INSTITUTE OF TEXTILE TECHNOLOGY

CHOUDWAR

SUB-STRUCTURAL DESIGN-II

BRANCH-CIVIL ENGG.

SEM-5^{th}

PREPARED BY

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INTRODUCTION

Design of a building consists of two parts viz. (i) functional design and (ii) structural design. The first part consists in planning the building to serve its requirements taking into account ventilation, lighting, aesthetic view etc.

For the transferring the loads to the ground various materials like asbestos sheets, tiles, bricks, cement concrete, Reinforced concrete, steel, aluminium etc, are used. However, 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.

COMMON STEEL STRUCTURES :-

Steel has high strength per unit mass. Hence it is used in constructing large column-free structures. The following are the common steel structures in use:

1). Roof trusses for factories, cinema halls, auditoriums, etc
2). Trussed bents, crane girders, columns etc, in industrial structures
3). Roof trusses and columns to cover platforms in railway stations and bus stands.
4). Plate girder and truss bridges for railways and roads
5). Single layer or double layer domes for auditoriums, exhibition halls, indoor stadiums, etc
6). Transmission towers for microwave and electric power
7). Water tanks
8). Chimneys etc

ADVANTAGES AND DISADVANTAGES OF STEEL STRUCTURES:-

The advantages of steel over other materials for construction are -

(1) It has high strength per unit mass. Hence even for large structures, the size of steel structural element is small, saving space in construction and improving aesthetic view.

2) It has assured quality and high durability.

3) Speed of construction is another important advantage of steel structure. Since standard sections of steel are available which can be prefabricated in the workshop/site, they may be kept ready by the time the site is ready and the structure erected as soon as the site is ready. Hence there is lot of saving in construction time.

(4) Steel structures can be strengthened at any later time, if necessary. It needs just welding additional sections.

(5) By using bolted connections, steel structures can be easily dismantled and transported to other sites quickly.

(6) Material is reusable.

(7) If joints are taken care, it is the best water and gas resistant structure. Hence can be used for making water tanks also.

The disadvantages of steel structures are :-

(1) It is susceptible to corrosion.

(2) Steel members are costly.

(3) Maintenance cost is high, since it needs painting to prevent corrosion.

TYPES OF STEEL :-

Steel is an alloy of iron and carbon. Apart from carbon by adding small percentage of manganese, sulphur, phosphorous, 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) Increased sulphur and phosphorus beyond 0.06 percent imparts 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 ~~as~~ 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 members, tubes, pipes, sheets, strips, reinforcements for R.C.C., rivets, bolts, nuts and for welding.

→ Structural steel may be mainly classified as mild steel and high tensile steel.

→ Structural steel is also known as standard quality steel.

$\left\{ \begin{array}{l} \text{E250 steel :- 250 MPa} \\ \text{E300 '' :- 440 MPa} \end{array} \right.$, E350 :- Fe490

PROPERTIES OF STRUCTURAL STEEL :-

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

(i) Physical properties

(ii) Mechanical properties

(i) Physical Properties :- Irrespective of its grade physical properties of steel may be taken as (clause 2.2.4 of IS 800-2007) (pg-12)

(a) Unit mass of steel, $(\rho) = 7850\ kg/m^3$

(b) Modulus of elasticity, $(E) = 2.0 \times 10^5\ N/mm^2$

(c) Poisson's ratio, $(\mu) = 0.3$.

(d) Modulus of rigidity, $(G) = 0.769 \times 10^5\ N/mm^2$

(e) Coefficient of thermal expansion, $(\alpha_t) = 12 \times 10^{-6}/^\circ C$.

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

(a) Yield stress (f_y)

(b) The tensile or ultimate stress (f_u).

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

(d) Notch toughness.

ROLLED STEEL SECTIONS :-

Like concrete, steel section of any shape and size cannot be cast on site, since steel needs very high temperature to melt it and roll it to required shape. Steel sections of standard shapes, size 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 listed below:

(i) Rolled steel I-sections (Beam section)

(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.

(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

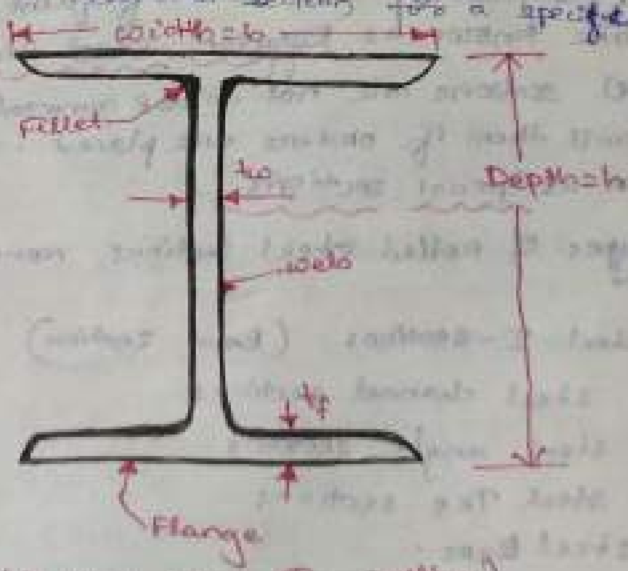
These sections are designated by the series to which they belong, followed by depth (in mm) and weight per metre run. e.g.,

ISMB 600 @ 0.852 kN/m

→ In case of ISWB and ISHB sections weight per unit length should always be specified since for the same depth in these series more than one sections are available with different weight and properties e.g.

ISWB 600 @ 1.423 kN/m
ISWB 600 @ 1.312 kN/m
ISHB 450 @ 0.855 kN/m
ISHB ~~450~~ 450 @ 0.907 kN/m

→ ~~But~~ In case of ISJB, ISLB, ISMB sections it ~~many~~ not matter much of weight per metre length is not written, since there is only one standard section for a specified depth.



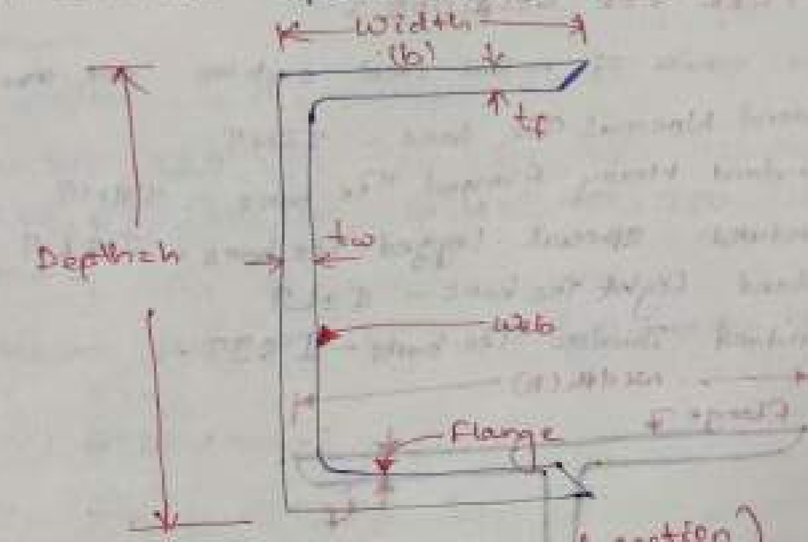
(Rolled Steel I – section)

(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 :-

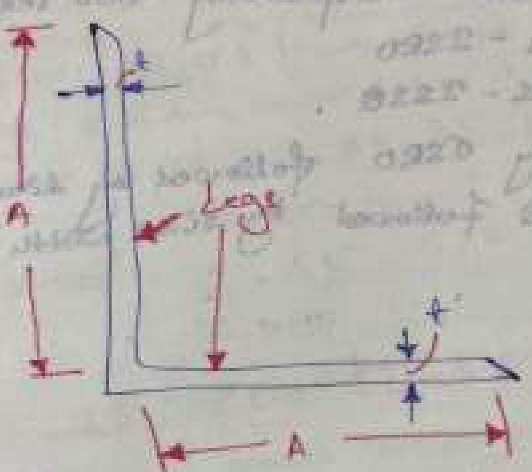


(Rolled steel channel section)

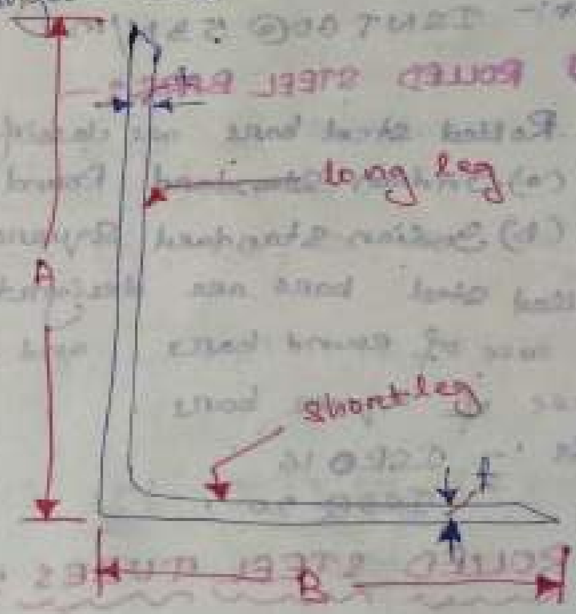
* Rolled steel channel sections are designated by the series to which they belong, followed by depth (in mm) and weight (in KN/m). e.g.,
ISMC 250 @ 0.251 KN/m

(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



(a) Rolled steel equal angle



(b) Rolled steel unequal angle.

Thickness of legs of equal and unequal angles are same. Rolled steel equal and unequal angles are designated by their series name ISA followed by length and thickness of legs e.g.:

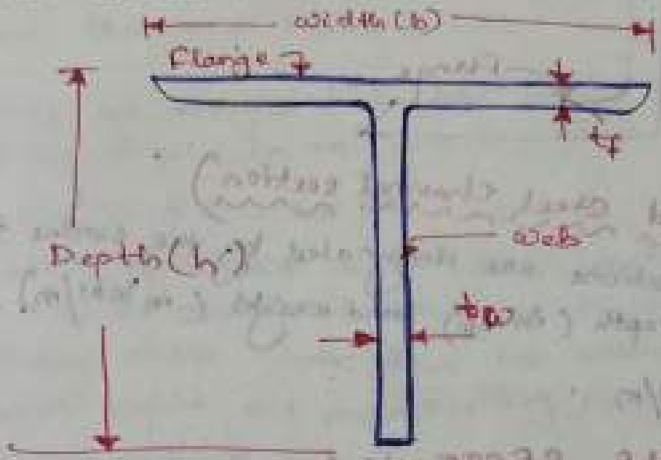
ISA 150 150, 12 mm thick or ISA 150 x 150 x 12

ISA 150 115, 10 mm thick or ISA 150 x 115 x 10

(4) ROLLED STEEL TEE-SECTIONS :-

Followed five series of rolled steel section are available:

(a) Indian Standard Normal Tee bars - ISNT
(b) Indian Standard Heavy flanged Tee bars - ISHT
(c) Indian Standard Special Legged Tee bars - ISLT
(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.

Ex:- ISNT 60 @ 53 N/m

(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

Ex :- ISRO 16
ISSQ 20.

(6) ROLLED STEEL TUBES :-

These sections are designated by their nominal bore sizes. In each sizes there are three classes, namely Light, Medium and Heavy. The difference is due to difference in their thicknesses. Hence there cross-sectional properties are also different.

(7) ROLLED STEEL PLATES :-

Rolled steel plates of the following thickness 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, 355, 400, 450, 500, 560, 630, 710, 800, 900, 1000, 1100, 1250, 1400, 1600, 1800, 2000, 2250, 2500 mm

→ These plates are designated by ISPL followed by length, width and thickness e.g.:

ISPL 2000 × 1000 × 6

(8) ROLLED STEEL STRIPS :

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

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

* The 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

(9) ROLLED STEEL FLATS :

Flats differs from strips in the sense that the thickness of flats is 5 mm onward and their width is limited.

Width :- 10, 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

SPECIAL CONSIDERATIONS IN STEEL DESIGN

The following special considerations are required in the steel design:

1.) Size and Shape.
2.) Buckling.
3.) Minimum Thickness.
4.) Connection Designs.

1.) Size and shape :- Steel is manufactured in steel mills and is available in certain shapes and sizes. Hence the member of a steel structure should be designed to consist of any of the available sections or a combination of them. For example, a beam section may be standard I-section or it may consist of builtup sections.

2.) Buckling Consideration :- The permissible load parameters in steel is much higher as compared to permissible values in concrete. Therefore, for the same load, the cross-sectional area of a steel member is smaller. As the members in a steel structure are more slender, the compression members in steel structure are liable to buckling.

3.) Minimum Thickness :- Corrosion needs special consideration in steel design. If very thin sections are used, a small amount of corrosion may result into a large percentage reduction in effective area. The members directly exposed to weather the following minimum thickness is to be used:

(a) If fully accessible for cleaning and painting — 6mm

(b) If not accessible for cleaning and painting — 8mm.

(c) The above limitations do not apply for rolled steel sections, tubes and cold formed light gauge sections.

4.) Need for Design of Connections :- A steel design is not complete if the following connections are not designed.

(a) Connections between various standard sections selected for a member.

(b) Connections between various members (like beam, column, foundation) of the structure.

The following three types of connections are commonly used:

(a) Riveted connection

(b) Bolted connection

(c) Welded connection

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 (DL) :

Dead loads include the weight of all permanent construction for example in a building weight of roofs, floors, wall, beams, columns etc.

(b) Imposed loads (IL) :

The following loads are grouped under imposed loads:

(i) Live load

(ii) Crane load

(iii) Snow load

(iv) Dust load

(v) Hydrostatic and earth pressure

(vi) Impact load

(vii) Horizontal loads on parapets and balustrades etc.

(ii) Live loads (LL) :-

The loads which keep on changing from time to time are called live loads.
Ex:- loads in a building are the weight of the persons, weight of movable partition, dust loads and weight of furniture.

(iii) Crane loads (CL) :-

These loads include loads from cranes and other machines acting on the structure.

(iv) Snow load :-

IS 875 (part-4) deals with snow loads on roof of the buildings. This load is to be considered for the buildings to be located in the regions where snow is likely to fall. The snow load acts vertically downward. It is expressed in KN/m^2.
The load on the roof due to accumulation of snow is obtained by the expression

$$\boxed{S = \mu \cdot S_0}$$

where
S = snow load on plan area of roof.
μ = shape coefficient
S_0 = Ground snow load.

(iv) Dust Load :-

In areas prone to settlement of dust on roof (e.g. steel plants, cement plants) provision for dust load equivalent to probable thickness of accumulation of dust may be made.

(v) Hydrostatic and Earth Pressure :-

In the design of structures partly or fully below ground level, the pressure exerted by soil or water or both shall be duly accounted. All foundation slabs and other footings subjected to water pressure shall be designed to resist uniformly uplift equal to the full uplift hydrostatic pressure.

(vi) Impact load :-

For structures supporting moving loads suitable additional allowance of load should be made by increasing imposed load.

(vii) Horizontal load :-

Parapets, balustrades and their supporting structures shall be designed for the horizontal forces acting at the handrail or coping level. These loads may be considered to act vertically also but not simultaneously with the horizontal forces.

(c) Wind load :- (WL) :-

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.

(d) Earthquake loads (EL) :-

Earthquake shocks cause movement of foundation of structures. Due to inertia additional force develop on superstructures. The total variation caused by earthquake may be resolved into three mutually perpendicular directions, usually taken as vertical and two directions.

(e) Erection loads (ER) :-

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

(f) Accidental loads :-

The following accidental loads on the structures are
(i) Impact and collision (due to vehicles, crane failure / lift failure, flying fragments)
(ii) Explosions (Internal gas explosion, external gas explosion, Boiler failure, High explosions (dynamic
(iii) Fire (Time temperature curve, energy balance method)

(g) Secondary effects :-

The following types of secondary effects should be looked into the design ;
→ Differential settlement of foundations.
→ Differential shortening of columns.
→ Eccentric connections.
→ Rigidity of joints differing from design assumptions.

Load Combinations :-

A judicious combinations 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 stress or deformation caused by the combination of various loads.

Load combinations for design purposes shall be those that produces maximum forces and effects and consequently maximum stresses and deformations. The following combination of loads with appropriate partial safety factors may be considered.
(a) Dead load + imposed load
(b) Dead load + imposed load + wind or earthquake load
(c) Dead load + wind or earthquake load.
(d) Dead load + erection load.

In case of structures supporting cranes, imposed loads shall include the crane effects. Wind load and earthquake loads shall not be assumed to act simultaneously. The effect of each shall be considered separately.

STRUCTURAL ANALYSIS :-

In order to find the effect of loads on a structure and its members and connections i.e., the internal forces or moments developed in the members of the structure, the structural analysis is carried out. The IS code permits the following methods of analysis
(a) Elastic Analysis
(b) Plastic Analysis
(c) Advance Analysis.
(d) Dynamic Analysis.

(a) Elastic Analysis :- This method of analysis is also known as working stress analysis. The worst combination of service (working) loads is ascertained and the members are proportioned on the basis of working stresses. These stress should never exceed the permissible stresses as laid down by the code.

The permissible stresses are some function of the yield stress of the material and may be defined as the ratio of the yield stress to the factor of safety. It may also be defined as the ratio of strength of the member to the expected force.

The analysis may be carried out in two stages i.e.,

(i) First order analysis which is based on the loads acting on the undeformed geometry of the structure where redistribution of 15% of peak moment is permitted in the code.

(ii) Second order analysis which is based on the deformation shape of the structure or using the amplification factors {cl - 4.4.3.3}

(b) Plastic Analysis :— In this method, it is assumed that a plastic hinge is formed when every fibre at a section reaches yield stress and after plastic hinge is formed, infinite rotation takes place without resisting any additional moment i.e., its resistance to moment remain constant.

(c) Advanced Analysis :— If the actual behaviour of a frame with full lateral restraints can be accurately modelled in respect of its actual behaviour, an advanced structural analysis may be carried out which takes into account relevant material properties, residual stresses, geometric imperfections, second order effects, erection procedure, reduction in stiffness due to axial compression and interaction with foundation.

(d) Dynamic Analysis :— Dynamic analysis is carried out as per IS 1893 either by seismic coefficient method or by response spectrum method.

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. The design philosophies used are listed below in the order of their evolution and they are -

(i) Working stress Method (WSM).

(ii) Ultimate Load Design (ULD).

(iii) Limit state Design (LSD).

(1) WSM :— This is the oldest systematic analytical design method. Though 800-2007 insists for the limit state design permits use of this method whenever LSD cannot be conveniently adopted.

permissible stress = $\frac{\text{Yield stress}}{\text{Factor of safety}}$.

(ii) ULD (Ultimate Load Design):-

The limitation of working stress method to assess actual load carrying capacity, made to develope ultimate load method, which is also known as load factor method (LFM). When applied to steel structure it is referred as plastic design method.

In this method a section is said to have formed 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 develope collapse mechanism.

(iii) LSD :- It is the comprehensive method which will take care of ~~design~~ both strength and serviceability requirements.

PRINCIPLES OF 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 violates the serviceability requirements of deflection, vibrations, cracks due to fatigue, corrosion and fire.

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 risks involved.

(a) 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):-

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

→ Have adequate durability under normal maintenance.

→ 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 collapse is considered disproportionate, if more than 15 percent of the floor or roof area of 70m² collapse 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 of permanent nature and 0.33 times wind load acting together. To avoid disproportionate collapse, the following conditions should be satisfied :-

(i) 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 :

(*) Expected tensile force subjected to a minimum of 75 kN.

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

(ii) 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.

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

(iv) Floor or roof units should be effectively anchored in the direction of their span either to each other or directly to the support.

(b) 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.

STRUCTURAL STEEL FASTENERS AND CONNECTIONS

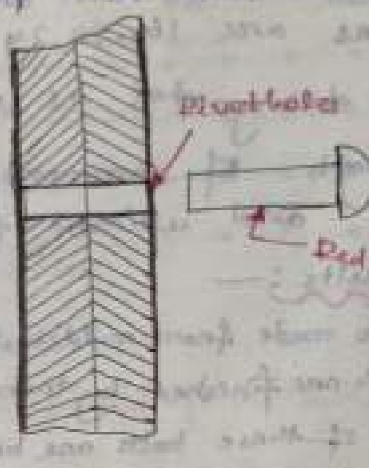
The design of connections is very important because the failure of joint is sudden and catastrophic.

- The following three types of connections may be made in steel structures
  - (a) Riveted.
  - (b) Bolted.
  - (c) Welded.

(a) 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.

Rivet holes are made in the structural members to be connected by punching or by drilling. The size of rivet hole is kept slightly more (1.5 to 2mm) than the size of 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 red hot rivet at shop head end, hammering is made.



Disadvantages :-

→ It is associated with high level of noise pollution.
→ It needs heating the rivel to red hot.
→ Inspection of connection is a skilled work.
→ Removing poorly installed rivets is costly.
→ Labour cost is high.

(b) BOLTED CONNECTIONS :-

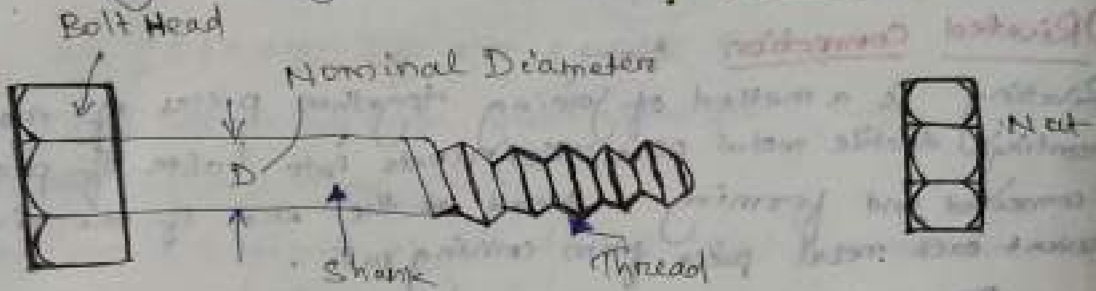
A bolt is a metal pin with a head formed at one end and shank threaded at the other in order to receive a nut. Bolts are used for joining together pieces of metals by inserting them through holes in the metal and tightening the nut at the threaded ends.

→ Bolts are classified as:

(a) Unfinished Bolts or Black Bolts.

(b) Finished Bolts or Turned Bolts.

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



(Bolt and Nut)

(a) 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 (dia. of shank) of size 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.

→ The yield strength of commonly used black bolts is 240 N/mm² and ultimate strength 400 N/mm².

(b) 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 1.2mm to 1.3mm larger than the nominal diameter.

→ As usual the bolt hole is kept 1.5mm larger than the nominal diameter.

(c) 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.

→ Commonly available nominal diameter of HSFG bolts are 16, 20, 24, 30, and 36mm.

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 Group Type

Unfinished bolts and finished 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 :—

Advantages and Disadvantages of Bolted Connections :—

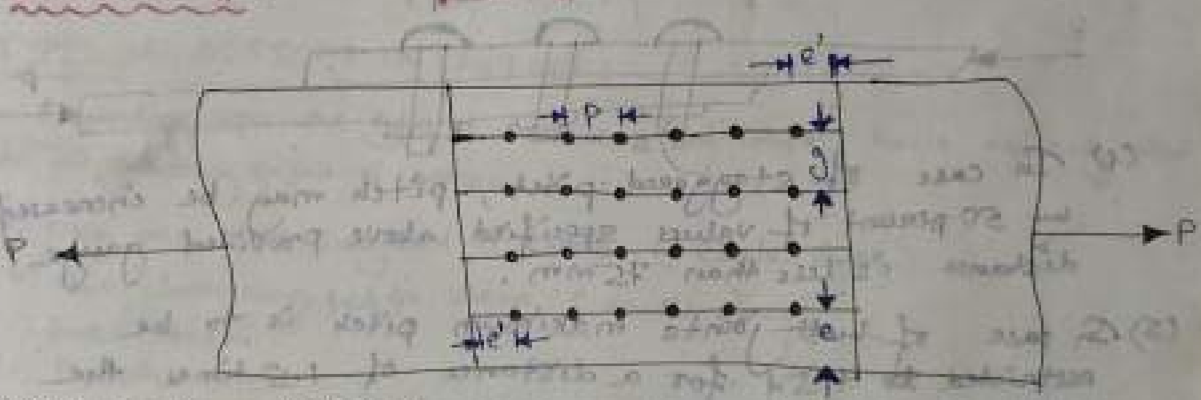
Advantages :—

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) Working area required in the field is less.

Disadvantages :—

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.

TERMINOLOGY :—



(i) Pitch of the bolts (p) :—

It is the centre to centre spacing of the bolts in a row, measured along the direction of load.

(ii) Gauge Distance (g) :— (100 + 4t) or 200 mm (cl:- 10.2.3.2)

It is the distance between the two consecutive bolts of adjacent rows and is measured at right angles to the direction of load.

(iii) Edge Distance (e) :— { 1.7 times - for hand flame cut edge ; 1.5 times hole dia - rolled, machine flame cut } min.

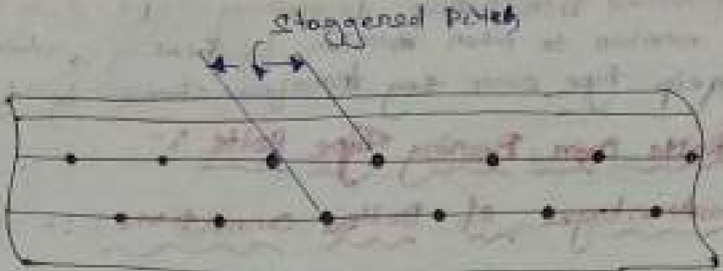
It is the distance of centre of bolt hole from the adjacent edge of plate

(iv) End Distance (e') :—

It is the distance of the nearest bolt hole from the end of the plate

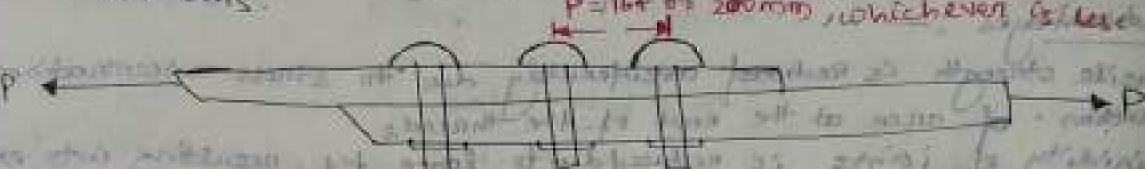
(v) Staggered Distance :—

It is the centre distance of staggered bolts measured obliquely on the member.

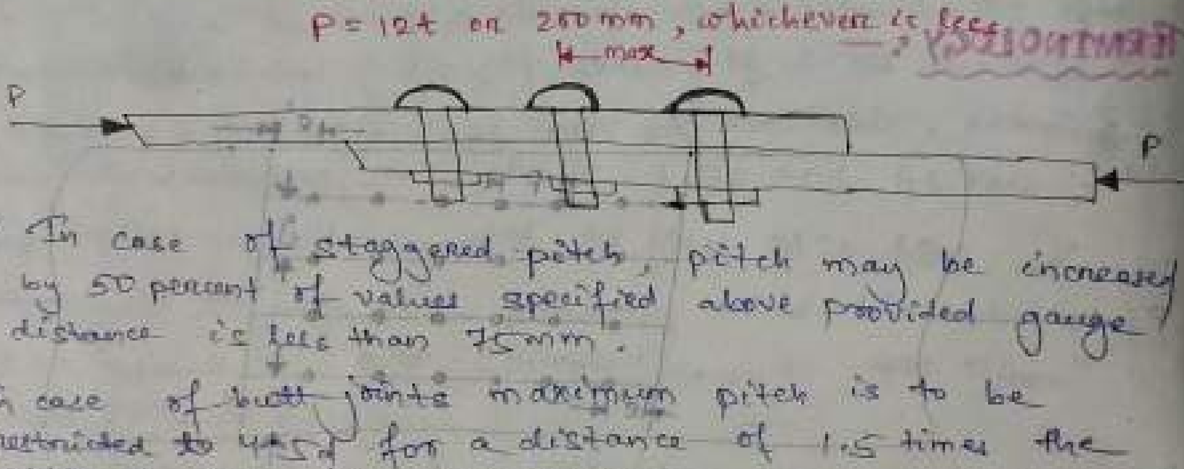


*IS 800-2007 Specifications for spacing and edge distance of bolt holes :-

1. Pitch 'p' shall not be less than 2.5 d, where 'd' is the nominal diameter of bolt.
2. Pitch 'p' shall not be more than
   (a) 16t or 200mm, whichever is less, in case of tension members.



