is more.The length of weld along either edge should not be less than the transverse spacing of welds.
Intermittent welds: Length shall not be less than 4 times the weld size or 40 mm whichever is more. The minimum
clear spacing of intermittent weld shall be 12t for compression joints and 16t for tensile joints, where t is the thickness of thinner plate joined. The intermittent welds shall not be used in positions subject to dynamic, repetitive and alternating stresses.

Plug Welds

The effective area of a plug weld shall be considered the nominal area of the hole.

DESIGN STRESSES IN WELDS

Learning Objectives
3.4 Stresses in Welds
3.5 Reduction in design stresses for long joints

3.4 DESIGN STRESSES IN WELDS:

Butt Welds

Butt welds shall be treated as parent metal with a thickness equal to the throat thickness, and the stresses shall not exceed those
permitted in the parent metal.

Fillet Weld, Slot or Plug Welds

Design strength shall be based on its throat area and shall be given by

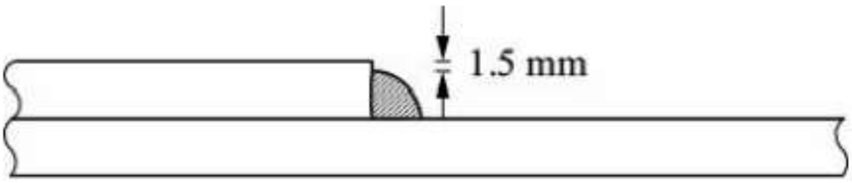
$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$$

where $$f_{wn} = \frac{f_u}{\sqrt{3}}$$

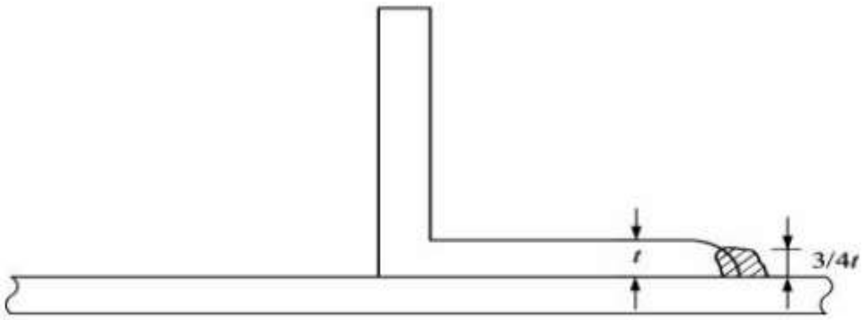
f_u = smaller of the ultimate stress of the weld or of the parent metal.
γ_{mw} = 1.25 for shop welds
= 1.5 for field welds.

The following provisions are made in the code for the fillet welds applied to the edge of a plate or section:

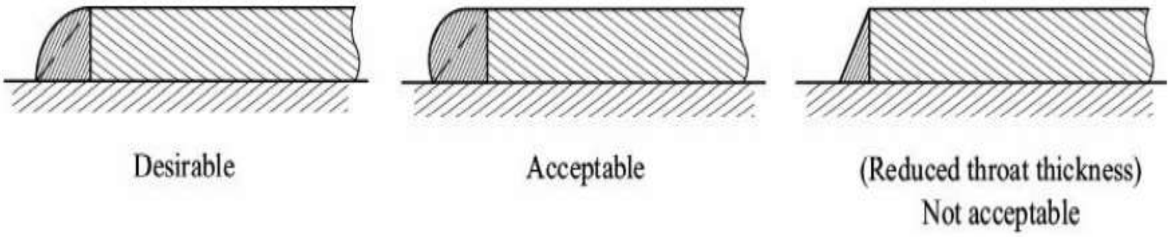
1. If a fillet weld is to the square edge of a part, the specified size of the weld should generally be at least 1.5 mm less than the edge thickness



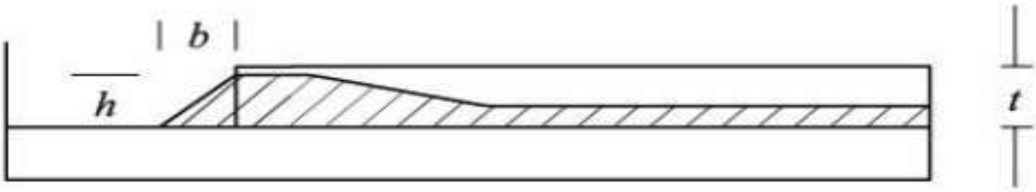
2. If fillet weld is to the rounded toe of a rolled section, the specified size of the weld should generally not exceed ¾th of the thickness of the section at the toe



3. In members subject to dynamic loading, the fillet weld shall be of full size with its leg length equal to the thickness of plate.



4. End fillet weld, normal to the direction of force shall be of unequal size with throat thickness not less than 0.5t as shown in Fig.



$h/b = 1/2$ or flatter

The difference in the thickness of weld shall be negotiated at a uniform slope.

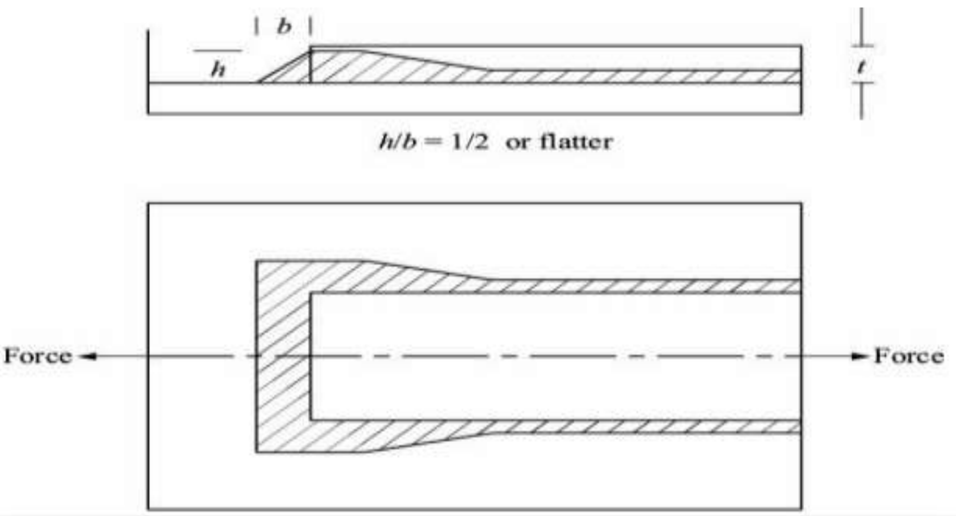
3.5 REDUCTION IN DESIGN STRESSES FOR LONG JOINTS

If the length of the welded joint lj is greater than 150t, where t is throat thickness, the design capacity of weld fwd shall be
by the factor

$$\beta_{lw} = 1.2 - \frac{0.2 l_j}{150t} \leq 1.0$$

Problem-4.

A 18 mm thick plate is joined to a 16 mm plate by 200 mm long (effective) butt weld.
Determine the strength of joint if
(i) a double V butt weld is used
(ii) a single V butt weld is used
Assume that Fe 410 grade plates and shop welds are used.



Solution:
Case (i): Double V butt weld joint
Since in such case complete penetration takes place,
throat thickness = thickness of thinner plate t = 16 mm.
Effective length L_w = 200 mm
f_u = 410 N/mm^2 , since it is shop weld γ_{mw} = 1.25
Effective area of weld = effective length × throat thickness
∴ Design strength of weld

$$= \frac{L_w t\, f_u / \sqrt{3}}{\gamma_m}$$

$$= \frac{200 \times 16 \times 410/\sqrt{3}}{1.25} = 605987 \text{ N}$$

= 605.987 kN Answer
Case (ii): Single V butt weld joint
Since penetration is not complete, effective throat thickness

$$t = \frac{5}{8} \times 16 = 10 \text{ mm.}$$

∴ Design strength

$$= \frac{L_w\ t\ f_u / \sqrt{3}}{\gamma_{mw}}$$

$$= \frac{200 \times 10 \times 410/\sqrt{3}}{1.25} = 378742 \text{ N}$$

= 378.742 kN Answer
Module – I

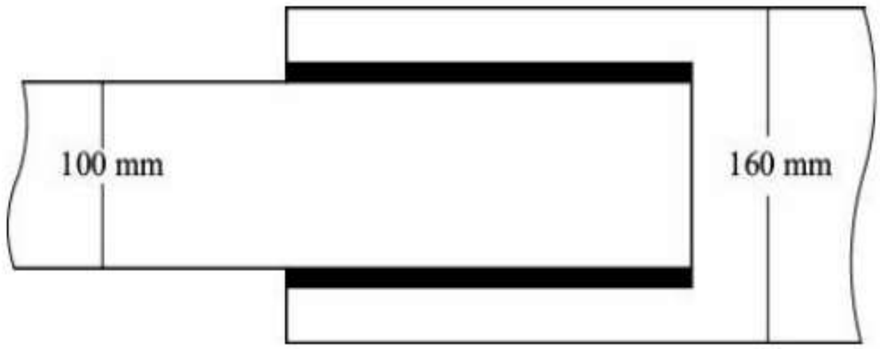
NUMERICALS

Learning Objectives

3.6 Numericals on fillet weld.

Problem - 5

suitable longitudinal fillet weld to connect the plates as shown in Figure below to transmit a pull equal to the full strength of small
plate. Given: Plates are 12 mm thick; grade of plates Fe 410 and welding to be made in workshop.



Solution:
Minimum size to be used = 5 mm
Maximum size = 12 – 1.5 = 10.5 mm
Use s = 10 mm fillet weld
fy = 410 N/mm2 ,
γ mw = 1.25,
thickness of plate = 12 mm,
breadth of plate = 100 mm
∴ Full design strength of smaller plate

$$= \frac{A_g f_y}{\gamma_{mo}}$$

fy = 250 MPa, γ mo = 1.1
∴ Full design strength

$$= 12 \times 100 \times \frac{250}{1.1} = 272727 \text{ N}$$

Let effective length of welds be Lw
Assuming normal weld, throat thickness t = 0.7 × 10 = 7 mm
∴ Design strength of weld

$$= L_w t \frac{f_u}{\sqrt{3}} \times \frac{1}{1.25}$$

Equating it to the strength of plate, we get Lw = 205.7 mm Provide effective length of 105 mm on each side.

NUMERICALS

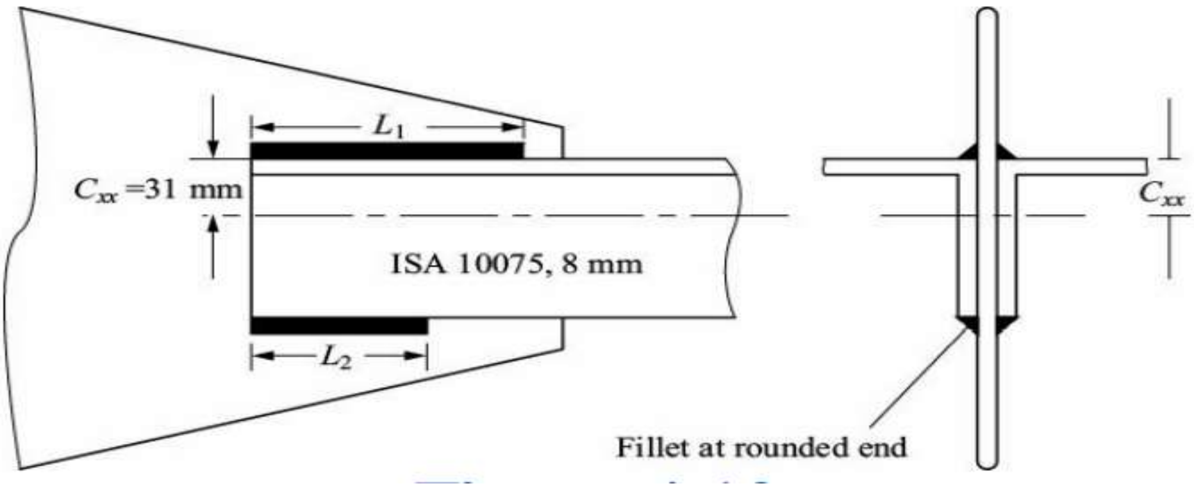
Learning Objectives

3.7 Numericals on tie members of roof truss.

Problem-6

A tie member of a roof truss consists of 2 ISA 10075, 8 mm. The angles are connected to either side of a 10 mm gusset plate and the member is subjected to a working pull of 300 kN. Design the welded connection. Assume connections are made in the workshop.

Solution:



Working Load = 300 kN

∴ Factored Load = 300 × 1.5 = 450 kN

Thickness of weld:

(i) At the rounded toe of the angle section, size of weld should not exceed = $\frac{3}{4}$ × thickness

s = $\frac{3}{4}$× 8 = 6 mm

(ii) At top (Ref. Fig) the thickness should not exceed

s = t – 1.5 = 8 – 1.5 = 6.5 mm.

Hence provide s = 6 mm, weld.

Each angle carries a factored pull of $\frac{450}{2}$ = 225kN

Let Lw be the total length of the weld required.

Assuming normal weld, t = 0.7 × 6 mm

∴ Design strength of the weld = Lwt $\frac{fu}{\sqrt{3}}$ x$\frac{1}{1.25}$

$$=Lw \text{ x}0.7 \text{ x}6 \text{ x}\frac{410}{\sqrt{3}} \text{ x}\frac{1}{1.25}$$

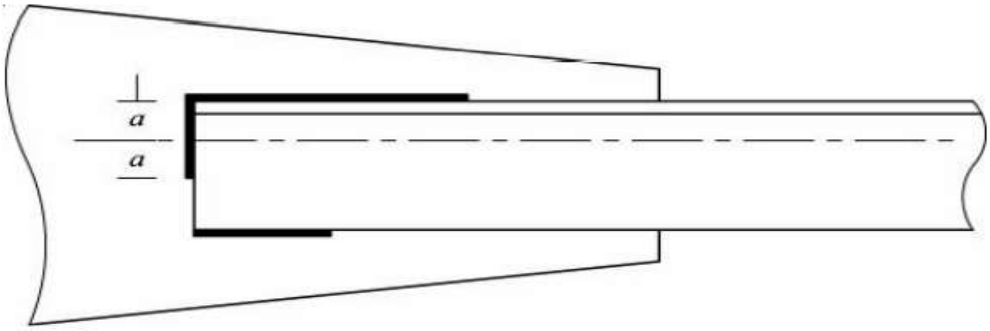
Equating it to the factored load, we get

Lw x0.7 x6 x $\frac{410}{\sqrt{3}}$ x $\frac{1}{1.25}$ = 225 x 10^3

∴ Lw = 283 mm.

Centre of gravity of the angle section is at a distance 31 mm from top.

Let L1 be the length of top weld and L2 be the length of lower weld.
To make centre of gravity of weld to coincide with that of angle,
L1 × 31 = L2 (100 − 31) L1 + L2 = 283
or L2 = 87 mm.
∴ L1 = 195 mm.
Provide 6 mm weld of L1 = 195 mm and L2 = 87 mm as shown in the above figure.
Note: In case the length available at the sides becomes insufficient, end fillet weld also may be provided. The length of end fillet should be the same on either side of centroidal axis of the angle, so that neutral axis of the weld and the section coincide as below.



Problem-7

Design the welded connection to connect two plates of width 200 mm and thickness 10 mm for 100 percent efficiency.

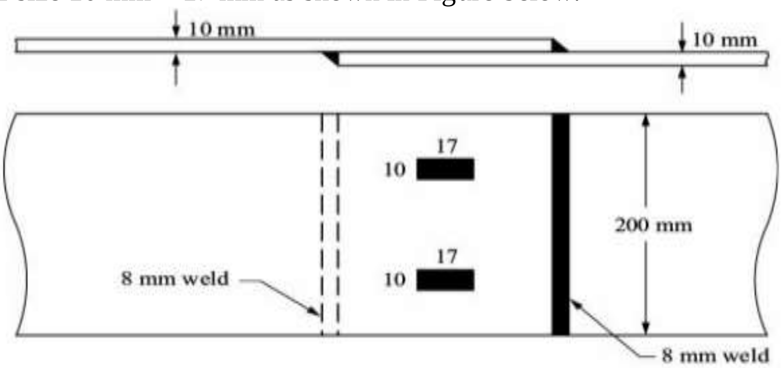
Solution:

Strength of plates $= \frac{A_g f_y}{\gamma_{mo}} = \frac{200 \times 10 \times 250}{1.1} = 454545$N
Minimum size of weld = 5 mm.
Maximum size = 10 – 1.5 = 8.5 mm.
Use s = 8 mm weld.
Effective length of fillet welds = 2 (200 – 2 × 8)
Lw = 368 mm.
Throat thickness t = 0.7 × 8

Design strength of fillet welds = Lwt $\frac{f_u}{\sqrt{3}} \times \frac{1}{\gamma_{mw}}$

$= 368 \times 0.7 \times 8 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25} = 390256$ N

∴ Slot welds are to be provided to resist a force of = 454545 – 390256 = 64289 N
Strength of the slot weld $= \frac{f_{wn}}{\gamma_{mw}} = \frac{f_u}{\sqrt{3}\gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.25} = 189.37$ N/mm2
∴ Area of the slot weld required $= \frac{64289}{189.37} = 339.5$ mm2
Provide two slot welds of size 10 mm × 17 mm as shown in Figure below.



* * *

Short Type Questions

1. Fillet weld fails in what?
2. What will be the maximum size for square edge of two plates of equal thickness getting full strength of fillet weld?
3. A butt weld is specified by __________ ?
4. What is k for 70^o angel of fusion faces?
5. Spot welding is used when two plates are placed __________ ?
6. The design nominal strength of fillet weld is __________ ?

Answers :

1. Shear
2. 1.5mm less than the thickness of plate
3. Leg length
4. k = 1.0
5. one below other
6. $\frac{f_u}{\sqrt{3}}$

Medium type Questions

1. What are the advantages and disadvantages of welded connections?
2. Neatly sketch the following welded connections:
   (a) Butt weld (groove weld) – single V, double V
   (b) Fillet weld
   (c) Slot weld
   (d) Plug weld.
3. Two 12 mm thick plates are joined by 160 mm long (effective) butt weld. Determine the strength of joint if
   (a) Single U butt weld is used.
   (b) Double U butt weld is used.

Long type questions

1. Design a suitable longitudinal fillet weld to connect 120 × 8 mm plate to 150 × 10 mm plate to transmit a pull equal to the full strength of small plate. Assume welding is to be made in the field.
2. A tie member of a roof truss consists of 2 ISA 9060, 10 mm. The angles are connected on the either side of 12 mm gusset plate and the member is subjected to a factored pull of 350 kN. Design the welded connection. Assume welding is to be made in the workshop.
3. Two plates 180 mm wide and 8 mm thick are to be connected by welding, using shop welds, design the connection.