   (b) 12t or 200mm, whichever is less, in case of compression members where 't' is the thickness of thinnest member.



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

(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 buttering surface.

TYPES OF BOLTED CONNECTIONS :-

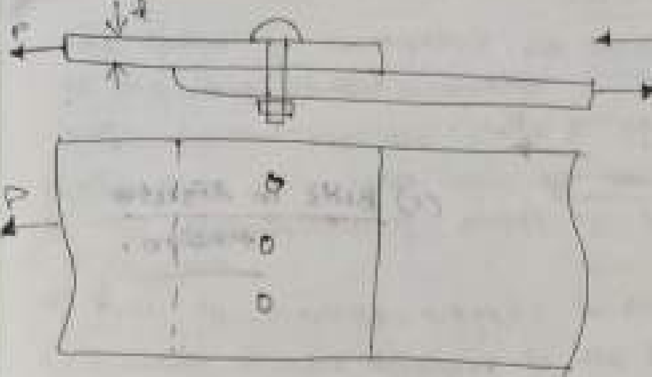
Types of joints may be grouped into the following two :-
(a) Lap Joint
(b) Butt Joint

(a) Lap Joint :-

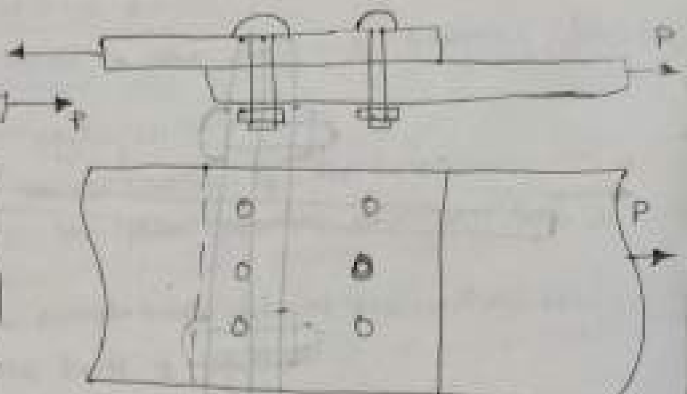
It is the simplest type of joints. In this the plates to be connected overlap one another.

(b) Butt Joint :-

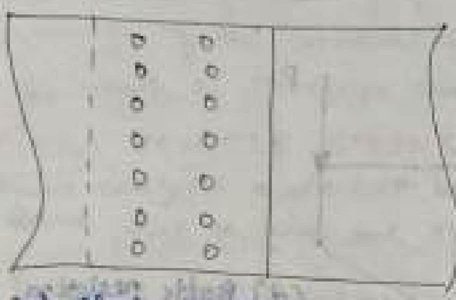
The two main plates a butt against each other and the connection is made by providing a single cover plate connected to main plate or by double cover plates, one on either side connected to the main plates.



(a) Single line bolted lap joint



(b) Double-bolted lap joint



(c) Chain Bolting

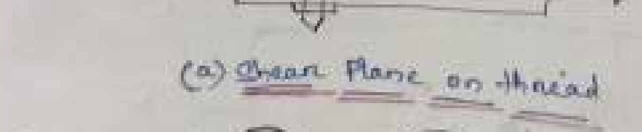
(d) Zig Zag Bolting.

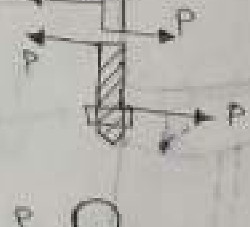
(Lap Joint)

TYPES OF ACTIONS ON FASTENERS :-

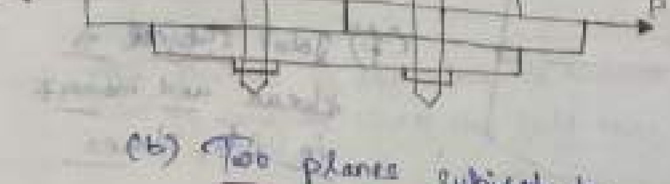
Depending upon the types of connections and loads, bolts are subjected to the following types of actions :

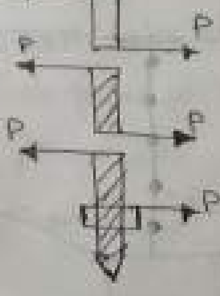
(a) Only one plane subjected to shear (Single shear)
(b) Two planes subjected to shear (double shear)
(c) Pure Tension.
(d) Pure Moment.
(e) Shear and moments in the plane of connection.
(f) Shear and tension.



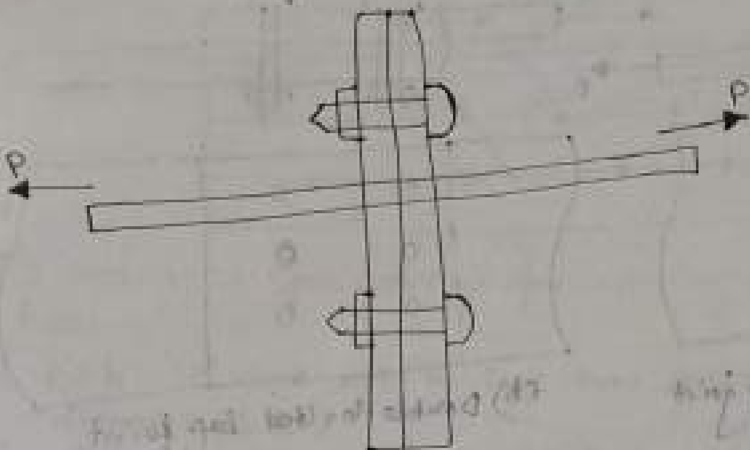


(a) Shear Plane on thread

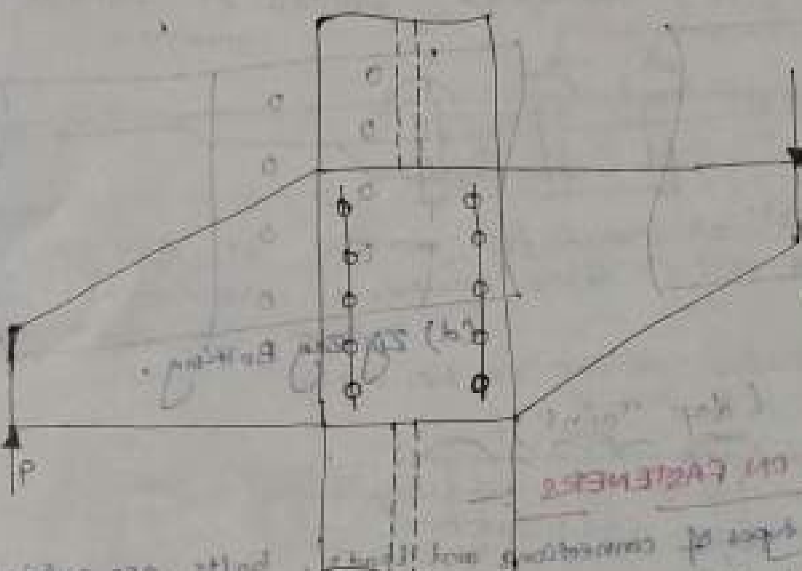




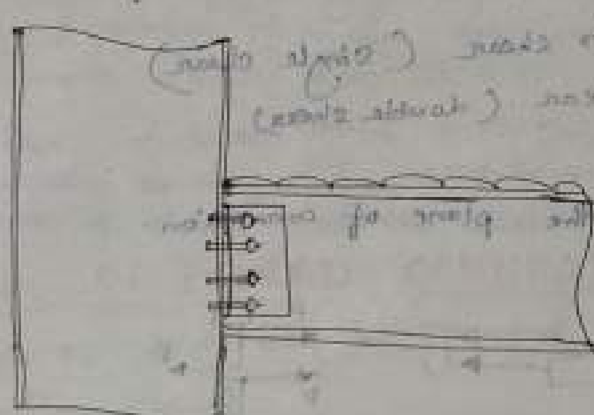
(b) Two planes subject to shear



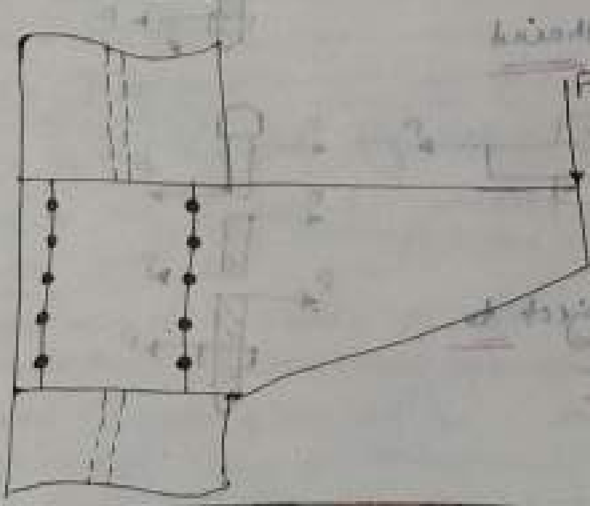
(c) Bolts in direction tension.



(d) Bolts resisting pure moments.



(e) Bolts subject to shear and tension



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

ASSUMPTIONS IN DESIGN OF BEARING BOLTS :-

The following assumptions are made in the design of bearing (finished or unfinished) bolted connections :

1) The friction betn the plate is negligible
2) The shear is uniform over the cross-section of the bolt.
3) The distribution of stress on the plate 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.

PRINCIPLES OBSERVED IN THE DESIGN :-

The following principles are observed in the design of connections :

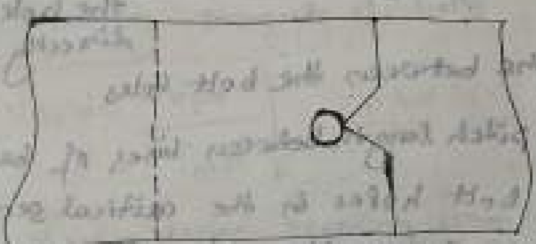
1) The centre of gravity of bolts should coincide with the centre of gravity of the connected members.
2) The length of connection should be kept as small as possible.

DESIGN TENSILE STRENGTH OF PLATES IN A JOINT :-

Plates in a joint made with bearing bolts may fail under tensile force ~~Base~~ due to any one of the following :

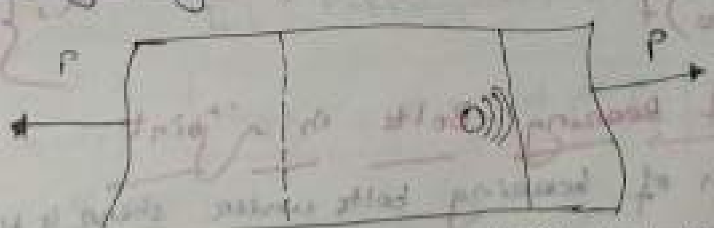
1) Bursting or shearing of the edge
2) Crushing of plates.
3) Rupture of plates.

The bursting or shearing and crushing failures are avoided if the minimum edge / end distances are provided. The plate will crack at the back of a bolt, if it is placed very near to the edge of the plate.



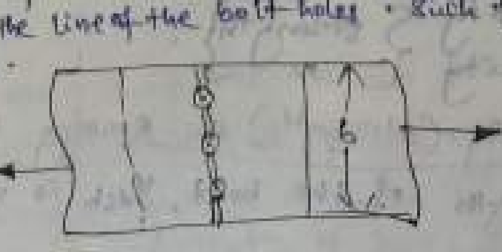
(Bursting or shearing of plates).

→ The plate or bolt is crushed if the compressive stress exceeds the bearing strength of the plate or bolt.



(Crushing of plates)

→ The strength of the plate is reduced by bolt holes and the plate may tear off along the line of the bolt holes. Such types of failure is for tension members.



(Rupture of plate)

Design strength of plates in a joint :-

Plates in a joint made with bearing type of bolts may fail due to bursting of the edge, crushing of plates in bearing, rupture of plate. The bursting of the edges and crushing failure of plates are generally avoided if the minimum edge/end distance are provided in a joint, the design tensile strength of plate in the joint is the strength of the thinnest member against rupture, this strength is given by :-

$$\boxed{T_{dn} = \frac{0.9\, A_n \cdot f_u}{\gamma_{m1}}}$$ {cl: 6.3.1. IS 800:2007 Pg: 32}

where,

γ_{m1} = Partial safety factor for failure at ultimate stress
$= 1.25$ (Pg - 30 (Table 5))

f_u = ultimate stress of the material

A_n = net effective area of the plate at critical section

$$\boxed{A_n = \left[b - n d_0 + \sum \frac{P_s^2 i}{4 g_i} \right] t}$$ (Pg-33, cl-6.3.1)

where,

b = width of plate

t = thickness of thinner plate in joint

d_0 = diameter of the bolt hole (2 mm in addition of the hole, in case of directly punched holes)

g = gauge lengths between the bolt holes.

P_s = 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.

{ If there is no staggering $P_s/g = 0$ and hence,
$A_n = (b - n d_0) t$ }

Design strength of Bearing Bolts in a joint

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

(a) Shear capacity } strength.
(b) Bearing capacity }

(a) Shear capacity (strength) of Bearing Bolts in a joint

Design strength of the bolt, V_{dsb} is given by

$$\boxed{V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}}$$ (Pg:- 75)

where,

V_{nsb}, nominal shear capacity of bolt

γ_{mb}, partial safety factor of material of bolt.

$$\boxed{V_{nsb} = \frac{f_{ub}}{\sqrt{3}} \left(n_n \, A_{nb} + n_s \, A_{sb} \right)}$$ (Pg- 75, cl-10.3.3)

where

f_{ub} = ultimate strength of the bolt.

n_n = number of ~~thread~~ 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.

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

$$\boxed{A_{nb} = \frac{\pi}{4}\left(d - 0.9382p\right)^2}$$ where p is pitch of thread

$$\simeq \frac{\pi}{4} \quad \boxed{= 0.78 \frac{\pi}{4} d^2}$$

(b) Bearing Capacity of Bolts :-

To find bearing strength of bolts

$$\boxed{V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}}$$ (Pg- 75, cl:- 10.3.4)

where,

V_{dpb} = design bearing strength.

V_{npb} = nominal bearing strength.

γ_{mb} = Partial safety factor of material = 1.25 } Table 2-2

$$\boxed{V_{npb} = 2.5\, k_b \cdot d \cdot t \cdot f_u}$$ (Pg- 75, cl- 10.3.4)

where,

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

where, e, p are end and pitch distances.

f_{ub}, f_u = ultimate tensile stress of the bolt and plate

d = nominal diameter of the bolt.

d_0 = diameter of bolt hole.

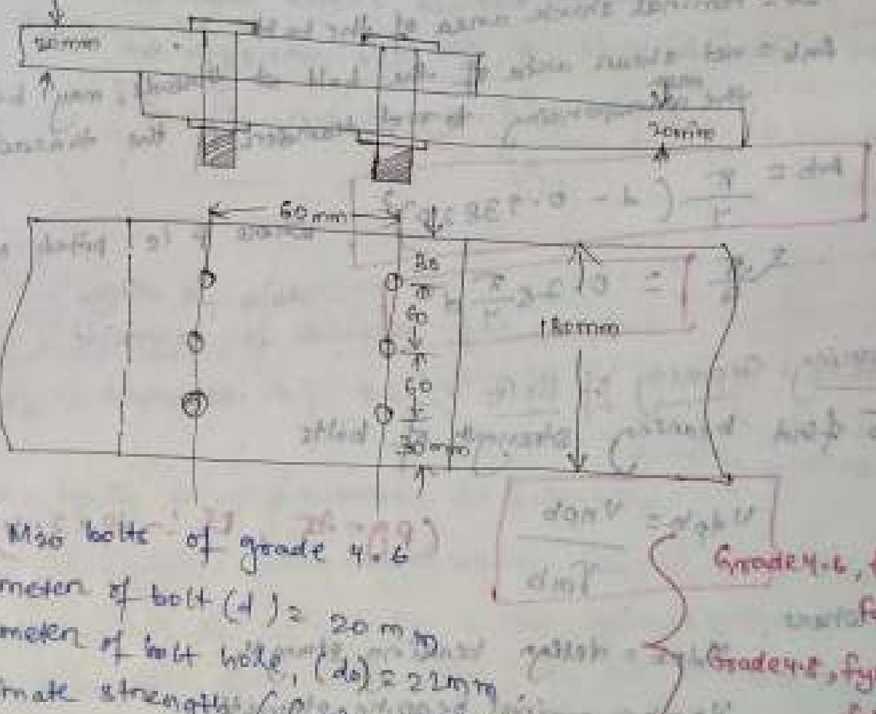
t = Summation of the thickness of the connected plates experiencing bearing stress in the same direction. If bolts are counter sunk, it is to be reduced by the half depth of counter sinking.

EFFICIENCY OF A JOINT

The ratio of strength of joint and strength of solid plate in tension. It is usually expressed in percentage.

$$\boxed{\text{Efficiency } (\eta) = \frac{\text{Strength of joint}}{\text{Strength of solid plate}} \times 100\%}$$

(Q) Find the efficiency of the lap joint. 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, (do) = 22 mm
Ultimate strength (fub) = 400 MPa.
Partial safety factor (γ_{mb}) = 1.25
For Fe 410 (E 250) plates,
Ultimate stress, fu = 410 MPa
Partial safety factor, γ_{ml} = 1.25

Grade 4.6, fyb = 240MPa
fub = 400MPa
Grade 4.8, fyb = 320MPa
fub = 420MPa
Grade 5.6, fyb = 300MPa
fub = 500 MPa
Grade 5.8, fyb = 400MPa
fub = 520 MPa

Strength of plates in the joint:

Thickness of thinner plate t = 20 mm
(width) b = 180 mm
There is no staggering, P_s = 0
No. of bolt holes on the weakest section = 3
∴ Net area of weakest section
$A_n = [b - nd_0 + 0] \times t$ (IS 800:2007, cl- 6.3.1)

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

Design strength of plates in the joint

$$T_{dn} = \frac{0.9 f_u \cdot A_n}{\gamma_{m1}} = \frac{0.9 \times 410 \times 2280}{1.25} = 673056 N$$

$= 673.056$ kN.

Strength of Bolts :-

Total no. of bolts = 6.

(i) Design strength in shear :-

No. of shear planes at thread $(n_n) = 1$ per bolts

No. of shear planes at shank $(n_s) = 0$ per bolt.

Total no. of $n_n = 1 \times 6 = 6$ & $n_s = 0$.

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

Nominal shear strength,

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}} (n_n \cdot A_{nb} + n_s \cdot A_{sb}) \quad \{ \text{Pg :- 75, cl- 10.3.3} \}$$

$$= \frac{400}{\sqrt{3}} (6 \times 245 + 0) = 339482 N = 339.482 \text{ kN}$$

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 :-

$V_{npb} = 2.5 \, k_b \cdot d \cdot t \cdot f_u$

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

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

(b) $\frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.6591$

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

(d) 1.0

Hence, $k_b = 0.4545$

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

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

Design strength of joint $= 6 \times 149076 = 894456.8\,N$

$\simeq 894.456\,kN$.

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

strength of joint $= 271.586\,kN$.

Efficiency of joint :—

Area of solid plate $= 180 \times 20 = 3600\,mm^2$ $(b \times t)$ (180×20)

Design strength of solid plate

$= \frac{f_y}{\gamma_{m0}} \times A_g = \frac{250}{1.1} \times 3600 = 818181.8\,N$

$\simeq 818.182$

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

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

(Q-2) Find the efficiency of the joint, if in the above example instead of lap, butt joint is made using two cover plates each of size 12mm and 6 numbers of bolts on each side

Ans:- In this case strength of plates and strength of bolts in bearing are same, strength in shear is different since in each bolt a section in root and another section at shank resist shear. Thus in this case total number of section resisting shear at shank $n_s = 6$

$A_{sb} = \frac{\pi}{4} d^2 = \frac{\pi}{4} \times 20^2 = 314.16\,mm^2$

$A_{nb} = 0.78 \frac{\pi}{4} d^2 = 0.78 \times \frac{\pi}{4} \times 20^2 = 245\,mm^2$ (Net area of bolt)

(Shear capacity of bolt)

∴ Nominal shear strength $= \frac{f_{ub}}{\sqrt{3}} (n_n \cdot A_{nb} + n_s \cdot A_{sb})$

$= \frac{400}{\sqrt{3}} (6 \times 245 + 6 \times 314.16)$

$= 774795\,N$

$= 774.795\,kN$.

∴ Design shear strength $= \frac{774.795}{1.25} = 619.836\,kN$

Design strength in bearing :-

$V_{npb} = 2.5\, k_b \cdot d \cdot t \cdot f_u$

$k_b = \dfrac{e}{3d_0} = \dfrac{30}{3\times22} = 0.4545$

$\rightarrow \dfrac{p}{3d_0} - 0.25 = \dfrac{60}{3\times22} - 0.25 = 0.6591$

$\rightarrow \dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.9756$

$\rightarrow 1.0$

$k_b = 0.4545$

$V_{npb} = 2.5\, k_b \cdot d\, t\, f_u$

$= 2.5\times0.4545\times20\times20\times410 = 186345$ N per bolt.

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

Design strength of joint $= 6\times149076 = 894456$ N

$= 894.456$ kN

$\therefore$ Design strength of bolt $= 619.8337$ kN)

Design strength of plates in the joint :-

$A_n = [b - n d_0] t = [180 - 3\times22]\times20 = 2280\ mm^2$

$T_{dn} = \dfrac{0.9 f_u A_n}{\gamma_{m1}} = \dfrac{0.9\times410\times2280}{1.25} = 673056$ N

$= 673.056$ kN

{ Since total thickness of cover plates $= 2\times12 = 24$ mm which is more than thickness of plates, strength of plates is the strength of main plate only. }

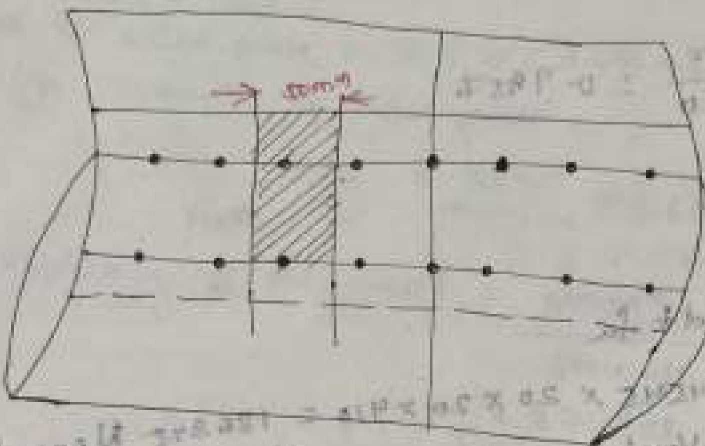
$\therefore$ Design strength of joint $= 619.836$ kN.

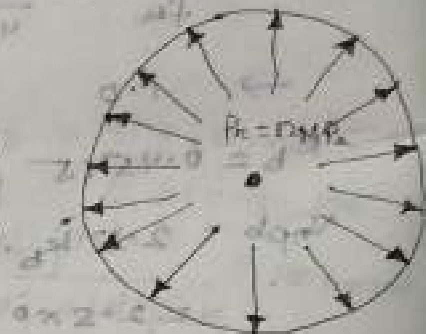
Design strength of solid plate $= \dfrac{250}{1.1}\times180\times20$

$= 818182$ N $\simeq 818.182$ kN

$\therefore$ Efficiency of joint $= \dfrac{619.835}{818.182} \times 100 = 75.76\%$.

(Q-3) A boiler shell is made up of 14mm thick Fe 410 plate. If the joint is double bolted lap joint with M16 bolts of grade 4.6 at distances of 50mm, determine the strength of the joint per pitch width. Is it a safe design of the internal diameter of bolt.





Ans:- Strength of plate per 50mm width:

Dia of bolts $d = 16mm$.

$\therefore$ diameter of bolt hole $(d_0) = 18mm$.

Strength of plate per 50mm width:

$t = 14mm$, $b = P = 50mm$, $f_u = 410$ MPa.

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

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

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

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

Strength of bolts per 50mm width:

Since it is lap joint, shear planes at shanks = 0.

As there are two bolts per pitch width considered 2.

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

$= 156.83\ mm^2$

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

$= 72436\ N$

$\therefore$ Design strength $(V_{dsb}) = \frac{V_{nsb}}{\gamma_{mb}} = \frac{72436}{1.25} = 57949 N$

$\simeq 57.949 kN$

<u>Design strength in bearing :-</u>

$k_b =$ — e is given in Pg-74, cl-10.2-4-2

(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'.

(b) $\frac{P}{3d_0} - 0.25 = \frac{50}{3\times18} - 0.25 = 0.6759$

(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

$= 2.5 k_b \cdot d\, t\, f_u = 2.5 \times 0.6759 \times 16 \times 14 \times 410 = 155187 N$

$= 155.187 kN$.

As there is 2 bolts, design strengths of bolt in bearing =

$2 \times 155.187 = 310.374 kN$

$\therefore 310.374 kN > V_{dsb}$

. Design strength of bolts $= 57.949 kN$

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

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

<u>To check the safety of joint :-</u>

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

where P_r is applied pressure and D is diameter of boiler

$= \frac{1.2 \times 1500}{2 \times 14} = 42.857 N/mm^2$

$\therefore$ Force per 50mm length $= 42.857 \times 14 \times 50 = 30,000 N$

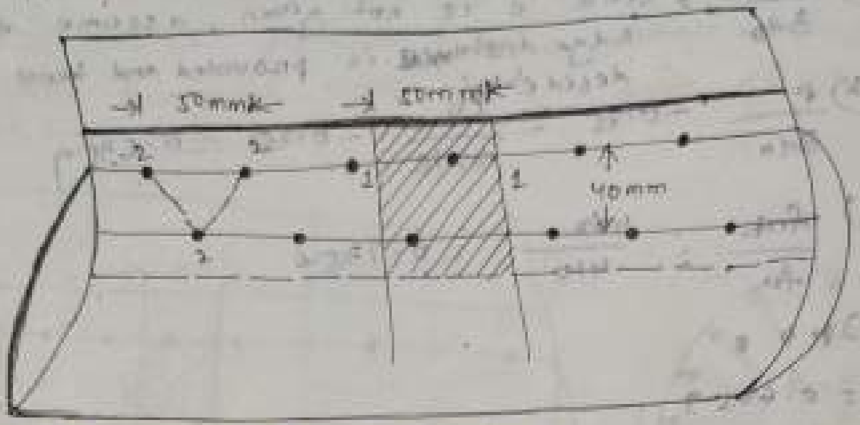
$= 30 kN$

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

Design strength $= 57.949 kN >$ Design action.

Hence the design is safe.

Q-4) A boiler is made up of 14mm thick Fe 415 plate. If the joint is double bolted lap joint with M16 bolts of grade 4.6 at distance of 50mm, determine the design strength of the joint per pitch width, if zig-zag bolting is used. Is it safe design if the internal diameter of boiler is 3m and steam pressure is 1.2 MPa?



Ans :- Consider the strength of joint per 50mm.

$d = 16mm$, $d_0 = 18mm$, $t = 14mm$.