DESIGN OF TENSION MEMBERS

Learning Objectives

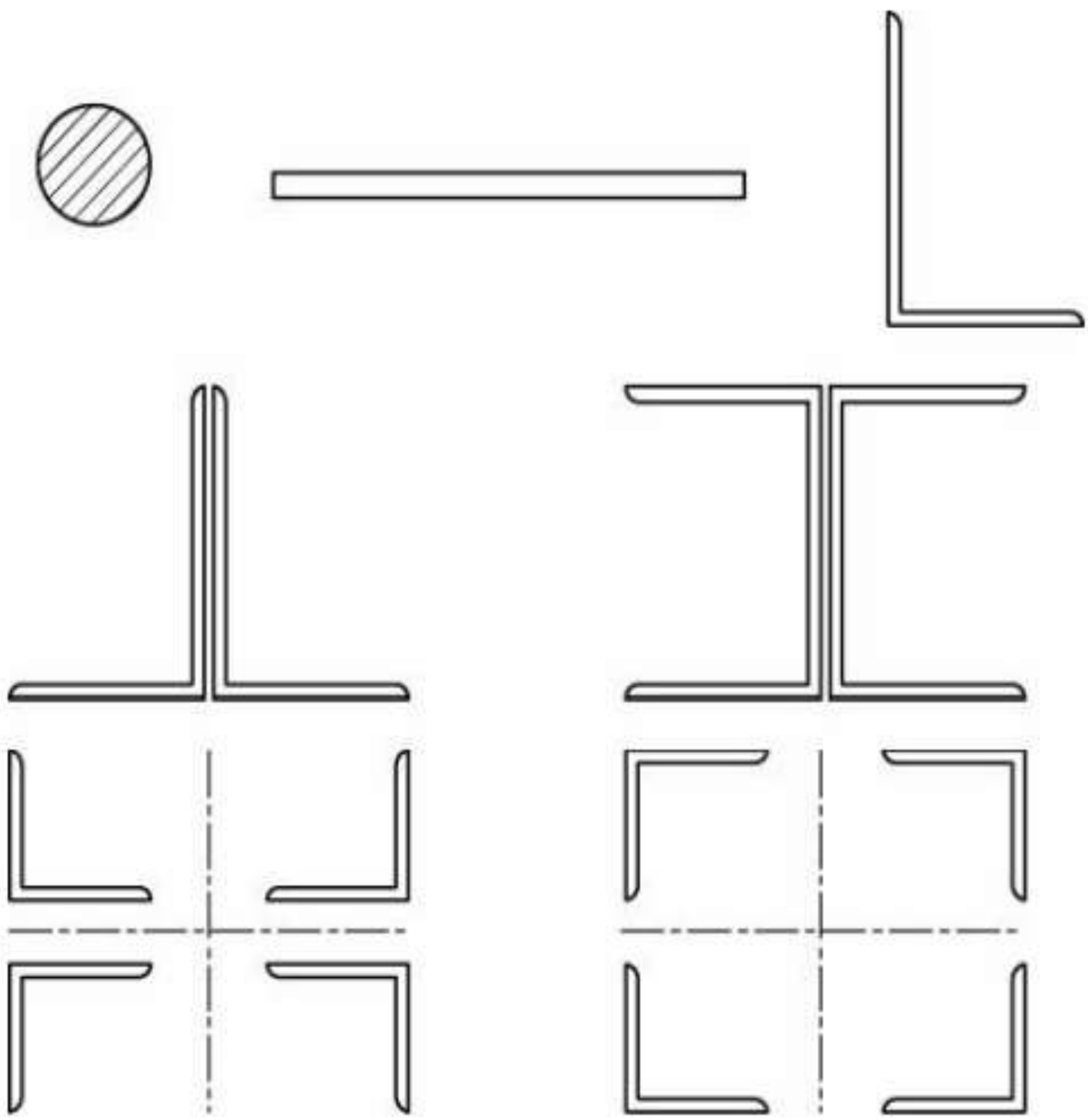
4.1 Design of Tension Members

4.2 *Design Strength of Tension Members*

4.1 DESIGN OF TENSION MEMBERS

Tension members are also known as tie members. The form of a tension member is governed to a large extent by the type of the structure of which it is a part and by the method of joining it to the adjacent member of the structure. In general the section should be compact and in order to minimize stress concentration it should be so arranged that as large portion of it as possible is connected to the gusset plates.

The common shapes used are shown in Figure below.



4.2 DESIGN STRENGTH OFA TENSION MEMBER

The design strength of a tension member is the lowest of the following:

(a) Design strength due to yielding of gross section Tdg .

(b) Rupture strength of critical section, Tdn and

(c) The block shear Tdb .

Design Strength Due to Yielding of Gross Section

This strength is given by

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}}$$

where, f_y = yield stress of the material

A_g = gross area of the cross-section

γ_{mo} = partial safety factor for failure in tension by yielding = 1.1.

DESIGN STRENGTH DUE TO RUPTURE AT CRITICAL SECTION

Learning Objectives

4.3 Design strength due to Rupture at Critical Section

4.4 Design strength due to block shear

Design Strength of Tension Members

4.3 Design Strength Due to Rupture at Critical Section

This strength for plates is

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{ml}}$$

where An net effective area at critical section

$$= \left[b - n d_o + \sum \frac{p_{si}^2}{4 g_i} \right] t$$

For threaded rods and bolts

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{ml}}$$

where An = net area at the threaded section.

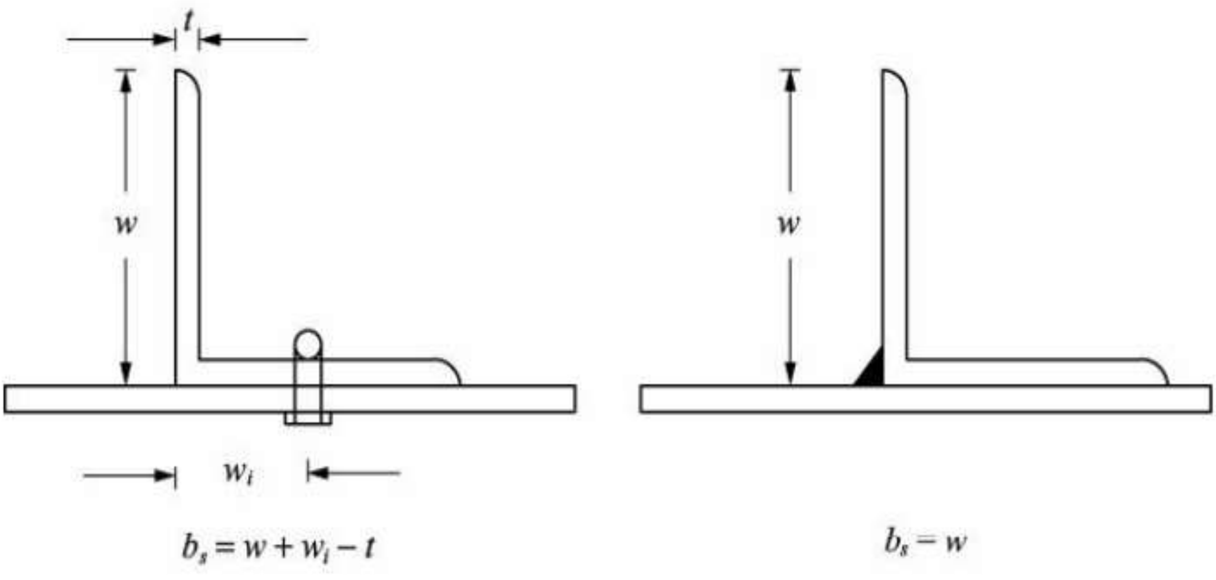
$$= \frac{\pi}{4} \left(d - 0.9382 p \right)^2$$

where p is pitch of thread

$$\approx 0.78 \frac{\pi}{4} d^2$$

for ISO threads

Single Angle [Ref. Figure below]



As the effectiveness of outstanding leg is less, the design strength as governed by rupture at net section is given by

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{ml}} + \frac{\beta A_{go} f_y}{\gamma_{mo}}$$

where, A_{nc} = net area of the connected leg

A_{go} = gross area of the outstanding leg and

$$\beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\frac{b_s}{L_c} \leq \frac{f_u \gamma_{mo}}{f_y \gamma_{ml}} \geq 0.7$$

where w = outstanding leg width.

b_s = shear leg width,

and L_c = length of the end connection,

that is, the distance between outermost bolt in the end joint measured along the load direction or length of the weld along the load

direction.

t = thickness of leg.

For preliminary design IS code recommends the following formula:

$$T_{dn} = \frac{\alpha A_n f_u}{\gamma_{ml}}$$

where α = 0.6 for one or two bolts

= 0.7 for three bolts

= 0.8 for four or more bolts along the length of connection or equivalent weld length.

However, if it is difficult to find equivalent weld length, designers have to judge this.

Other Sections

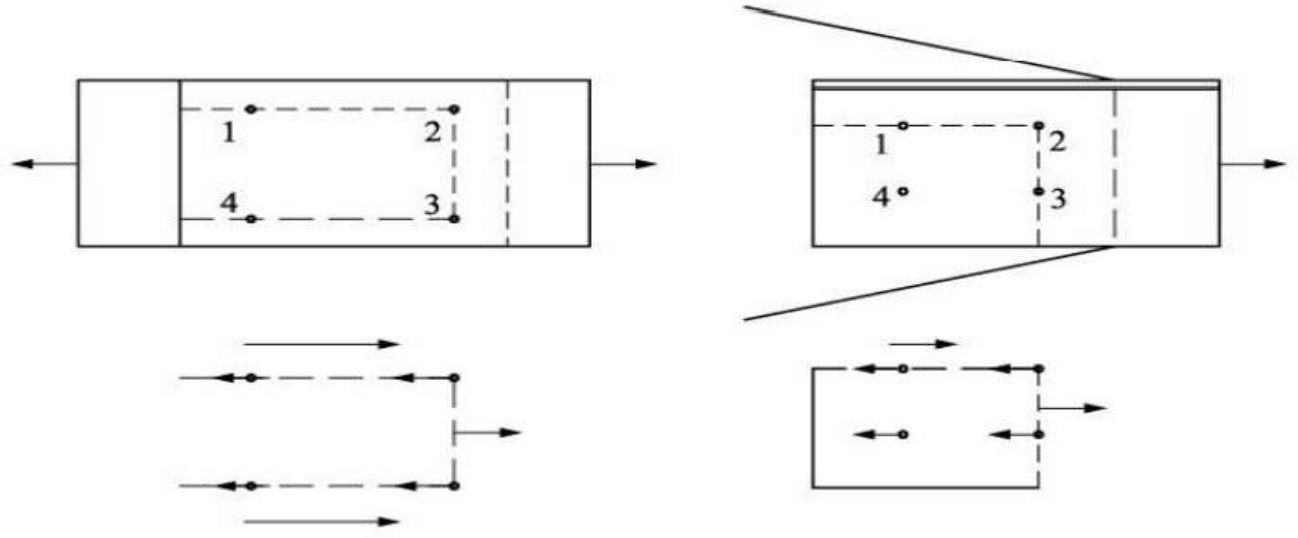
The rupture strength, T_{dn} of the double angles, channels, I-sections etc., may be calculated by the same equation as for single angle,

but with bs taken from the farthest edge of the outstanding leg to the nearest bolt /weld line in the connected leg.

4.4 Design Strength Due to Block Shear:

At the connected end, failure of a tension member may occur along a path involving shear along one plane and tension on a

perpendicular plane along the fastener. This type of failure is known as block failure.



(a) (b)

Referring to above fig(a), shear failure occurs along 1-2 and 3-4 whereas tension failure occurs along 2-3.

Referring to above fig(b), shear failure occurs along 1-2 and tension failure along 2-3.

IS 800-2007, recommends the following block shear strength Tdb if bolted connections are used.

It shall be smaller of

$$T_{db} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{mo}} + \frac{0.9 A_{tn} f_u}{\gamma_{ml}}$$

Or

$$T_{db} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{ml}} + \frac{A_{tg} f_y}{\gamma_{mo}}$$

where, A_{vg} and A_{vn} = Minimum gross and net area in shear (1-2, 3-4 in Fig. (a), 1-2 in Fig. (b))

A_{tg} and A_{tn} = Minimum gross and net area in tension [2-3 as shown in above Fig.].

Note: The block shear strength, T_{db} shall be checked for welded end connections by taking an appropriate section around the end weld.

NUMERICALS

Learning Objectives
4.5 *Numericals on tensile strength.*

4.5 Problem-7
Determine the design tensile strength of the plate 130 mm × 12 mm with the holes for 16 mm diameter bolts as shown in below.
Steel used is of Fe 410 grade quality.
Solution:
Strength of the plate is the least of
(a) Yielding of gross section
(b) Rupture of critical section
(c) The block shear strength
(a) From consideration of yielding:

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}}$$

Now, $A_g = 130 \times 12 = 1560$ mm^2 ,
$f_y = 250$ N/mm^2 ,
$\gamma_{mo} = 1.1$

$$\therefore \quad T_{dg} = \frac{1560 \times 250}{1.1} = 354545 \text{ N} = 354.545 \text{ kN}$$

(b) From the consideration of rupture along the critical section:
Critical section is having two holes.
Diameter of holes = 16 + 2 = 18 mm.
∴ $A_n = (130 - 2 \times 18) \times 12 = 1128$ mm^2
Strength of member from the consideration of rupture

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{ml}} = \frac{0.9 \times 1128 \times 410}{1.25}$$

= 332986 N = 332.986 kN
(c) Block shear strength:
$A_{vg} = 2 \times (35 + 60) \times 12 = 2280$ mm^2 $A_{tg} = 60 \times 12 = 720$ mm^2
$A_{vn} = (35 + 60 - 1.5 \times 18) \times 12 \times 2 = 1632$ mm^2 $A_{tn} = (60 - 18) \times 12 = 504$ mm^2

The block shear strength is the least of the following two:

(1)

$$T_{db} = \left[\frac{A_{vg} f_y}{\sqrt{3}\gamma_{mo}} + \frac{0.9 A_{tn} f_u}{\gamma_{ml}} \right]$$

$$= \frac{2280 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 504 \times 410}{1.25}$$

= 447953 N = 447.953 kN.

(2)

$$T_{db} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\gamma_{ml}} + \frac{A_{tg} f_y}{\gamma_{mo}}$$

= 441784 N = 441.784 kN

∴ Tdb = 441.784 kN

Strength of plate = 332.986 kN

DESIGN PROCEDURE FOR TENSION MEMBER

Learning Objectives
4.6 Design Procedure for Tension members
4.7 Numerical on tension member.
DESIGN PROCEDURE:
The following design procedure may be adopted.
1. Find the required gross area to carry the factored load considering the strength in yielding. i.e., where Tu = factored tensile force.
2. Select suitable shape of the section depending upon the type of structure and the location of the member such that gross area is
25 to 40 per cent more than Ag calculated.
3. Determine the number of bolts or the welding required and arrange.
4. Find the strength considering:
(a) Strength in yielding of gross area
(b) Strength in rupture of critical section and
(c) Strength in block shear.
Usually, if minimum edge distance and minimum pitch are maintained, strength in yielding is the least value, hence the design is
safe if Ag provided > Ag required.
5. The strength obtained should be more than factored tension. If it is too much on higher side or the strength is less than factored
tension, the section may be suitably changed and checked.

IS 800-2007 also recommends the check for slenderness ratio of tension members as per table-3.

4.7 Problem-8
Design a double angle tension member connected on each side of a 10 mm thick gusset plate, to carry an axial factored load of 375
kN. Use 20 mm black bolts. Assume shop connection.
Solution:
Area required from the consideration of yielding.
Try 2 ISA 7550, 8 mm thick which has gross area = 2 × 938 = 1876 mm^2.
Strength of 20 mm black bolts:
In double shear =

$$\left[\frac{\pi}{4}\times 20^2 + 0.78\times\frac{\pi}{4}\times 20^2\right]\times\frac{400}{\sqrt{3}}\,\frac{1}{1.25}$$

= 103314 N.
Strength in bearing:
Taking e = 40 mm, p = 60 mm,
K_b is smaller of

$$\frac{40}{3\times 22},\ \frac{60}{3\times 22}-0.25,\ \frac{400}{410},1.0$$

i.e., K_b = 0.606

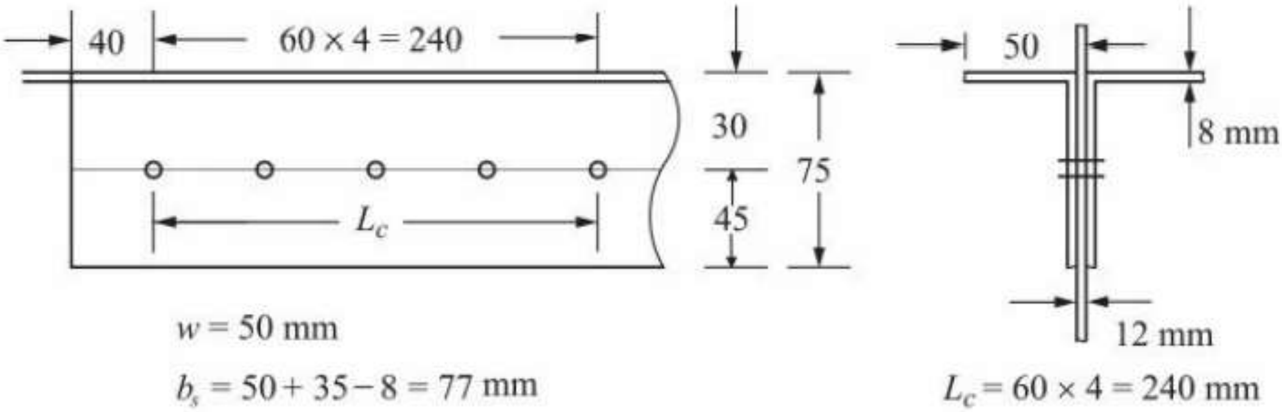
$$\therefore \quad V_{dpb} = \frac{1}{1.25} \times 2.5 \times 0.606 \times 20 \times 8 \times 400 = 77568 \text{ N}$$

∴ Bolt value = 77568 N

Number of bolts required

$$= \frac{375000}{77568} = 4.83$$

Provide 5 bolts in a row as shown in Figure below.



Checking the design:

(a) Strength against yielding

$$= \frac{A_g f_y}{\gamma_{mo}} = \frac{1876 \times 250}{1.1} = 426364 \text{ N} > 375 \times 1000$$

ok

(b) Strength of plate in rupture:

Area of connected leg,

$$A_{nc} = 2\left(75 - 22 - \frac{8}{2}\right) \times 8 = 784 \text{ mm}^2$$

Area of outstanding leg,

$$A_{go} = 2 \times \left(50 - \frac{8}{2}\right) \times 8$$

$$= 736 \text{ mm}^2$$

$$= 1.4 - 0.076 \times \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_c}$$

$$= 1.4 - 0.076 \times \frac{50}{8} \times \frac{250}{410} \times \frac{77}{240}$$

$$= 1.307$$

$$\therefore \; T_{dn} = \frac{0.9 f_u A_{nc}}{\gamma_{ml}} + \beta \frac{A_{go} f_y}{\gamma_{mo}}$$

$$= \frac{0.9 \times 410 \times 784}{1.25} + 1.307 \times \frac{736 \times 250}{1.1} = 450062 > 375000 \text{ N}$$

O.K.

(c) Strength against block shear failure:

Per angle: $A_{vg} = (40 + 60 \times 4) \times 8 = 2240 \text{ mm}^2$

$A_{vn} = (40 + 60 \times 4 - 4.5 \times 22) \times 8 = 1448 \text{ mm}^2$

$A_{tg} = (75 - 35) \times 8 = 320$ mm2

$A_{tn} = (75 - 35 - 0.5 \times 22) \times 8 = 232 \text{ mm}^2$

Strength against block failure of each angle is the smaller of the following two values:

$$\text{(i)} \; = \frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{mo}} + \frac{0.9 \times A_{tn} f_u}{\gamma_{ml}}$$

$$= \frac{2240 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 232 \times 410}{1.25}$$

$$= 362410 \text{ N}$$

$$\text{(ii)} = \frac{0.9 A_{vn} f_u}{\sqrt{3} \times \gamma_{ml}} + \frac{A_{tg} f_y}{\gamma_{mo}}$$

$$= \frac{0.9 \times 1448 \times 410}{\sqrt{3} \times 1.25} + \frac{320 \times 250}{1.1}$$

$$= 319515 \text{ N}$$

∴ Strength of two angles against block failure = 2 × 319515 > 375000 O.K.