No. of bolts per 50mm width = 2

∴ Design strength of plate :

It should be check along section (1)-(1) and (2)-(2)

Net cross sectional area resisting tearing along

(a) Section (1)-(1) :-

$$A_{n1} = (b - nd_0)t = (50 - 1 \times 18) \times 14 = 448 mm^2$$

(b) Section (2)-(2) :-

$$A_{n2} = \left(b - nd_0 + \sum \frac{P_s^2}{4g}\right)t$$

$$= \left[50 - 2 \times 18 + \frac{2 \times 40^2}{4 \times 25}\right] \times 14$$

$$= 644 mm^2$$

∴ Section (1)-(1) is weaker

Hence plate strength is —

$$T_{dn} = 0.9 \times 448 \times \frac{410}{1.25} = 132.250 kN \approx 132.250 kN$$

Strength of bolt per 50mm width of joint :-

(a) In shear = 57.949 kN

(b) In bearing = 155.127 kN

∴ Strength of joint = 44.367 kN

∴ Design action = 45 kN

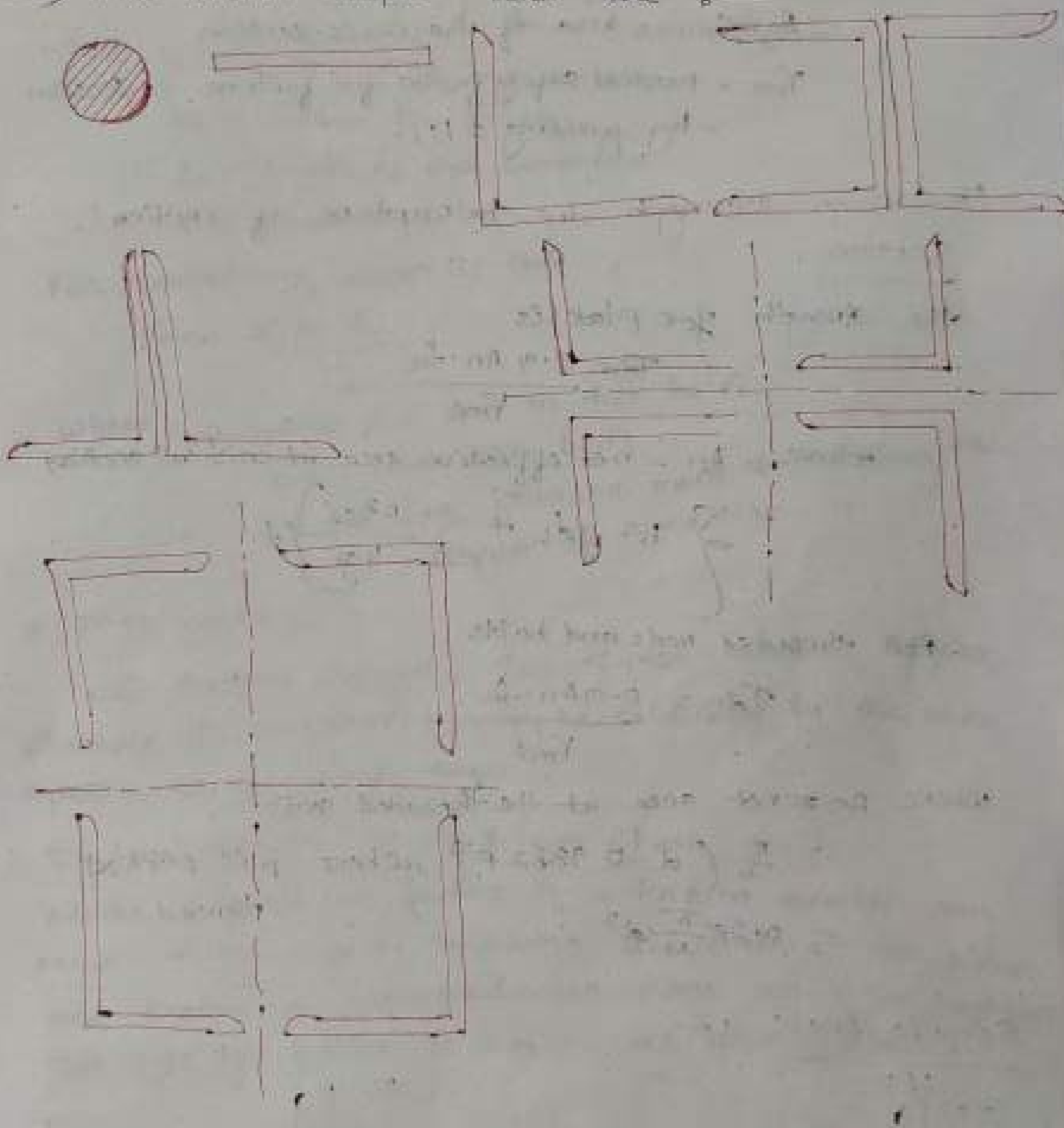
Design strength (57.949 kN) > Design action (45 kN)

∴ Design is safe

DESIGN OF TENSION MEMBER

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.

→ The common shapes used are :—



* DESIGN STRENGTH OF A TENSION MEMBER :—

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

(a) Design strength due to yielding of gross section (T_{dg})
(b) Rupture strength of critical section, (T_{dn}) and
(c) The block shear (T_{db})

(a) Design strength due to yielding of gross-section

This strength is given by

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}}$$

where, f_y = yield stress of the material

A_g = Gross area of the cross-section

γ_{m0} = partial safety factor for failure in tension by yielding = 1·1.

(b) Design strength due to rupture of critical section.

The strength for plate is

$$T_{dn} = \frac{0.9\, A_n \cdot f_u}{\gamma_{m1}}$$

where, A_n = net effective area at critical section

$$= \left\{ b - n d_h + \sum \frac{p^2 s_i}{4 g_i} \right\} t$$

For threaded rods and bolts

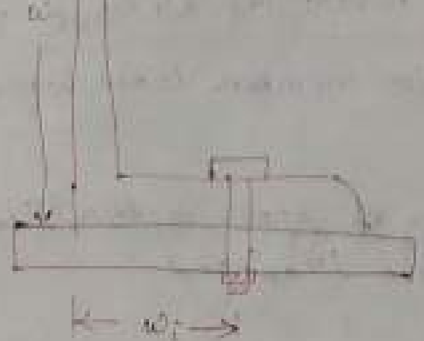
$$T_{dn} = \frac{0.9 A_n \cdot f_u}{\gamma_{m1}}$$

where A_n = net area at the threaded section

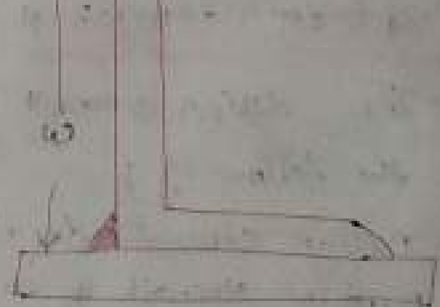
$= \frac{\pi}{4}(d - 0.9382\, p)^2$, where p is pitch of thread

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

Single Angle :—



$b_s = w + w_1 - t$



$b_s = w$

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

where,

A_{nc} = net area of the connected leg.

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

$$\beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\frac{b_s}{L_c} \le \frac{f_u \cdot \gamma_{m0}}{f_y \cdot \gamma_{m1}} \ge 0.7$$

where, w = outstanding leg width.

b_s = shear leg width.

L_c = length of end connection.

t = thickness of leg.

For preliminary design IS code,

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

where, $\alpha = 0.6$ for one or two bolts.

$= 0.7$ for three bolts.

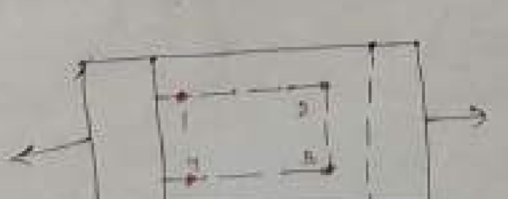
$= 0.8$ for four or more bolts along the length of connection.

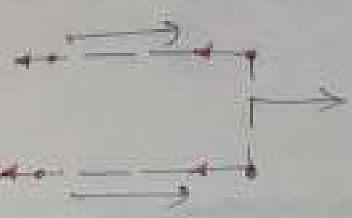
* other sections :-

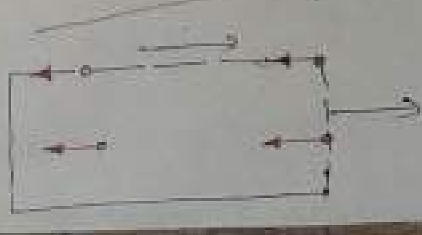
The rupture strength, T_{dn} of the double angles, channels, I-sections etc., may be calculated by the same equation as for single angle.

(c) 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.







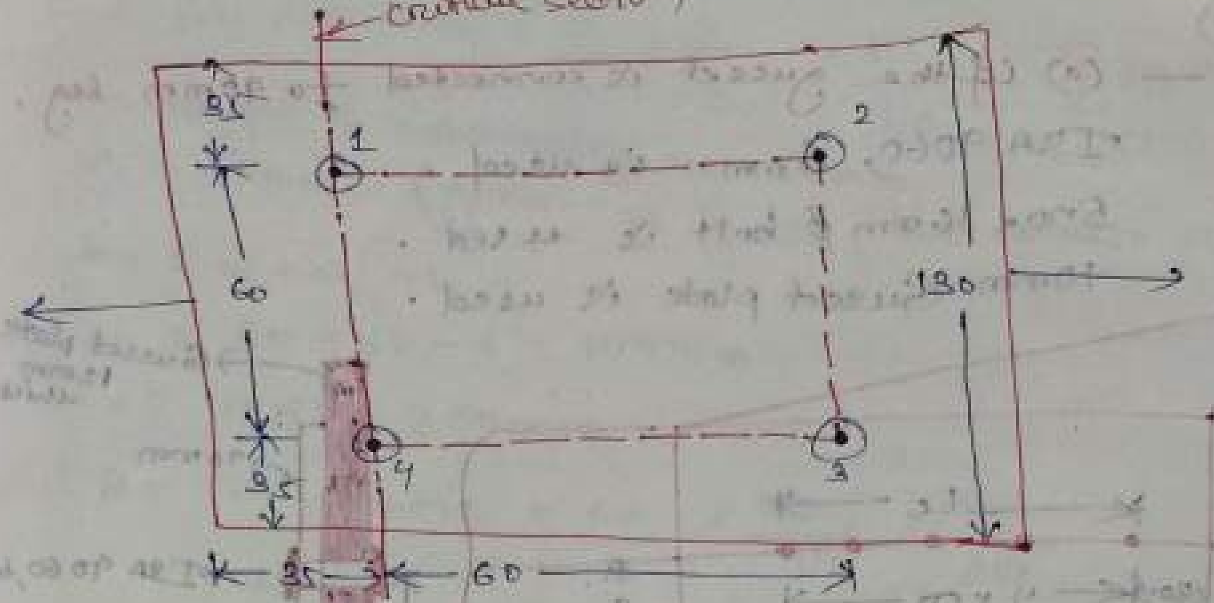
$$T_{db} = \frac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{m0}} + \frac{0.9 A_{tn} \cdot f_u}{\gamma_{m1}}$$

OR

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

A_{vg} and A_{vn} = minimum gross & net area in shear

A_{tg} & A_{tn} = minimum gross & net area in tension



(b) Rupture critical sectⁿ :— $d_h = 16+2 = 18$ mm

$A_n = (b - n \cdot d_h)\, t$

$= (130 - 2 \times 18) \times 12 = 1128 \text{ mm}^2$

(no. of bolt along critical sectⁿ is 2)
(The section goes through sectⁿ 1–4)

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

(c) Block shear strength :— (pg – 33)

A_{vg} = minimum gross area along 1–2 & 3–4

A_{vn} = Net area in shear along 1–2 & 3–4

A_{tg} = minimum gross area along 2–3

A_{tn} = Net area in tension from the bolt hole to the toe of angle in 2–3 sectⁿ

$A_{vg} = 2 \times (35 + 60) \times 12 = 2280 \text{ mm}^2$

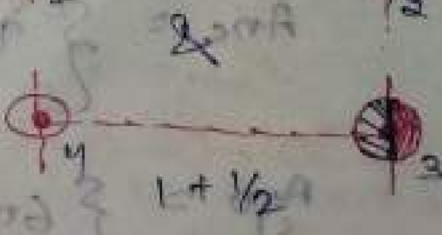
2× is for 1–2 & 3–4 sectⁿ

(35 + 60) → width till the last bolt; 12 → t

$A_{vn} = 2 \times \{(35+60) - 1.5 \times 18\} \times 12 = 1632 \text{ mm}^2$

2× for 1–2 & 3–4 $\left(\frac{1}{2}+1\right)$

1.5 → $\left(1+\frac{1}{2}\right)$



$A_{tg} = 60 \times 12$
$(b \times t)$

$A_{tn} = (60 - 18) \times 12$

18 → d_h

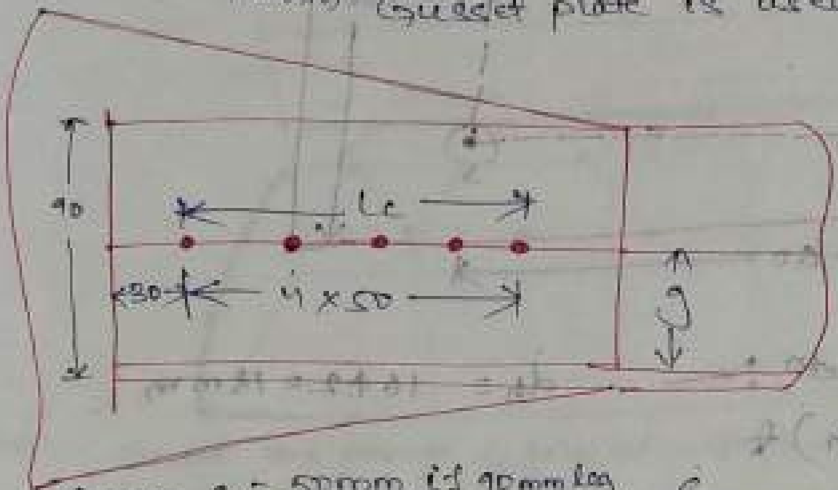
(Q-5.2)

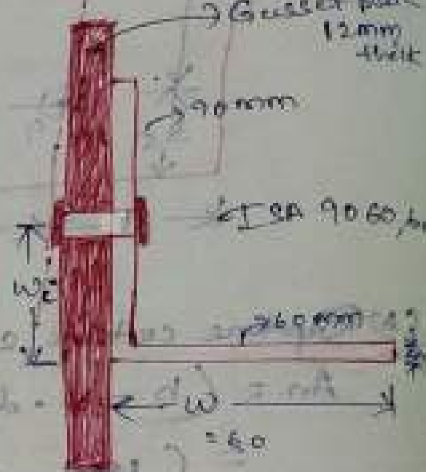
Ans: — (a) if the gusset is connected to 90mm leg.

ISA 9060, 6mm is used.

5no. 16mm ϕ bolt is used.

10mm Gusset plate is used.





Given, $g = 50mm$, if 90mm leg is connected

$g = 30mm$, if 60mm leg is connected

{ 1 leg is 90
2nd leg is 60
Thickness is 6mm }

(i) Strength as governed by yielding of gross section:

$$T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}} \quad \{Pg:-32\}$$

$A_g = 865\ mm^2$ { From steel table Pg 1-14 }

As ISA section 9060, 6mm is used

$$T_{dg} = \frac{864 \times 250}{1.1} = 196.364\ kN$$

(Pg 1-30)

(ii) Strength as governed by tearing at critical section

A_{nc} = net area of connected leg.

A_{go} = Gross area of the outstanding leg.

{ Here, 90mm leg is connected }

$$A_{nc} = \left\{ \underbrace{90}_{\text{connected leg}} - \frac{6}{2} \right\} \times 6 = 522mm^2 \quad \{Pg:-33\}$$

$\frac{6}{2}$ → t/2

$$A_{go} = \left\{ \underbrace{60}_{\text{Outstanding leg}} - \frac{6}{2} \right\} \times 6 = 342mm^2$$

$\frac{6}{2}$ → t/2

$\beta = 1.4 - 0.076 \times \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_c}$ {pg 1-33) (cl-6.3.3)

w = length of outstanding leg = 60mm

w_i = 50mm (given as $g = 50$, w_i is also 50)

$b_s = w + w_i - t$

$= 60 + 50 - 6 = 104$mm

$L_c = 4 \times 50 = 200$mm

$\therefore \beta = 1.4 - 0.076 \times \frac{60}{6} \times \frac{250}{410} \times \frac{104}{200}$

$= 1.159$

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

$= 1.159 \leq \left(\frac{410 \times 1.10}{250 \times 1.25}\right) \geq 0.7$

$1.159 \leq 1.4432 \geq 0.7$

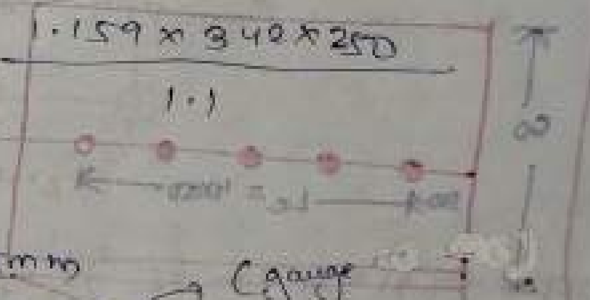
Hence take,

$\beta = 1.159$

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

$= \frac{0.9 \times 522 \times 410}{1.25} + \frac{1.159 \times 342 \times 250}{1.1}$

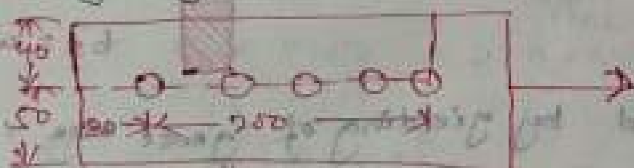
$= 244.180$ kN



(ii) Block shear strength :-

$d = 16$mm $d_h = 16 + 2 = 18$mm

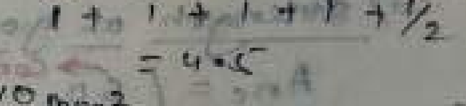
Tearing length in tension = $90 - 50$ (gauge)



$A_{vg} = (200 + 30) \times 6 = 1380$ mm^2

$A_{vn} = \{(200+30) - (4.5 \times 18)\} \times 6 = 894$ mm^2

{4 + ½}



$A_{tg} = \{90 - 50\} \times 6 = 40 \times 6 = 240$ mm^2

$A_{tn} = \{(90 - 50) - \frac{1}{2} \times 18\} \times 6 = 186$ mm^2 (Perpendicular only taken)

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

$= \dfrac{1380 \times 250}{\sqrt{3} \times 1.10} + \dfrac{0.9 \times 186 \times 410}{1.25}$

$= 235.985$ kN

or

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

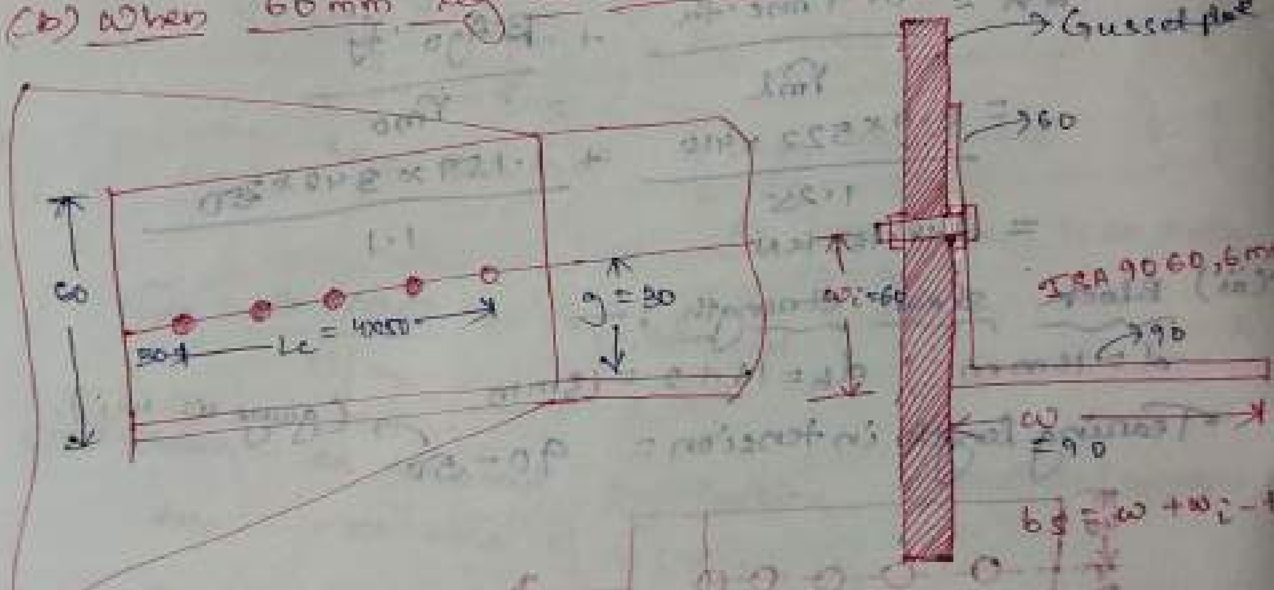
$= \dfrac{0.9 \times 894 \times 410}{\sqrt{3} \times 1.25} + \dfrac{240 \times 250}{1.1}$

$= 206.913$ kN

Thus, when 90 mm leg is connected, the strength of the plate is the least of 196.364 kN, 244.180 kN & 206.913 kN.

So, strength of plate i.e 196.364 kN.

(b) When 60 mm leg is connected to gusset plate:



(i) Strength as governed by yielding of gross area

$T_{dg} = \dfrac{A_g \cdot f_y}{\gamma_{m0}}$, $A_g = 865$ mm² (from steel table)

$\therefore T_{dg} = \dfrac{864 \times 250}{1.10} = 196.364$ kN

(ii) Strength as governed by tearing at critical section:

$A_{nc} = \left(60 - \dfrac{6}{2}\right) \times 6 = 342$ mm² (connected leg)

$A_{go} = \left(90 - \dfrac{6}{2}\right) \times 6 = 522$ mm² (outstanding leg; 6 = thickness)

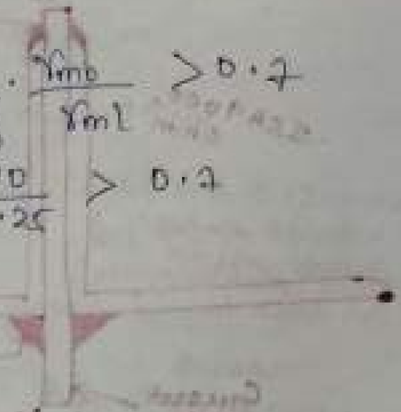
$$\beta = 1.4 - 0.076 \times \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_c}$$

$w = 90\,mm$ (outstanding leg)

$w_i = 30\,mm$

$b_s = w + w_i - t = 90 + 30 - 6 = 114\,mm$

$L_c = 50 \times 4 = 200\,mm$



$$\beta = 1.4 - 0.076 \times \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_c} < \frac{f_u}{f_y} \cdot \frac{\gamma_{m0}}{\gamma_{m1}} > 0.7$$

$$= 1.4 - 0.076 \times \frac{90}{6} \times \frac{250}{410} \times \frac{114}{200} < \frac{410}{250} \times \frac{1.10}{1.25} > 0.7$$

$= 1.0037 < 1.4432 > 0.7$

$= 1.004 < 1.4432 > 0.7$

$\beta = 1.004$

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

$$= \frac{0.9 \times 342 \times 410}{1.25} + \frac{1.004 \times 522 \times 250}{1.10}$$

$\simeq 220.067\,kN$

(iii) Block shear strength :—

Tearing length in tension $= 60 - 30 = 30\,mm$

$A_{vg} = (200 + 30) \times 6 = 1380\,mm^2$

$A_{vn} = \{(200 + 30) - (4.5 \times 18)\} \times 6 = 894\,mm^2$

$4 + \frac{1}{2} = 4.5$

$A_{tg} = \{(60 - 30) \times 6\} = 180\,mm^2$

$A_{tn} = \{(60 - 30) - \frac{1}{2} \times 18\} \times 6 = (30 - 0.5 \times 18) \times 6 = 126\,mm^2$

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

$$= \frac{1380 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 126 \times 410}{1.25} = 218.273\,kN$$

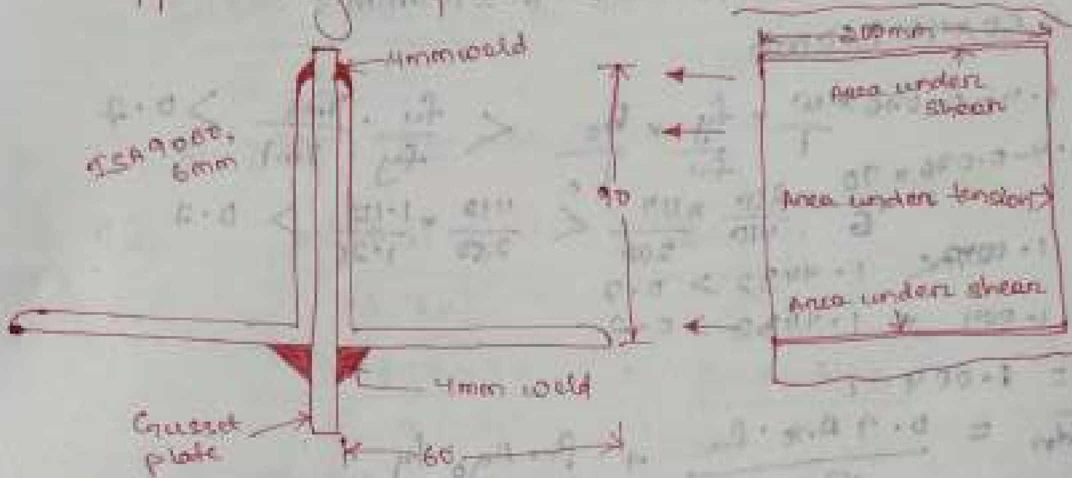
or

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

$$= \frac{0.9 \times 894 \times 410}{\sqrt{3} \times 1.25} + \frac{180 \times 250}{1.10} = 193.27\,kN$$

$\therefore$ strength of member is $193.27\,kN$.

(Q-) Determine the tensile strength of a roof truss member 2 ISA 9060, 6mm connected to the gusset plate of 8mm thickness by 4mm weld as shown in Fig. The effective length of weld is 200mm.