Hence use 2 ISA 7550, 8 mm with 5 bolts of 20 mm diameter.

TENSION MEMBER SPLICE

Learning Objectives

4.8 Tension Member Splice

4.9 *Lug Angels*

4.8 TENSION MEMBER SPLICE:

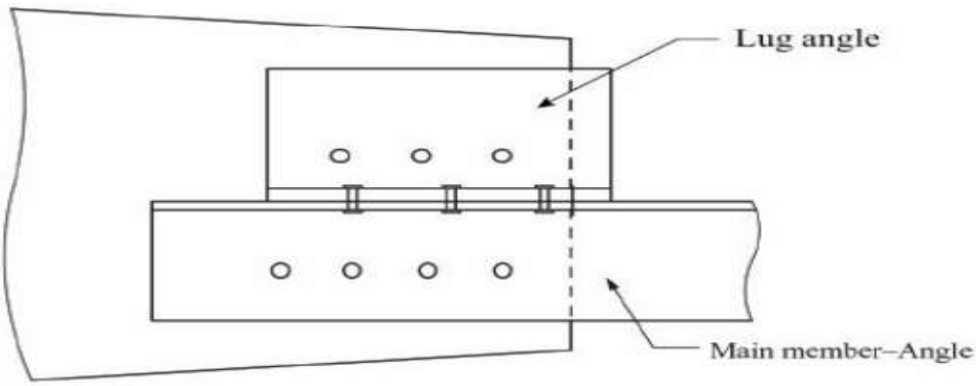
If a single piece of required length is not available tension members are spliced to transfer required tension from one piece to
another. The strength of the splice plates and the bolts/weld connecting them should have strength at least equal to the design load. When
tension members of different thicknesses are to be connected, filler plates may be used to bring the members in level. The design shear capacity of bolts carrying shear through a packing plate in excess of 6 mm shall be decreased by a factor (clause 10.3.3.3 in IS 800- 2007)

$\beta_{pk} = 1 - 0.0125\, t_{pk}$

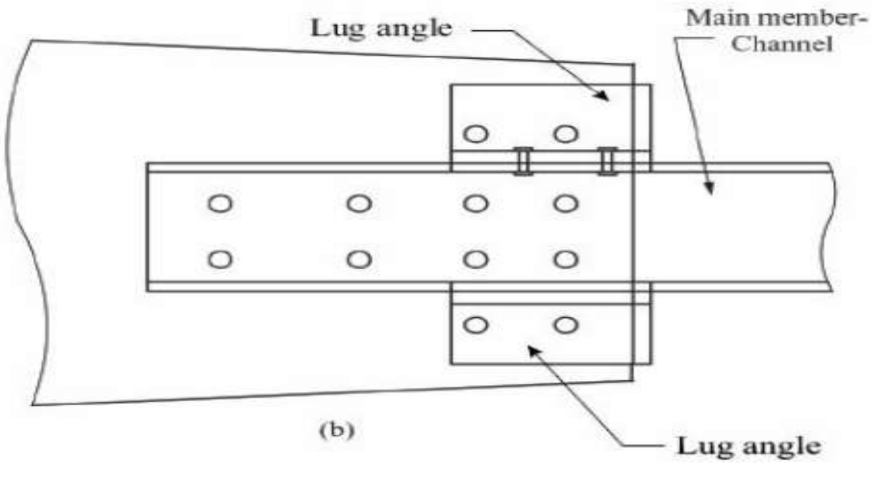
where t_{pk} = thickness of the thicker packing plate

4.9 LUG ANGLES:

Length of the end connection of a heavily loaded tension member may be reduced by using lug angles as shown in Figure below.



(a)



(b)

By using lug angles there will be saving in gusset plate, but it is upset by additional fasteners and angle required. Hence nowadays
it is not preferred. IS 800-2007 specifications for lug angles are (clause 10.12)

1. The effective connection of the lug angle shall as far as possible terminate at the end of the member.
2. The connection of lug angle to main member shall preferably start in advance of the member to the gusset plate.
3. Minimum of two bolts, rivets or equivalent welds be used for attaching lug angle to the gusset.
4. If the main member is an angle
   (a) the whole area of the member shall be taken as the effective rather than net effective section (i.e., with reduction for outstanding leg area). The whole area of the member is the gross area less deduction for bolt holes.
   (b) the strength of lug angles and fastener connecting lug angle to gusset plate should be at least 20 percent more than the force in outstanding leg.
   (c) the strength of the fastener connecting lug angle and main member shall be at least 40% more than the force carried by the outstanding leg.
5. In case the main member is a channel and like:
   (a) as far as possible should be symmetric.
   (b) the strength of fasteners connecting lug angle to the gusset should be at least 10% more than the force in outstanding leg.
   (c) the strength of fasteners connecting lug angle and main member shall be at least 20% more than the force in outstanding leg.

NUMERICALS

Learning Objectives
4.10 Numerical on tension member of roof truss.

4.10 Problem-9
A tension member of a roof truss carries a factored axial tension of 430 kN. Design the section and its connection
(a) without using lug angle
(b) using lug angle.
Solution:
Tensile force in the main member = 430 kN.
Considering the strength in yield, gross area required is given by,

$$430 \times 1000 = \frac{A_g f_y}{1.1} = \frac{A_g \times 250}{1.1}$$

$A_g = 1892\ mm^2$
Select ISA 100100, 10 mm which has
$A_g = 1903\ mm^2$
Using 20 mm diameter black bolts,
Strength in single shear:

$$T_{dg} = 0.78 \times \frac{\neq}{4} \times 20^2 \times \frac{400}{\sqrt{3}} \times \frac{1}{1.25} = 45272 \text{ N}$$

Strength in bearing:
$e_{min} = 1.5 \times 20 = 30$ mm
$p_{min} = 2.5 \times 20 = 50$ mm
Let e = 30 mm p = 50 mm
Then K_b is smaller of

$$\frac{30}{3 \times 22}, \quad \frac{50}{2 \times 22} - 0.25, \quad \frac{400}{410}, \; 1.0$$

$\therefore K_b = 0.4545$

$$\therefore \; T_{dn} = 2.5 \times 0.4545 \times 20 \times 10 \times \frac{400}{1.25} = 72720 \text{ N.}$$

$\therefore$ Bolt value = 45272 N.
Note: In case of single shear, bolt value is usually governed by value in single shear.
Connection without lug angle:
Number of bolts required

$$= \frac{430000}{45270} = 9.5$$

Provide 10 bolts.

Length of connection, Lc = 9 × 50 = 450 mm
15 d = 15 × 20 = 300 mm.
∴ Lc > 15 d ,   It is long connection.
∴ blj = 1.075 – 0.005$\frac{450}{20}$ = 0.9625
Shear strength of bolt (after reducing for long connection)
= 0.9625 × 45272 = 435743 N
∴ No. of bolts required =$\frac{430000}{4357.3}$ = 9.87
Hence 10 bolts are sufficient.
Yield strength =

$$\frac{A_g f_y}{1.1} = \frac{1903 \times 250}{1.1}$$

= 432500 N > 430 × 10 3 N.
Hence O.K.
Rupture strength:
A_{nc} = $(100 - \frac{10}{2} - 22) \times 10 = 730$ mm^2
A_{go} = $(100 - \frac{10}{2}) \times 10 = 950$ mm^2
b = 1.4 – 0.076$(\frac{95}{10}) \times \frac{250}{410} \times \frac{130}{450}$ = 1.2728 > 0.7 and <$\frac{f_u \gamma_{mo}}{f_y \gamma_{ml}}$
∴ b = 1.2728
Strength in rupture =$\frac{730 \times 0.9 \times 410}{1.25} + \frac{1.2728 \times 950 \times 250}{1.1}$ = 490305 N
Block shear strength:
A_{vg} = (450 + 30) × 10 = 4800 mm2 , A_{tg} = 70 × 10 = 700 mm^2
A_{vn} = (480 – 9.5 × 22) × 10 = 2710 mm^2 , A_{tn} =$(70 - \frac{22}{2})$ × 10 = 490 mm^2
∴ Block shear strength =$\frac{4800 \times 2500}{\sqrt{3} \times 1.1} + \frac{490 \times 0.9 \times 410}{1.25}$ = 774485 N
Or=$\frac{0.9 \times 2710 \times 410}{\sqrt{3} \times 1.25} + \frac{700 \times 0.9 \times 250}{1.1}$ = 605057 N
∴ Block shear strength = 605057 N
Hence strength of angle is 432500 N > 430 × 10^3 N. Hence O.K.

Connection with lug angle:
Gross area of connected leg = Gross area of outstanding leg
∴ Load is shared equally.
i.e., Load in outstanding leg = Load in connected leg = $\frac{430}{2}$ = 215 kN
Lug angle is to be designed to take a load of = 1.2 × 215 = 258 kN.
Gross area of lug angle required = $\frac{258 \times 1000}{250/1.1}$ = 1135 mm2
Provide ISA 100100, 6 mm.
∴ A_g provided = 1167 mm^2
The strength of lug angle in rupture =$\frac{0.9 \times (100+100-10-22) 6 \times 410}{1.25}$
= 297562 N > 258000 N. O.K
Bolt value:
In single shear = 45272 N
In bearing =$\frac{2.5 \times 0.4545 \times 20 \times 6 \times 400}{1.25}$ =43632
∴ Bolt value = 43630 N

Number of bolts required $=\frac{258x1000}{43630.}$ = 5.91

Provide 6 bolts.

Design force for connected leg = 1.4 × 215 kN

∴ Number of bolts required to connect lug angle with main angle

$$= \frac{1.4x215x1000}{43630} = 6.89$$

Provide 7 bolts.

Connection of main angle to gusset plates:

Force to be transferred = 215 kN

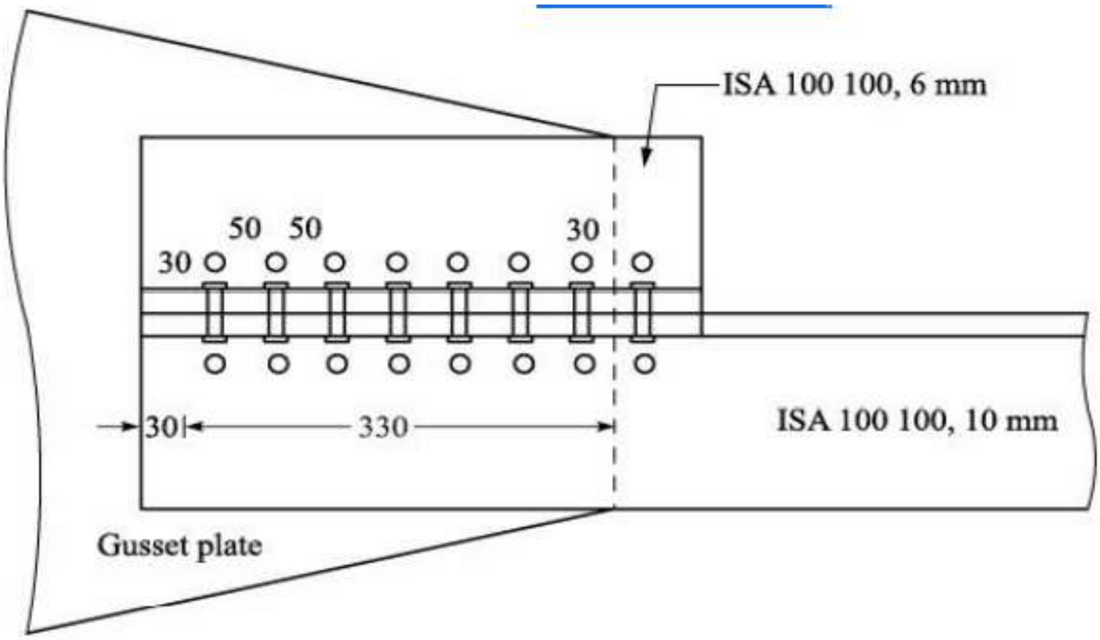
Bolt value for this is 45272 N.

∴ No. of bolts required = $\frac{215000}{45270}$ = 4.75

Provide 5 bolts.

Required length of gusset plate = 30 + (7 – 1) × 50 = 330 mm (compared to 480 mm required without lug angle)

[Block shear strength may be checked. It is safe.] The connection detail is shown in the Figure below.



* * *

Short type questions

1. Which one is the best tension member section ?
2. A steel plate is 30cm wide and 10mm thick. If the diameter of the bolt hole is 20mm, the net section of the area of the plate is _______________
3. Limits are placed on slenderness ratio of tension members to check ____________.
4. A tension member splice is designed for ___________.
5. For the block shear failure of a tension member, the failure occurs along the path through the connections involving ____________.

Answers

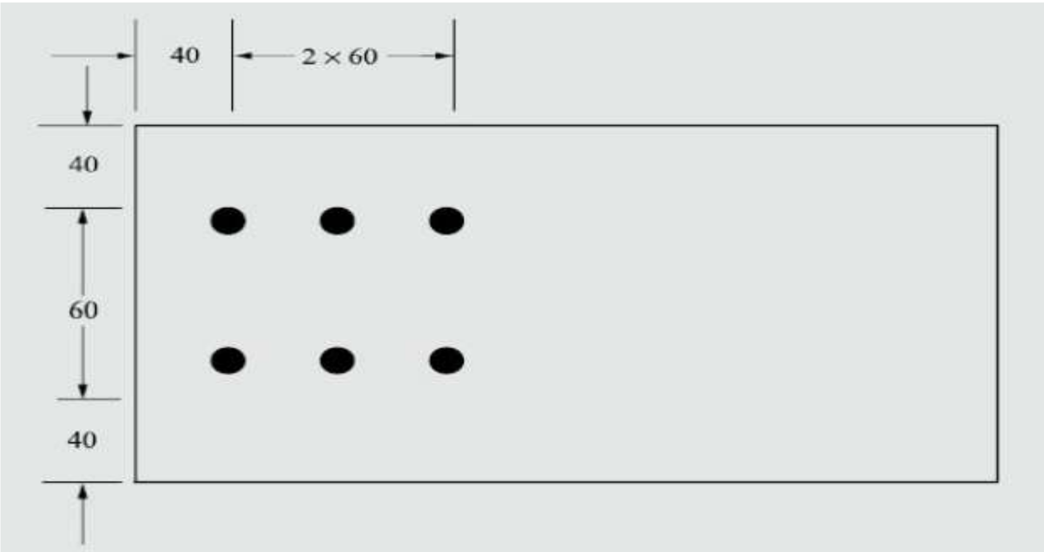
1. Double angle section on opposite side of gusset plate.
2. $28 cm^2$
3. Lateral vibration of the member.
4. Maximum of factored tensile force.
5. Tension on one plane and shear on the other perpendicular plane.

Medium type Questions

1. Explain the different modes of failure of tension members.
2. Write short note on block shear failure.
3. What is a lug angle? Illustrate with sketch. Why lug angles are used?
4. Write short notes on tension member splices.

Long type questions

1. Determine the tensile strength of the plate 140 mm × 10 mm with the holes for 24 mm bolts as shown in figure below.
2. Determine the tensile strength of a roof truss diagonal 100 × 75 × 10 mm. The longer leg is connected to 12 mm thick gusset plate with 20 mm diameter bolts in one row. Number of bolts used is 6, the edge/end distance = 30 mm and pitch = 50 mm
3. A member consists of a single angle ISA 150 × 75. It is to be connected to the gusset plate by two rows of 20 mm diameter bolts at a pitch of 80 mm with a stagger of 40 mm. The first line of bolt is located at their centres 50 mm from the back of the angle while the second row is located at 60 mm from the first row. The tensile force (working) is 200 kN. Calculate the thickness of angle.



DESIGN OF COMPRESSION MEMBERS

Learning objectives:

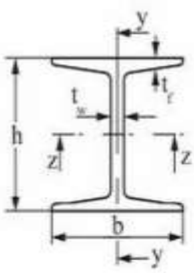
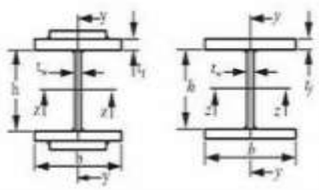
5.1. Learning of buckling class.
5.2. To know about slenderness ratio.

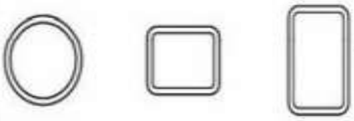
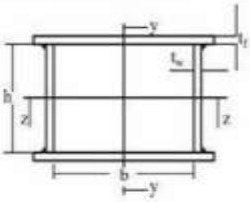
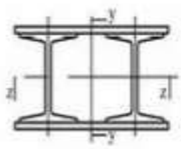
Many structural members are in compression. Vertical compression members in buildings are called columns, posts or stanchions. Compression members in trusses are called struts. The jib of crane which carries compression is called boom. Whatever care taken by the engineers to transfer load axially unexpected eccentricity of load unavoidable due to imperfection. This eccentricity causes lateral bending moment which results into bending compression also. As the axial compression increases the lateral deflection increases resulting into additional bending stresses. A stage of instability is reached at a load much below crushing strength of compression members. This phenomenon is called buckling of columns. Because of buckling tendency, the load carrying capacity of columns is reduced considerably. The load carrying capacity depends upon the end conditions and also on slenderness ratio of the column sections. Here different buckling classes based on possible imperfections in the column is discussed.

5.1 BUCKLING CLASS OF CROSS-SECTION:

Imperfections of fabrication resulting into accidental eccentricity largely depends upon the cross-section of the compression members. Based on such imperfection buckling tendency varies. IS 800-2007 divides various cross-sections into four buckling classes a, b, c and d as shown below. (Table 10 in IS 800 2007)

Buckling class of cross-sections [Refer Table 10 in IS 800]

Cross-Section (1)	Limits (2)	Buckling About Axis (3)	Buckling Class (4)
Rolled I-Sections 	$h/b_f > 1.2$: $t_f \leq 40$ mm	z-z	a
		y-y	b
	40 mm $< t_f \leq 100$ mm	z-z	b
		y-y	c
	$h/b_f > 1.2$: $t_f \leq 100$ 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 $b/t_f < 30$ $h/t_w < 30$	z-z y-y	*c* *c*
Channel, Angle, *T* and Solid Sections 		Any	*c*
Built-up Member 		Any	*c*

5.2 SLENDERNESS RATIO:

Slenderness ratio of a column is defined as the ratio of effective length to corresponding radius of gyration of the section.

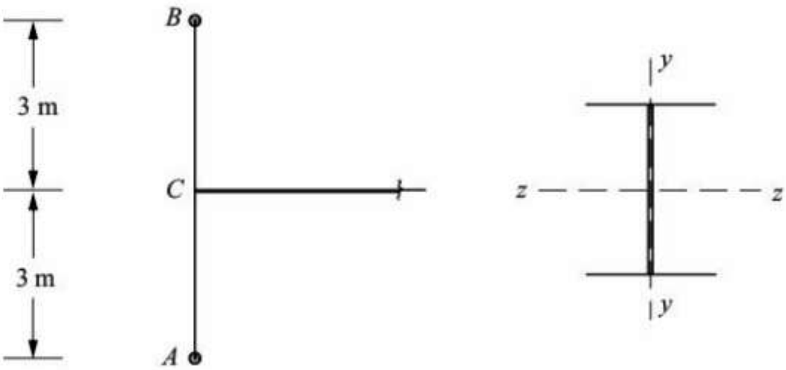
Thus, slenderness ratio = $\frac{l_e}{r} = \frac{KL}{r}$

where, L = actual length of compression member

l_e = KL, effective length

r = appropriate radius of gyration.