Solution

Gross Area of one angle = 865 mm^2 (from steel table (Pg-14))

Gross area of angles, $(A_g) = 2 \times 865 = 1730 mm^2$

Area of the connected leg, $(A_{nc}) = 2\left(90 - \frac{6}{2}\right) \times 6$
$= 1044 mm^2$ for two angles

Area of the outstanding leg, $(A_{go}) = 2\left(60 - \frac{6}{2}\right) \times 6$ (thickness)
$= 684 mm^2$ for two angles

(i) Strength governed by yielding

$$\frac{A_g \cdot f_y}{\gamma_{m0}} = \frac{1730 \times 250}{1.10} = 393.182 \text{ KN}$$

(ii) Strength of the plate in rupture at critical section:-

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

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

In this case,
$b_s = 90mm$, $t = 6mm$, (shear leg width) $f_u = 410 MPa$, $f_y = 250 MPa$,
$w = 60mm$, $L_w = 200mm$.

b_s = (Shear leg width)
w = Outstanding leg.

$$\therefore \beta = 1.4 - 0.076 \times \frac{60}{6} \times \frac{250}{410} \times \frac{90}{200} < \left(\frac{f_u \cdot \gamma_{m0}}{f_y \cdot \gamma_{m1}}\right) > 0.7$$

$= 1.191$

$\simeq 1.2 < 1.443 > 0.7$

$\beta = 1.2$

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

$$= \frac{0.9 \times 410 \times 1044}{1.25} + \frac{1.2 \times 684 \times 250}{1.1}$$

$$= 494.734 \text{ kN}$$

(iii) Strength governed by block shear :—

One shear leg and one tension failure can occur

For each angle area under shear, $A_{vg} = A_{vn} = 200 \times 6 = 1200 \text{ mm}^2$

(Length of weld along parallel direction) (Length of weld under shear)

(min gross & net area in shear)

Area under tension at failure = $A_{tg} = A_{tn} = 90 \times 6 = 540 \text{ mm}^2$

(Length along tension (Perpendicular))

(min gross & net area in tension)

Strength in block shear is the smaller of the following two:

$$T_{db} = \frac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$$

$$= \frac{1200 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 540 \times 410}{1.25} = 316.867 \text{ kN}$$

$\simeq 2 \times 316.867$
$= 633.734$ kN
for two angle

or

$$T_{db} = \frac{0.9 A_{vn} \cdot f_u}{\sqrt{3} \cdot \gamma_{m1}} + \frac{A_{tg} \cdot f_y}{\gamma_{m0}}$$

$$= \boxed{\frac{0.9 \times 1200 \times 410}{\sqrt{3} \times 1.25}} + \frac{540 \times 250}{1.1} = 327.248 \text{ kN}$$

$\simeq 2 \times 327.248$
$= 654.496$ kN
for two angle.

∴ Strength of tension member = 392.727 kN

DESIGN PROCEDURE

The design procedure may be adopted

1.) Find the required gross area to carry the factored load considering the strength in yielding i.e.,

$$\boxed{A_g = \frac{T_u}{(f_y / \gamma_{m0})} = \frac{1.1 T_u}{f_y}}$$ → Pg (32)

T_u = 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 percent more than "A_g" 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

(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 A_g (provided) > A_g (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.

6.> The check for slenderness ratio of tension member as per table 3 (Pg-20, Table No-3)

Example (5.6)

(Q) Design a single angle section for a tension member of a roof truss to carry a factored tensile force of 225kN. The member is subjected to the possible reversal of stress due to the action of wind. The effective length of the member is 3m. Use 20mm shop bolts of grade 4.6 for the connection.

Ans:-

From the consideration of yield strength, gross area of the angle required $\boxed{(A_g) = \frac{T_u}{f_y/\gamma_{m0}}}$ (Pg-32)

$$A_g = \frac{225\times10^3}{250/1.10} = 990\ mm^2$$

Try ISA 100 75, 8mm which has gross area $A_g = 1336\ mm^2$

{From steel table, Pg-14}

No. of bolt required :-

Use gusset plate of thickness 10mm

$d = 20mm$, $d_h = 20+2 = 22\ mm$

Strength of one bolt in single shear

$$= \frac{f_{ub}}{\sqrt{3}}\frac{(n_n A_{nb} + n_s A_{sb})}{1.25} \quad \left\{\begin{matrix} n_s = 0 \\ n_n = 1 \end{matrix}\right\}$$

$$= \frac{400}{\sqrt{3}}\frac{(0 + 1\times\frac{\pi}{4}\times(20)^2\times0.78)}{1.25}$$

$= 45272\ N$

Adopting edge distance (e) = 40 mm

pitch distance (p) = 60mm.

Pitch distance, minimum = 2.5 d
= 2.5 × 20 = 50 mm

Maximum = 16t or 200

Edge distance, Minimum = 1.7 × d_h or 1.5 d_h
= 1.7 × 22 = 37.4 mm

Edge distance = 37.4 (min) < 40 (Provided) < 16tε (max), $\varepsilon = \sqrt{\frac{250}{f_y}}$

Pitch distance = 50 mm (min) < 60 mm Provided < 16t or 200 (max)

K_b is smaller of $\frac{e}{3d_h}$, $\frac{p}{3d_h} - 0.25$, $\frac{f_{ub}}{f_u}$, 1.0

$\rightarrow \frac{e}{3d_h} = \frac{40}{3 \times 22} = 0.6060$

$\rightarrow \frac{p}{3d_h} - 0.25 = \frac{60}{3 \times 22} - 0.25 = 0.6590$

$\rightarrow \frac{400}{410} = 0.9756$

$\rightarrow$ 1.0

Hence, $K_b = 0.606$

strength of one bolt in bearing

$$V_{dpb} = \frac{2.5 \times K_b \times d \cdot t \cdot f_u}{\gamma_{mb}}$$

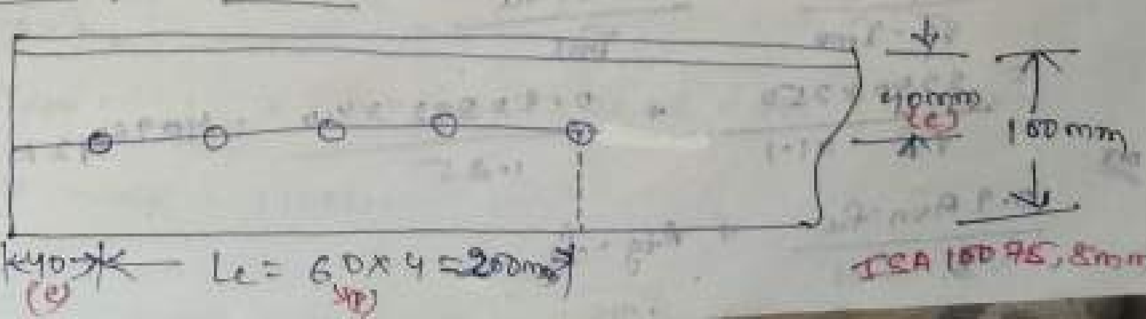
$$= \frac{2.5 \times 0.606 \times 20 \times 8 \times 400}{1.25} = 77568\,N$$

∴ Bolt strength = 45272 N

Number of bolts required = $\frac{\text{Force}}{\text{strength}}$

$$= \frac{225 \times 10^3}{45272} = 5 \text{ nos}$$

Provide the bolts :—



Checking the design :-

(a) Strength against yielding :- ($A_g = 1336\ mm^2$, pg = 14 steel table)

$$\frac{A_g f_y}{\gamma_{m0}} = \frac{1336 \times 250}{1.1} = 303636\,N$$

$303636\,N > 225000\,N$

(b) Strength of plate in rupture :-

Area of connected leg = A_{nc} =

$$\left(100 - \overset{d_h}{22} - \frac{8}{2}\right) \times 8 \quad \text{thickness}$$

$= 592\,mm^2$ outstanding leg

{ d_h is substracted }

Area of outstanding leg = A_{go} = $\left(75 - \frac{8}{2}\right) \times 8$

$= 568\,mm^2$

$$\beta = 1.4 - 0.076 \times \frac{w}{t} \times \frac{250}{410} \times \frac{b_s}{L_c}$$

$$= 1.4 - 0.076 \times \frac{75}{8} \times \frac{250}{410} \times \frac{(75 + 40 - 8)}{240}$$

$= 1.206$

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

$$= \frac{0.9 \times 410 \times 592}{1.25} + \frac{1.206 \times 568 \times 250}{1.1}$$

$= 330442\,N > 225000\,N$

(c) Strength against block shear failure :

$A_{vg} = \{(40 + 60 \times 4)\} \times 8 = 2240\,mm^2$

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

$A_{tg} = (100 - 40) \times 8 = 480\,mm^2$

$A_{tn} = \{(100 - 40) - 0.5 \times 22\} \times 8 = 392\,mm^2$

Strength against block shear facture is smaller of

$$= \frac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{m0}} + \frac{0.9 A_{tn} \cdot f_u}{\gamma_{m1}}$$

$$= \frac{2240 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 392 \times 410}{1.25} = 409642\,N$$

or

$$\frac{0.9 A_{vn} \cdot f_u}{\sqrt{3} \cdot \gamma_{m1}} + \frac{A_{tg} \cdot f_y}{\gamma_{m0}}$$

$= \dfrac{0.9 \times 1448 \times 410}{\sqrt{3} \times 1.25} + \dfrac{480 \times 250}{1.1}$

$= 355879\ N$

$\therefore T_{db} = 355879\ N > 225000\ N$

Hence safe.

Check for maximum values of effective slenderness ratio:

$\dfrac{l}{r}$ where, r = least radius of gyration = 12.7mm {from steel table}

$\therefore \dfrac{l}{r} = \dfrac{3000}{12.7} = 236$ {l = 3m (given)}

$\dfrac{l}{r} < 250$

$236 < 250$ (Hence ok).

(Q) Design a double angle tension member connected on each side of a 10mm thick gusset plate, to carry an axial factored load of 375 KN. Use 20mm black bolts. Assume shop connection.

Ans :- Design strength due to yielding $= \dfrac{A_g \cdot f_y}{\gamma_{m0}}$

Area required from the consideration of yielding $= \dfrac{1.1 \times 375 \times 1000}{250}$

$= 1650mm^2$

Try 2ISA 9550, 8mm thick which has gross area $= 2 \times 938$ {from steel table}

$= 1856mm^2$

Strength of 20mm black bolts :—

(a) In double shear $= \dfrac{400}{\sqrt{3}} \left\{ \dfrac{1 \times \frac{\pi}{4} \times 20^2 + 1 \times 0.78 \times \frac{\pi}{4} \times 20^2}{1.25} \right\}$

$= 103314\ N$

(b) Strength in bearing :

Taking, e = 40mm, p = 60mm.

K_b is smaller of $\dfrac{40}{3 \times 22} = 0.606$

$\rightarrow \dfrac{60}{3 \times 22} - 0.25 = 0.6590$

$\rightarrow \dfrac{400}{410}$

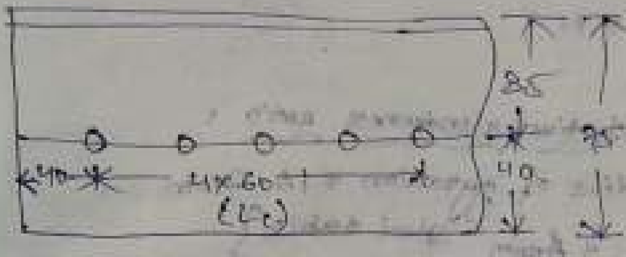
$\rightarrow 1.0$

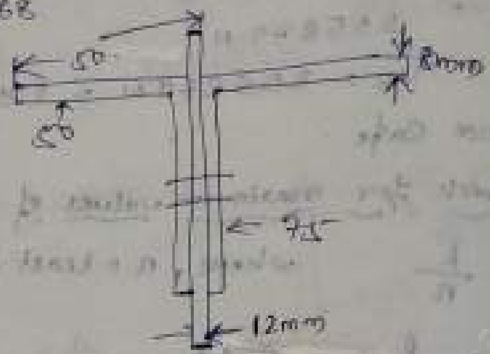
i.e, $K_b = 0.606$

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

Bolt value = 77568N

Number of bolt required $= \dfrac{375000}{77568} = 4.83 \approx 5$





$w = 50\,mm$

$b_s = 50 + 35 - 8 = 77mm$.

$L_c = 4 \times 60 = 240mm$.

Checking the design :-

(a) Strength against yielding $= \dfrac{A_g \cdot f_y}{\gamma_{m0}} = \dfrac{1876 \times 250}{1.1}$

$= 426364N > 375000N$ (OK)

(b) Strength of plate in rupture,

Area of connected leg,

$A_{nc} = 2\left\{75 - 22 - \dfrac{8}{2}\right\} \times 8 = 784\,mm^2$

Area of outstanding leg,

$A_{go} = 2 \times \left\{50 - \dfrac{8}{2}\right\} \times 8 = 736\,mm^2$

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

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

$= 1.307$

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

$= \dfrac{0.9 \times 410 \times 784}{1.25} + 1.307 \times \dfrac{736 \times 250}{1.10}$

$= 450062 > 375000N$ (ok)

(c) Strength against block shear failure :-

$A_{vg} = (40 + 60 \times 4) = 2240\,mm^2$

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

$A_{tg} = (75 - 35) \times 8 = 320\,mm^2$

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

per angle

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

(i) $\dfrac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{m0}} + \dfrac{0.9 \times A_{tn} \times f_u}{\gamma_{m1}}$

$$= \frac{2240 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 282 \times 410}{1.25} = 362410\ N$$

iii) $$\frac{0.9\, A_{vn} \cdot f_u}{\sqrt{3} \cdot \gamma_{m1}} + \frac{A_{tg} \cdot f_y}{\gamma_{m0}}$$

$$= \frac{0.9 \times 1448 \times 410}{\sqrt{3} \times 1.25} + \frac{320 \times 250}{1.10} = 319515\ N$$

$\therefore$ Strength of two angles block failure $= 2 \times 319515$

$$= 639030 > 375000$$

Hence use 2 ISA 9550, 8mm with 5 bolts of 20mm diameter.

DESIGN OF COMPRESSION MEMBER

A compression member is a structural member which is straight and subjected to two equal and opposite compressive forces applied at its ends.

→ An ideal compression member is one which is perfectly straight, has no crookedness, no imperfections; and the loads are applied uniformly across it, with the centre of gravity of loads coinciding with centre of gravity of the member. Such a compression member will be a truly axially loaded member.

→ Different terms are used to designate a compression member depending upon its position in structures. Column, stanchion or post is a vertical compression member supporting floors or girders in a building. These compression members are subjected to heavy loads.

→ Strut is a compression member used in the roof truss and bracing. It is small span and lightly loaded compression member. A strut may be continuous or discontinuous.

A continuous strut is a compression member which is continuous over a number of joints, such as a top chord member of a truss bridge girder, principal rafter of a roof truss, etc.

A discontinuous strut is a compression member which extends between two adjacent joints only, e.g., vertical or inclined compression members in a roof truss. The principal rafter is a top chord member in a roof truss and boom is the principal compression member in a crane.

EFFECTIVE LENGTH :-

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.

→ According to IS 800-2007 the cross-section is divided into four buckling class a, b, c, & d.

→ Buckling class of cross-section in table-10 in IS 800 page-44.

SLENDERNESS RATIO

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

$$\boxed{\text{Slenderness Ratio} = \frac{l_e}{r} = \frac{KL}{r}}$$

where, L = actual length of compression member.

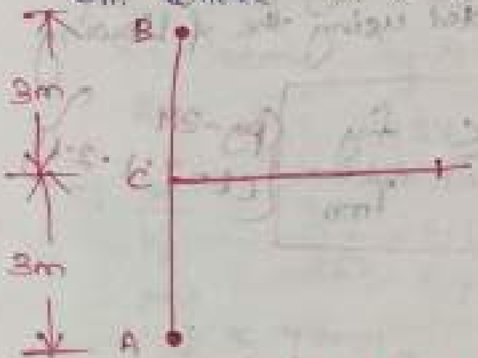
l_e = effective length = KL

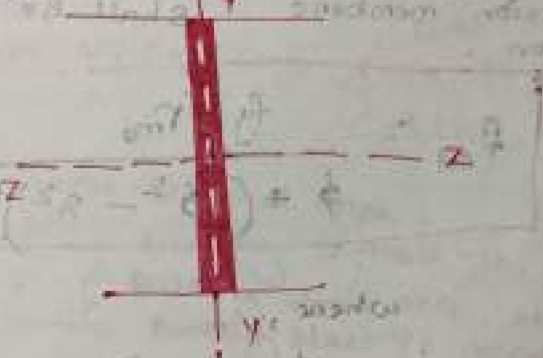
r = appropriate radius of gyration

ACTUAL LENGTH :-

It is the centre to centre distance of compression member between the restrained ends.

* Ex 1:- A 6m 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 6m while in Z-Z direction it is equal to AC = 3m only.





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.

(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 using the Table - 11 in IS - 800 :- {Pg - 45}

(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 restrains provided.
(Pg - 95)(Cl - 7.2.4)

(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, the effective length depends upon stiffness of the members meeting at the joint.

APPROPRIATE RADIUS OF GYRATION :-

Appropriate radius of gyration means the radius of gyration of compression member about the axis of buckling. 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 COMPRESSIVE STRESS AND STRENGTH :-

The design compressive stress, fcd of axially loaded compression members shall be calculated using the following equation.

$$\boxed{f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + (\phi^2 - \lambda^2)^{0.5}} \leq \frac{f_y}{\gamma_{m0}}}$$ {pg-34 Cl-7.1.2.1}

where,

$$\phi = 0.5\left[1 + \alpha(\lambda - 0.2) + \lambda^2\right]$$

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 (Table 7) (Pg-35)

γ_{m0} = 1.1 for Fe 415 steel.

The design compressive strength 'P_d' of a member is given by

$$\boxed{P_d = A_e \cdot f_{cd}}$$ {Pg-34, Cl-7.1.2}

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 filled with bolts.

{ Effective length of prismatic compression members is in pg-45, Table-11 }

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

Ans :- For rolled steel section,

$f_y = 250\ N/mm^2$, $f_u = 410\ N/mm^2$, $E = 2\times10^5\ N/mm^2$ ↳ Modulus of elasticity.

For both end pinned columns,

$KL = L = 3m$.

For ISHB 300 @ 577 N/m

$h = 300mm$, $b_f = 250\ mm$, $t_f = 10.6mm$ { Data given pg-20, steel table ISHB 576.8 }

$A_e = A = 7484\ mm^2$

↳ effective area = gross-sectional area.

$\therefore \frac{h}{b_f} = \frac{300}{250} = 1.2$ { According to table-10, Pg-44 Buckling about axis, and buckling class is calculated }

and $t_f < 40mm$

Hence according to Table 6.1

It falls under buckling class 'b' for buckling about z-z axis and under 'c' for buckling about y-y axis.

From steel table,

$r_{min} = r_{yy} = 54.1mm$

$$\therefore f_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2\times2\times10^5}{\left(\frac{3000}{54.1}\right)^2} = 641.92\ N/mm^2$$

(Pg-34, cl-7.1.2.1)

Non-dimensionalised effective slenderness ratio

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{641.92}} = 0.624 \quad \{Pg-34\}$$

For buckling class 'b',

$\alpha = 0.34$ { Table - 7, Pg - 35 }

$$\therefore \phi = 0.5\left[1 + \alpha(\lambda - 0.2) + \lambda^2\right]$$

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

$$= 0.767$$

$$\therefore f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + (\phi^2 - \lambda^2)^{0.5}} \quad \{Pg-34\}$$

$$= \frac{250/1.1}{0.767 + (0.767^2 - 0.624^2)^{0.5}} = 187.36\ N/mm^2$$

∴ Strength of column,

$P_d = A_e f_{cd} = 784 \times 182.96$
$= 1402237$ N
≈ 1402.237 kN

∴ Working Load $= \dfrac{1402.237}{1.5} = 934.822$ kN

SHAPES OF COMPRESSION MEMBERS

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.

→ Requirements for compression members are more demanding than those for tension members, for here the carrying capacity is a function of the shape as well as the area and also the material properties. The material must be disposed so as to resist effectively and tendency towards general or local instability. Thus, the member must be sufficiently rigid to prevent general buckling in any possible direction, and each plate element of the member must be thick enough to prevent local buckling.

→ Rolled steel section cost less than the built-up sections per unit weight and therefore preferred.

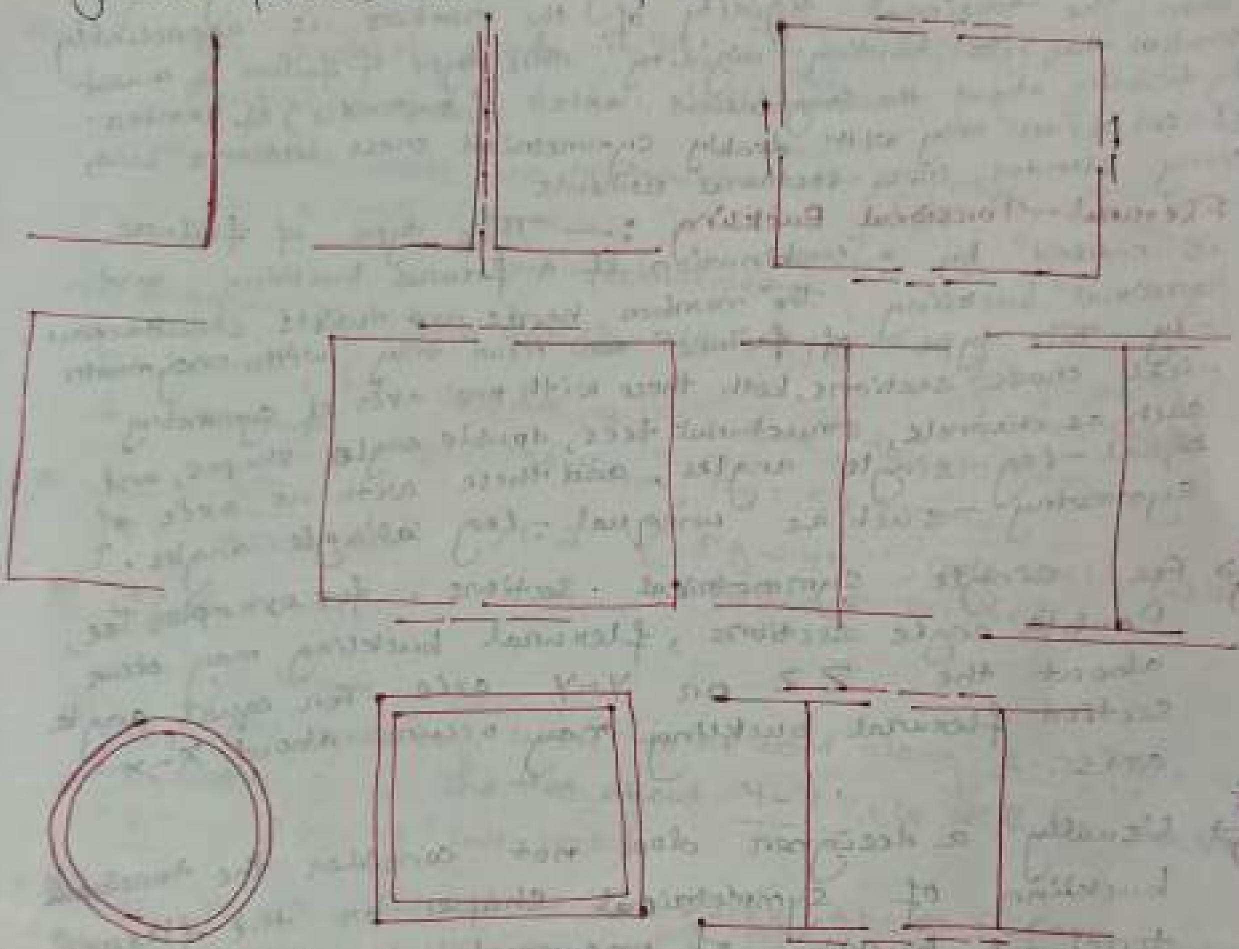
→ Rods and bars withstand very little compression when length is more. Hence these are recommended for lengths less than 3m only.

→ Tubular sections are most suitable for small loads and lengths. These sections are usually provided for roof trusses and bracings. The use of tubes as compression members was limited due to the difficulty in making connections with rivets / Bolts. But with the development of welding techniques its use has become frequent for the following reasons:

1) Tubes have the same radius of gyration in all directions and have a high local buckling strength. These are usually very economical unless moments are too large for the sizes available.
2) Tubes have excellent torsional resistance.
3) In case of members subjected to wind, round tubes are subjected to less force than flat sections.
4) They have less surface area to paint or fireproof.
5) Tubes donot have the problem of dirt collection or cleaning.

The least radius of gyration of a single angle section is small as compared to channels and I-sections and hence, it is not suitable for long lengths.

→ Equal angles are more desirable and economical than unequal ones, because their least radius of gyration is greater for the same area of steel.



TYPES OF BUCKLING :-

The individual elements of a column i.e., flange or web may buckle locally forming wrinkles. This type of buckling causing column failure is called Local buckling and can be prevented by providing suitable width-thickness ratios of the elements. When an axially loaded compression member becomes unstable overall, it can buckle in one of the following of three ways.

1) Flexural Buckling :— It is a deflection caused by bending, or flexure, about the axis corresponding to the largest slenderness ratio. This is usually the minor principal axis - the one with small radius of gyration. Compression members with any type of cross-sectional configuration can fail in this way.

2) Torsional Buckling :— The flexural buckling considered is due to bending alone; that is, the sections displace from their original position by translation without rotation. Thin wall members with open cross-sectional shapes are sometimes weak in torsion and hence may buckle by twisting rather than bending. Torsional buckling occurs when the torsional rigidity of the member is appreciably smaller than its bending rigidity. This type of failure is caused by twisting about the longitudinal axis (x-x axis) of member. It can occur only with doubly symmetrical cross sections with very slender cross-sectional elements.

3) Flexural-Torsional Buckling :— This type of failure is caused by a combination of a flexural buckling and torsional buckling. The member bends and twists simultaneously. This type of failure can occur only with unsymmetrical cross-sections, both those with one axis of symmetry such as channels, structural tees, double angle shapes, and equal-leg single angles, and those with no axis of symmetry - such as unequal-leg single angles.

* (1) For single symmetrical sections, for example, Tee, Double angle sections, flexural buckling may occur about the ZZ or Y-Y axis. For equal angle section flexural buckling may occur about X-X axis.

* (2) Usually a designer does not consider the torsional buckling of symmetrical shapes or the flexural torsional buckling of unsymmetrical shapes. The feeling is that these conditions do not control the critical column loads, which is far from the truth.

(8) Calculate the design compressive load for a stanchion 350 @ 710.2 N/m, 3.5m high. The column is restrained in direction and position at both the ends. It is to be used as an unscaced column in a single-storey building. Use steel grade Fe 410.