5.2.1 Actual Length: It is the centre-to-centre distance of the compression member between the restrained ends. In figure below, 6 m column is restrained at ends A and B in both y-y and z-z direction. At C it is restrained in z-z direction only. Hence its actual length in y-y direction is 6 m while in z-z direction it is equal to AC = 3 m only.



5.2.2 Effective Length:

The effective length KL is calculated from the actual length L, of the member considering the rotational and relative translational boundary conditions at the ends. IS 800-2007 recommends the following:

(a) If end conditions can be assessed:

Where the boundary conditions in plane of buckling can be assessed the effective length KL can be calculated on the bases of the Table below (Refer to Table 11 in IS 800

Effective length of prismatic compression members [Refer to Table 11 in IS 800]

Boundary Conditions				Schematic Representation	Effective Length
At One End		At the Other End			
Translation (1)	Rotation (2)	Translation (3)	Rotation (4)	(5)	(6)
Restrained	Restrained	Free	Free		2.0*L*
Restrained	Free	Free	Restrained		
Restrained	Free	Restrained	Free		1.0*L*

Restrained	Restrained	Free	Restrained		1.2*L*
Restrained	Restrained	Restrained	Free		0.8*L*
Restrained	Restrained	Restrained	Restrained		0.65*L*

Note: L is the unsupported length of the compression member.

(**b) Compression members in trusses**:

(i) In the case of bolted, riveted or welded trusses and braced frames, the effective length, KL, shall be taken as 0.7 to 1.0 times the actual length, depending upon the degree of end restraints provided.

(ii) For buckling in the plane perpendicular to the plane of truss, the effective length may be taken as actual length.

(c) In frames:

In the frame analysis, if deformed shape is not considered (second order or advanced analysis is not used), the effective length depends upon stiffnesses of the members meeting at the joint. The method of finding effective length factor K are shown in Annex D of IS 800. One can use the graphs given in the annexure.

(d) In case of stepped columns:

Expressions for finding effective length factor for various stepped columns are presented in IS 800 annexure D2 and D3.

5.2.3 Appropriate Radius of Gyration:

Appropriate radius of gyration means the radius of gyration of compression member about the axis of buckling. **For example**, in case of column shown in above figure, when length of the column is taken 6 m, the radius of gyration about z-z axis should be considered. For buckling about y-y axis, the length of column is 3 m and radius of gyration about y-y axis is to be considered. The maximum slenderness ratio governs the design strength. If the length of the column to be considered is the same for buckling about any axis, naturally the governing slenderness ratio is

$$\frac{KL}{r_{min}}$$

.

DESIGN OF COMPRESSION MEMBERS

Learning objectives:

5.3. Design of compressive stress and strength.
5.4. Table for design stress.

5.3 DESIGN COMPRESSIVE STRESS AND STRENGTH

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

$$f_{cd} = \frac{f_y / \gamma_{mo}}{\phi + \left(\phi^2 - \lambda^2\right)^{0.5}} \leq \frac{f_y}{\gamma_{mo}}$$

where φ = 0.5 [1 + α (λ − 0.2) + λ^2]
λ = non-dimensional effective slenderness ratio

$$= \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{f_y \left(\frac{KL}{r}\right)^2}{\pi^2 E}}$$

f_{cc} = Euler buckling stress
α = imperfection factor, given in Table below.
γ_{mo} = 1.1 for Fe 415 steel.

The design compressive strength P_d of a member is given by

$$P_d = A_e f_{cd}$$

where A_e effective sectional area, which is the same as gross area if bolt holes are filled with bolts. Deductions for bolt holes may be made only if the holes are not fitted with bolts.

Imperfection factor α

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

Problem 5.1

Determine the design axial load capacity of the column ISHB 300 @ 577 N/m if the length of column is 3 m and its both ends pinned.

Solution:

For rolled steel sections,
f_y = 250 N/mm2 , f_u = 410 N/mm2 and E = 2 × 10 5 N/mm2 .
For both ends pinned columns, KL = L = 3 mm.
For ISHB 300 @ 577 N/m. h = 300 mm, b_f = 250 mm, t_f = 10.6 mm, Ae = A = 7484 mm2

$$\therefore \quad \frac{h}{b_f} = 1.2$$

And t_f < 40 mm.

Hence according to Table 10 in IS 800 it falls under buckling class 'b' for buckling about z-z axis and under class 'c' for buckling about y-y axis. From steel table $r_{min} = r_{yy}$ = 54.1 mm.

$$\therefore \quad f_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 \times 2 \times 10^5}{\left(\frac{3000}{54.1}\right)^2} = 641.92 \text{ N/mm}^2$$

Non-dimensionalised effective slenderness ratio

$$= \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{641.92}} = 0.624$$

For buckling class b, α = 0.34.

$\therefore \varphi = 0.5\,[1 + \alpha\,(\lambda - 0.2) + \lambda^2]$

$= 0.5\,[1 + 0.34\,(0.624 - 0.2) + 0.624^2]$

$= 0.767$

$$\therefore \quad f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + \left(\phi^2 - \lambda^2\right)^{0.5}}$$

$$= \frac{250/1.1}{0.767 + \left(0.767^2 - 0624^2\right)^{0.5}}$$

$$= 187.36 \text{ N/mm}^2$$

$\therefore$ Strength of column $P_d = A_c f_{cd}$ = 7484 × 187.36 = 1402237 N = 1402.237 kN

$\therefore$ Working load = $\frac{1402.237}{1.5}$

= 934.823 kN Answer

5.4 I.S. TABLES FOR DESIGN STRESS: For the benefit of users tables are given in IS 800-2007 (Refer Table 9) to find design stress f_{cd} , if $\frac{KL}{r}$ is determined for all the four (a, b, c and d) classes of buckling. It may be verified that in the above problem $\frac{KL}{r} = \frac{3000}{51.8} = 57.9$ and from the Table 9(c) from IS 800-2007, f_{cd} = 171.4.

Design compressive stress, f_{cd} (MPa) for column buckling class a (Refer Table 9(a) in IS 800)

Design compressive stress, f_{cd} (MPa) for column buckling class b (Refer Table 9(b) in IS 800)

Design compressive stress, f_{cd} (MPa) for column buckling class c (Refer Table 9(c) in IS 800)

Design compressive stress, f_{cd} (MPa) for column buckling class d (Refer Table 9(d) in IS 800)

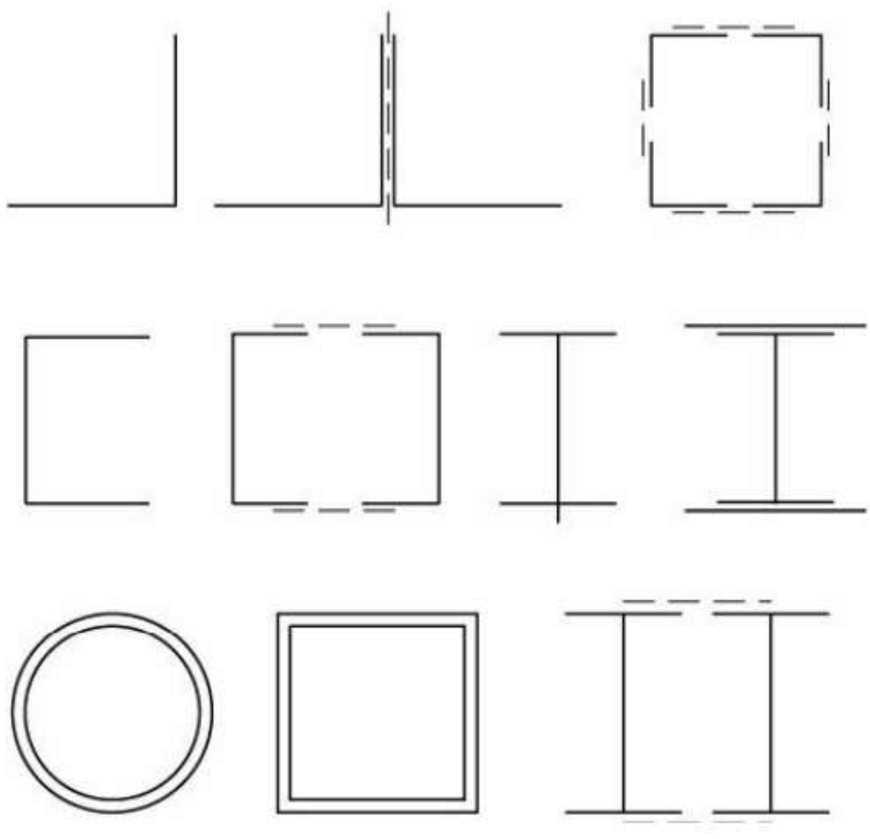
DESIGN OF COMPRESSION MEMBERS

Learning objectives:

5.5. Knowledge about shapes of compression members.
5.5.1. Numerical.

5.5 SHAPES OFCOMPRESSION MEMBERS:

Since the design stress in compression member decreases with the least radius of gyration, the section should be proportioned to have maximum moment of inertia for the same sectional area. This can be achieved by concentrating the area away from centroid of the section. As far as possible the section should have approximately the same radius of gyration about any axis. This requirement is fulfilled by circular tubes. Due to difficulties in making end connections, they were not commonly used earlier. But nowadays due to improvements in welding technology tubular sections are getting popularity as compression members. Next best shape may be square tubing. Among I -sections ISHB sections are preferable as columns, since they have better r_{min} values for the same area of cross-sections. If built up areas are required strengthening should be made by connecting plates on flanges so as to increase r_{zz} value which is lower compared to r_{yy} value. In roof trusses and transmission towers angle sections are commonly used. It is preferable to use equal angles instead of unequal angles as compression members since such angles have higher r_{min} values for the same cross sectional areas. Various shapes of commonly used compression members are shown in Figure below.



Problem 5.2

In a truss a strut 3 m long consists of two angles ISA 100100, 6 mm. Find the factored strength of the member if the angles are connected on both sides of 12 mm gusset by

(i) one bolt

(ii) two bolts

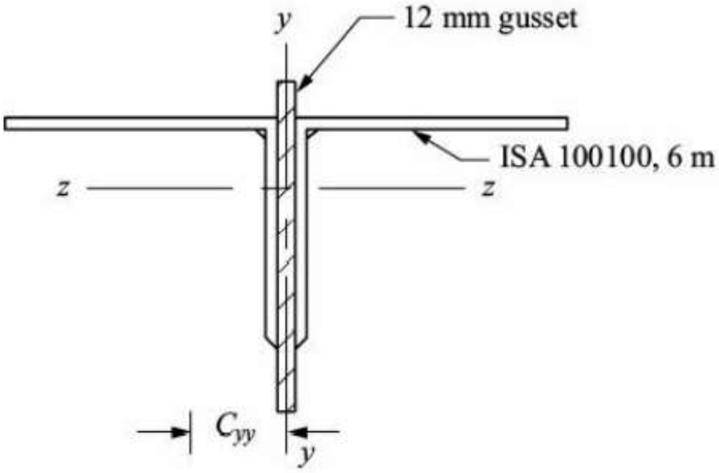
(iii) Welding, which makes the joint rigid.

Solution:

From steel table for an ISA 100100, 6 mm,

area = 1167 mm^2 ; $C_{zz} = C_{yy}$ = 26.7 mm. $r_{zz} = r_{yy}$ = 30.9 mm.

Figure below shows the details of the member.



r_{zz} of the member is the same as r_{zz} of single angle, since the z-z axis for both is the same, resulting into doubling of I_{zz} and area.

$\therefore r_{zz}$ = 30.9 mm.

$I_{yy} = 2$ [I_{yy} of one angle + Area of one angle × $(C_{yy} + 6)$ 2].

From steel table, I_{yy} of one angle = 111.3×10^4

I_{yy} of the member

$$= 2\left[111.3\times10^4 + 1167\times(26.7+6)^2\right]$$

$$= 4721723 \text{ mm}^4$$

$$\therefore \quad r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{4721723}{2\times1167}} = 44.98$$

$\therefore r_{zz}$ is governing the strength of member.

Case (i): When a single bolt is used

$r = r_{zz}$ = 30.9 mm KL = L = 3000 mm.

$$\therefore \frac{KL}{r} = \frac{3000}{30.9} = 97$$

The member belongs to buckling class c . Hence referring to Table 9(c) of IS code, for $\frac{KL}{r}$ =97,

corresponding to f_y = 250 MPa,

$$f_{cd} = 121 - \frac{7}{10}(121-107)$$

$$= 111.2 \text{ N/mm}^2 .$$

$\therefore P_d = A_e f_{cd} = 2 \times 1167 \times 111.2 = 259541$ N. i.e. Pd = 259.541 kN Answer

Case (ii): When two bolts are used

The effective length is reduced. It may be taken as 0.85 times actual length.

$\therefore$ KL = 0.85 × 3000 = 2550 mm.

Hence in this case

$$\frac{KL}{r} = \frac{2550}{30.9} = 82.5$$

From Table 9(c), for steel with f_y = 250 N/mm2

$$f_{cd} \quad \text{for} \quad \frac{KL}{r} = 80 \quad \text{is} \quad 136 \text{ N/mm}^2$$

$$\text{for} \quad \frac{KL}{r} = 90 \quad \text{is} \quad 121 \text{ N/mm}^2$$

$$\therefore \quad \text{Linearly interpolating, } f_{cd} \text{ for } \frac{KL}{r} = 82.5 \text{ is}$$

$$f_{cd} = 136 - \frac{2.5}{10} \times (136 - 121)$$

$$= 132.25 \text{ N/mm}^2$$

$$\therefore \quad P_d = 2 \times 1167 \times 132.25$$

$$= 308672 \text{ N} = 308.672 \text{ kN}$$ **Answer**

Case (iii): Rigid joint by welding

Effective length KL = 0.7 × L = 0.7 × 3000 = 2100 mm

$$\therefore \quad \frac{KL}{r} = \frac{2100}{30.9} = 67.96$$

From the table, f_{cd} values are

$$\text{for} \quad \frac{KL}{r} = 60 \quad f_{cd} = 168 \text{ N/mm}^2$$

$$\frac{KL}{r} = 70 \quad f_{cd} = 152 \text{ N/mm}^2$$

$$\therefore \quad \text{For} \quad \frac{KL}{r} = 67.96, \quad f_{cd} = 168 - \frac{7.96}{10}(168 - 152)$$

$$= 155.26 \text{ N/mm}^2$$

$\therefore P_d = 2 \times 1167 \times 155.26 = 362386$ N = 362.386 kN Answer

.

DESIGN OF COMPRESSION MEMBERS

Learning objectives:

5.6. Design of compression member.
5.7. To know about Laced and Battened columns.

5.6 DESIGN OFCOMPRESSION MEMBERS:

The following are the usual steps in the design of compression members:

1. Design stress in compression is to be assumed. For rolled steel beam sections the slenderness ratio varies from 70 to 90. Hence design stress may be assumed as 135 N/mm^2 . For angle struts, the slenderness ratio varies from 110 to 130. Hence design stress for such members may be assumed as 90 N/mm2 . For compression members carrying large loads, the slenderness ratio is comparatively small. For such members design stress may be assumed as 200 N/mm^2 .

2. Effective sectional area required is .

$$A = \frac{P_d}{f_{cd}} .$$

3. Select a section to give effective area required and calculate r_{min} .

4. Knowing the end conditions and deciding the type of connection determine effective length.

5. Find the slenderness ratio and hence design stress f_{cd} and load carrying capacity P_d .

6. Revise the section if calculated P_d differs considerably from the design load.

Thus, the design of compression member is by a trial-and-error process.

Problem5.3

Design a single angle strut connected to the gusset plate to carry 180 kN factored load. The length of the strut between centre-to-centre connection is 3 m.

Solution:

Assuming f_{cd} = 90 N/mm^2 ,

$$A = \frac{180 \times 10^3}{90} = 2000 \text{ mm}^2$$

Try ISA 9090, 12 mm, which has A = 2019 mm2

$r_{min} = r_{vv}$ = 17.4 mm.

Assuming the strut will be connected to the gusset plate with at least 2 bolts (Note: Strength of 20 mm bolt in single shear is about 45 kN)

KL = 0.85L = 0.85 × 3000 = 2550 mm

$$\therefore \quad \frac{KL}{r} = \frac{2550}{17.4} = 146.55$$

From the Table 9 (c),

for fy = 250 N/mm^2

when $\frac{KL}{r}$ =140 f_{cd} = 60.2

when $\frac{KL}{r}$ = 150 f_{cd} = 59.2

∴ when$\frac{KL}{r}$ = 146.55

$f_{cd} = 58.4 - \frac{6.55}{10}(58.4 - 52.6) = 54.6$ N/mm^2

∴ Pd = A f_{cd} = 2019 × 54.6 = 110239 < 180000 N

Hence revise the section.

Try ISA 130130, 8 mm.

Area provided = 2022 mm^2 , r = r_{vv} = 25.5

∴ $\frac{KL}{r} = \frac{2550}{25.5}$ = 100

∴ f_{cd} = 107 N/mm2 ∴ P_{cd} = 2022 × 107 = 216354 > 180,000 N. **O.K.**

Provide ISA 130130, 8 mm

5.7 LACED AND BATTENED COLUMNS:

To achieve maximum value for minimum radius of gyration, without increasing the area of the section, a number of elements are placed away from the principal axis using suitable lateral systems. The commonly used lateral systems are

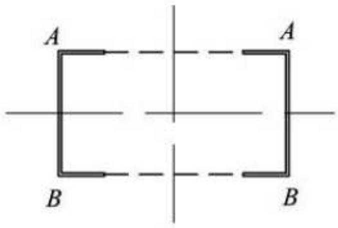
(a) lacing or latticing

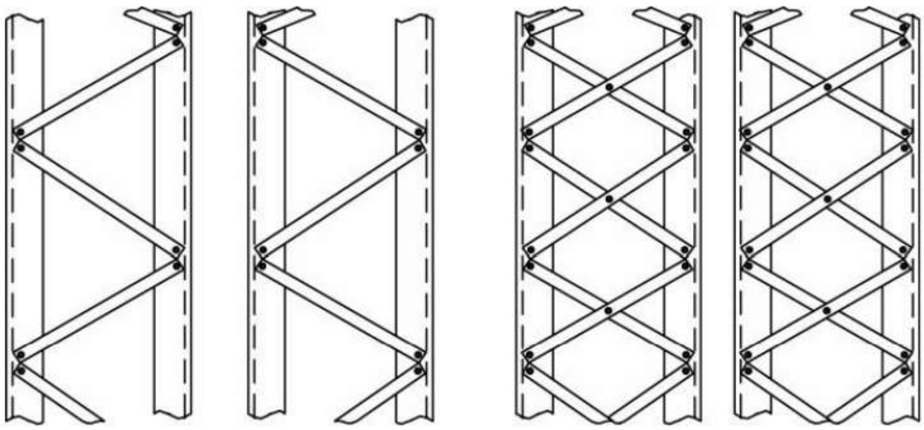
(b) battening.

Perforated cover plates are also used for this purpose. However, IS 800 do not give any specifications for the design of such plates.