Ans:– For steel of grade Fe 410; $(f_y) = 250$ MPa

Partial Safety factor of material; $(\gamma_{m0}) = 1.10$

The column ends are restrained in direction and position

$K = 0.65$ (Pg:- 45) (for restrained)

{Effective length, $KL = 0.65L = 0.65 \times 3.5$}

The properties of ISHB 350 @ 710.2 N/m

$h = 350mm$, $b_f = 250mm$, $t_f = 11.6mm$, $t_w = 10.1mm$ {Pg - 20, 21 (Steel table)}

$A = 9221 mm^2$

$r_z = 146.5mm$, $r_y = 52.2mm$

* $\frac{h}{b_f} = \frac{350}{250} = 1.4 > 1.2$ (Pg - 44, code book, Table-10) (Rolled I-section)

* $t_f \le 40mm$

$11.6 \le 40mm$

Hence, the buckling curve to be used along z-z axis will be curve 'a', and that about y-y axis will be curve 'b'. (Pg - 44, code-book)

* {Since $r_y < r_z$ the column will buckle about y-y axis and the design compressive strength will be governed by effective slenderness ratio (λ_y). However, to make the design compressive strength calculations will be made in example about both yy and z-z axis}

(i) About y-y axis :-

$r_y = 52.2mm$

$$f_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 \times 2 \times 10^5}{\left(\frac{0.65 \times 3.5 \times 10^3}{52.2}\right)^2} = 1039.221 \, N/mm^2$$

{f_{cc} :- Pg :- 34, Cl :- 7.1.2.1}

Non dimensional effective slenderness ratio

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{1039.221}} = 0.490 \quad \{Pg:-34\}$$

For buckling class - 'b',

$\alpha = 0.34$ {Table :- 7, Pg:- 35}

$$\therefore \phi = 0.5\left[1 + \alpha(\lambda - 0.2) + \lambda^2\right]$$

$$= 0.5\left[1 + 0.34(0.490 - 0.2) + 0.490^2\right] = 0.669$$

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + (\phi^2 - \lambda^2)^{0.5}} \quad (\text{Pg-34})$$

$$= \frac{250/1.1}{0.669 + (0.669^2 - 0.490^2)^{0.5}}$$

$$= 202.11\ N/mm^2$$

∴ Strength of column,

$P_d = A_e \cdot f_{cd} = 9221 \times 202.11$

$= 1863.65\ kN$

(ii) About z-z axis

$r_z = 146.5\ mm$ (from steel table)

Non-dimensional effective slenderness ratio,

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}}$$

$$= \sqrt{f_y\left(\frac{KL}{r_z}\right)^2 / \pi^2 E} = \sqrt{\frac{250 \times \left(\frac{0.65 \times 3.5 \times 10^3}{146.5}\right)^2}{\pi^2 \times 2 \times 10^5}}$$

$= 0.1747$

$\simeq 0.175 < 0.2$

* When $\lambda < 0.2$, then the magnitude of λ to be considered will be 0.2

$$\phi_z = 0.5\left[1 + \alpha(\lambda_z - 0.2) + \lambda_z^2\right]$$

$$= 0.5\left[1 + 0.21(0.2 - 0.2) + 0.175^2\right]$$

$= 0.52$

($\alpha = 0.21$ (from Table-7 Pg-35) for buckling class 'a' (z-z axis))

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi_z + (\phi_z^2 - \lambda_z^2)^{0.5}} \quad (\text{Pg-34})$$

$$= \frac{250/1.1}{0.52 + (0.52^2 - 0.2^2)^{0.5}}$$

$= 227.2\ N/mm^2$

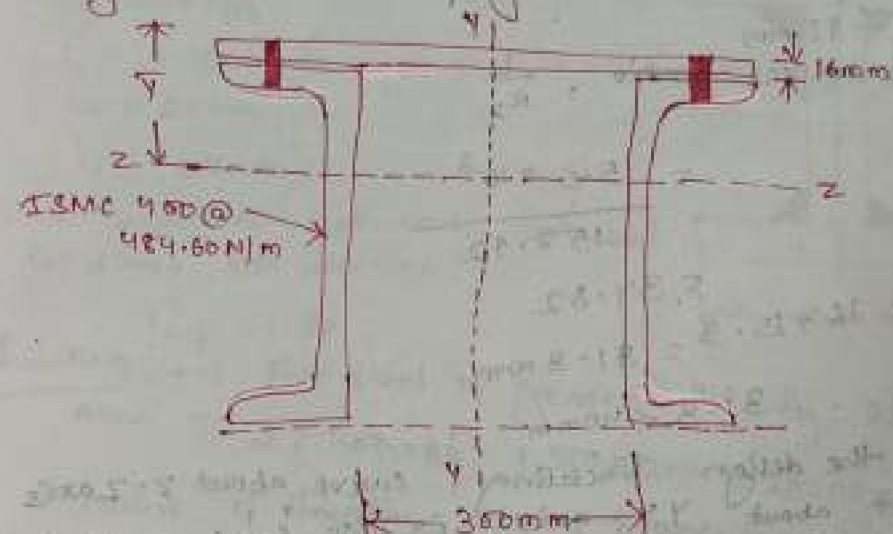
The design compressive strength $= A_e \cdot f_{cd}$

$= 9221 \times 227.2$

$= 2095\ kN$

Hence, the design compressive strength is the least of compressive strength about y-y axis and z-z axis is 1863.65 kN

(8) Calculate the design compressive load which the member shown in fig. can support, if the member is of 5.5m effective length. Use steel of grade Fe 410.



Solution :-

For steel of grade Fe 410 : ($f_y = 250$ MPa)

for ISMC 400 @ 484.60 N/m are, { Steel table, Pg - 22, 23 }

$A = 6293 mm^2$, $b_f = 100$, $t_f = 15.3$, $t_w = 8.6 mm$, $r_z = 154.8 mm$

$g = 60 mm$, $I_z = 15082.8 \times 10^4 mm^4$, $I_y = 504.8 \times 10^4 mm^4$

$C_{yy} = 24.2 mm$

Area of channel sections = $2 \times 6293 = 12,586 mm^2$ → there are 2 channel section

Area of plate section = $\{300 + 2 \times 100\} \times 16 = 8000 mm^2$

Total area provided = $12586 + 8000 = 20,586 mm^2$

Let the distance of neutral axis from top be $\bar{y}$.

$$\bar{y} = \frac{\underbrace{2 \times 6293}_{\text{area of channel section}} \times \left(\frac{\overset{\text{height}}{400}}{2} + 16\right) + \underbrace{8000}_{\text{area of plate section}} \times \frac{\overset{\text{thickness}}{16}}{2}}{\underbrace{20,586}_{\text{Total area}}}$$

$= 135.168 mm$

$$I_z = 2 \times \left[\overset{I_z}{15082.8 \times 10^4} + \overset{\text{area of channel section}}{6293} \times \left(\overset{\left(\frac{400}{2}+16\right)}{216} - \overset{\bar{y}}{135.168}\right)^2 \right] + \frac{\overset{(300+100+100)}{500} \times 16^3}{12} + \overset{(300+100+100)\,(b)}{500} \times \overset{\text{thickness}}{16} \times (\overset{\bar{y}}{135.168} - 8)^2$$

$= 51,342.82 \times 10^4 mm^4$

$$I_y = 2 \times \overset{I_y}{504.8 \times 10^4} + 16 \times \frac{500^3}{12} + 2 \times 6293 \times (500 + 24.2)^2$$

$= 55,869.22 \times 10^4 mm^4$

I_z is less than I_y, the minimum radius of gyration will be r_z.

Minimum radius of gyration,

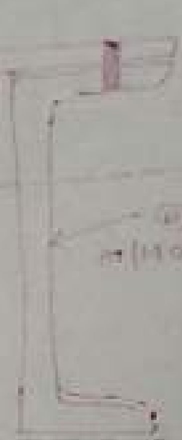
$$r_z = \sqrt{\frac{51942.82 \times 10^4}{20,588}}$$

$= 157.92$ mm

Effective slenderness ratio, $\frac{L}{r_z}$

$$= \frac{5.5 \times 10^3}{157.92}$$

$= 34.82$



$t_f = 16 + 15.3 = 31.3$ mm

$31.3 < 40$ mm

From table, the design buckling curve about Z-Z axis is 'c' and that about Y-Y axis is also 'c'. (for channel section)

For $KL/r = \frac{5.5 \times 10^3}{157.92}$

$f_y = 250$ N/mm², and buckling curve c, the design compressive stress can be calculated from the Table 9(c), pg: 42, code book.

KL/r	f_y (for 250)
30	211
34.82	x
40	198

$$\frac{34.82 - 30}{40 - 30} = \frac{x - 211}{198 - 211}$$

$0.483 \times (-13) = x - 211$

$x = -6.279 + 211$

$x = 204.721$

$(f_{cd}) = 204.721$ N/mm²

The design compressive load, $P_d = A_e \cdot f_{cd}$

$= 20,588 \times 204.721$

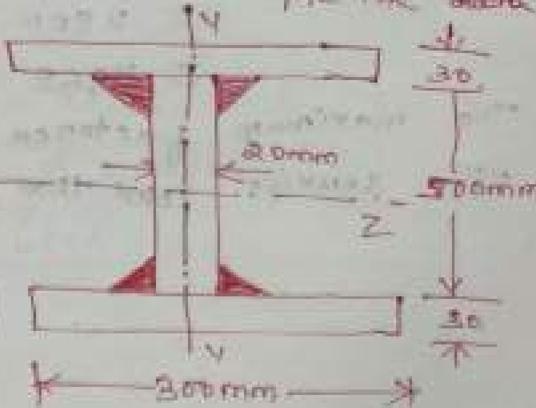
$= 4214.65$ KN.

(a) For a column section built up of shape shown in fig. Determine the axial load capacity in compression for the data indicated against the figure.

$f_y = 250$ MPa.
$L = 6.0$ m.
$t_w = 20$ mm.
$t_f = 30$ mm.

End conditions: Both ends restrained in direct and position.

$\gamma_{mf} = 1.50$.



Solution: For steel of grade Fe 410; $f_y = 250$ MPa.

Area $= 2(300 \times 30) + 500 \times 20 = 28000\ mm^2$

Moment of Inertia about Z-Z axes.

$$(I_z) = 2\left[\frac{300 \times 30^3}{12} + 300 \times 30 \times (250 + 15)^2\right] + \frac{20 \times 500^3}{12}$$

$$= 142373.33 \times 10^4\ mm^4$$

Moment of Inertia about Y-Y axis

$$I_y = 2 \times \frac{30 \times 300^3}{12} + \frac{500 \times 20^3}{12}$$

$$= 13533.33 \times 10^4\ mm^4$$

$I_y < I_z$

Since moment of Inertia about Y-Y axis is minimum, the column will buckle about Y-Y axis

$$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{1353333 \times 10^4}{28000}} = 69.52\ mm$$

Effective slenderness ratio, $\frac{KL}{r} = \frac{0.65 \times 6 \times 10^3}{69.52}$

$= 56.09$

This is a welded I-section with $t_f < 40$ mm. Therefore, from table - 10 the buckling curve to be used along Z-Z axis is "b" and the along Y-Y axis is "c"

For $\frac{KL}{r} = 56.09$, $f_y = 250\ N/mm^2$, and buckling curve 'c', the design compressive stress from Table - 9(c)

$f_{cd} = 173.865\ N/mm^2$

The design compressive strength

$P_d = A_e \cdot f_{cd}$

$= 2800 \times 173.865 \times 10^{-3}$

$= 4868.22$ kN

The maximum factored load, $P \le P_d$ (= 4868.22 kN)

The service load that can be applied $= \dfrac{P}{\gamma_{mf}}$

$= \dfrac{4868.22}{1.50}$

$= 3245.48$ kN

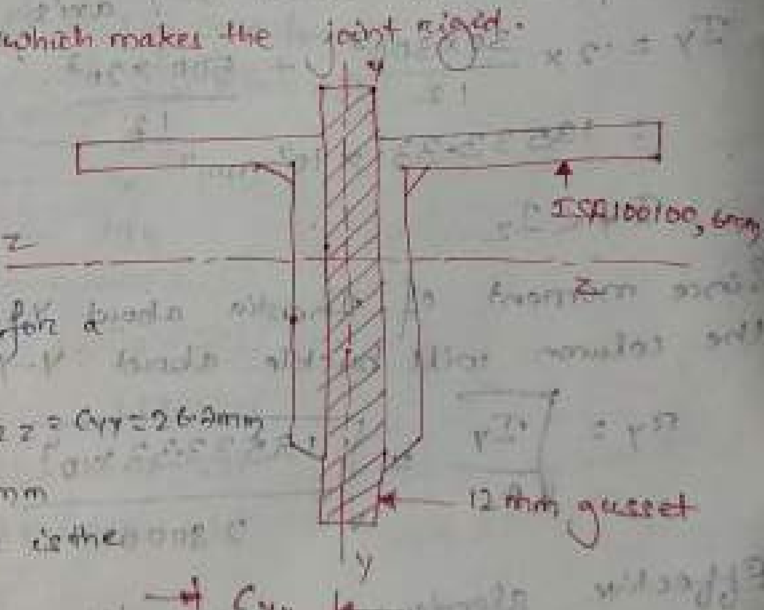
(Q) In a truss a strut 3m long consists of two angles ISA 100100, 6mm. Find the factored strength of the member if the angles are connected on both sides of 12 mm gusset by

(i) one bolt.

(ii) two bolts

(iii) welding, which makes the joint rigid.

Solution:—



From steel table for a ISA100100, 6mm

Area = 1167 mm², $C_{zz} = C_{yy} = 26.7$mm

$r_{zz} = r_{yy} = 30.7$mm

→ 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.

∴ $r_{zz} = 30.7$mm

$I_{yy} = 2\left[I_{yy} \text{ of one angle} + \text{Area of one angle} \times (C_{yy}+6)^2\right]$

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

I_{yy} of the member $= 2\left[111.3\times10^4 + 1169\times(26.7+6)^2\right]$

$= 4721723\, mm^4$

$\therefore r_{yy} = \sqrt{\dfrac{I_{yy}}{A}} = \sqrt{\dfrac{4721723}{2\times1169}} = 44.98$

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

Case (I) : When a single bolt is used

$r = r_{zz} = 30.9\,mm$

$KL = L = 3000\,mm$

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

The member belongs to buckling class "c"

Hence reffering Table 9(c), $\dfrac{KL}{r} = 97$, $f_y = 250\,MPa$,

$f_{cd} =$

90 — 121 {Pg – 42}

97 — x

100 — 107

$\dfrac{97-90}{100-90} = \dfrac{x-121}{107-121}$

$x = 111.2\, N/mm^2$

$f_{cd} = 111.2\, N/mm^2$

$\therefore P_d = A_e f_{cd}$ (Pg 34 – Cl – 7.1.2)

$= 2\times1169\times111.2 = 259754\,N$

$P_d = 259.54\,kN$. (Ans)

Case (II) :- When ~~rotten~~ bolts are used

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

$\therefore KL = 0.85\times3000 = 2550\,mm$

{Pg – 35 0.7 to 1.0 times length}

Hence in this case,

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

From Table 9(c), for steel with $f_y = 250\,N/mm^2$

f_{cd} for $\dfrac{KL}{r} = 82.5$

80 — 136

82.5 — x

90 — 121

$f_{cd} = 132.25\ N/mm^2$

$\therefore P_{cd} = 2 \times 1167 \times 132.25 = 308672\ N$

$= 308.672\ kN$

Case-III :- Rigid joint by welding

Effective length $= (KL) = 0.7 \times L$

$= 0.7 \times 3000$

$= 2100\ mm$

$\therefore \frac{KL}{r} = \frac{2100}{30.1} = 69.96$

$f_{cd} =$

60 — 168

69.96 — x

70 — 152

$$\frac{69.96 - 60}{70 - 60} = \frac{x - 168}{152 - 168}$$

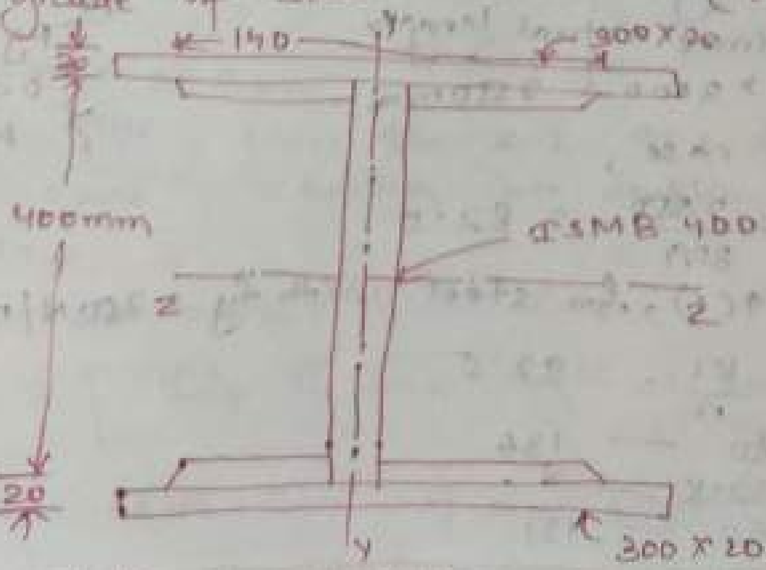
$$x = 168 - \frac{7.96}{10}(168 - 152)$$

$= 155.26\ N/mm^2$

$\therefore P_d = 2 \times 1167 \times 155.26 = 362386\ N$

$\simeq 362.386\ kN$

(5) Determine the load carrying capacity of the column section shown in fig., if its actual length is 4.5m. Its one end may be assumed fixed and the other end hinged. The grade of steel is Fe 415 (i.e., E 250)



Solution

For ISMB 400,, $I_{zz} = 20458.4 \times 10^4 mm^4$

$h = 400mm$, $b_f = 140mm$, $t_f = 16mm$,

$I_{yy} = 622.1 \times 10^4 mm^4$, Area = $7846 mm^2$

Buckling class: Built up section. Hence it belongs to class "c"

Sectional properties:

$I_{zz} = 20458.4 \times 10^4 + 2 \times 300 \times 20 \times (200+10)^2$

$= 733784000 mm^4$

$I_{yy} = 622.1 \times 10^4 + \frac{1}{12} \times 20 \times 300^3 \times 2$

$= 96221000 mm^4$

$\therefore I_{yy} < I_{zz}$

Buckling about y-axis governs the design

$r = r_{yy} = \sqrt{\frac{I_{yy}}{A}}$

$A = 7846 + 2 \times 300 \times 20 = 19846 mm^2$

$\therefore r = r_{yy} = \sqrt{\frac{96221000}{19846}} = 69.63 mm$

Effective length $KL = 0.8L = 0.8 \times 4500 = 3600mm$

$\therefore$ Slenderness ratio $= \frac{KL}{r} = \frac{3600}{69.63} = 51.70$

from Table - 9 - (c) (Pg - 42)

50 —— 183

51.70 —— x

60 —— 168

$\frac{51.70 - 50}{60 - 50} = \frac{x - 183}{168 - 183}$

$\Rightarrow 0.17 \times (-15) = x - 183$

$\Rightarrow x = 183 - 2.55$

$x = 180.45 N/mm^2$

$f_{cd} = 180.45 N/mm^2$

$\therefore P_d = A \cdot f_{cd} = 19846 \times 180.45 = 3581210N$

$\therefore$ (Working Load) Load carrying capacity of the

Column = $\frac{3581.210}{1.5} = 2387.474$ KN

DESIGN OF COMPRESSION MEMBERS

The steps for the design of compression members are:

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 may be assumed as 90 N/mm^2.

* 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

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

3) Select a section to give effective area required and calculated "r_{min}".

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

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

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

(6) 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 3m.

Solution

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

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

Try ISA 9090, 12mm, which has A = 2019 mm²
$r_{min} = r_{vv} = 17.4$ mm.

Assuming the strut will be connected to the gusset plate with atleast 2 bolts. {Strength of 20mm bolt in single shear is about 45kN}

$KL = 0.85L = 0.85 \times 3000 = 2550$ mm

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

From table 9(c)

KL/r		fcd
140	—	60.2
146.55	—	x
150	—	59.2

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

$\therefore P_d = A \cdot f_{cd} = 2019 \times 54.6 = 110239 < 180000$N

Hence revise the section.

Try ISA 130130, 8mm

Area provided = 2022 mm², $r = r_{vv} = 25.5$

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

$\therefore f_{cd} = 107$ N/mm²

$\therefore P_{cd} = 2022 \times 107 = 216354 > 180,000$N

Provide ISA 130130, 8mm.

Q) A column 4m long has to support a factored load of 6000kN. The column is effectively held at both ends and restrained in direction at one of the ends. Design the column using beam sections and plates.

Solution:-

Assuming $f_{cd} = 200$ N/mm²

$$\text{Area required} = \frac{6000 \times 10^3}{200} = 30000 \text{ mm}^2$$

Using ISHB 450 @ 907 N/m,

Area provided = 11789 mm², width of flange = 250mm

$\therefore$ Area to be provided by plates = 30000 − 11789
= 18211 mm²

Selecting 20mm plates, breadth required 'b' is obtained from,

$2b \times 20 = 18211$

$b = 455.3$

Provide 20mm × 500mm plate

<u>Check</u> <u>for</u> <u>overhang</u>:

Overhang $\frac{500-250}{20} = 12.5 < 12\varepsilon$ (Clause - 10.2.3.2 Pg-74)

Total area provided

$Ae = 11289 + 500 \times 20 \times 2$

$= 31789 \ mm^2$

For ISHB 450 @ 907 N/m

$I_{zz} = 40349.9 \times 10^4 \ mm^4$

$I_{yy} = 3045 \times 10^4 \ mm^4$

∴ For the section selected,

$I_{zz} = 40349.9 \times 10^4 + 2 \times 500 \times 20 (225+10)^2$

$= 1507.994 \times 10^6 \ mm^4$

$I_{yy} = 3045 \times 10^4 + 2 \times \frac{1}{12} \times 20 \times 500^3$

$= 447.1167 \times 10^6 \ mm^4$

$\therefore r = r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{447.1167 \times 10^6}{31789}}$

$= 118.6 mm$

Effective length $(KL) = 0.8L = 0.8 \times 4000$

$= 3200 \ mm$

$\therefore \frac{KL}{r} = \frac{3200}{118.6} = 26.98$

$t_f = t_f$ of I section + 20

$= 13.7 + 20 = 31.7 \leq 40 mm$

It belongs to buckling class 'c' for buckling about Y-Y axis.

$f_{cd} = 224 - \frac{6.98}{10}(224-211)$

$= 214.9 \ N/mm^2$

$\therefore P_d = Ae \cdot f_{cd} = 31789 \times 214.9 = 6831456N$

6831.456 KN > Factored load (Hence safe)

DESIGN OF STEEL BEAMS

A beam is a structural member subjected to bending. It is subjected to transverse loads normal to its axis. Beams of light sections that support floor construction are termed as joists. Horizontal beams spanning between the adjacent trusses are known as purlins. Lintel is a beam that spans over openings in buildings. Header is a beam framed to two beams at right angles and supports joist on one side of it. Beam that supports the headers is termed as trimmer. The beam supporting the stair steps is termed as stringer. Rolled I sections with and without cover plates are normally used for floor beams. Channel tee and angle sections are used in roof trusses as purlins and common rafters.

Laterally stable steel beams can fail only by (a) Flexure (b) Shear or (c) Bearing, assuming that local buckling of slender components does not occur.

BASIC CONCEPTS OF PLASTIC THEORY

To understand plastic theory, consider a rectangular shape steel beam, simply supported at both ends, subjected to a concentrated load at the centre. Consider the section at the midspan.

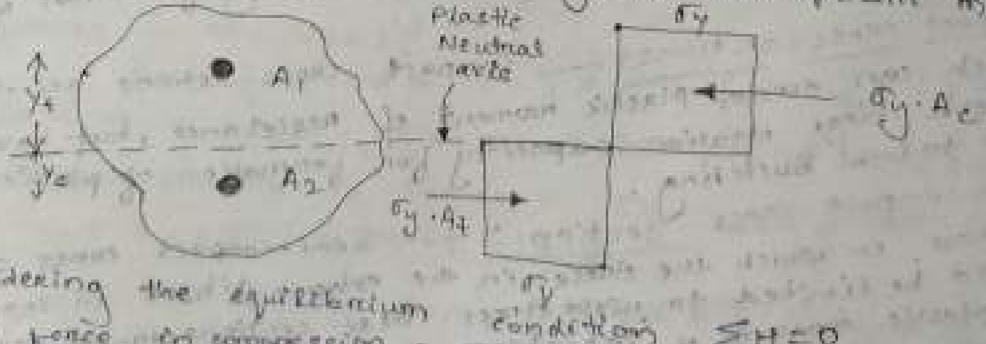
(a) When maximum bending moment (M) is less than yield moment (M_y), all the extreme fibres are stressed below yield point.

(b) With increase in load, maximum bending moment equals to the yield moment i.e., $M = M_y$. The stress in extreme fibres reach the value of yield stress and begins to yield.

(c) With further increase in load, the maximum bending moment lies between yield moment and plastic moment (M_p).

(d) Practically, all the fibres at the section reach the yield stress and the section become fully plastic. The moment corresponding to this state is called the plastic moment of the section.

Plastic moment may also be defined as the magnitude of the bending moment at which a plastic hinge is formed. Consider an arbitary section subjected to a plastic moment (M_p)



Considering the equilibrium condition $\Sigma H = 0$

Total force in compression = Total force in tension

$\sigma_y \cdot A_t = \sigma_y \cdot A_c$ or $A_t = A_c$

The neutral axis that divides the cross-section into two equal halves is known as plastic neutral

$A_t = A_c = A/2$

where, A_t = Area in tension, A_c = Area in compression,

A = Total Area.

Considering equilibrium condition $\Sigma M = 0$

$M_p = (\sigma_y \cdot A_c) Y_c + (\sigma_y \cdot A_t) Y_t = \sigma_y (A_c \cdot Y_c + A_t \cdot Y_t) = \sigma_y \cdot Z_p$

where,

Z_p = Plastic section modulus $= A_c \cdot Y_c + A_t \cdot Y_t$

Shape factor of a cross-section is defined as the ratio between plastic moment to yield moment

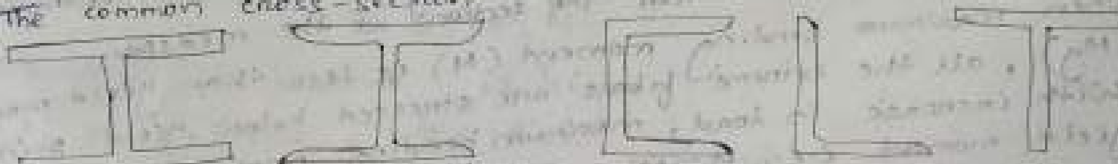
$$\text{Shape factor} = \frac{M_p}{M_y} = \frac{\sigma_y \cdot Z_p}{\sigma_y \cdot Z_e} = \frac{Z_p}{Z_e}$$

LOCAL BUCKLING

Any plate element subjected to direct compression, bending or shear stress or combination of these stresses may buckle prematurely. Plate elements may fail in buckling locally before overall column buckling or overall beam failure due to yielding or lateral buckling. This type of failure is called local buckling. To prevent local buckling, IS code limits on width to thickness ratio of plate elements of different class

COMMON CROSS-SECTION AND THEIR CLASSIFICATION

The common cross-section to be used as beam



Wide Flange   standard beam   Channel   Angle   Tee

Tubular   Box

Sections are also classified depending on their moment-rotation characteristics.

* Plastic cross-sections :- Plastic cross-sections are those which can develop plastic hinges and have the rotation capacity required for failure of the structure by formation of plastic mechanism.
* Compact cross-sections :- Compact cross-sections are those which can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of plastic mechanism due to local buckling.
* Semi-compact cross-section :- Semi-compact cross-sections are those in which the stress in the extreme fibers in compression should be limited to yield stress. These sections cannot develop the plastic moment of resistance due to local buckling.
* Slender cross-sections :- Slender cross-sections are those in which the elements buckle locally even before reaching yield stress.

{* Limits on width to thickness Ratio of plate Elements in Table-2, pg-18}

(B) Determine the plastic moment capacity and plastic section modulus of

(a) The rectangular section of size $b \times t$ about z-z axis (fig-a)

(b) The I-Section about z-z axis (fig.b)

(c) The I-section about y-y axis (fig.c)

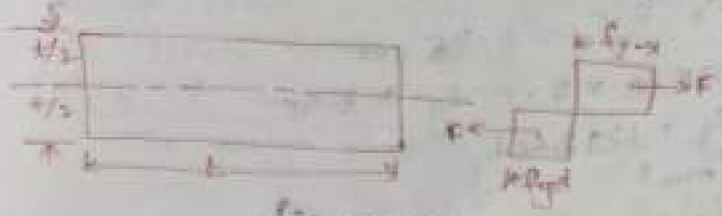


figure (a)

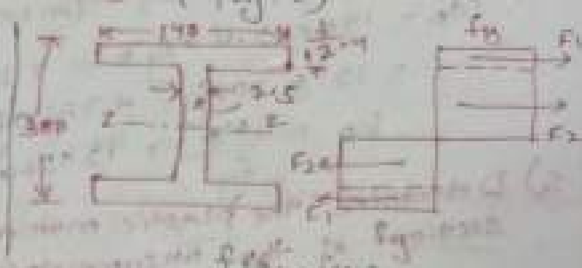
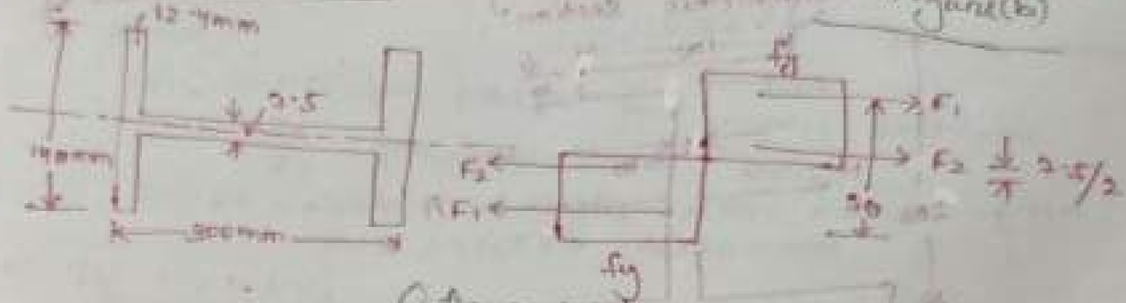


figure (b)



(figure (c))

Solution :- (a) Rectangular Section :

Due to symmetry plastic neutral axis is at mid depth.