5.7.1 Lacings

Rolled steel flats and angles are used for lacing. One can use single lacing or double lacing system





Lacing on Face A | Lacing on Face B | Lacing on Face A | Lacing on Face B

Single laced system **doubled laced system**

The object of providing lateral system is to keep the main members of the column away from principal ones. In doing so, the lacings are subjected to shear forces due to horizontal forces on columns.

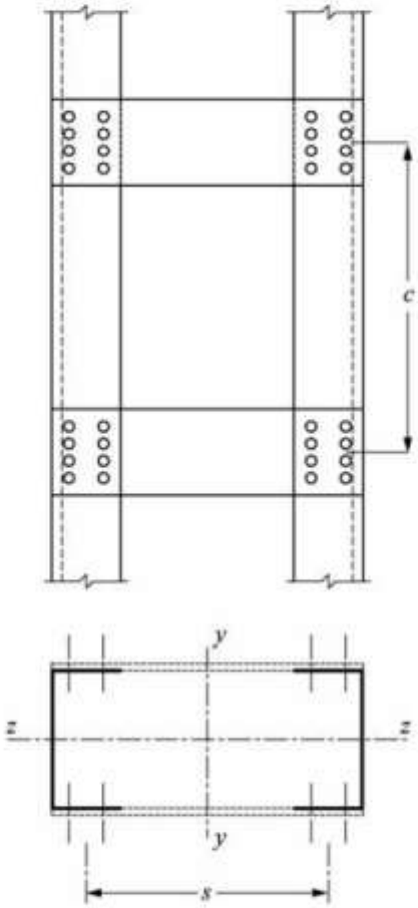
. **DESIGN OF COMPRESSION MEMBERS**

Learning objectives:

5.7.2 To know about Battened columns.
5.8. Design of laced columns.

5.7.2 Battens

Instead of lacing one can use battens to keep members of columns at required distances. Figure below shows the use of batten plates.



5.8 DESIGN OF LACED COLUMNS:

IS 800-2007 specifies the following rules for the design of latticed columns:

1. As far as possible, the latticing system shall be uniform throughout.

2. In single laced system the direction of lattices on opposite faces should be shadow of the other. It should not be mutually opposite.

3. In bolted / riveted construction, the minimum width of lacing bars shall be three times the nominal diameter of the bolt / rivet.

4. The thickness of flat lacing bars shall not be less than $\frac{1}{40}$ th of its effective length for single lacing and $\frac{1}{16}$ th of the effective length for double lacings.

5. Lacing bars shall be inclined at 40° to 70° to the axis of built-up member.

6. The distance between the two main members should be kept so as to get $r_{yy} > r_{zz}$ where, r_{yy} is the radius of gyration about weaker axis and r_{zz} is the radius of gyration of stronger axis of individual member.

7. Maximum spacing of lacing bars shall be such that the maximum slenderness of the main member between consecutive lacing connection is not greater than 50 or 0.7 times the most unfourable slenderness ratio of the member as a whole.

8. The lacing shall be designed to resist transverse shear Vt = 2.5% of axial force in columns. If there are two transverse parallel systems then each system has to resist $\frac{Vt}{2}$ shear force.

9. If the column is subjected to bending also, Vt = bending shear + 2.5% column force.

10. Effective length of single laced system is equal to the length between the inner end fastener. For welded joints and double laced, effectively connected at intersection effective length may be taken as 0.7 times the actual length.

11. The slenderness ratio $\frac{KL}{r}$ for lacing bars should not exceed 145.

12. Laced compression members shall be provided with end tie plates.

13. The effective slenderness ratio of laced columns shall be taken as 1.05 times the actual maximum slenderness ratio, in order to account for shear deformation effects.

Problem-5.4

Design a laced column with two channels back to back of length 10 m to carry an axial factored load of 1400 kN. The column may be assumed to have restrained in position but not in direction at both ends (hinged ends).

Solution:

Assuming f_{cd} = 135 N/mm2

Area required = $\frac{1400 \times 1000}{135}$ = 10370mm^2

Try 2 ISMC 350 @ 413 N/m.

Area provided = 2 × 5366 = 10732 mm^2 $\quad r_{zz}$ = 136.6 mm

Distance will be maintained so as to get $r_{yy} > r_{zz}$.

$$\therefore \text{ Actual } \frac{KL}{r} = \frac{1 \times 10000}{136.6} = 73.206$$

Since it is a laced column

$$\frac{KL}{r} = 1.05 \times 73.206 = 76.87$$

From Table 9(c)

$$f_{cd} = 152 - \frac{6.87}{10}(152 - 136)$$

$$= 141.0 \text{ N/mm}^2$$

Load carrying capacity = 10732 × 141.0
= 1513.297 × 10 3
= 1513.297 kN > 1400 kN O.K.

Spacing between the channels:

Let it be a clear distance 'd',

Now: I xx = 2 × 10008 × 10 4 = 20016 × 10 4 mm^4

$$I_{yy} = 2\left[430.6\times10^4 + 5366\left(\frac{d}{2}+24.4\right)^2\right]$$

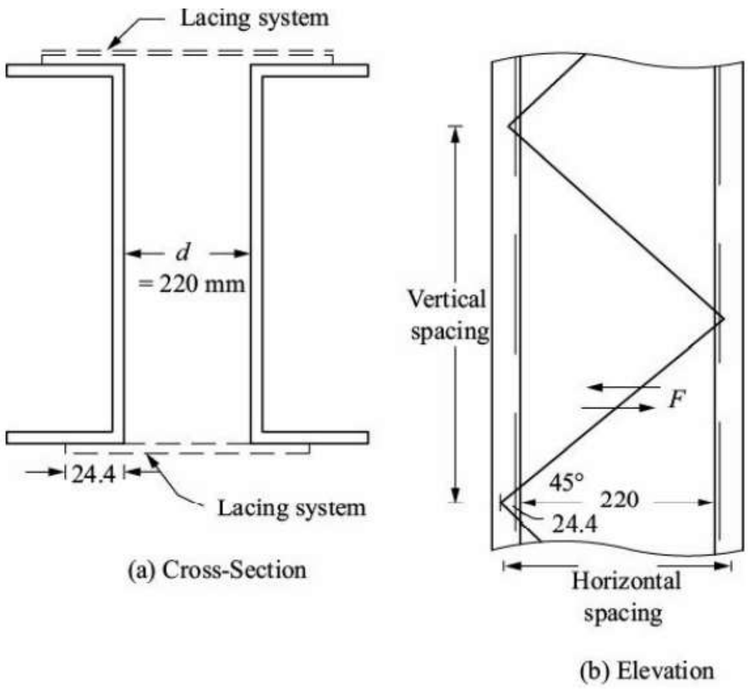
Equating I yy to I xx, we get

$$2\left[430.6\times10^4 + 5366\left(\frac{d}{2}+24.4\right)^2\right] = 20016\times10^4$$

$$\text{i.e.,}\quad \left(\frac{d}{2}+24.4\right)^2 = 17848.3$$

∴ d = 218.4 mm

Provide d = 220 mm as shown in Figure below.



(a) Cross-Section

(b) Elevation

Lacings:

Let the lacings be provided at 45° to horizontal.

Horizontal spacing of lacing = 220 + 60 + 60 = 340 mm

[Note: g = 60 is gauge distance]

∴ Vertical spacing = 340 tan 45° × 2 = 680 mm

Least r of one channel = r_{yy} = 28.3

∴ $\frac{KL}{r}$ of channel between lacing= $\frac{680}{28.3} = 24.03 < 50$ O.K.

Transverse shear to be resisted by lacing systems $=\frac{2.5}{100} \times 1400 \times 10^3$=3500N
Shear to be resisted by each lacing systems $=\frac{35000}{2}$ = 17500N
Length of lacing = (220 + 60 + 60) $\frac{1}{cos45}$ = 480.83mm.
Minimum thickness of lacing $=\frac{1}{40}$ × 480.83 = 12.02 mm.
Use 14 mm flats
Minimum width of lacing, if 20 mm bolts are used = 3 × 20 = 60 mm.
Use 60 ISF 14
Sectional area = 60 × 14 = 840 mm^2 .

$$r_{\min} = \sqrt{\frac{\frac{1}{12}\times 60\times 14^3}{60\times 14}} = 4.041 \text{ mm}$$

$\therefore \frac{KL}{r} = \frac{480.83}{4.041}$ = 118.97 < 145 OK

Strength of 20 mm shop bolt:
(a) in single shear

$$= 0.78\times\frac{\pi}{4}\times 20^2 \times \frac{400}{\sqrt{3}\times 1.25} = 45272 \text{ N}$$

Edge distance = $\frac{60}{2}$ =30

$\therefore K_b = \frac{30}{3\times 22}$ = 0.4545

(b) Strength in bearing = $\frac{2.5\, Kb\, dt\, fu}{1.25}$

$= \frac{2.5 \times 0.4545\times 20\times 10\times 400}{1.25}$
= 101808 N

∴ Bolt value = 45272 N.
Number of bolt required = $\frac{17500}{45272}$ = 0.387
Provide one bolt.

Check for the strength of lacing:
$\frac{KL}{r}$ = 118.97
A flat belongs to bucking class c.

$\therefore f_{cd} = 94.6 - \frac{8.97}{10}(94.6 - 83.7) = 84.82$ N/mm^2
Load carrying capacity in compression = 84.82 × 60 × 14 = 71251 N

Force in lacing= $\frac{17500}{sine\ 45}$ = 24749N < 71251 N
∴ Safe.
Hence provide 60 ISF 14 flats at 45^o and connect them to centre of gravity of channels with one bolt of 20 mm nominal diameter.

DESIGN OF COMPRESSION MEMBERS

Learning objectives:

5.9. Design of Battened columns.
5.10. Design of laced columns.

5.9 DESIGN OF BATTENED COLUMNS:

IS 800-2007 specifies the following rules for the design of battened columns:
1. Batten plates should be provided symmetrically.

2. At both ends batten plates should be provided. They should be provided at points where the member is stayed in its length.

3. The number of battens should be such that the member is divided into not less than three bays. As far as possible they should be spaced and proportioned uniformly throughout.

4. Battens shall be of plates, angles, channels, or I-sections and at their ends shall be riveted, bolted or welded.

5. By providing battens distance between the members of columns is so maintained that radius of gyration about the axis perpendicular to the plane of battens is not less than the radius of gyration about the axis parallel to the plane of the batten ($r_{yy} > r_{xx}$, in Fig. 6.6).

6. The effective slenderness ratio of battened columns shall be taken as 1.1 times the maximum actual slenderness ratio of the column, to account for shear deformation.

7. The vertical spacing of battens, measured as centre to centre of its end fastening, shall be such that the slenderness ratio of any component of column over that distance shall be neither greater than 50 nor greater than 0.7 times the slenderness ratio of the member as a whole about its z-z axis.

8. Battens shall be designed to carry the bending moments and shear forces arising from transverse shear force Vt equal to 2.5% of the total axial force.

9. In case columns are subjected to moments also, the resulting shear force should be found and then the design shear is sum of this shear and 2.5% of axial load.

10. The design shear and moments for batten plates is given by

$$V_b = \frac{V_t C}{NS} \quad \text{and} \quad M = \frac{V_t C}{2N}$$

at each connection.

where, Vt = transverse shear force as defined in 8 and 9.
C = distance between centre to centre of battens longitudinally.
N = number of parallel planes.
S = minimum transverse distance between the centroid of the fasteners connecting batten to the main member.

11. The effective depth of end battens (longitudinally), shall not be less than the distance between the centroids of main members.

12. Effective depth of intermediate battens shall not be less than ¾ th of above distance.

13. In no case the width of battens shall be less than twice the width of one member in the plane of the batten. It is to be noted that the effective depth of a batten shall be taken as the longitudinal distance between the outermost fasteners.

14. The thickness of battens shall be not less than $\frac{1}{50}$ th of the distance between the innermost connecting lines of rivets, bolts or welds.

15. The length of the weld connecting batten plate to the member shall not be less than half the depth of batten plate.

Problem 5.5

Design the built-up column for problem 5.4 using battens instead of the lacing system.

Solution:

The design of the column is the same as in the previous example
i.e., use 2ISMC 350 @ 413 N/m
with a clear spacing of 220 mm. $\frac{KL}{r} = 1.1\times\frac{10000}{136.6} = 80.52$

Distance between centres of channels S = 220 + 60 + 60 = 340 mm

Design of battens:

Let C be the spacing of battens, longitudinally.
Radius of gyration of one channel = 28.3 mm

$\therefore \frac{C}{28.3} < 50$ i.e., C < 1415.
It should also satisfy the condition,

$\therefore \frac{C}{28.3} < 0.7\times 80.52$ i.e., C < 1595.
Let us select C = 1200 mm.

$$V_t = \frac{2.5}{100}\times 1400\times 10^3 = 35000 \text{ N}$$

$$\therefore V_b = \frac{V_t C}{NS} = \frac{35000\times 1200}{2\times 340} = 61765 \text{ N}$$

$$M = \frac{V_t C}{2N} = \frac{35000\times 1200}{2\times 2} = 10500000 \text{ N-mm}$$

∴ Size of battens:

Effective depth of end batten $\nless$ 268.8 mm and also $\nless$ 2 × 100 mm.

∴ Provide 270 mm depth for end battens, overall depth = 270 + 2 × 35 = 340 mm.

For intermediate battens it is $\nless \frac{3}{4} \times 270$mm and $\nless 200$mm
Provide depth = 210 mm

Giving edge distance of 35 mm, Overall depth = 210 + 2 × 35 = 280 mm

Thickness of battens $\nless \frac{1}{50} \times 340$
$\nless 6.8$
Use 8 mm thick plates.

Check for stresses in batten plates:

Shear stress

$$= \frac{61765}{280 \times 8} = 27.57 \text{ N/mm}^2 < \frac{f_y}{\sqrt{3}} \times \frac{1}{1.1}$$

Shear stress < 0.6 × permissible stress

Bending stress

$$= \frac{6M}{td^2} = \frac{6 \times 10500000}{8 \times 280^2} < \frac{f_y}{1.1} \times 1.2$$

= 100.45 < 227.27 N/mm2 O.K.
Obviously end plate satisfies these requirements since it is deeper.

Connections:

It is to be designed to transmit both shear and bending moment.
Using 20 mm bolts,

Strength in single shear

$$= 0.78 \frac{\pi}{4} \times 20^2 \times \frac{400}{\sqrt{3} \times 1.25} = 45272 \text{ N}$$

Strength in bearing is much higher.

∴ Bolt value = 45272 N.

Number of bolts required $= \frac{78125}{45272} = 1.72$
Let us provide 3 bolts to take into account stresses due to bending also.

Check:
Force in each bolt due to shear $= \frac{78125}{3} = 26042N$
Let the pitch be $\frac{210}{2}$ =105mm.
Force due to moment in extreme bolt $= \frac{Mr}{\sum r2}$

$$= \frac{10500000 \times 105}{105^2 + 105^2} = 50000$$

Resultant force in extreme bolt $=\sqrt{26042^2+50000^2} > 45272N$

Try 5 bolts as shown in Fig. below.

Force in each bolt due to shear $=\frac{78125}{5}= 15625$N

Force due to moment in extreme bolt

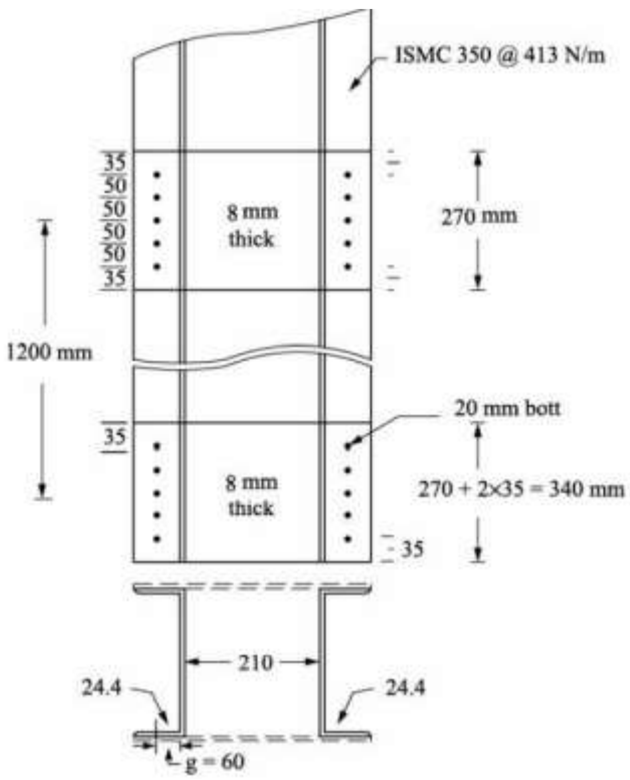
$$=\frac{Mr}{\Sigma r^2}=\frac{10500000\times 100}{2\left(50^2+100^2\right)}=42000 \text{ N}$$

∴ Resultant force

$$=\sqrt{15625^2+42000^2}$$

$$= 44812 \text{ N} < 45272 \text{ N}$$

Provide the bolts as shown in Fig. below.



DESIGN OF COMPRESSION MEMBERS

Learning objectives:

5.11. Column bases.
5.12. Design of slab base.

5.11. COLUMN BASES:

Column bases transmit the column load to the concrete or masonry foundation blocks. The column base spreads the load on wider area so that the intensity of bearing pressure on the foundation block is within the bearing strength.

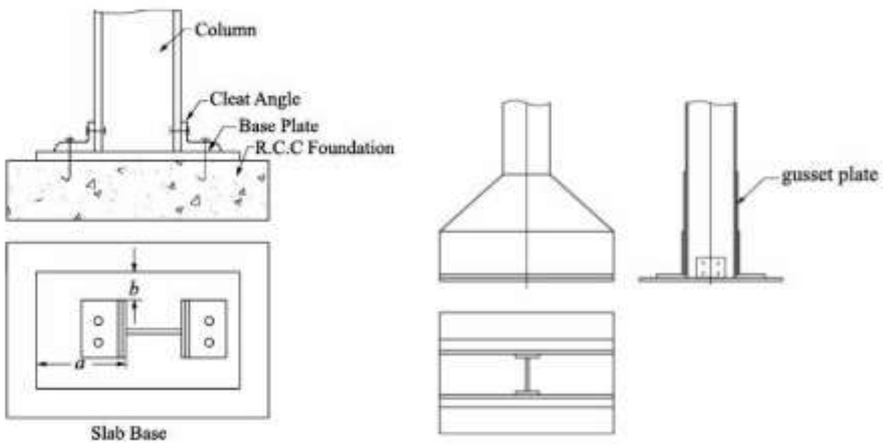
There are two types of column bases commonly used in practice:
1. Slab Base
2. Gusseted Base.

5.11.1 Slab Base

These are used in columns carrying small loads. In this type, the column is directly connected to the base plate through cleat angles as shown in Fig. below. The load is transferred to the base plate through bearing.

5.11.2 Gusseted Base

For columns carrying heavy loads gusseted bases are used. In gusseted base, the column is connected to base plate through gussets. The load is transferred to the base partly through bearing and partly through gussets. Figure below shows a typical gusseted base connection.



Slab Base

5.12.DESIGN OF SLAB BASE

The design of slab base consists in finding the size and thickness of slab base. In the procedure given below it is assumed that the pressure is distributed uniformly under the slab base.

Size of Base plate:

(1) Find the bearing strength of concrete which is given by = 0.45 f_{ck} .

(2) Therefore, area of base plate required $=\frac{Pu}{0.45\ fck}$, where Pu is factored load.