$\therefore A_t = A_c = \frac{b}{2} \times t$

$F = \frac{b}{2} \cdot t \cdot f_y$

The distance between tensile & compressive forces $= \frac{t}{2}$

$\therefore M_p = F \times \frac{t}{2} = \frac{b}{2} t \cdot f_y \cdot \frac{t}{2}$

$= \frac{1}{4} bt^2 f_y$

$\therefore Z_p = \frac{M_p}{f_y} = \frac{1}{4} bt^2$

(b) I-section about z-z axis :

Plastic NA 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,

$F_2 = \frac{1}{2}(150 - 12.4) \times 7.5\, f_y = \frac{1}{2} \times 137.6 f_y$

Distance between F_1 forces $= 300 - 12.4 = 287.6$ mm

Distance between F_2 forces $= 150 - 12.4 = 137.6$ mm

$\therefore M_p = F_1 \times 287.6 + F_2 \times 137.6$

$= 140 \times 12.4 f_y + \frac{1}{2} \times 137.6 \times 7.5 f_y \times 137.6$

$= 499274 f_y + 71001.6 f_y$

$= 570275.6 f_y$

$Z_p = 570.276 \times 10^3$ mm^3

(c) I-section about y-y axis :-

Plastic N-A is in mid-depth. Let F_1 be force on flange of size 140 × 12.4 mm and F_2 be force in web of size (300 − 2×12.4) × 7.5 mm. Then

$F_1 = 140 \times 12.4 f_y$; $F_2 = 275.2 \times 7.5 f_y$

Distance between F_1 force $= \frac{140}{2} = 70$ mm

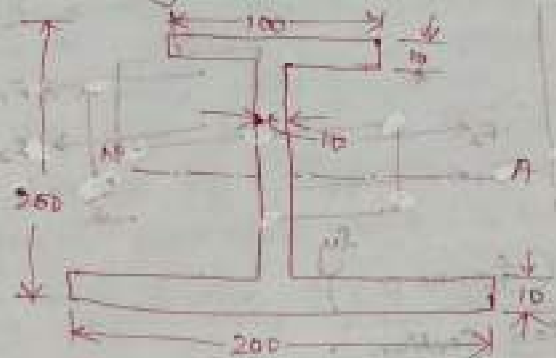
Distance between F_2 force $= \frac{7.5}{2} = 3.75$ mm

$\therefore M_p = 140 \times 12.4 \times f_y \times 70 + 275.2 \times 7.5 f_y \times 3.75$

$= 121520 f_y + 7740 f_y = 129260 f_y$

$\therefore Z_p = \frac{M_p}{f_y} = 129260 \text{ mm}^3$

Q) Determine the plastic moment capacity and plastic modulus of section of the unsymmetric section



Solution

Total area $= 100 \times 10 + 200 \times 10 + (250 - 20) \times 10 = 4800 \text{ mm}^2$

$A_c = A_t = \frac{4800}{2} = 2400 \text{ mm}^2$

Plastic N-A is at a depth 'h' from top fibre where 'h' is given by $100 \times 10 + (h - 10) \times 10 = 2400$

$\therefore h = 150$ mm

When plastic hinge is formed, one half is subjected to compressive stresses f_y and another half to tensile stresses f_y. Taking moment of all such forces about plastic NA, we get

$$M_p = \left(100 \times 10 \times (150 - 5) + 10 \times (150 - 10)\frac{(150-10)}{2} + 10 \times (50 - 10)\left(\frac{50 - 10}{2}\right) + 200 \times 10\,(50 - 5)\right) \times f_y$$

$= 331000 f_y \text{ mm}^3$

$\therefore Z_p = \frac{M_p}{f_y} = 331000 \text{ mm}^3$

DESIGN PROCEDURE

1) A trial section is selected assuming it is going to be plastic section (class I 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.

BENDING STRENGTH OF A LATERALLY SUPPORTED BEAM

If $\frac{d}{t_w} < 67\varepsilon$. IS 800-2007 considers two cases one with design shear strength less than $0.6V_d$ and other with design shear strength more than $0.6V_d$ where, V_d is design shear. When $\frac{d}{t_w} > 67\varepsilon$, shear buckling of web is likely to take place.

(a) If $V \le 0.6V_d$:

The design bending strength M_d shall be taken as

$$M_d = \beta_b \cdot Z_p \cdot f_y \times \frac{1}{\gamma_{m0}} \le 1.2 Z_e f_y \times \frac{1}{\gamma_{m0}}$$ for simply supported beam

$$\le 1.5 Z_e \frac{f_y}{\gamma_{m0}}$$ for cantilever beam.

where,

$\beta_b = 1.0$ for plastic and compact sections.

$= \frac{Z_e}{Z_p}$ for semi-compact sections.

Z_p, Z_e = plastic and elastic section modulii of the cross-section respectively.

(b) If $V > 0.6V_d$

In such cases,

$$\boxed{M_d = M_{dv}}$$

where M_{dv} is design strength under high shear.

(i) Plastic or compact section :

$$\boxed{M_{dv} = M_d - \beta(M_d - M_{fd}) \le 1.2 Z_e \times f_y \times \frac{1}{\gamma_{m0}}}$$

where, $\beta = \left(\frac{2V}{V_d} - 1\right)^2$

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 γ_{m0}.

(ii) Semicompact section

$$\boxed{M_{dv} = \frac{Z_e f_y}{\gamma_{m0}}}$$

SHEAR STRENGTH OF A LATERALLY SUPPORTED BEAM

The design shear strength of a section is given by

$$V_d = \frac{A_v \cdot f_{yw}}{\sqrt{3}} \times \frac{1}{\gamma_{m0}}$$

where, A_v = Shear area, 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 $(A_v) = h t_w$, welded $(A_v) = d \cdot t_w$

(ii) Minor Axis bending
Hot rolled or welding $(A_v) = 2b \cdot t_f$

(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: $A_v = \frac{2A}{\pi}$

(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
t_w = thickness of the web.

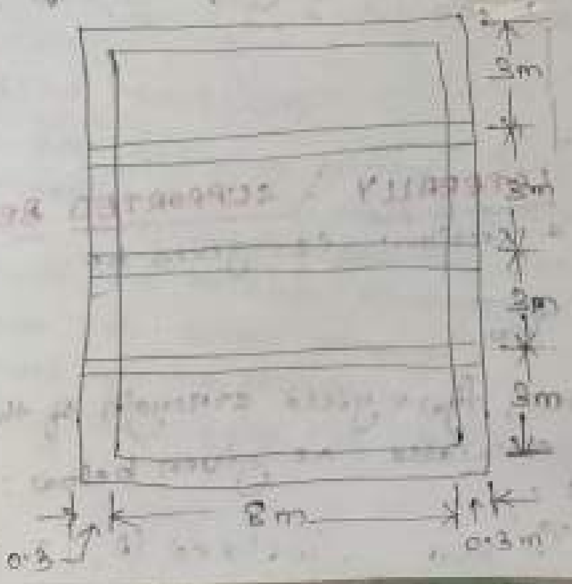
DEFLECTION LIMITS

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 condition.
{ Deflection limits are given in Table-6, pg-31 }

(Q) A roof of a hall measuring 8m × 12m consists of 100m thick R.C. slab supported on steel I-beam spaced 3m apart. 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 8m and takes care of 3m width of slab. Hence the load per metre length of the beam is as follows:



Weight of RC slab = $0.1 \times 1 \times 3 \times 25$
$= 7.5$ KN/m
Finishing load = $1.5 \times 3 = 4.5$ KN/m
Self weight (assumed) $= 0.8$ KN/m
$\therefore$ Total dead load $= 12.8$ KN/m
Live load $= 1 \times 3 \times 1.5 = 4.5$ KN/m
$\therefore$ Factored dead load $= 1.5 \times 12.8$
$= 19.2$ KN/m
Factored live load $= 1.5 \times 4.5$
$= 6.75$ KN/m
$\therefore$ Total factored load $= 25.95$ KN/m

Effective span of the simply supported beam = centre to centre distance of supports

Assuming width of support = 0.3m

Effective span = 8 + 0.3 = 8.3m

∴ Design moment, $M = \frac{wL^2}{8}$

$= \frac{25.95 \times 8.3^2}{8} = 223.46 \, kN\text{-}m$

Design shear force $(V) = \frac{25.95 \times 8.3}{2} = 107.69 \, kN$

∴ Section modulus required $= \frac{M}{f_y} \cdot \gamma_{m0}$ → Pg 53, cl - 8.2.1.2

$Z_p = \frac{223.46 \times 10^6}{250} \times 1.1 = 983224 \, mm^2$

Try ISMB 400 which has $Z_p = 1176.3 \times 10^3 \, mm^3$

The properties of the section are as follows:-

Depth of section (h) = 400mm, width of flange (b) = 140mm

Sectional area (A) = 7846mm², Thickness of web (t_w) = 8.9mm

Thickness of flange (t_f) = 16.0mm

Depth of web $(d) = h - 2(t_f + R_1)$

$= 400 - 2(16 + 14) = 340 \, mm$

Moment of inertia about z-z axis

$I_{zz} = 20458.4 \times 10^4 \, mm^4$

Elastic Section Modulus $(Z_e) = 1020 \times 10^3 \, mm^4$.

Outstanding leg of comp. flange $(b) = \frac{140}{2} = 70$

Section classification :-

$\epsilon = \left(\frac{250}{f_y}\right)^{1/2} = \left(\frac{250}{250}\right)^{1/2} = 1.0$ → {Pg - 18, Table 2}

$\frac{b}{t_f} = \frac{70}{16} = 4.38 < 8$

$\frac{d}{t_w} = \frac{340}{8.9} = 38.2 < 84\epsilon$

Hence the section is classified as plastic section.

Weight of the section = 0.604 kN/m

Assumed weight = 0.81 kN/m

Difference is not so much. Hence the design is continued with moments and shear calculated as earlier.

Check for shear strength :-

Design shear $(V) = 107.69 \, kN$

Design shear strength of the section

$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{1.1} \times$ shear area → {Pg 59, cl - 8.4}

$= \frac{f_y}{\sqrt{3}} \times \frac{1}{1.1} \times h \times t_w$

$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 400 \times 8.9 = 467128 N$

$\simeq 467.128 kN > 107.61 kN$

Hence the section is adequate.

$0.6 V_d = 0.6 \times 467.128 = 280.277 > 107.61 kN$

Check for moment capacity :

$\frac{d}{t_w} = 38.2$ which is less than 67.6, since $\epsilon = 1$

Hence, $M_d = \beta_b Z_p \frac{f_y}{\gamma_{mo}}$

$\beta_b = 1.0$ since it is plastic section

$M_d = 1.0 \times 1175.2 \times 10^3 \times \frac{250}{1.1} = 267.091 \times 10^6$ N.mm

Hence adequate $\simeq 267.091$ kN.m > 223.46 kN.m

Check for deflection :-

Total working load $= 12.8 + 4.5 = 17.3$ kN/m

$= 17.3$ N/mm

Maximum deflection

$$\delta = \frac{5}{384} \frac{wL^4}{EI}$$

$$\therefore \delta = \frac{5}{384} \times \frac{17.3 \times (8300)^4}{2 \times 10^5 \times 20458.4 \times 10^4} = 26.129 mm$$

Permissible deflection for a beam in building

$= \frac{le}{300} = \frac{8300}{300} = 27.67 mm \rightarrow$ {Pg-31, Table-6}

Hence deflection is within the permissible limit.

$\therefore$ Provide ISMB 400

(6) Design a simply supported beam of effective span 1.5m carrying a factored concentrated load of 360 kN at mid span.

Solution

Maximum moment occurs at mid span,

$$M = \frac{WL}{4} = \frac{360 \times 1.5}{4} = 135 \text{ kN/m} \simeq 135 \times 10^6 \text{ N.mm}$$

$\therefore Z_p$ required is obtained from the relation

$$f_y \frac{Z_p}{\gamma_{mo}} = M$$

$$Z_p = \frac{135 \times 10^6}{250} \times 1.1 = 594 \times 10^3 \; mm^3$$

Select trial section as ISMB 300 which has $Z_p = 651.7 \times 10^3$

The sectional properties of ISMB 300 are

Overall depth $(h) = 300$ mm, Width of flange $(b) = 140$ mm

Thickness of flange $(t_f) = 12.4mm$

Depth of web $(d) = h - 2(t_f + r_1) = 300 - 2(12.4 + 14)$
$= 247.2mm$

Thickness of web $(t_w) = 7.5mm$

$I_{zz} = 8603 \times 10^4 \; mm^4$, $Z_e = 573.6 \times 10^3 \; mm^4$

$Z_p = 651.7 \times 10^3 \; mm^4$

Self weight of beam $= 0.452 \; kN/m$

$\therefore$ Factored weight $= 1.5 \times 0.452 = 0.678 \; kN/m$

$\therefore$ Additional factored moment due to self wt $= \frac{wl^2}{8}$

$= 1.5 \times 0.452 \times \frac{(1.5)^2}{8} = 0.190 \; kN.m$

$\therefore$ Total factored moment,

$M = 135 + 0.190$
$= 135.190 \; kN.m$

Factored shear force due to self weight $= 1.5 \times 0.452 \times \frac{1.5}{2}$
$= 0.508 \; kN$.

$\therefore$ Total factored shear force on section $= \frac{360}{2} + 0.508$
$= 180.508 \; kN$

Section classification (Pg 18, Table-2)

$\epsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$, overhang $b = \frac{140}{2}$

$\frac{b}{t_f} = \frac{140/2}{12.4} = 5.64 < 9.4\epsilon$

$\frac{d}{t_w} = \frac{247.2}{7.5} = 32.96 < 84\epsilon$

It is classified as a plastic (class I) section.

Shear capacity of the section :-

$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{\gamma_{m0}} \times h \times t_w$ (Pg-59, cl-8.4)

$= \frac{250}{\sqrt{3}} \times \frac{1}{1.1} \times 300 \times 7.5 = 295.235 \times 10^3 \; N$
$\simeq 295.235 \; kN$

Section is adequate to resist shear

$\therefore 0.6 V_d = 0.6 \times 295.235 = 177.145 \; kN$ (Pg-53, cl-8.2.1.2)

$\therefore V > 0.6 V_d$

Moment capacity of the section:

Since $V > 0.6 V_d$ and section belongs to plastic category

$M_{dv} = M_d - \beta(M_d - M_{fd}) \leq 1.2 Z_e f_y \times \frac{1}{\gamma_{m0}} \rightarrow \{$Pg-70, cl-9.2.2 (a)$\}$

$M_d = Z_p \cdot f_y \cdot \frac{1}{\gamma_{m0}} \leq 1.2 Z_e f_y \times \frac{1}{\gamma_{m0}} \rightarrow \{$Pg-53, cl-8.2.1.2$\}$

Now, $Z_p f_y \times \frac{1}{\gamma_{m0}} = 651.7 \times 10^3 \times 250 \times \frac{1}{1.1} = 148.114 \times 10^6 \; N.mm$

$1.2 Z_e f_y \times \frac{1}{\gamma_{m0}} = 1.2 \times 573.6 \times 10^3 \times 250 \times \frac{1}{1.1} = 156.436 \times 10^6 \; N.mm$

$M_d = 148.114 \times 10^6$

$$\beta = \left(\frac{2V}{V_d} - 1\right)^2 = \left(\frac{2\times 180.554}{295.235} - 1\right)^2 = 0.05$$

Since it is double symmetric section, M_{fd} may be obtained from Table-14 of the code → (pg-57)

$$\frac{KL}{r} = \frac{1500}{28.4} = 52.8 \text{ and } \frac{h}{t_f} = \frac{300}{12.4} = 24.19$$

Referring to table 14, by double interpolation we get

$f_{crb} = 916$

$$f_{cd} = 204.5 + \frac{16}{150}(209.1 - 204.5)$$

$$= 205.94$$

$\therefore M_{fd} = f_{cd} \times A = 205.24 \times 5626 = 1.15468 \times 10^6\ N\cdot mm$

$\therefore M_{dv} = 148.114\times10^6 - 0.05(148.114\times10^6 - 1.15468\times10^6)$

$= 140.77 \times 10^6\ N\cdot mm$

$\simeq 140.77\ KN\cdot m > 135.190\ KN\cdot m$

Maximum deflection corresponding to working load.

$$\delta = \frac{WL^3}{48EI} = \frac{360\times10^3 \times 1500^3}{48\times2\times10^5\times 8603\times10^4}$$

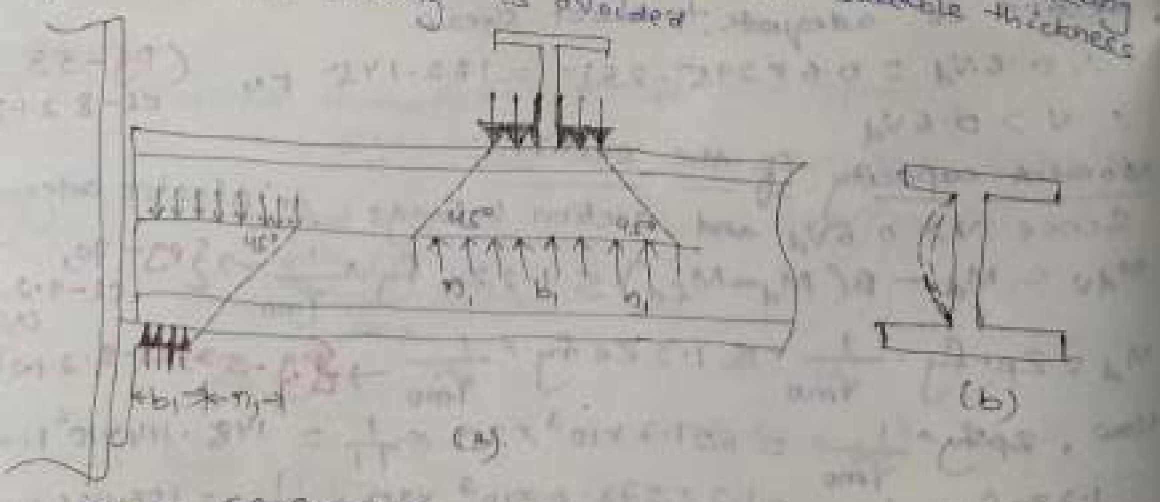
$$= 1.68\ mm < \frac{1500}{300}$$

Hence section is adequate.

Use ISMB 300 as beam.

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. 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 buckling is avoided.



As per IS 800-2007, effective web buckling strength is to be found based 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

$$\therefore \boxed{F_{cdw} = (b_1 + n_1) t_w \cdot f_c}$$

where, F_{cdw} = web buckling strength

and f_c is the allowable compressive stress corresponding to the assumed web column.

$\therefore$ Effective length = $0.7d$ of web column.

$$r_y = \sqrt{\frac{I_y}{A}} \text{ of web}$$

$$= \sqrt{\frac{\frac{1}{12}(b_1+n_1)t_w^3}{(b_1+n_1)t_w}} = \frac{t_w}{2\sqrt{3}}$$

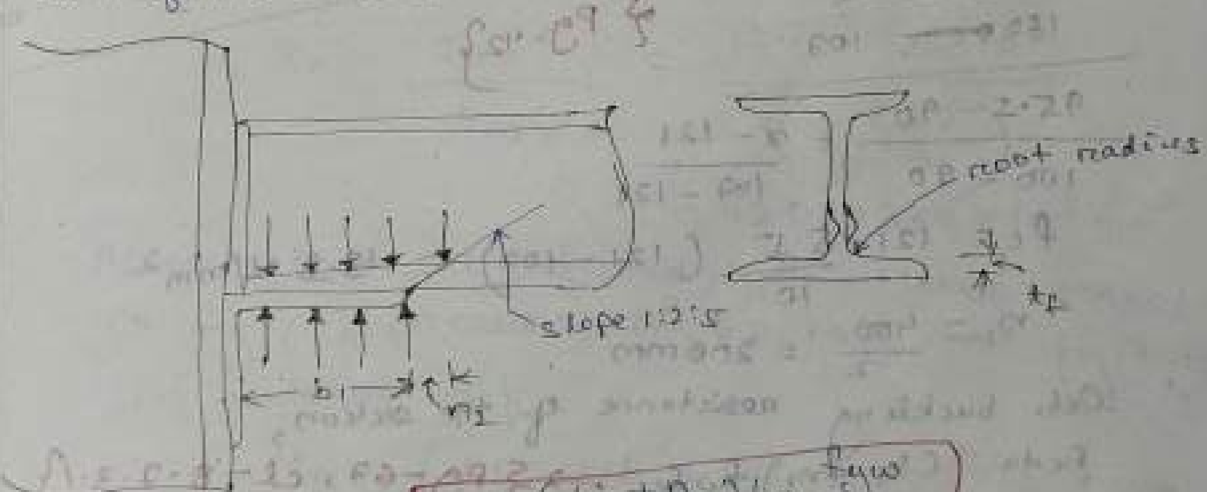
$$\therefore \boxed{\lambda = \frac{\text{Effective length}}{r_y}} = 0.7d \cdot \frac{2\sqrt{3}}{t_w} \simeq 2.5\frac{d}{t_w}$$

corresponding to this slenderness ratio from Table buckling stress 'f_c' can be found and hence

$$\boxed{F_{cdw} = (b_1 + n_1) t_w \cdot f_c}$$

WEB CRIPPLING

Near the support web of the beam may cripple due to lack of bearing capacity. The crippling occurs at the root of the radius



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

where, b_1 = Stiff bearing length,

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

f_{yw} = yield stress of the web

In the design $F_w >$ Load transferred by bearing.

(c) Check the section (ISMB 400) for web buckling and web crippling if stiff bearing is over a length $b_1 = 95mm$. (Question-1)

Solution

Section Selected was ISMB 400

End reaction = End shear = 107.61 kN

Stiff bearing at ends = 75mm

From steel table,

$t_w = 8.9mm$, $t_f = 16.0mm$

radius of root = 14.0mm.

Depth of section (h) = 400mm.

$\therefore$ Depth of web $= 400 - 2(t_f + r_1)$

$= 400 - 2(16 + 14)$

$= 340mm$.

Check for web buckling :

$$\lambda = 2.5 \frac{d}{t_w}$$ {Pg-53, cl-8.2.1}

$$= 2.5 \times \frac{340}{8.9} = \boxed{95.5}$$

Hence from Table 9(c) of IS 800-2007

f_c =

90 — 121

95.5 — x

100 — 107

{Pg-42}

$$\frac{95.5 - 90}{100 - 90} = \frac{x - 121}{107 - 121}$$

$$f_c = 121 - \frac{5.5}{10}(121 - 107) = 113.3\ N/mm^2$$

$$n_1 = \frac{400}{2} = 200mm.$$

$\therefore$ Web buckling resistance of the section,

$$F_{cdw} = (b_1 + n_1) t_w f_c$$ → {Pg-67, cl-8.7.3.1}

$= (75 + 200) \times 8.9 \times 113.3$

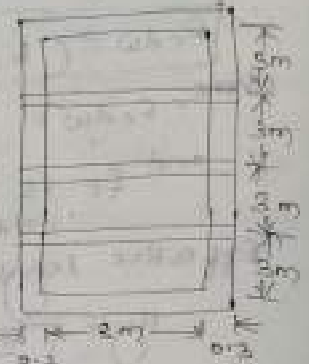
$= 277.302 \times 10^3 N$

$\simeq 277.302 kN > 107.61 kN$

Hence the section is safe against web buckling.

Check for web crippling :-

Flange thickness = 16.0, radius root = 14.0



Each beam is 8m & width of slab is 3m.

Wt. of RC 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 live load = 1.5×4.5

Effective span = 8+0.3 = 8.3

Total load = 25.95 kN/m

Shear force (V) = $\frac{wl}{2}$

$= \frac{25.95 \times 8.3}{2} = 107.6$ k

$\therefore n_2 = 2.5(16+14) = 75\ mm$

$\therefore$ strength of web against web crippling

$$F_w = (b_1 + n_2)\, t_w\, f_{yw} \times \frac{1}{\gamma_{m0}} \rightarrow \{pg\ 67,\ cl-8.7.4\}$$

$$= (75+75)\ 8.9 \times 250 \times \frac{1}{1.1}$$

$$= 303.407 \times 10^3\ N$$

$$\simeq 303.409\ kN > \text{load transferred by bearing in this case } (107.61\ kN)$$

Hence safe.

(Q) Determine the uniformly distributed load carrying capacity of the welded plate girder, when it is used as a cantilever beam of 4m effective span and check it for shear, deflection, web buckling and web crippling. Assuming stiff bearing length as 100mm.

Solution

Section Moduli

$$I_{zz} = \frac{1}{12}\left[200 \times 832^3 - 184 \times 800^3\right]$$

$$= 1748.173 \times 10^6\ mm^4$$

$$\therefore Z_e = \frac{I_{zz}}{y_{max}} = \frac{1748.173 \times 10^6}{\left(\frac{832}{2}\right)}$$

$$= 4202.338 \times 10^3\ mm^3$$

Plastic N-A is at mid depth. Hence stress is fy (compression) in top half and fy (tensile) in bottom half

$$M_p = (200 \times 16 \times 816 + 400 \times 16 \times 400)\, f_y$$

$$\therefore Z_p = \frac{M_p}{f_y} = \frac{5171200 \times 10^3 f_y}{f_y}$$

$$= 5171200 \times 10^3\ mm^3$$

DESIGN OF TUBULAR STEEL STRUCTURES

* INTRODUCTION :—

Tubular structures are hollow ~~cylinder~~ circular sections which form the most efficient sections for some of the structural elements.

→ It has smaller self-weight compared to the solid section.

→ The economy of steel ~~structure~~ tube construction is incomparable.

* USES OF STEEL TUBES IN STRUCTURE :—

→ Roofing system of industrial buildings and warehouse.

→ Transmission line towers.

→ Offshore drilling installations etc.

* FOR THE DESIGN OF TUBULAR STEEL STRUCTURES, WE HAVE TO USE THE FOLLOWING INDIAN STANDARD CODES :—

→ IS: 806-1968, code of practice for use of steel tubes in general building construction.

→ IS: 1161-1998, steel tubes for structural purposes — specification.

TYPES OF TUBES FOR STRUCTURAL USE AS PER IS:1161-1998 :-

* TYPES OF TUBES :

- HFW :— Hot finished welded.
- HFS :— Hot finished seamless.
- ERW :— Electric resistance welded or induction welded.

* CLASSIFICATION OF STEEL TUBES AS PER IS:806-1968 :—

IS 1161:1998 (Table-2) :—

Steel tubes are classified as light, medium and heavy depending upon the wall thickness. On the basis of yield stress, these are classified as :-

Grade	Tensile strength (Min), MPa	Yield stress (Min) MPa
Yst 210	330	210
Yst 240	410	240
Yst 310	450	310

* ADVANTAGES OF STEEL TUBE :—

- These have small self-weights. Also because of direct connections, gusset plates are eliminated further reducing dead loads.
- Tubes have uniform radius of gyration and for the same weight their torsional strength is more than any other rolled section.
- For the same load the surface area of a tube is about 60 to 70% of that for other rolled sections consequently wind forces are small. Also, because of less surface area considerable economy is achieved in maintenance, painting and fire proofing.
- For dynamic loads tubes have high frequency of vibration than other rolled sections.
- Due to a smooth finished surface, dirt and moisture do not collect over the surface, reducing the possibili-ty of corrosion.
- The change in load with the floor levels can be accomodated by varying the tube thickness and the external tube dimension may be maintained.
- The internal hollow space of tubular columns may be used for carrying drain pipes, wires, cables etc. Also these spaces may be filled with concrete to increase the load carrying capacity and to improve fire resistance.

* DISADVANTAGES OF STEEL TUBE :—

- Manufacturing cost is high.
- Difficult to cut the tube end to a correct profile, for making joints between steel tubes at angles.
- Difficult to have a smooth cut surface.
- Problem during connections.

* MINIMUM THICKNESS :—

→ For tube painted with one prime coat of red oxide and then painted periodically, the thickness should not be less than .

(i) For construction exposed to weather :— 4mm .

(ii) For construction not exposed to weather :— 3.2mm

(iii) For members not readily accessible for maintenance :— 5mm.

→ For tubes painted with one coat of zinc primer followed by two coats of paint, the thickness should not be less than

(i) For construction exposed to weather — 3.2mm

(ii) For construction not exposed to weather — 2.6mm

* PERMISSIBLE STRESSES :—

The provision as regards permissible stresses on the net or gross cross-sectional areas, is applicable to steel tubes for which the minus tolerance on the weight per unit length of tube is not more than 4 percent. If one the steel tubes used the minus tolerances on the weight per unit length are larger than 4 percent, a corresponding reduction in cross-sectional areas is required to be made in applying the permissible stresses.

⟹ Axial Stress in Tension :—

The direct stress in axial tension on the net cross-sectional area of tubes shall not exceed the values of Ft .

PERMISSIBLE AXIAL STRESS IN TENSION

GRADE	Ft in (kgf/cm^2)
Yst 22	1250
Yst 25	1500
Yst 32	1900

[pg-5, Table-1 ,
(IS code : 806 : 1968)

→ Axial Stress in Compression :—

The direct stress in compression on the cross-sectional area of axially loaded steel tubes shall not exceed the value of 'F_c' in which 'L/r_2' is equal to the effective length of the member divided by the radius of gyration (Pg - 6, Table - 2)

→ Bending stresses :— (Pg-7, T-3)

In tubes, the tensile bending stress and the compressive bending stress in the extreme fibre shall not exceed the values of F_b.

Permissible bending stress in extreme Fibres in Tension and Compression	
Grade	F_b Kgf/cm^2
Yst 22	1400
Yst 25	1655
Yst 35	2050

→ Shear stresses :— (T-4, Pg-7)

The maximum shear stresses in a tube calculated by dividing the total shear by an ~~equal~~ area equal to half the net cross-sectional area of the tube shall not exceed F_s. The net cross-sectional area shall ~~not~~ be derived by deducting areas of all holes from the gross-cross-sectional area.

Permissible maximum shear stress	
Grade	F_s Kgf/cm^2
Yst 22	960
Yst 25	1150
Yst 35	1350

→ Bearing stresses :— (T-5, Pg-7)

The average bearing stress on the net projected area of contact shall not exceed the value of F_p

Permissible Grade	Maximum Bearing stress P_b kgf/cm^2
Yst 22	1900
Yst 25	1950
Yst 35	2500

* EFFECTIVE LENGTH OF COMPRESSION MEMBERS :-

Effective length (l) of a compression member for the purpose of determining allowable axial stresses shall be assumed according to the Table-7, pg-10. IS:- 806-1968.

* CONNECTIONS :- (pg-14, cl-6.7)

Connections in structures using steel tubes shall be provided by welding, riveting, or bolting. Whenever possible, connections between tubes shall be made directly tube to tube without gusset plates and other attachments.

* DETERMINATION OF THE LENGTH OF THE CURVE OF INTERSECTION OF A TUBE WITH ANOTHER TUBE OR WITH A FLAT PLATE :- (pg- 18,19)

The length of the curve of intersection may be taken as

$$P = a + b + 3\sqrt{a^2 + b^2}$$

where,

$$a = \frac{d}{2} \operatorname{cosec}\theta$$

$$b = \frac{d}{3} \times \frac{3 - (d/D)^2}{2 - (d/D)^2}$$, for intersection with a tube.

d = outside diameter of branch.

θ = angle between branch and main.

D = outside diameter of main.

Q) A tubular column consist of IS 161 grade Yst 32 steel. The column is hinged at both ends. The outside diameter of tube is 219.1mm. The length of column is 4.5m. Weight of 1m length of tube is 310N. Determine the safe load carrying capacity of the column.

Given data :-

* Outside dia = 219.1 mm
* Unit weight = 310 N/m.
* Nominal Bore = 200.
* Section is heavy - (Table-1) → {IS 1161 : 1998, Table-1, pg-2}
* Area = 39.5 cm^2 = 39.5 × 10 × 10 = 3950 mm^2
* r = 7.54 cm ≃ 75.4 mm

Grade = Yst 32

Length = 4.5 m

Step: 2 :-

Calculation of $\frac{l}{r}$

Length = 4.5 m

r = 75.4

$\frac{l}{r} = \frac{4500}{75.4} = 59.82$

Step: 3 :-

Permissible stress in compression :- (From 806 code)

→ (Table-2) {IS 806 : 1968, pg-6}

$\frac{l}{r}$	f_c
50	1539
59.82	f_c
60	1468

$\frac{59.82-50}{60-50} = \frac{f_c - 1539}{1468 - 1539}$

$= \frac{9.82}{10} = \frac{f_c - 1539}{-71}$

$\Rightarrow (-71) \times \frac{9.81}{10} = f_c - 1539$

$\Rightarrow -69.651 = f_c - 1539$

$\Rightarrow f_c = 1539 - 69.651$

$f_c = 1469.298$ kgf/cm^2

$$f_c = \frac{1469.278 \times 9.81}{100} \ N/mm^2$$

$$= 0.0981 \times 1469.278$$

$$f_c = 144.136 \ N/mm^2$$

Step-4 :- Load Carrying Capacity

$$P = A \times f_c$$

$$= 39.5 \times 10^2 \times 144.136$$

$$= 569337.2 N$$

$$\simeq \frac{569337.2}{1000}$$

$$\simeq 569.337 \ KN.$$

(8) Design member AB, BC, and joint A of a roof truss as shown in figure, for the following data:-

Member	Length	Compressive force	Tensile force
AB	2.3m	60kN	55kN
AC	1.8m	55kN	80 kN

Given data :-

Member AB :-

Assume Yst 22 grade.

Let's try :- 65mm nominal bore light section.

Area (A) = 738mm², $r = 25.8 mm$, $L = 2.3m$

$$\lambda = \frac{L}{r} = \frac{0.7 \times 2.3 \times 10^3}{25.8} = 62.50$$

$$\left\{ \frac{KL}{r} = \frac{0.7L}{r} \right\}$$

* f_c, (Permissible stress in compression) (Pg-6, Table-2 I.S: 806-1968)

60 —— 1002

62.50 —— f_c

70 —— 970

$$\frac{62.50-60}{70-60} = \frac{f_c - 1002}{970-1002}$$

$= \frac{2.50}{10} = \frac{f_c - 1002}{-32}$

$\Rightarrow \frac{2.50}{10} \times -32 = f_c - 1002$

$\Rightarrow f_c = 1002 - \frac{2.50}{10} \times 32$

$\Rightarrow f_c = 994 \text{ kgf/cm}^2$

$\Rightarrow f_c = \frac{994 \times 9.81}{100}$

$f_c = 97.511 \text{ N/mm}^2$

* Load carrying capacity :

$P = A \times f_c$

$= 732 \times 97.511$

$= 71378.052 \text{ N}$

$= \frac{71378.052}{1000}$

$= 71.737 \text{ kN} > 60 \text{kN}$

* Check for tension member :—

$f_t = 1250 \text{ kgf/cm}^2$ → (Pg-5, T-1, IS·806-1968)

$= \frac{1250 \times 9.81}{100}$

$= 122.625 \text{ N/mm}^2$

* $\frac{1 \text{kgf}}{\text{cm}^2} = \frac{9.81}{100} \frac{\text{N}}{\text{mm}^2}$

$P = A \times f_t$

$= 732 \times 122.625$

$= 89761.5 \text{ N}$

$= \frac{89761.5}{1000} = 89.761 \text{ kN}$.

$= 89.761 \text{ kN} > 55 \text{kN}$.

* For AC member :—

$l = 1.8$ m,

Tension load = 80 kN

Compression load = 55 kN.

$\sigma_{st} = 1250 \text{ kgf/cm}^2$

$\text{Stress} = \frac{\text{Load}}{\text{Area}}$

$\text{Area} = \frac{\text{Load}}{\text{stress}}$

$= \frac{80 \times 10^3}{\frac{1250 \times 9.81}{100}} = 652.395 \text{ mm}^2$

Try, Nominal diameter = 50 mm, Heavy section.

Outside diameter = 60.3 mm

Area = 788 mm²

$r = 1.98 \text{ cm} \simeq 19.8 \text{ mm}$

$\frac{Kl}{r} = \frac{0.7L}{r} = \frac{0.7 \times 1.8 \times 1000}{19.8}$

$= 63.63$

f_c, permissible stress for compression :-

$\frac{Kl}{r}$	f_c
60	1002
63.63	f_c
70	970

$$\frac{63.63 - 60}{70 - 60} = \frac{f_c - 1002}{970 - 1002}$$

$$\Rightarrow \frac{3.63}{10} = \frac{f_c - 1002}{-32}$$

$$\Rightarrow \frac{3.63}{10} \times (-32) = f_c - 1002$$

$$\Rightarrow f_c = 1002 - \frac{3.63 \times (32)}{10}$$

$$\Rightarrow f_c = 990.384 \ Kgf/cm^2$$

$$f_c = \frac{990.384 \times 9.81}{100}$$

$$= 97.156 \ KN/mm^2$$

$P = A \times f_c$

$= 788 \times 97.156$

$= 76.55 KN > 55 KN$

* Permissible stress for tension :-

$Ys\,t\,22 = 1250 \ Kgf/cm^2$

$= 1250 \times \frac{9.81}{100} = 122.625 \ N/mm^2$

$P = A \cdot e$

$= 788 \times 122.625 = 96628.5 N$

$\simeq 96.628 KN$

* Joint A

$D = 76.1mm$ (outside diameter of main)

$d = 60.33mm$ (outside diameter of branch)

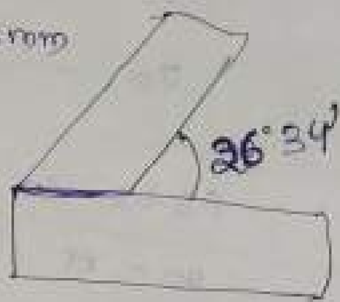
$\theta = 26°34'$

The length of curve of inter-section

$$\boxed{L = a + b + 3\sqrt{a^2 + b^2}}$$ → (pg-18, 19, IS:806-1968)

$a = \frac{d}{2} \operatorname{cosec}\theta$

$= \frac{60.33}{2} \operatorname{cosec}(26°34') = 67.413mm$



$b = \frac{d}{3} \times \frac{3 - (d/D)^2}{2 - (d/D)^2}$

$= \frac{60.33}{3} \times \frac{3 - (60.33/76.1mm)^2}{2 - (60.33/76.1)^2}$

$= 34.746 mm$

$L = a + b + 3\sqrt{a^2 + b^2}$

$= 67.413 + 34.746 + 3\sqrt{67.413^2 + 34.746^2}$

$= 329.680 mm.$

Let 's' be the weld size.

Permissible stress in weld $= 108 N/mm^2$.

strength of weld $= K \cdot S \cdot Lw \cdot f \times$ permissible stress

$80 \times 10^3 = 0.7 \times S \times 329.680 \times 108$

$S = \frac{80 \times 10^3}{0.7 \times 329.680 \times 108}$

$S = 3.20mm$

$S \simeq 4mm$

Hence, provide 4mm fillet weld.

(Q) The principal rafter in a round tubular truss carries a maximum force of 84KN. A tension member meeting at right angle to the principal rafter carries a force of 20KN. Design the member using IS:1161 grade st 25 steel for the tube. The panel length along the principal rafter is 1.80m.

*{st 35 = Yst 22}

Ans:- Load of compression member = 84KN

Load of tension member = 20KN.

Assuming maximum allowable stress = 80 N/mm²

$A_g = \frac{load}{stress} = \frac{84 \times 10^3}{80} = 1050mm^2$ {stress = $\frac{Load}{Area}$}

$\simeq \frac{1050}{100} = 10.50cm^2$

Taking a trial section 88.9 mm outer diameter and weight = 8.36kg/m

= 83.6 N/m

Area provided = 10.7cm² = 1070mm² (Medium)

{IS 1161 - 1998, T-1, pg-2}

Radius of gyration = 30mm.

Length of rafter = 1.8m = 1800mm

Effective length = 1800mm

slenderness ratio = $\frac{l}{r} = \frac{1800}{30} = 60$ → {IS 806-1968, T-2, pg-6}

* For $\frac{l}{r} = 60$, f_c = ~~1002~~ = ~~100.2 N/mm²~~

$f_c = 1002$ kgf/cm² $= \frac{1002 \times 9.81}{100} = 98.296$ N/mm².

* Safe load carrying capacity :—

Load = Stress × area = 98.296 × 1070

= 105176.72 N

= 105176 KN > 84 KN (ok)

* Tension member (IS-806-1968 (Pg-5, T-1))

st (35) = Yst 22 = 1250 kgf/cm² = 122.625 N/mm²

Permissible axial tensile stress (ft) = 122.625 N/mm².

Area = $\frac{Load}{stress} = \frac{20 \times 10^3}{122.625} = 163.09 mm^2 = 1.63cm^2$

Here the take outside diameter of tube → (Pg-2, T-1, IS:-1161-1998)

= 26.9 @ 1.98kg/m

area = 178mm²

area provided > area required .... (ok)

Length of curve intersection :-

Here diameter of main tube $(D) = 88.9mm$

$(d) = 26.9m$

length of curve intersection, , $\theta = 90°$

$l = a + b + 3\sqrt{a^2 + b^2}$

$a = \frac{d}{2} \operatorname{cosec}\theta = \frac{26.9}{2} \operatorname{cosec}(90°) = 13.45 mm$

$b = \frac{d}{3} \times \frac{3 - (d/D)^2}{2 - (d/D)^2}$

$= \frac{26.9}{3} \times \frac{3 - \left(\frac{26.9}{88.9}\right)^2}{2 - \left(\frac{26.9}{88.9}\right)^2}$

$= 13.66mm$

$\therefore l = 13.45 + 13.66 + 3\sqrt{13.45^2 + 13.66^2}$

$= 84.62mm$

Here allowable shear stress $= 900 kgf/cm^2$ (IS 806:1968, 9-4, pg-2)

$= \frac{900 \times 9.81}{100}$

$= 88.29 N/mm^2$

$Area = \frac{load}{stress}$

$\Rightarrow l \times t \times s = \frac{20 \times 10^3}{90}$

$\Rightarrow 84.62 \times t \times s = \frac{20 \times 10^3}{90}$

$\Rightarrow ks = \frac{20 \times 10^3}{90 \times 84.62}$ $\{k = 0.7s\}$

$\Rightarrow ks = 2.626mm$

$\Rightarrow s = \frac{2.626}{k}$

$\Rightarrow s = \frac{2.626}{0.7} = 3.75mm \simeq 4mm$.

$\therefore$ Provide size of weld = 4mm and length of weld = 84.62 mm.

DESIGN OF MASONRY STRUCTURES

* Choice of masonry units is generally made from the consideration of (a) local availability, (b) compressive strength, (c) durability (d) cost, (e) ease of construction.
* Brick has the advantage over stone that it lends itself to easy construction and requires less labour for laying.
* Stone masonry, because of practical limitations of dressing to shape and size, usually has to be thicker and results in unnecessary extra cost.

DESIGN CONSIDERATION FOR MASONRY WALLS & COLUMNS :-

In order to ensure uniformity of loading, openings in walls should not be too large and these should be of 'hole in wall' type as far as possible; bearings for lintels and bed blocks under beams should be for lintels and bed blocks under beams should be liberal in sizes; heavy concentration of loads should be avoided by judicious planning and sections of load bearing members should be varied where feasible with the loadings so as to obtain more or less uniform stress in adjoining parts of members. One of the commonly occuring causes of cracks in masonry is wide variation in stress in masonry in adjoining parts.

* Lateral Supports and stability :-

⟹ Stability :- The stability force equal to "2.5 percent" of all vertical loads acting above that lateral support is assumed for checking the adequacy of that support. This horizontal force is in addition to any other lateral force, namely wind or seismic that the structure may be subjected to.

⟹ Thickness and spacing of stiffening walls (Brick size 23×11.5×7.7cm)

Sl No.	Thickness of load bearing wall to be stiffened (cm)	Height of storey (m)	Stiffening wall: Minimum Thickness (cm)	Stiffening wall: Maximum spacing (m)
1.	11.5	3.25	11.5	4.50
2.	23	3.25	11.5	6.00
3.	34.5 and above	5.00	11.5	8.00

→ Cross walls in conjunction with floors in a building provide stability to the structure against the effect of lateral loads that is, wind, etc.

If wall is longer than 8.0m, the end walls may not be able to provide adequate stability and necessary to check stability and stresses by structural analysis.

→ Effective Height :—

Minimum thickness of Basement Walls (Bricksize 23 x 11.5 x 7.7cm)

Sl.No.	Minimum thickness of basement wall	Height of the Ground Above Basement Floor with wall Loading (Permanent Load) of — More than 50 KN/m	Less than 50 KN/m
	(cm)	(m)	(m)
1.	34.5	2.50	2.00
2.	23	1.35	1.00

* Wall :—

Actual height of a wall for the purpose of working out its effective height should be taken to be the clear distance between the supports.

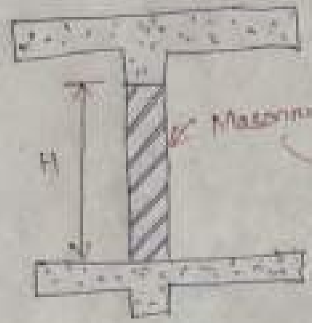
→ When wall thickness is not less than 2/3 of the thickness of pier, a concentrated load on the pier, will be borne by the pier as well as the wall.

→ In case thickness of wall is less than 2/3 of the thickness of pier, we have to design the pier just like a column, for which permissible stress is less because of greater effective height and further supporting area will be only that of the pier that is, without getting any benefit in design of the adjoining walls on either side.

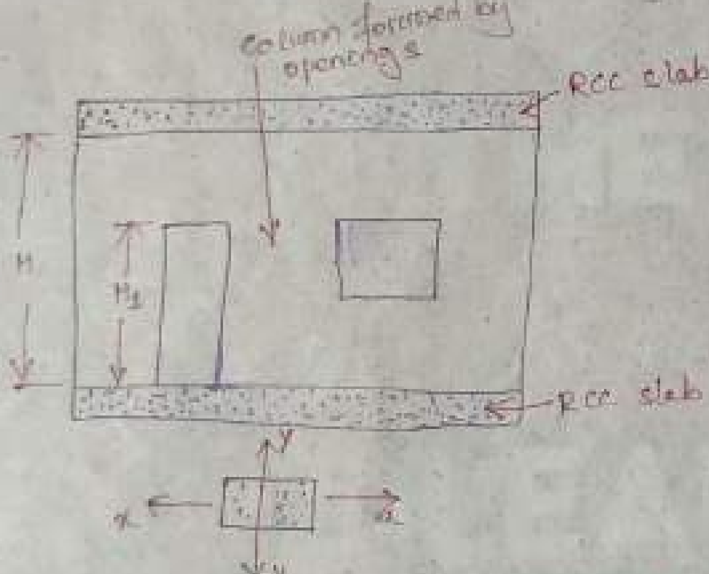
* Column :—

→ An RCC slab bearing on a wall is assumed to provide full restraint to the wall while a timber floor comprising timber joints and planking is assumed to provide is assumed to provide only partial restraint.

(a) When wall has full restraint at top and bottom.
When wall has partial restraint at top and bottom



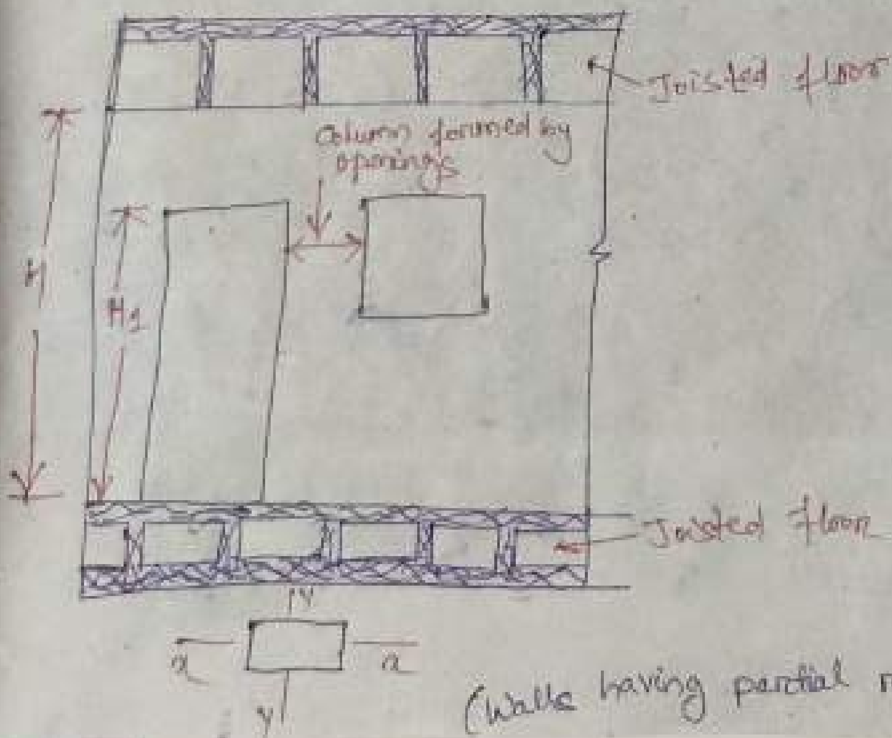
* Actual Height, (H) = clear distance between support



Effective Height ⇒ $h_{xx} = 0.75H + 0.25H_1$

$h_{yy} = H$

(Walls having full restraint)



For $H_1 \leq 0.5H$

$h_{xx} = H$

$h_{yy} = 2H$

For $H_1 > 0.5H$

$h_{xx} = 2H$

$h_{yy} = 2H$

(Walls having partial restraint)

→ If height of neither opening exceeds "0·5H", wall masonry would provide some support to the column formed by openings exceeds 0·5H, wall masonry would provide some support to the column formed by openings in the direction parallel to the wall and for this reason effective height for the axis perpendicular to the wall is taken as "H" and otherwise it is to be taken as "2H".

→ For the direction perpendicular to the wall, there is a likelihood of a situation when no joist rests on the column formed between the openings and thus effective height is taken as "2H" that is, for a column having no lateral support at the top.

* Effective length :—

→ When a wall has more than one opening such that there is no opening within a distance of "H/8" from a cross wall and wall length between openings are not columns by definition, the design of the wall should be based on the value of Slenderness Ratio (SR) obtained from the consideration of height or length, whichever is less.

* Effective thickness :—

→ In case of masonry using modular bricks, actual thickness of a one-brick wall for design calculation is taken as 19 cm, though nominal thickness is 20 cm.
Similarly in case of brick masonry with bricks of old size actual thickness of one brick wall would be taken as 22 cm though nominal size of brick is 23 cm.

→ When ratio t_p/t_w is 1·5 or less and the wall is having distributed load. Table-4 would be applicable. (Pg-10)

→ When ratio t_p/t_w exceeds 1·5, Table-6 are valid

Slenderness Ratio :—

→ Under a vertical load a wall would buckle either around a horizontal axis parallel to the length of the wall or around a vertical axis. Buckling is resisted by horizontal supports such as floor and roofs, as well as by vertical supports such as cross walls, piers and buttresses. Thus capacity of the walls to take vertical loads depends both on horizontal supports that is, floor or roof as well as on vertical supports that is cross walls, piers and buttresses. Thus capacity of the walls to take vertical ~~support~~ loads depends both on horizontal supports that is, floor or roof as well as on vertical supports that is, cross walls, piers and buttresses.

→ Load carrying capacity of a masonry member depends upon its slenderness ratio. As the ratio increases, crippling stress of the member gets reduced because of limitations of workmanship and elastic instability. A masonry member may fail, either due to excessive stress or due to buckling.

→ According to Sahlin for materials of normal strength with SR less than 30, the load carrying capacity of a member at ultimate load is limited by stress, while for higher value of SR failure is initiated by buckling.

→ Further, mode of failure of a very short member having h/t ratio of less than 4 is predominated through shear action, while with $h/t = 4$ or more, failure is by vertical tensile splitting.

→ Stress in masonry is worked out at a depth of $\frac{H}{8}$ from the bottom of the beam. This should not exceed the permissible compressive stress in masonry.

→ Full value of stress reduction factor is applicable only for central one-fifth height of the member.

* In case of walls, $\boxed{SR = \frac{h}{t \times K_n}}$ or $\boxed{\frac{L}{t}}$ for design lesser of the two values is considered.

* In case of columns if SR is different for the two horizontal axes, greater of the two values of 'SR' is considered in design.

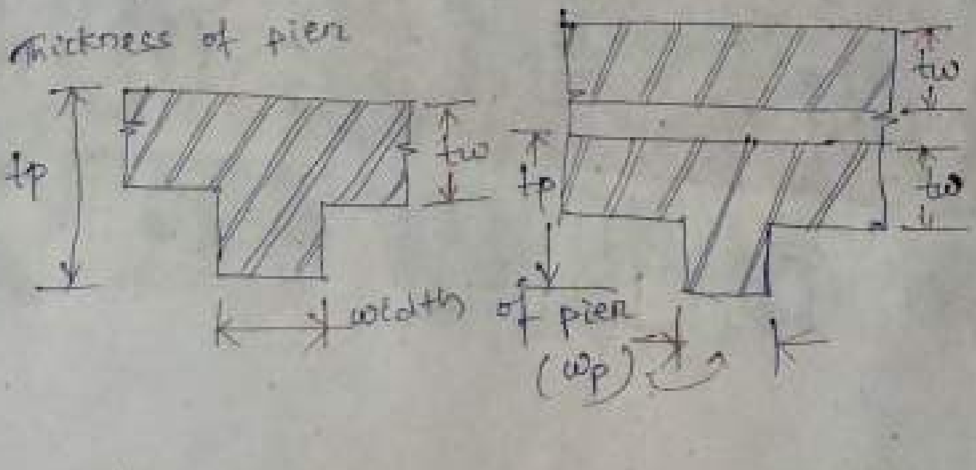
→ K_n = stiffening coefficient from Table-6, Pg-14, IS 1905:-1987

Stiffening coefficient for walls.				
Sl No.	$\frac{SP}{\omega p}$ ratio	Stiffening coefficient $\frac{tp}{tw}=1$	$\frac{tp}{tw}=2$	$\frac{tp}{tw}=3$
1.	6	1.0	1.4	2.0
2.	8	1.0	1.3	1.7
3.	10	1.0	1.2	1.4
4.	15	1.0	1.1	1.2
5.	20 or more	1.0	1.0	1.0

tp = centre to centre spacing of the pier or cross wall.

tw = actual thickness of the wall proper.

ωp = width of the pier in the direction of the wall or the actual thickness of the cross wall.