(3) Select the size of base plate. For economy, as far as possible keep the projections a and b equal.

Thickness of Base Plate:

(1) Find the intensity of pressure

$$w = \frac{P_u}{\text{Area of base plate}}$$

(2) Minimum thickness required is given by

$$t_s = \left[\frac{2.5w\left(a^2 - 0.36^2\right)\gamma_{mo}}{f_y}\right]^{0.5} > t_f$$

where t_s = thickness of base plate and t_f = thickness of flange.

The above formula may be derived by taking μ = 0.3 and using plate theory for finding bending moment.

Connections:

(1) Connect base plate to foundation concrete using four 20 mm diameter and 300 mm long anchor bolts.
(2) If bolted connection is to be used for connecting column to base plate, use 2 ISA 6565, 6 mm thick angles with 20 mm bolts.
(3) If weld is to be used for connecting column to base check the weld length of fillet welds.

Problem 5.6.

Design a slab base for a column ISHB 300 @ 577 N/m carrying an axial factored load of 1000 kN. M20 concrete is used for the foundation. Provide welded connection between column and base plate.

Solution:

Bearing strength of concrete = 0.45 f_{ck} = 0.45 × 20 = 9 N/mm^2

Factored load P_u = 1000 KN.

∴ Area of base plate required $= \frac{1000\times10^3}{9}$

$= 111111$ mm^2

Provide 360 × 310 size plate.

Area provided = 360 × 310 = 111600 mm^2.

Pressure $= \frac{1000\times10^3}{111600}$ = 8.96N/mm^2

Projections are

$$a = \frac{360-300}{2} = 30 \text{ mm}$$

$$b = \frac{310-250}{2} = 30 \text{ mm}$$

$$\therefore \quad t_s = \left[\frac{2.5\times8.96\left(30^2 - 0.3\times30^2\right)\times1.1}{250}\right]^{0.5}$$

= 7.88 mm.

Thickness of flange of ISHB 300 @ 577 N/m is 10.6 mm.

Provide 12 mm thick plate.

Connecting 360 × 310 × 12 mm plate to concrete foundation:

Use 4 bolts of 20 mm diameter 300 mm long to anchor the plate.

Welds:
Properly machined column is to be connected to base plate using fillet weld.

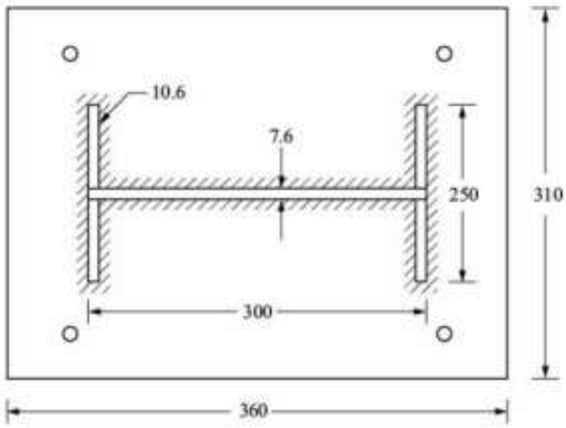
Total length available for welding (Ref. Fig. below)

= 2(250 + 250 – 7.6 + 300 – 2 × 10.6) = 1542.4 mm.

Strength of weld $= \frac{410}{\sqrt{3}} \times \frac{1}{1.25} = 189.37 N/mm2$

Let ‘s’ be the size of weld. Then effective area of weld = 0.7 s L_e
where L_e is effective length.

∴ The design condition is 0.7 s L_e × 189.37 = 1000 × 10^3
s L_e = 7543.8
Using 6 mm weld, Le = 1257 mm.



After deducting for end return of the weld at the rate of twice the size of the weld at each end.

Available effective length = 1542.4 – 2 × 6 × No. of returns
= 1542.4 – 2 × 6 × 12
= 1398.4 > 1257 mm.
Hence 6 mm weld is adequate.

.

DESIGN OF COMPRESSION MEMBERS

Learning objectives:

5.13. Design of gusseted base.
5.12. Numericals.

5.13.DESIGN OF GUSSETED BASE:

IS 800-2007 specifies that the gusset plates, angle cleats, stiffeners and fastenings etc., in combination with the bearing area, shall be sufficient to take the loads, bending moments and reactions to the base plate without exceeding specified strength. All the bearing surfaces shall be machined to ensure perfect contact.

The following design procedure may be followed:

1. Area of base plate

$$= \frac{\text{Factored Load}}{0.45\, f_{ck}}$$

2. Assume various members of gusset base.
 (a) Thickness of gusset plate is assumed as 16 mm.
 (b) Size of the gusset angle is assumed such that its vertical leg can accommodate two bolts in one vertical line. Corresponding to this leg the other leg is assumed in which one bolt can be provided.
 (c) Thickness of angle is kept approximately equal to the thickness of gusset plate.
3. Width of gusset base is kept such that it will just project outside the gusset angle and hence length.

$$= \frac{\text{Area of plate}}{\text{width}}.$$

4. When the end of the column is machined for complete bearing on the base plate, 50 percent of the load is assumed to be transferred by the bearing and 50 percent by the fastenings. When the ends of the column shaft and gusset plates are not faced for complete bearing, the fastenings connecting them to the base plate shall be designed to transmit all the forces to which the base is subjected.
5. The thickness of the base plate is computed by flexural strength at the critical sections.

Problem.5.7.

Design a gusseted base for a column ISHB 350 @ 710 N/m with two plates 450 mm × 20 mm carrying a factored load of 3600 KN. The column is to be supported on concrete pedestal to be built with M20 concrete.

Solution:

$f_{ck} = 20$ N/mm^2

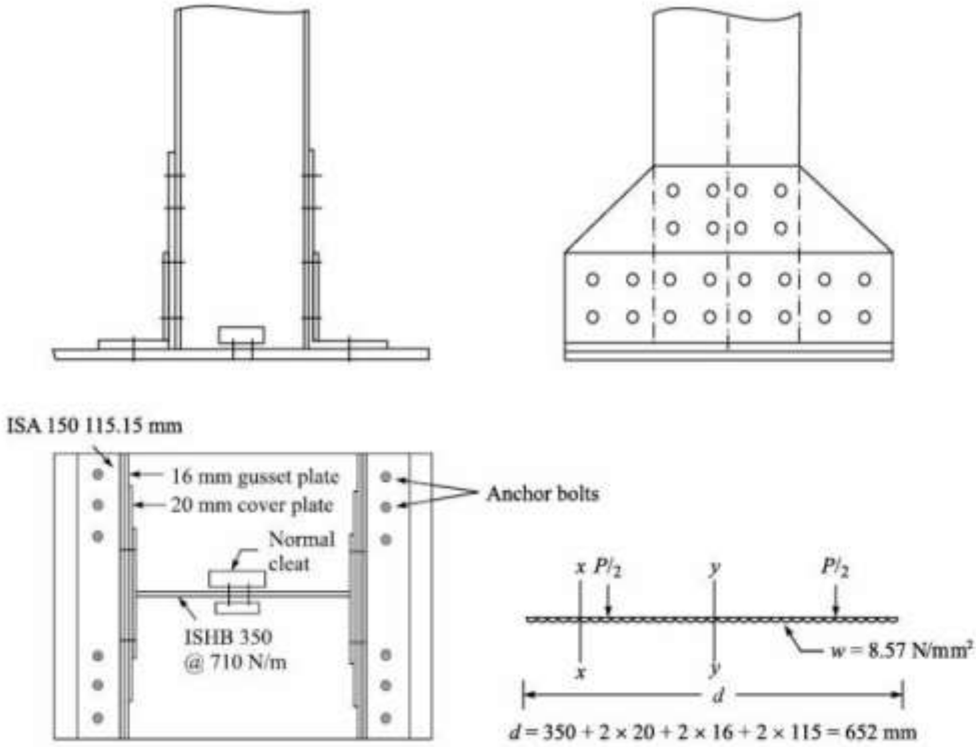
$$A = \frac{P_u}{0.45 f_{ck}} = \frac{3600\times10^3}{0.45\times 20} = 400000 \text{ mm}^2$$

Selecting ISA 150115, 15 mm angle and 16 mm thick gusset plate (Fig. below).

Minimum width required = 350 + 2 × 20 + 2 × 16 + 2 × 115 = 652 mm.

Use 700 mm wide plate.

$\therefore$ Length of base plate $=\frac{400000}{700} = 571mm.$



Provide 700 × 600 mm plate.

Pressure under base plate

$$= \frac{3600 \times 10^3}{700 \times 600} = 8.57 \text{ N/mm}^2$$

$$a = \frac{700 - (350 + 20 \times 2 + 16 \times 2 + 2 \times 15)}{2} = 124 \text{ mm}$$

BM at section X-X per mm width

$$= 8.57 \times \frac{124^2}{2} = 65886 \text{ N-mm}$$

At section Y-Y, bending moment

[Note: Per mm width P = 8.57 × 350 N]

$$M_{yy} = 8.57 \times \frac{350^2}{2} - \frac{700}{2} \times 8.57 \times \left(\frac{350}{2} + 20 + \frac{16+15}{2}\right)$$

= – 106482 N-mm

$\therefore$ Design moment = 106482 N-mm (tension at top).

Bending strength

$$= \frac{f_y}{\gamma_{mo}} = \frac{250}{1.1} = 227.27 \text{ N/mm}^2$$

Equating moment of resistance to bending moment we get,

$$1.2 \times \frac{1}{6} \times 1 \times t^2 \times 227.27 = 106482$$

∴ t = 48.4 mm.

∴ Use 56 mm base plate of size 700 × 600 mm.

Assuming ends of columns are faced for complete bearing,
the connection between gusset plate and column will be designed for 50 percent of axial load.

Design load = 0.5 × 3600 = 1800 KN.

Load on each splice = $\frac{1800}{2} = 900KN$
Using 24 mm shop bolts,

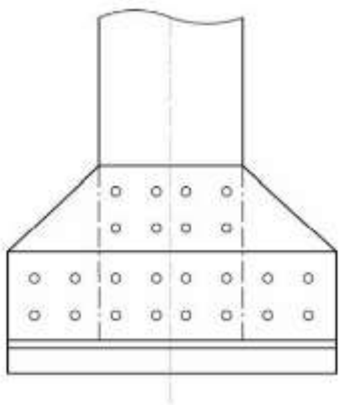
Strength of bolt in single shear

$$= 0.78 \times \frac{\pi}{4} \times 24^2 \times \frac{400}{\sqrt{3}} \times \frac{1}{1.25}$$

= 65192 N.

Strength in bearing is higher.
∴ Bolt value = 65192 N.
∴ No. of bolts required $= \frac{900 \times 10^3}{65192} = 13.8$

Provide 16 bolts as shown as shown in figure below, for connecting column to gusset plate.
Use another 8 bolts to connect cleat angle to gusset plate.



SHORT TYPE QUESTIONS AND ANSWERS:

1. A compression member may fail due to buckling or crushing?
2. What do you mean by slenderness ratio?
3. What is radius of gyration?
4. What do you mean by effective length of column?
5. Which sections are used for lacing?
6. What is column splice?
7. What are the types of bases used in column?
8. The thickness of the base plate is determined from what?
9. Anchor bolts are provided in column bases to resist what type of force?
10. Minimum no of battens required to built- up column are ------.

ANSWERS:

1. Buckling.
2. Slenderness ratio of a column is defined as the ratio of effective length to the corresponding radius of gyration of the section.
$$\lambda = \frac{Le}{r} = \frac{KL}{r}$$
3. It is the distance axis of rotation to a point where the total body is supposed to concentrate.
$$\kappa = \sqrt{\frac{I}{A}}$$
4. The effective length KL is calculated from the actual length L of the member considering the rotational and relative translational boundary conditions at the ends.

.

DESIGN OF BEAMS

Learning objectives:

6.*1. Introduction.*
6.2. Plastic moment.

Beam is a structural member with length considerably larger than cross sectional dimensions subject to lateral loads which give rise to bending moment shear forces in the member. Purlins which rest between the trusses and support roof sheets are beams. For this, angles or channels are commonly used. T-sections are used in water tanks to support steel plates. In buildings, I-sections are commonly used as beams. For heavier loads I-sections with additional plates connected on flanges are used. If still heavier sections are required built up sections like plate girders are used.

Based on the lateral supports to compression flanges there are mainly two types of beams viz.,

(a) Laterally supported beams and

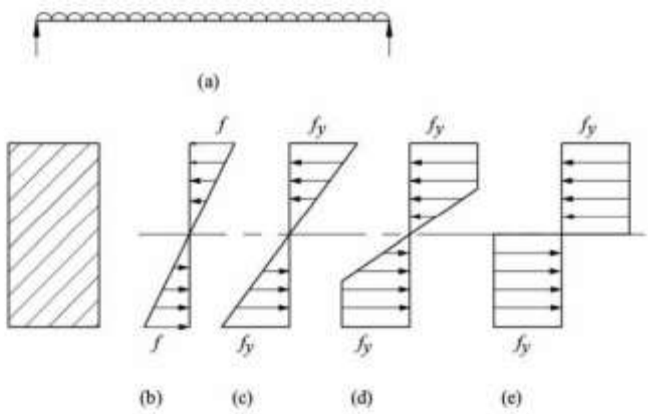
(b) Laterally unsupported beams.

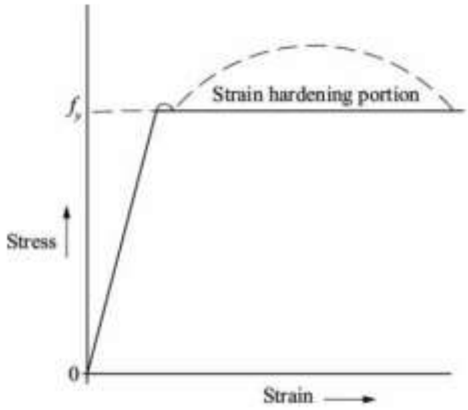
If the compression flanges are laterally supported by flooring, it is mainly subjected to bending and shear. If the compression flange of beam is not laterally supported, the lateral buckling of the compression flange reduces the load carrying capacity of the beam.

In this chapter the design of both type of beams is presented based on limit state consideration as recommended by IS 800-2007.

6.2. PLASTIC MOMENT CARRYING CAPACITYOFA SECTION:

Consider the cross section of a simply supported beam where the bending moment is maximum for the given loading. Within the elastic limit the stress varies linearly from compression to tension as shown in Figure below. As the load is gradually increased stresses increase proportionately till extreme fibre is subjected to yield stress. Then extreme fibre yields as in figure below. For simplicity of analysis, stress strain for steel is assumed as shown in below in which strain hardening part of the curve is ignored and it is assumed that after yield point is reached fibre goes on yielding without resisting any additional load. Hence according to theory of plastic analysis highly stressed fibre once yields is not capable of resisting any moment. But interior fibres are not yet yielded and hence additional loads are resisted by unyielded portion of the section. As the load is gradually increased one by one fibre reach yield stress and stop resisting additional load. Figure below shows partially yielded case. However, resistance to load continues till all fibres are yielded as shown below. After this condition the section will not resist further moment due to increase in load. This condition when all fibres at a section yield is called formation of plastic hinge. After this stage the rotation at section will take place without resisting additional moment but the moment corresponding to yielding of all fibres is resisted. This moment capacity is called plastic moment capacity of the section and is denoted as Mp .





The expression of M_p of a section can be easily derived as given below:

Let the area of the section in compression be Ac , in tension be At

and total area A. Equating the horizontal forces for the equilibrium condition, we get

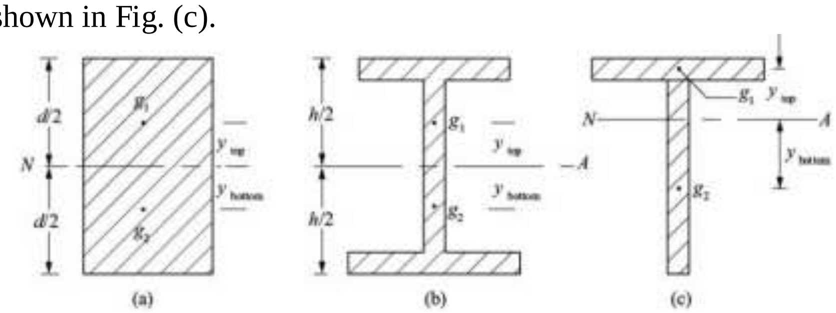
$$A_c f_y = A_t f_y$$

$$\therefore \quad A_c = A_t = \frac{A}{2}$$

Denoting the section where stress changes the sign as plastic neutral axis, we can conclude plastic neutral axis divides the total area into two equal parts. Obviously, such section is at mid depth for symmetric sections as shown in Fig. (a and b).
For unsymmetric sections it is to be found from the condition that.

$$A_c = A_t = \frac{A}{2}$$

This is shown in Fig. (c).



Plastic moment capacity may be found by taking moment of horizontal forces about plastic N-A. It may be noted that the total moment of resistance is additive of moment resisted by compressive forces and tensile forces, since the moments are having same sign (clockwise or anticlockwise).
Three examples are solved to illustrate the method of finding Mp and hence Zp

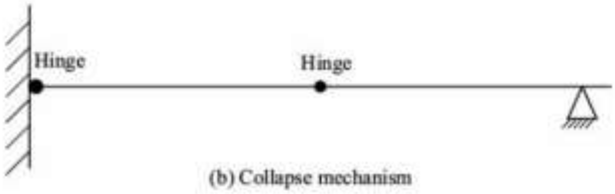
$$Mp = fy\ Zp\ .$$

For standard rolled sections it is found that Zp is of the order of 1.125 to 1.14 times Zxx for I sections and about 1.7 to 1.8 for channel sections. If these rolled sections are treated as the sections with rectangular parts, it is possible to determine Zp values but they will be slightly on higher side. Indian rolled steel sections consist of sloping flanges, fillets at junctions and rounded edges. The author and K. V. Promod considered all these complexities in the shapes and determined plastic modulus of rolled steel sections and standard builtup sections and have brought out Steel Tables published by IK International Publishing House.

It may be noted that due to formation of one hinge the beam need not fail in all cases. In case of determinate structures formation of first hinge itself causes collapse of structure, since in this case it goes on rotating without resisting additional load.

If we consider a propped cantilever highly stressed section is at fixed end and hence plastic hinge is first formed here. After the formation of first hinge beam behaves like simply supported beam and it resists further increase in load, till one more hinge is formed. After formation of this hinge the collapse mechanism is formed i.e., rotation of beam takes place without resisting any more load.

(a) Formation of first hinge



(b) Collapse mechanism

In case of fixed beam subject to symmetric loading, end hinges may form simultaneously. After that beam start acting as a simply supported beam for further load and fails only after one more hinge is formed in the middle portion. Figure below shows this case.

(a) Formation of earlier hinges

(b) Formation of collapse mechanism

A frame fails only after collapse mechanism is formed. This type of analysis, known as plastic analysis is usually taught to the students in a separate course.

.

DESIGN OF BEAMS

Learning objectives:

6.*3. Numerical on plastic moment.*
6.4. Cross-sections.

Problem 6.1.

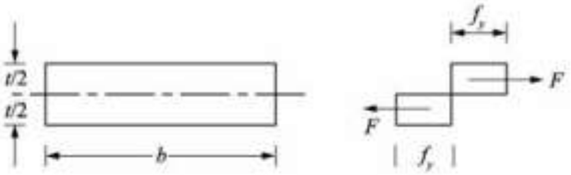
Determine the plastic moment capacity and plastic section modulus of

(a) the rectangular section of size b × t about z-z axis as shown below

(b) the I-section about z-z axis as shown below.

(c) the I-section about y-y axis as shown below.

(a) Rectangular section:



Due to symmetry plastic neutral axis (axis of equal areas) is at mid depth.

$$\therefore \quad A_t = A_c = \frac{b}{2} \times t$$

$$F = \frac{b}{2} t f_y$$

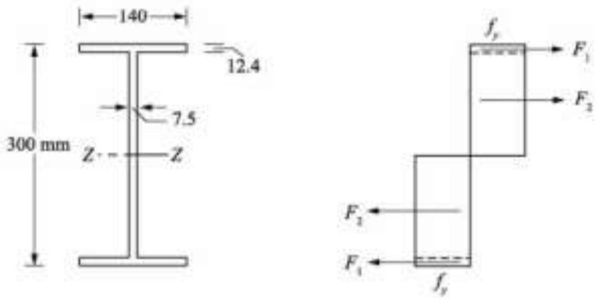
The distance between tensile and compressive forces = t/2

$$\therefore \quad M_p = F \times \frac{t}{2}$$

$$= \frac{b}{2} t f_y \frac{t}{2} = \frac{1}{4} b t^2 f_y$$

$$\therefore \quad Z_p = \frac{M_p}{f_y} = \frac{1}{4} b t^2$$

(b) I-section – About z-z axis:



Plastic N-A is at mid depth.

When plastic hinge is formed

forces in flanges, $F_1 = 140 \times 12.4 \times f_y$

forces in the webs,

$$F_2 = \frac{1}{2}(150 - 124) \times 7.5 f_y$$

Distance between F_1 forces = 300 – 12.4 = 287.6 mm
Distance between F_2 forces = 150 – 12.4 = 137.6
$\therefore M_p$ = F1 × 287.6 + F2 × 137.6

$$= 140 \times 12.4 f_y \times 287.6 + \frac{1}{2} \times 137.6 \times 7.5 f_y \times 137.6$$

= 499274 f_y + 71001.6 f_y = 570275.6 f_y
$\therefore Z_p$ = 570.276 × 10 3 mm^3.
[Note: Contribution of flanges = $\frac{49274}{570276} \times 100 = 87.5\%$]

(c) I-section about y-y axis:
Plastic N-A is in mid depth.
Let F1 be force in flange of size 140 × 12.4 mm and F2 be force in web of size (300 – 2 × 12.4) × 7.5 mm.
Then F1 = 140 × 12.4 fy
F2 = 275.2 × 7.5 fy
Distance between F1 force = $\frac{140}{2} = 70mm.$
Distance between F2 force = $\frac{7.5}{2} = 3.75mm.$
$\therefore M_p$ = 140 × 12.4 × fy × 70 + 275.2 × 7.5 fy × 3.75
= 121520 fy + 7740 fy = 129260 fy

$$\therefore \quad Z_p = \frac{M_p}{f_y} = 129260 \text{ mm}^3$$

[Note: Contribution of flange is $\frac{121520}{129260} \times 100 = 94\%$]

6.4. CLASSIFICATION OF CROSS-SECTIONS:

When the plastic analysis is used, the members should be capable of forming plastic hinges with sufficient rotation capacity without local buckling. Hence it is necessary to see that plate elements of a cross section do not buckle locally due to compressive stresses before plastic hinges are formed. The local buckling can be avoided before the limit state is achieved by limiting the width to thickness ratio of each element of a cross section subjected to compression due to axial force, moment or shear. On this basis IS: 800-2007, classifies various cross sections as follows (clause 3.7):

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

2. Class 2 (Compact) Cross-Sections: Such sections can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of plastic mechanism, due to local buckling. The sections having width to thickness ratio of plate elements between those specified for class 2 and class 1 shown in Table 2 belong to this class of sections.

3. Class 3 Cross-Sections (Semi Compact): These are the sections in which the extreme fibre in compression can reach yield stress, but cannot develop the plastic moment of resistance, due to local buckling. The sections having width to thickness ratio in the range between those shown for class 2 and class 3 in Table 2 belong to this class.

4. Class 4 Cross-Sections (Slender): The cross sections the elements of which buckle locally even before reaching yield stress belong to this category. They are having width to thickness ratio more than those specified for class 3.

Limiting width to thickness ratio are given in Table 2 in IS 800 (Clauses 3.7.2 and 3.7

.

DESIGN OF BEAMS

Learning objectives:

6.*5. Notes on different types of cross-sections.*
6.6. Design procedure.
6.7. Bending strength of the Laterally supported beam.
6.8. Shear strength of the laterally supported beam.

6.5 Notes:

1. Elements which exceed semi-compact limits are to be taken as of slender cross-section.
2. $\varepsilon = (250 / f_y)$ 1/2 .
3. Webs shall be checked for shear buckling in accordance with 8.4.2. when d/t > 67ε, where, b is the width of the element (may be taken as clear distance between lateral supports or between lateral support and free edge, as appropriate), t is the thickness of element, d is the depth of the web, D is the outer diameter of the element (See Figs. 2, 3.7.3 and 3.7.4).
4. Different elements of a cross-section can be in different classes. In such cases the section is classified based on the least favourable classification.
5. The stress ratio r_1 and r_2 are defined as:

$$r_1 = \frac{\text{Actual average axial stress (negative if tensile)}}{\text{Design compressive stress of web alone}} \qquad r_2 = \frac{\text{Actual average axial stress (negative if tensile)}}{\text{Design compressive stress of overall section}}$$

IS 800-2007 considers the design of members belonging to class 4 (slender sections) as beyond its scope and hence here also that is treated beyond the scope. For the design of such sections reference may be made to IS 801.

6.6.DESIGN PROCEDURE:

1. A trial section is selected assuming it is going to be plastic section (class 1 section).
2. Then it is checked for the class it belongs.
3. Check for bending strength.
4. Check for shear strength.
5. Check for the deflection.

If any check fails the section is revised.

6.7. BENDING STRENGTH OF A LATERALLY SUPPORTED BEAM:

If $\frac{d}{t_w} \leq 67\ e.$

IS 800-2007 considers two cases one with design shear strength less than 0.6Vd and other with design shear strength more than 0.6Vd where V_d is design shear.

When $\frac{d}{t_w} > 67$ e, shear buckling of web is likely to take place. For such case Ref. Art. 10.5.

(a) If $V \leq 0.6Vd$:

The design bending strength Md shall be taken as:

$$M_d = \beta_b Z_p f_y \times \frac{1}{\gamma_{mo}} \leq 1.2 Z_e f_y \times \frac{1}{\gamma_{mo}} \text{ for simply supported beam}$$

$$\leq 1.5 Z_e \frac{f_y}{\gamma_{mo}} \text{ for cantilever beam}$$

where β_b = 1.0 for plastic and compact sections

= $\frac{Ze}{Zp}$ for semi-compact sections.

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

(b) If V > 0.6Vd

In such cases,

$M_d = M_{dv}$

where M_{dv} is design bending strength under high shear.

This reduced value is recommended to account for the effect of higher shear on the bending strength of the sections. M_{dv} is to be calculated as given below (clause 9.2.2 in IS 800- 2007):

(a) Plastic or Compact Section:

$$M_{dv} = M_d - \beta\left(M_d - M_{fd}\right) \leq 1.2 Z_e \times f_y \times \frac{1}{\gamma_{mo}}$$

where $\beta = \left(\frac{2v}{Vd} - 1\right)^2$

M_d = Plastic design moment of the whole section.

V = Factored applied shear force.

V_d = Design shear strength.

M_{fd} = Plastic design strength of the area of the cross-section excluding the shear area, considering partial safety factor g_{mo} . (For finding shear area ref. Art. 7.5)

(b) Semi-Compact Section:

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

6.8. SHEAR STRENGTH OF A LATERALLY SUPPORTED BEAM:

The design shear strength of a section is given by (clause 8.4 of IS 800-2007):

$$V_d = \frac{A_v f_{yw}}{\sqrt{3}} \times \frac{1}{\gamma_{mo}}$$

where Av = shear area and f_{yw} = yield strength of the web.

The shear area may be calculated as given below:

(a) I and channel sections:

(i) Major Axis bending:

Hot-Rolled: Av = ht_w

Welded: Av = dt_w

(ii) Minor Axis bending:

Hot rolled or welded: $A_v = 2bt_f$

(b) Rectangular hollow sections of uniform thickness:

(i) Loaded parallel to depth (h): $A_v = \frac{Ah}{b+h}$

(ii) Loaded parallel to width (b): $A_v = \frac{Ab}{b+h}$

(iii) Circular hollow tubes of uniform thickness: $A_v = 2A/\pi$

(iv) Plates and solid bars: $A_v = A$

where, A = cross section area

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

d = clear depth of web between flanges, h = overall depth of the section

t_f = thickness of the flange and

t_w = thickness of the web.

DESIGN OF BEAMS

Learning objectives:

6.9. Deflection limits.
6.6. Design procedure.
6.7. Bending strength of the Laterally supported beam.
6.8. Shear strength of the laterally supported beam.

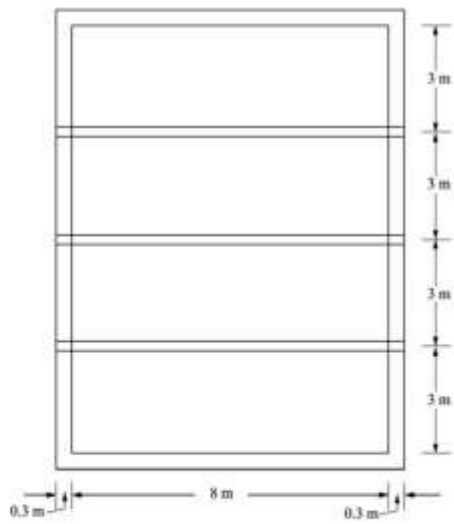
6.9. DEFLECTION LIMITS:

Deflection limits should be checked before accepting a design. In special situations, the other serviceability limits like vibration limit, durability considerations, and fire resistance also should be checked.

The deflection should be calculated by elastic theory for working load conditions. The maximum deflection in the beam should not exceed the limits specified in (Table 6 in IS 800-2007).

Problem 6.2.

A roof of a hall measuring 8 m × 12 m consists of 100 mm thick R. C. slab supported on steel I-beams spaced 3 m apart as shown in Figure below. The finishing load may be taken as 1.5 KN/m2 and live load as 1.5 KN/m2 . Design the steel beam.



Solution:

Each beam has a clear span of 8 m and takes care of 3 m width of slab. Hence the load per metre length of the beam is as follows:

Weight of R.C. slab = 0.1 × 1 × 3 × 25 = 7.5 kN/m
Finishing load = 1.5 × 3 = 4.5 kN/m
Self weight (assumed) = 0.8 kN/m
∴ Total dead load = 12.8 kN/m.
Live load = 1 × 3 × 1.5 = 4.5 kN/m.
∴ Factored dead load = 1.5 × 12.8 = 19.2 kN/m
Factored live load = 1.5 × 4.5 = 6.75 kN/m
∴ Total factored load = 25.95 kN/m.
Effective span of the simply supported beam = centre to centre distance of supports
Assuming width of support = 0.3 m,
Effective span = 8 + 0.3 = 8.3 m.

$\therefore$ Design moment M = $\frac{w\,L^2}{8}$

$$= \frac{25.95\times8.3^2}{8} = 223.46KNm.$$

Design shear force V= $\frac{25.95\times8.3}{2}$ =107.69KN.

$\therefore$ Section modulus required = $\frac{M}{f_y}\gamma_{mo}$

$$Z_p = \frac{223.46\times10^6\times1.1}{250} = 983224\ \text{mm}^3$$

Try ISMB 400 which has Zp = 1176.163 × 10 3 mm3 .

The properties of the section are as follows:
Depth of section h = 400 mm
Width of flange b = 140 mm
Sectional area A = 7845.58 mm2
Thickness of flange t_f= 16.0 mm
Thickness of web t_w= 8.9 mm
Depth of web d = h − 2 (h_2) = 400 − 2(32.8) = 333.4 mm
Moment of inertia about z-z axis I zz = 20458.4 × 10 4 mm4
Elastic section Modulus Ze = 1022.7 × 10 3 mm4
Outstanding leg of comp. flange, b = $\frac{140}{2}$ = 70

Section Classification:

$$\epsilon = \left(\frac{250}{f_y}\right)^{1/2} = \left(\frac{250}{250}\right)^{1/2} = 1.0$$

$$\frac{b}{t_f} = \frac{70}{16} = 4.38 < 9.4\epsilon$$

$$\frac{d}{t_w} = \frac{333.4}{8.9} = 37.57 < 84\epsilon$$

Hence the section is classified as plastic section:
Weight of the section = 0.604 kN/m.
Assumed weight = 0.8 kN/m.
Difference is not much.
Hence the design is continued with moments and shears calculated as earlier.

Check for shear strength:

Design shear V = 107.69 kN
Design shear strength of the section

$$V_d = \frac{f_y}{\sqrt{3}}\times\frac{1}{1.1}\times\text{shear area}$$

$$= \frac{f_y}{\sqrt{3}}\times\frac{1}{1.1}\times h\times t_w$$

$$= \frac{250}{\sqrt{3}}\times\frac{1}{1.1}\times400\times8.9$$

$$= 467128\ \text{N} = 467.128\ \text{kN} > 107.61\ \text{kN}$$

Hence the section is adequate.
0.6 Vd = 0.6 × 467.128 = 280.277 > 107.61 kN
Hence it is not high shear case.

Check for moment capacity:

$\frac{d}{t_w} = 38.2$ which is less than 67 ∈, since ∈ = 1.

Hence,

$$M_d = \beta_b Z_p \frac{f_y}{\gamma_{mo}}$$

b_b = 1.0 since it is plastic section.

$$\therefore \quad M_d = 1.0 \times 1176.163 \times 10^3 \times \frac{250}{1.1} \leq 1.2 \times 1022.7 \times 10^3 \times \frac{250}{1.1}$$

$$= 267.310 \times 10^6 \leq 278.918 \times 10^6$$

Md = 267.310 × N-mm = 267.310 kN-m.

Hence adequate.

Check for deflection:

Total working load = 12.8 + 4.5 = 17.3 kN/m. = 17.3 N/mm

Maximum deflection

$$\delta = \frac{5}{384} \frac{wL^4}{EI}$$

$$\therefore \quad \delta = \frac{5}{384} \times \frac{17.3 \times (8300)^4}{2 \times 10^5 \times 20458.4 \times 10^4}$$

= 26.127 mm.

Permissible deflection for a beam in building (Ref. Table 7.2)

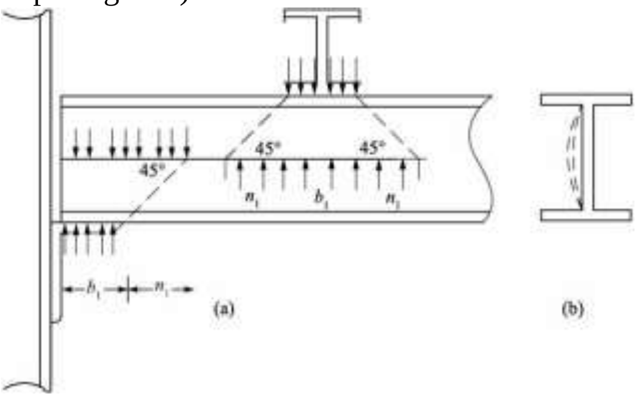
$$\frac{l_e}{300} = \frac{8300}{300} = 27.67 \text{ mm}$$

Hence deflection is within the permissible limit.

∴ Provide ISMB 400.

7. WEB BUCKLING STRENGTH

Certain portion of beam at supports acts as column to transfer the load from beam to the support. Hence under this compressive force the web may buckle [Ref. Fig. below]. This may happen under a concentrated load on the beam also. The load dispersion angle may be taken as 45°. Hence there is need to check for web buckling. However, the rolled section are provided with suitable thickness for web so that web buckling is avoided. In case of built up sectionsit is necessary to check for buckling of web and provide web stiffeners (whichis explained in the chapter on plate girder).



Hence as per IS 800-2007, effective web buckling strength is to be foundbased on the cross-section of web

$$= (b_1 + n_1)\, t_w.$$

Where b_1 = width of stiff bearing on the flange and $n_1 = \frac{1}{2}h$, where h is the depth of section.

$$\therefore F_{cdw} = (b_1 + n_1)\ t_w\ f_c$$

where F_{cdw} – web buckling strength

and f_c is the allowable compressive stress corresponding to the assumed webcolumn.

$$r_y = \sqrt{\frac{I_y}{A}} \text{ of web}$$
$$= \sqrt{\frac{\frac{1}{12}(b_1+n_1)t_w^3}{(b_1+n_1)t_w}} = \frac{t_w}{2\sqrt{3}}$$

$$\frac{l_e}{300} = \frac{8300}{300} = 27.67 \text{ mm}$$

Effective length = 0.7d of web column

$$\therefore \text{ Slenderness ratio} = = \frac{\text{Effective length}}{r_y} = 0.7d \cdot \frac{2\sqrt{3}}{t_w} = 2.5\frac{d}{t_w}$$

Corresponding to this slenderness ratio from Table 9 of IS 800-2007 buckling stress f_c can be found and hence

$F_{cdw} = (b_1 + n_1)\ t_w\ f_c$ may be found.

Near the support web of the beam may cripple due to lack of bearing capacityas shown in the Fig. .The crippling occurs at the root of the radius. IS 800-2007 has accepted the following formula to find crippling strength of web [Ref. clause 8.7.4].

where,

b_1 = stiff bearing length

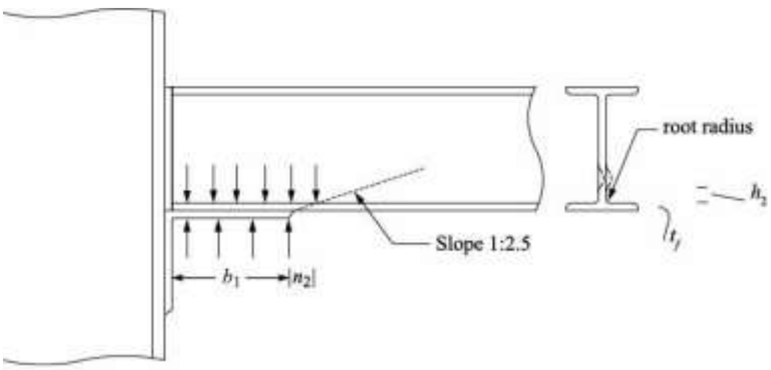
$$F_w = (b_1 + n_2)t_w \frac{f_{yw}}{\gamma_{mo}}$$

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

f_{yw} = yield stress of the web.

In the design F_w > Load transferred by bearing.

The care is taken in fixing the web thickness of rolled steel sections to avoid such failures. Hence if rolled steel section is selected as a beam section there is no need to check for this failure. However when built up sections are selected the web should be checked for this local failure.



Problem 6.3

Check the section selected in problem 6.2 for web buckling and web cripplingif stiff bearing is over a length b_1 = 75 mm.

Solution:

Section selected was ISMB 400.
End reaction = End shear = 107.61 kN . Stiff bearing at ends = 75 mm. From steel table,

t_w = 8.9 mm, t_f = 16.0 mm, radius at root = 14.0 mm.
Depth of section h = 400 mm.

∴ Depth of web = h_1=334.2

Check for web buckling:

$$\text{Slenderness ratio} = \lambda \approx 2.5\frac{h_1}{t_w} = \frac{2.5 \times 334.2}{8.9} = 93.88$$

Since cross section of web is rectangle, it falls under buckling class *C*.Hence from Table 9.c of IS 800-2007 we get,

$$f_c = 121 - \frac{3.88}{10}(121 - 107) = 115.568 \text{ N/mm}^2$$

$$n_1 = \frac{400}{2} = 200 \text{ mm}$$

∴ Web buckling resistance of the section,

$F_{cdw} = (b_1 + n_1)\ t_w\ f_c$

= (75 + 200) × 8.9 × 115.568 = 282.852 × 10^3 N = 282.852 kN > 107.61 kN
Hence the section is safe against web buckling.

Check for web crippling:

Flange thickness = 16.0 radius at root = 14.0

∴ n_2 = 2.5 (h_2) = 2.5 × 32.8 = 82 mm.

∴ Strength of web against web crippling

$$F_w = (b_1 + n_2)\,t_w\,f_{yw} \times \frac{1}{\gamma_{mo}}$$

$$= (75 + 82)\,8.9 \times 250 \times \frac{1}{1.1} = 317.568 \times 10^3 \text{ N}$$

= 317.568 kN > load transferred by bearing in this case (107.61 kN).
Hence safe.

Learning objective:

7.1. Introduction.

7.2. Elements of plate girder.

7.1. Introduction: When span and load increase, the available rolled section may not be sufficient, even after strengthening with cover plates. Such situations are common in the following:

1. Larger column free halls are required in the lower floor of a multistorey building.

2. In a workshop, where girders are required to carry crane beams.

3. In road or railway bridges.

In such situations one of the remedies is to go for a built up I-section with two flange plates connected to a web plate of required depth. The depth of such I beams may vary from 1.5 m to 5.0 m. This type of I-beams are known as ‘Plate Girder’.

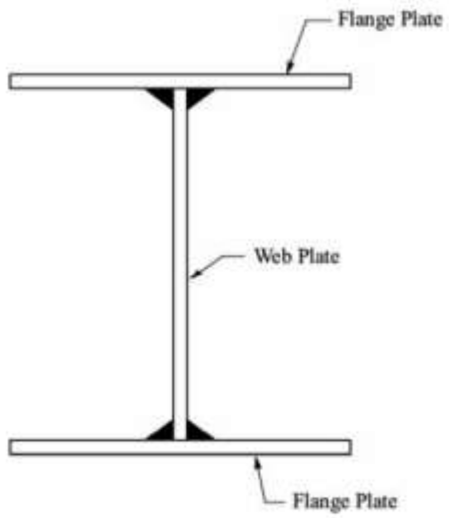
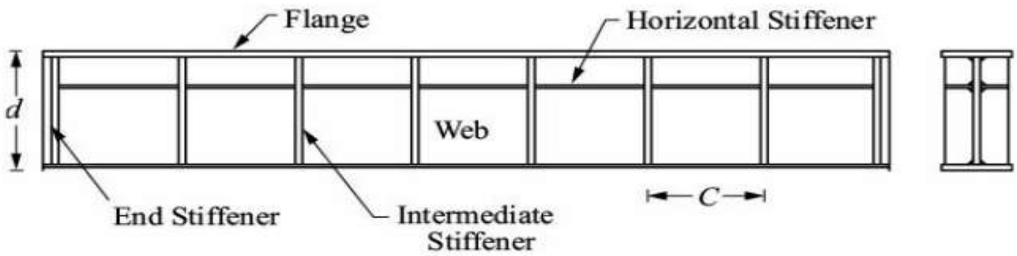


Figure above shows a typical plate girder. Before welding technology advanced, it was common practice to use riveted / bolted plate girders. Flange and web plates were connected to each other using angles and rivets / bolts. Many railway bridges of span 24 m to 46 m were built like this. This practice of using riveted / bolted plate girder is given up in 1960s. Nowadays only welded plate girders are built which are aesthetically good and at the same time light compared to riveted plate girders. Hence in this chapter design of only welded plate girder is presented.

7.2. ELEMENTS OF PLATE GIRDERS The following are the elements of a typical plate girder

1. Web. 2. Flanges. 3. Stiffeners.



Webs of required depth and thickness are provided to:

(a) keep flange plates at required distances

(b) resist the shear in the beam.

Flanges of required width and thickness are provided to resist bending moment acting on the beam by developing compressive force in one flange and tensile force in another flange.

Stiffeners are provided to safeguard the web against local buckling failure. The stiffeners provided may be classified as:

(a) Transverse (vertical) stiffeners and

(b) Longitudinal (horizontal) stiffeners.

Transverse stiffeners are of two types:

(i) Bearing stiffener (ii) Intermediate stiffener

End bearing stiffeners are provided to transfer the load from beam to the support. At the end certain portion of web of beam acts as a compression member and hence there is possibility of crushing of web. Hence web needs stiffeners to transfer the load to the support. If concentrated loads are acting on the plate girder (may be due to cross beams), intermediate bearing stiffeners are required.

To resist average shear stress, the thickness of web required is quite less. But use of thin webs may result into buckling due to shear. Hence when thin webs are used, intermediate transverse stiffeners are provided to improve buckling strength of web.

Many times longitudinal (horizontal) stiffeners are provided to increase the buckling strength of web. If only one longitudinal stiffener is provided, it will be at a depth of 0.2 d from the compression flange where 'd' is the depth of web. If another longitudinal stiffener is to be provided it will at mid depth of web.

Web, flange and stiffeners are all plates. They are to be connected suitably by welding to form a single structural system i.e., plate girder. The plate girder has to resist shear force and bending moment acting on it. No plate should fail under any of the designed load. In this chapter, the design principle is explained to select suitable sizes of plates and also to design their welded connections. The procedure is illustrated with solved example.

Learning objective:

7.3. Self-weight of plate girder.

7.4. Economic depth.

7.5. Size of flange.

7.3. SELFWEIGHT OF PLATE GIRDER

The following empirical formula may be used to assess the self weight of the beam:

$$w = \frac{W}{200} KN/m$$

where w – factored self-weight and W – total factored load on the girder. Considering this value of self-weight and the other applied loads, moment M and shear force F to be considered for the design is found.

7.4. ECONOMICAL DEPTH Assuming that the moment M is carried out by flanges only, the economical depth ' of the girder may be found as given below:

$$M = f_y b_f t_f d$$

where b_f and t_f are the breadth and thickness of the flange.

$$\therefore\ b_f t_f = \frac{M}{f_y d}$$

∴ The gross sectional area of girder A is given by

$$A = 2 b_f t_f + d t_w$$

$$= \frac{2M}{f_y d} + d\, t_w$$

Taking $\frac{d}{t_w} = k$, where k is assumed constant, then,

$$A = \frac{2M}{f_y d} + \frac{1}{k} d^2$$

For A to be minimum, the above expression is to be differential w.r.t. 'd ' and equated to zero. Hence

$$0 = -\frac{2M}{f_y d^2} + \frac{1}{k} 2d$$

or $d^3 = \frac{Mk}{f_y}$

or $d = \left[\frac{Mk}{f_y} \right]^{1/3}$

The above expression may be used to get the idea about economical depth. To avoid labour cost of cutting or welding, the available plate size is used. It may be slightly less than the economical depth, since in deriving bending resistance, the contribution of web has been neglected.

In selecting the value of $k = \frac{d}{t_w}$, the following codal provisions will be useful:

I. If,

$$\frac{d}{t_w} \leq 67 \in$$

It may be designated as an ordinary beam.

$$\in = \left(\frac{250}{f_y} \right)^{1/2}$$

ii. Minimum web thickness based on serviceability requirement [clause 8.6.1.1 in IS 800

(a) When transverse stiffeners are not provided,

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

, web connected to flanges along both longitudinal edges.

(b) When the transverse stiffeners are provided;

1. when $3\ d \geq c \geq d$

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

2. when $0.74\ d \leq c < d$

$$\frac{c}{t_w} = 200 \in_w \qquad \text{where} \quad \in_w = \sqrt{\frac{250}{\text{yield stress of web}}}$$

3. when $c < 0.74\ d$

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

4. when $c > 3\ d$, the web shall be considered unstiffened.

(c) When transverse stiffeners and longitudinal stiffeners at one level only are provided

(0.2 d from compression flange)

1. when 2.4 d ≥ c ≥ d

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

2. when 0.74 d ≤ c ≤ d

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

3. when c < 0.74 d

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

(d) When a second longitudinal stiffeners (located at neutral axis is provided)

$$\frac{d}{t_w} \leq 400 \epsilon_w .$$

III. Minimum web thickness based on compression flange buckling requirement (clause 8.6.1.2 in IS 800):

In order to avoid buckling of the compression flange into the web, the web thickness shall satisfy the following:

(a) When transverse stiffeners are not provided

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

where ∈f = yield stress ratio of flange

$$= \sqrt{\frac{250}{f_{yf}}}$$

(b) When transverse stiffeners are provided and

1. when c ≥ 1.5 d

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

2. when c < 1.5 d,

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

From the above clauses, if Fe 415 (E250) steel is used (∈ = 1), the following points may be observed.

(1) If k= $\frac{d}{t_w} \leq 67$, the plate girder may be designed as ordinary beam, i.e., without any stiffener (except end bearing stiffener). But such sections will be uneconomical

(2) If k= $\frac{d}{t_w}$ is between 67 to 200, it may be possible to have the plate girder without intermediate stiffeners. However, designer has to check for the shear buckling of web. For k values upto 100–110, intermediate transverse stiffeners may not be required. But for larger k values consideration of web buckling may force to go for transverse stiffener.

(3) For k value upto 250, longitudinal stiffener is also required.

(4) k value more than 345 should not be taken to avoid compression flange failure.

In the past stiffener webs have been used. But present-day tendency is to avoid stiffeners to reduce fabrication cost and time. Hence it is preferable to go up to k = 100 to 110, so that real economical girder is obtained, provided it is safe in shear buckling of web without transverse stiffeners.

It may be noted that in all plate girders end transverse stiffeners are required to transfer the load to the support.

Another practical guide line for selecting the depth of plate girder is given below:

$\frac{D}{L} = \frac{1}{15}\ to\ \frac{1}{25}$ for girders in buildings

$= \frac{1}{12}\ to\ \frac{1}{18}$ for highway bridges

$= \frac{1}{10}\ to\ \frac{1}{15}$ for railway bridges.

where D = depth of girder (including flange thicknesses)

and L= equivalent span of the girder.

7.5. SIZE OF FLANGES

Assuming moment is resisted by flanges only, and using material partial safety factor for a plastic section,

$$\frac{A_f \times f_y \times d}{1.1} = M$$

Hence area of flange Af may be found.

Select 9.4 ∈ < t_f < 13.6 b_f ∈ so that bending strength can be found by the formula for semi compact section as per the clause 8.2.1.2 in IS 800. Thus

$b_f\, t_f = A_f$

i.e., 13.6 ∈ t_f 2 = A_f .

Hence t_f is found. Then.

$$b_f = \frac{A_f}{t_f}.$$

Learning objective:

7.6. Shear buckling of web.

7.7. Simple post critical method.

7.8. Tension field method.

7.6. SHEAR BUCKLING RESISTANCE OF WEB:

For thin webs, it is necessary to check the shear resistance of web for buckling. IS 800-2007, clause 8.4.2 specify that this check is necessary when;

$$\frac{d}{t_w} > 67\in$$

for a web without stiffeners, and

$$> 67 \in \sqrt{\frac{K_v}{5.35}}$$

for a web with stiffeners

where Kv = 5.35 when transverse stiffeners are provided at support

$$= 4.0 + \frac{5.35}{\left(\frac{c}{d}\right)^2} \quad \text{for} \quad \frac{c}{d} < 1.0$$

$$= 5.35 + \frac{4.0}{\left(\frac{c}{d}\right)^2} \quad \text{for} \quad \frac{c}{d} \geq 1.0$$

where c and d are the spacing of transverse stiffeners and depth of the web, respectively. The nominal shear strength $V_n = V_{cr}$ may be calculated by any one of the following two methods.

(a) Simple post-critical method

(b) Tension field method.

7.7. Simple Post-Critical Method

This method can be used for plate girders with or without transverse stiffeners. According to this method $V_n = V_{cr} = A_v\tau_b$

where τ_b = shear stress corresponding to web buckling which is to be determined as follows:

1. When λ w ≤ 0.8.

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

f_{yw} = yield stress of web material

2. When $0.8 < \lambda_w < 1.2$

$$\tau_b = \left[1 - 0.8(\lambda_w - 0.8)\right]\frac{f_{yw}}{\sqrt{3}}$$

3. When $\lambda_w \geq 1.2$

$$\tau_b = \frac{f_{yw}}{\left(\sqrt{3}\,\lambda_w^2\right)}$$

where λ_w = non-dimensional web slenderness ratio for shear buckling stress

$$= \left[\frac{f_{yw}}{\sqrt{3}\,\tau_{cr}}\right]^{1/2}$$

τ_{cr} = elastic critical shear stress of web

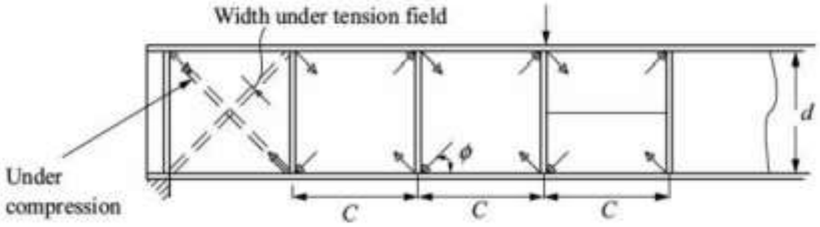
$$= \frac{K_v \pi^2 E}{12\left(1-\mu^2\right)\left(\frac{d}{t_w}\right)^2}$$

The Poisson's ratio m for steel may be taken as 0.3.

7.8. Tension Field Method

This method of finding shear buckling strength of web may be used if end and intermediate transverse stiffeners are provided. It accounts for post buckling strength provided by the stiffeners. As the web begins to buckle, it loses ability to resist diagonal compression. At this stage, the transverse stiffeners and the flanges come into action to resist the diagonal compression. The vertical component of this compression is resisted by transverse stiffeners and horizontal component by the flange [Ref. Fig.below].

The web resists only diagonal tension. Thus there is additional strength for resisting shear buckling. IS 800-2007 has accepted the following expression for computing shear resistance of web if end and intermediate stiffeners are provided and [clause 8.4.2.2]



$V_n = V_{tf}$.

where $V_{tf} = [A_v\ \tau_b + 0.9\ w_{tf}\ t_w\ f_v \sin\varphi\ ] \le V_p$

where τ_b = buckling strength as obtained in simple post-critical method.

fv = yield strength of the tension field obtained from

$= [\ f_{yw}\ 2 - 3\tau b\ 2 + \psi\ 2\ ]\ 0.5 - \psi$

$\psi = 1.5\ \tau_b \sin 2\varphi$

φ = inclination of the tension field

$$\approx \tan^{-1}\frac{d}{1.5}$$

wtf = the width of the tension field

$= d\cos\varphi> - (c - s_c - s_t)\sin\varphi> s_c$,

st = anchorage lengths of tension field along the compression and tension flanges obtained from

$$s = \frac{2}{\sin\varphi}\left[\frac{M f_t}{f_{yw} t_w}\right]^{0.5} \le c$$

where, M_{fr} = reduced plastic moment capacity of the respective flange plate

$$= 0.25\, b_f\, t_f^2\, f_{yf}\left[1 - \left\{\frac{N_f}{b_f t_f\, f_{yt} / _{mo}}\right\}^2\right]$$

N_f = Axial force in the flange.

SHORT TYPE QUESTIONS:

1.What do you mean by plate girder?

2. When plate girders are used?

3. Why end posts are provided in plate girder?

4. In a bolted plate girder flange, the angle section used to be -----------.

5. An ideal bolted plate girder section consists of ----------.

6. The optimum depth of plate girder is given by ---------.

ANSWERS

1. Plate girder is a flexural member made up of plate and angle section and is employed to carry loads that can not be supported by rolled beams.
2. These are used when loads and spans are large for buildings ,road bridges etc.
3. The end posts are provided to carry the tension field.
4. Unequal angle with long leg horizontal.
5. Flange angles and cover plates for compression flange and only flange angle for tension flange.
6. $\left(\frac{M}{f_y k^2}\right)^{0.33}$

LONG TYPE QUESTIONS:

1. What is plate girder? Where it is used? Explain its various components with sketches.

2. Derive the expression for the economical depth of a plate girder. Assume moment is resisted by flanges only.

3. A plate girder is subjected to a maximum factored moment of 4000 KN-m and a factored shear force of 600 KN. Find the preliminary sections for the following conditions:

(a) Girder without any stiffener

(b) Girder with end stiffeners only

(c) Girder with end as well as intermediate transverse stiffeners.

4. Explain the tension field action of thin web plates.

5. A plate girder with Fe 415 steel plates is having 12 mm × 1500 mm web plates and 56 mm × 500 mm flange plates. Determine the

(a) Design flexural strength, if the compression flange is supported laterally.

(b) Design strength in shear, if no intermediate stiffeners are used.

(c) Design shear strength, if stiffeners are provided at every 2 m interval.

* * *

Learning objectives:

7.9. End panel design.

7.10.

7.8.

7.9.END PANEL DESIGN:

In a plate girder with transverse stiffeners, if the web is designed using tension field action, special care should be taken in the design of end panel. In IS 800- 2007, clause 8.5 covers these provisions. The code permits the design of end panels both by simple post buckling method and by tension field action with additional provisions given below.

7.9.1. If Simple Post Buckling Method is Used in the Design of End Panel

In this case the end panel along with the stiffeners (Fig. 10.4) should be checked as a beam spanning between the flanges to resist a shear force Rtf and a moment, Mtf due to tension field forces. Apart from this end stiffener should be capable of resisting the reaction plus a compressive force due to the moment equal to Mtf. 10.6.2 If End Panel is Designed Using Tension Field Action If the end panel is also designed using tension field action, it should be provided with an end post consisting of a single or double stiffener (see Fig. 10.5 and Fig. 10.6). (a) Single Stif ener. (i) The top of the end post should be rigidly connected to the flange using full strength weld. (ii) The end post should be capable of resisting the reaction plus a moment from the anchor forces equal to due to tension field forces. (iii) The width and thickness of the end post are not to exceed the width and thickness of the flange. (b) Double Stif ener If double stiffeners as shown in Fig. 10.6 are provided, the end post should be checked as a beam spanning between the flanges of the girder and capable of resisting a shear force Rtf and a moment Mtf . Figure 10.5 Figure 10.6 10.7 ANCHOR FORCES The resultant longitudinal shear Rtf and a moment Mtf from the anchor of tension field forces are to be evaluated as given below: where, d = web depth. If the actual factored shear force, V (using tension field approach) is less than the shear strength, Vtf , then the values of Hq may be reduced by where, Vtf = the basic shear strength for the panel utilizing tension field action Vcr = critical shear strength for the panel designed utilizing tension field action. 10.8 DESIGN OFCONNECTION BETWEEN FLANGE AND WEB PLATES If ‘V ’ is the shear force acting on the section, then shear stress at the junction is, ∴ Shear force per unit length = If weld of throat thickness ‘t’ is provided on both side, then strength of shop weld per unit length Equating the force to strength we get Hence throat thickness of weld ‘t’ can be found, from which size of the weld is obtained as . In finding shear stress, moment of inertia of flange alone may be considered i.e. If weld size comes out too small intermittent welding may be adopted.