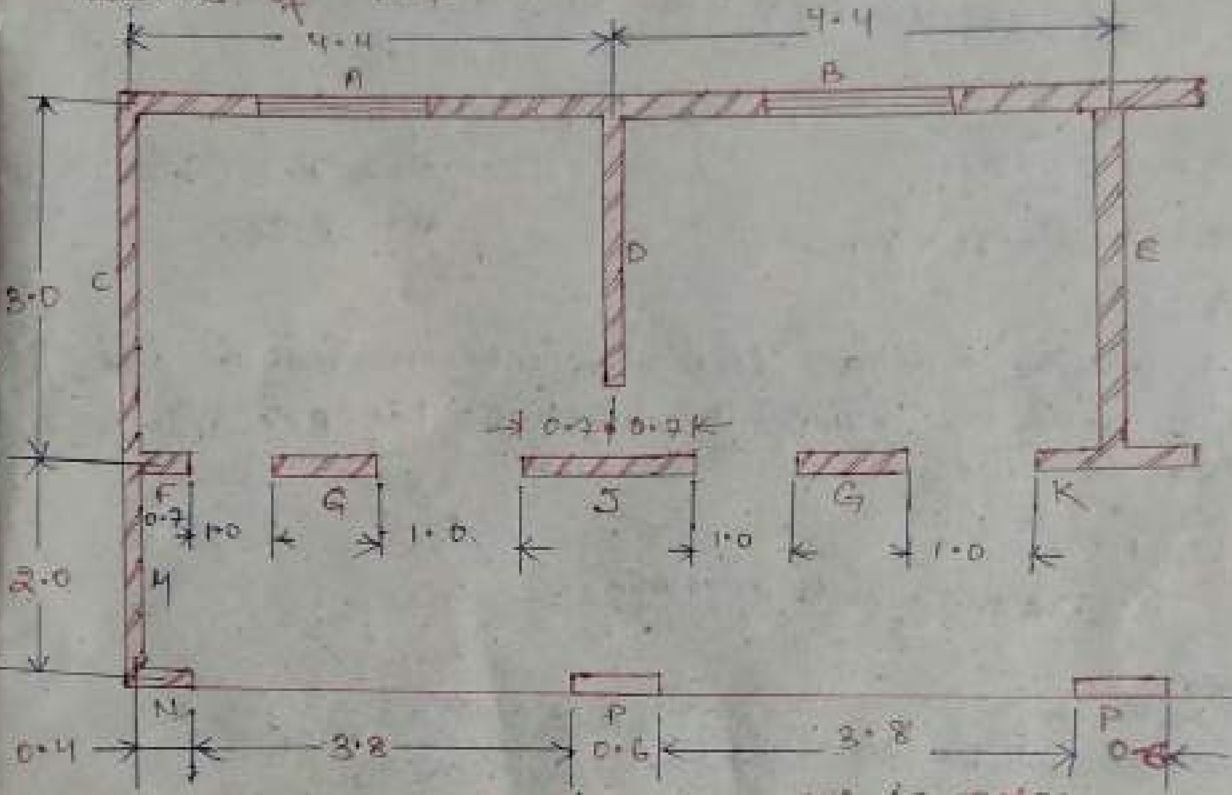
Eccentricity :-

→ Eccentricity of vertical loading on a masonry element increases its tendency to buckling and reduces its load carrying capacity; its effect is thus similar to that of slenderness of the member.

→ Eccentricity caused by an eccentric vertical load is maximum at the top of a member, that is, at the point of loading and it is assumed to reduce linearly to zero at the bottom of the member that is just above the bottom lateral support, while eccentricity on account of slenderness of a member is zero at the supports and is maximum at the middle. Taking the combined effect of eccentricity of loading and slenderness critical stress in masonry occurs at a section 0.6H above the bottom support.

→ It is assumed that critical section in a storey height is at the top of bottom support and masonry is designed accordingly. In other words the design method commonly adopted includes extra self weight of 0.6H of the member and thus errs on the safe side to some extent.

Q-1 In a double storeyed building walls are 20cm thick, clear height of floors is 3.0m, plinth is 0.7m above the foundation footing, floor and roof are of RCC 12cm thick, door height is 2.1m, window height is 1.5m and plan of the building is shown in fig. workout effective height, effective length, effective thickness and slenderness ratio of walls and columns of First and Second floors.



All dimensions are in metres.

Solution:—

1) Effective Height

(a) First floor

plinth = 0.7m
clear height = 3.0m
RCC roof = 12cm

Actual height (H) = 0.7 + 3.0 + 0.06 → $\frac{0.12}{2}$

= 3.76m

A, B, C, D, E, J and M are all walls, having length more than 4 times thickness and thus;

Effective height; → Pg-12, IS : 1905-1987

$h = \boxed{0.75H}$

$= 0.75 \times 3.76$

$= 2.82m$

In case of F, K and N wall, even though length is less than 4t, these are not to be treated as columns, because they are supported on one side by cross walls. There effective height, therefore, will be determined as in the case of wall and will be 2.82m.

$\frac{h}{t} < 4$ or $h < 4t$ (Pg-12)-SP-20

Brickwork G - Since length is less than 4t, it is a column by definition.

$h = \boxed{0.75H + 0.25H_1}$ → SP-20, Pg-11

$H_1 = 2.1$ (Door height)

$H = 3.76$

$= 0.75 \times 3.76 + 0.25 \times 2.1$

$= 3.34m.$

Brickwork P - is a column being less than '4t' in length, and is supported in both the horizontal directions by RCC beam/slab. Effective height is thus :-

$h = H$

$= 3.76m$ in both directions.

(b) Second floor :-

$H = 0.06 + 3.0 + 0.06$

$= 3.12m$

*RCC slab, $\frac{0.12}{2} = 0.06$

clear span = 3m

'h' for A, B, C, D, E, J, K, M and N

$= 0.75H$ → Pg-12, IS 1905-1987

$= 0.75 \times 3.12$

$= 2.34m$

'h' for G $= \boxed{0.75H + 0.25H_1}$ → SP-20, Pg-11, (E-13A)

$= 0.75 \times 3.12 + 0.25 \times 2.1$

$= 2.87m$

'h' for P in both directions $= H$

$= 3.12m.$

(8) Effective length of walls :—

Effective length of walls will be same in first floor and second floor

* Wall A. It is continuous on one end and discontinuous on the other end is supported by cross walls.

$L = 4.4m$

{ Pg-12, IS:- 1905-1987 T-5 }

$\therefore\ l = 0.9L$

$= 0.9 \times 4.4$

$= 3.96m$

* Wall B- It is continuous on both ends and is supported by cross-walls.

$\therefore\ l = 0.8L$

$= 0.8 \times 4.4$

$= 3.52m$

* Wall C - It is discontinuous on one side and continuous on the other and is supported by cross walls on both sides.

$\therefore\ l = 0.9L$

$= 0.9 \times 3.0$

$= 2.7m$

* Wall D - It is discontinuous on one side and has an opening on the other side which is taller than $0.5H$

$\therefore\ l = 1.5 \times 3.0$

$= 4.5m$

* Brickwork F - This wall, because of opening taller than $0.5H$ is free on one end and is supported by cross wall on the other end is discontinuous

$\therefore\ L = 2L$

$= 2 \times 0.7$

$= 1.4m$

* Brickwork G - It has no support on either side and thus slenderness for this element will be governed by its height

* Brickwork J - same as brickwork G

* Brickwork K - This is free at one end and supported by

a cross wall but continuous at the other.

$\therefore$ $l = 1.5L$
$= 1.5 \times 0.7$
$= 1.05\,m$

Note:- The element K has been taken as continuous on one end, because length of wall between cross wall and opening is more than

$\frac{H}{8} = \frac{3}{8}$ (SP-20, pg-12, cl-4.6, v(c))

$\frac{H}{8} = 0.37\,m$

* Wall M- The wall is continuous on one side and discontinuous on the other. On one side it is supported by a cross wall which is more than (SP-20, pg-12, cl-4.6(v))

$\frac{H}{5} = \frac{3}{5} = 0.6\,m$

in length, being $0.70 - 0.095 = 0.605\,m$

* On the other side, length of cross wall is less than $H/5$ and thus this side is not adequately supported. The brickwork is thus supported and continuous on one side and free on the other, thus

$l = 1.5L$
$= 1.5 \times 2$
$= 3.0\,m$

* Brickwork N - This element is discontinuous and supported on one end and free at the other, thus

$l = 2L$
$= 2 \times 0.40$
$= 0.80\,m$

* Column P is same as Q.

(3) Effective Thickness :-

Assume joints are not raked. Actual thickness will therefore be 19 cm. From examination of plan we find that walls

... continuous at the ...

A, B, C, D, E and M stiffened by cross walls. Thus effective thickness will be actual thickness multiplied by stiffening coefficient.

Walls A and B :— $t_w = 0.19$; (19cm)

$\frac{t_p}{t_w} = 3$ {Pg-17, IS: 1905-1987 T-6}

$\frac{S_p}{W_p} = \frac{4.4}{0.19} = 23$

This being more than 20, stiffening coefficient.

$k_s = 1$

Walls C and E

$\frac{t_p}{t_w} = 3$

$\frac{S_p}{W_p} = \frac{3.0}{0.19} = 15$ (approximately)

$k_s = 1.2$

Wall M :—

$\frac{t_p}{t_w} = \frac{0.40 + 0.095}{0.19}$

$= 2.6$ say 2.5

$\frac{S_p}{W_p} = \frac{2.0}{0.19}$

$= 10.5 = 10$ (say)

$k_s = \frac{1.2 + 1.4}{2} = 1.3$ (By interpolation).

* Slenderness ratio :- $SR = \frac{h}{t \times k_n}$ or $\frac{h}{t}$ or $\frac{l}{t}$ (whichever is less)

Brick work element	First floor, t=0.19				Second floor, t=0.19			
	h	l	kn	SR	h	l	kn	SR
A	2.82	3.96	1.0	14.8	2.34	3.96	1	12.3
B	2.82	3.52	1	14.8	2.34	3.52	1	12.3
C	2.82	2.7	1.2	12.4	2.34	2.7	1.2	10.3
D	2.82	4.5	1	14.8	2.34	2.7	1	12.3
E	2.82	3.0	1.2	12.4	2.34	3.0	1.2	10.3
F	2.82	1.4	1	7.4	2.34	1.4	1	7.4
G	3.34	—	1	17.6	2.87	—	1	15.1
J	2.82	—	1	14.8	2.34	—	1	12.3
K	2.82	1.05	1	5.5	2.34	1.05	1	5.5
M	2.82	3.0	1.3	11.4	2.34	3.0	1.3	0.5
N	2.82	0.80	1	4.2	2.34	0.80	1	4.2
P	3.76	—	1	19.8	3.12	—	1	16.4

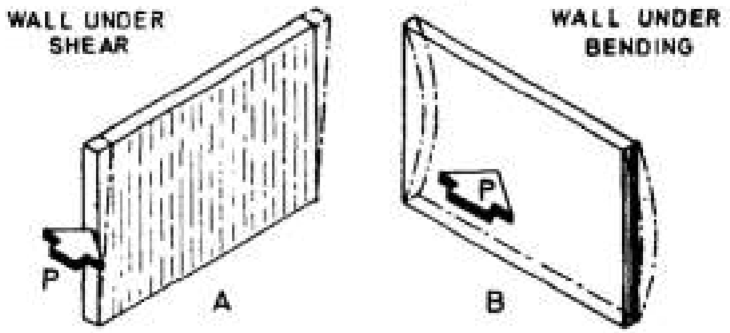
i) Some general guidance on the design concept of load bearing masonry structures is given in the following paragraphs.

ii) A building is basically subjected to two types of loads, namely:

a) vertical loads on account of dead loads of materials used in construction, plus live loads due to occupancy; and

b) lateral loads due to wind and seismic forces. While all walls in general can take vertical loads, ability of a wall to take lateral loads depends on its disposition in relation to the direction of lateral load. This could be best explained with the help of an illustration.

, the wall *A* has good resistance against a lateral load, while wall *B* offers very little resistance to such load. The lateral loads acting on the face of a building are transmitted through floors (which act as horizontal beams) to cross walls which act as horizontal beams) to cross walls which act as shear walls. From cross walls, loads are transmitted to the foundation.

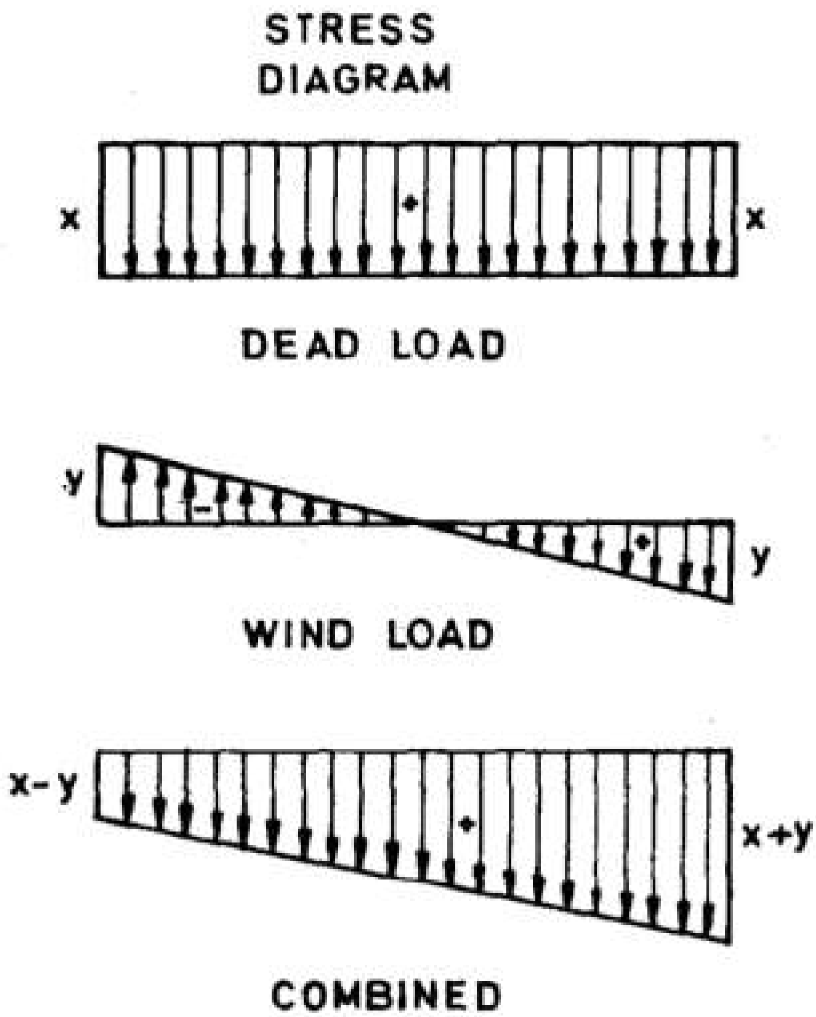


Resistance of brick wall to take lateral loads is greater in case of wall *A* than that in case of wall *B*.

ABILITY OF A WALL TO TAKE LATERAL LOADS

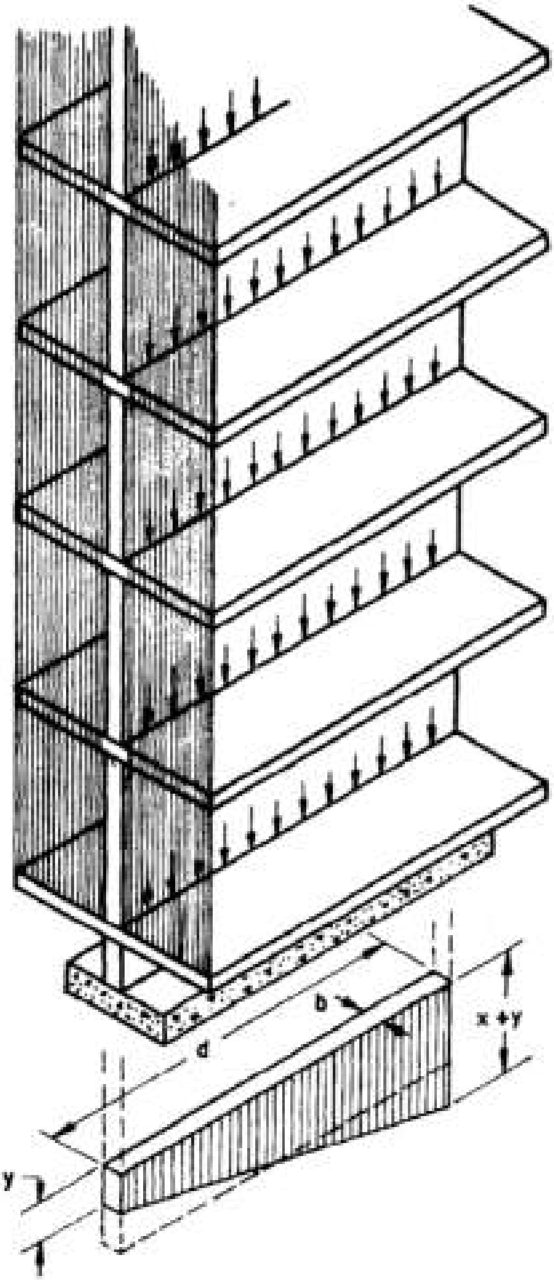
iii) As a result of lateral load, in the cross walls there will be an increase of compressive stress on the leeward side, and

WIND LOAD ON SHADED AREA IS RESISTED BY THE CROSS WALL

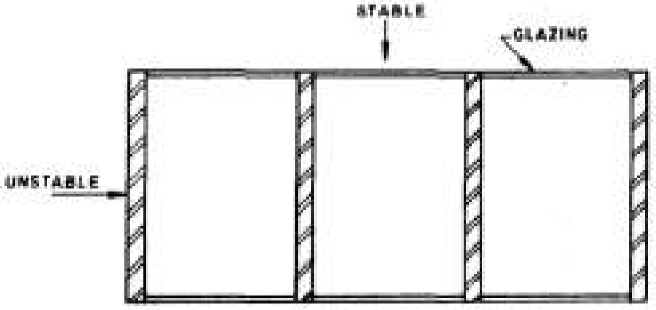


decrease of compressive stress on the windward side. These walls should be designed for 'no tension' and permissible compressive stress. It will be of interest to note that a wall which is carrying greater vertical loads, will be in a better position to resist lateral loads than the one which is lightly loaded in the vertical direction. This point should be kept in view while planning the structure so as to achieve economy in structural design.

iv) A structure should have adequate stability in the direction of both the principal axes. The so called 'cross wall' construction may not have much lateral resistance in the longitudinal direction. In multi-storeyed buildings, it is desirable to adopt 'cellular' or 'box type' construction from consideration of stability and economy

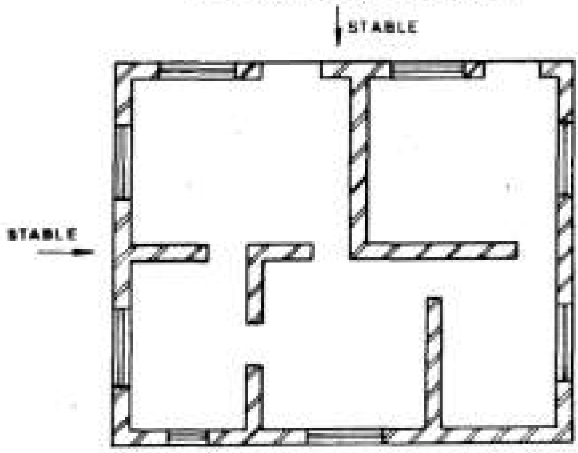


STRESS PATTERN IN CROSS WALL ACTING AS SHEAR WALL



PLAN

E-19 A CROSS WALL CONSTRUCTION-UNSTABLE IN LONGITUDINAL DIRECTION



PLAN

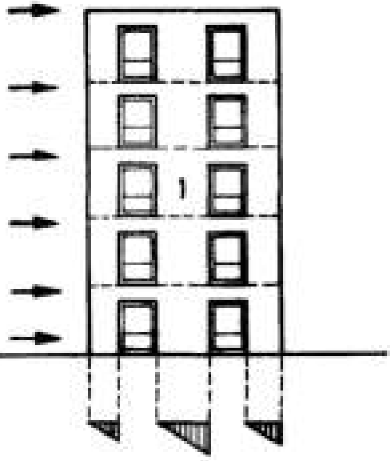
CELLULAR OR BOX TYPE CONSTRUCTION STABLE IN BOTH DIRECTIONS

. STABILITY OF CROSS WALL AND CELLULAR (BOX TYPE) CONSTRUCTION

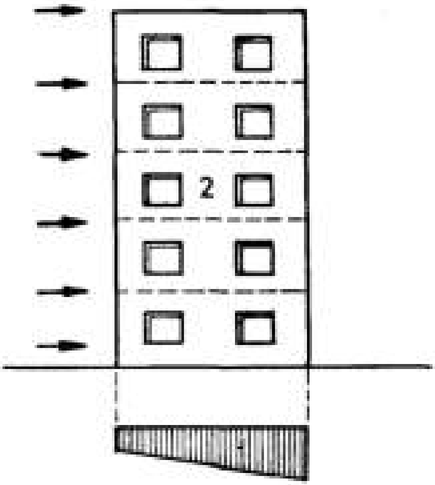
v) Size, shape and location of openings in the external walls have considerable influence on stability and magnitude of stresses due to lateral loads.

vi) If openings in longitudinal walls are so located that portions of these walls act as flanges to cross walls, the strength of the cross walls get considerably increased and structure becomes much more stable.

vii) Ordinarily a load-bearing masonry structure is designed for permissible compressive and shear stresses (with no tension) as a vertical cantilever by accepted principles of engineering mechanics. No moment transfer is allowed for, at floor to wall connections and lateral forces are assumed to be resisted by diaphragm action of floor, roof slabs, which acting as horizontal beams, transmit lateral forces to cross walls in proportion to their relative stiffness (moment of inertia).



This wall will not resist lateral loading as successfully as wall 2; it tends to act as three separate short lengths rather than one.



This wall will tend to act as one long portion of brickwork and will be more resistant to lateral loading.

EFFECT OF OPENINGS ON SHEAR STRENGTH OF WALLS

viii) For working out stresses in various walls, it is faster to tabulate stresses floor-wise for such walls carrying greater loads. Computations for vertical loads and lateral loads are made separately in the first instance, and the results from the two computations are superimposed to arrive at the net value of stresses.

ix) In any particular floor, from practical considerations, generally, quality of bricks and mix of mortar is kept the same throughout. Also in the vertical direction change in thickness of walls is made only at floor levels.

Arching Action

i) Arching in masonry is a well known phenomenon by which part of the load over an opening in the wall gets transferred to the sides of the opening. For good arching action masonry units should have good shear strength and these should be laid in proper masonry bond using a good quality mortar. Further, portions of the wall on both sides of the opening should be long enough to serve as effective abutments for the arched masonry above the opening since horizontal thrust for the arch is to be provided by the shear resistance of the masonry at the springing level on both sides of the opening. If an opening is too close to the end of a wall, shear stress in masonry at springing level of imaginary arch may be excessive and thus no advantage can be taken of arching in masonry for design of lintels.

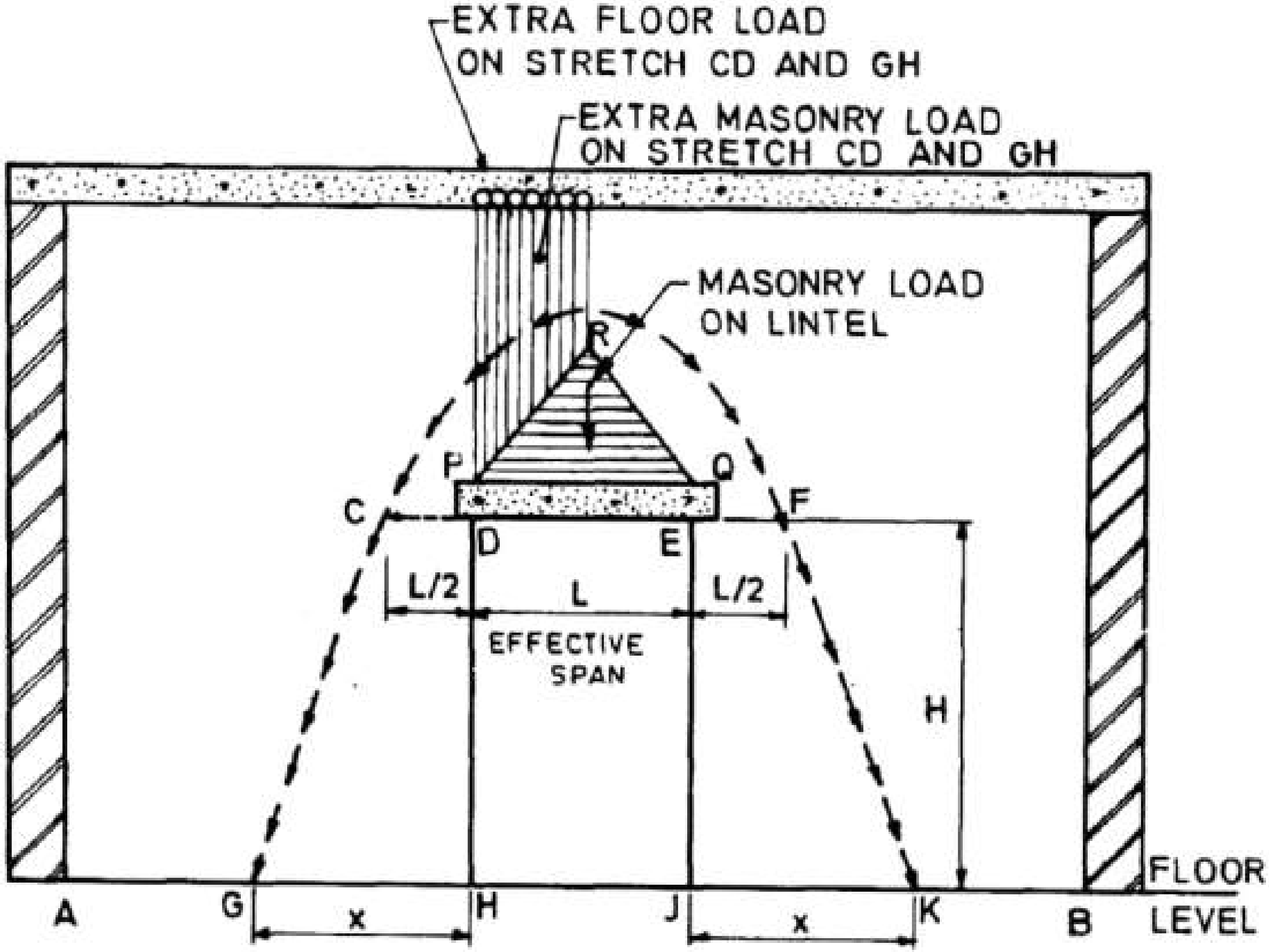
To work out approximate stress in masonry in various stretches, it is assumed that:

a) load from the lintel gets uniformly distributed over the supports,

b) masonry and floor loads above the triangle PRQ get uniformly distributed over the stretches of masonry CD and EF at the soffit level of the lintel, CD and EF being limited in length to $L/2$ and over the stretches GH and JK at the floor level, limited in length to L or $\frac{L-H}{2}$ whichever is less, H being the height of top of the opening from the floor level.

In case some other opening occurs between the lintel and horizontal plane 25 cm above the apex R of the triangle, arching action gets interrupted because of inadequate depth of masonry above the triangle to function as an effective arching ring. Also if there is some other load between the lintel and horizontal plane 25 cm above the apex *R* of the triangle, loading on the lintel gets affected.

iii) In case of buildings of conventional design with openings of moderate size which are reasonably concentric, some authorities on masonry recommend a simplified approach for design. In simplified approach, stress in masonry at plinth level is assumed to be uniformly distributed in different stretches of masonry, taking loadings in each stretch as indicated without making any deduction in weight of masonry for the openings. It is assumed that the extra stresses obtained in masonry by making no deduction for openings, compensates more or less for concentrations of stresses due to openings.

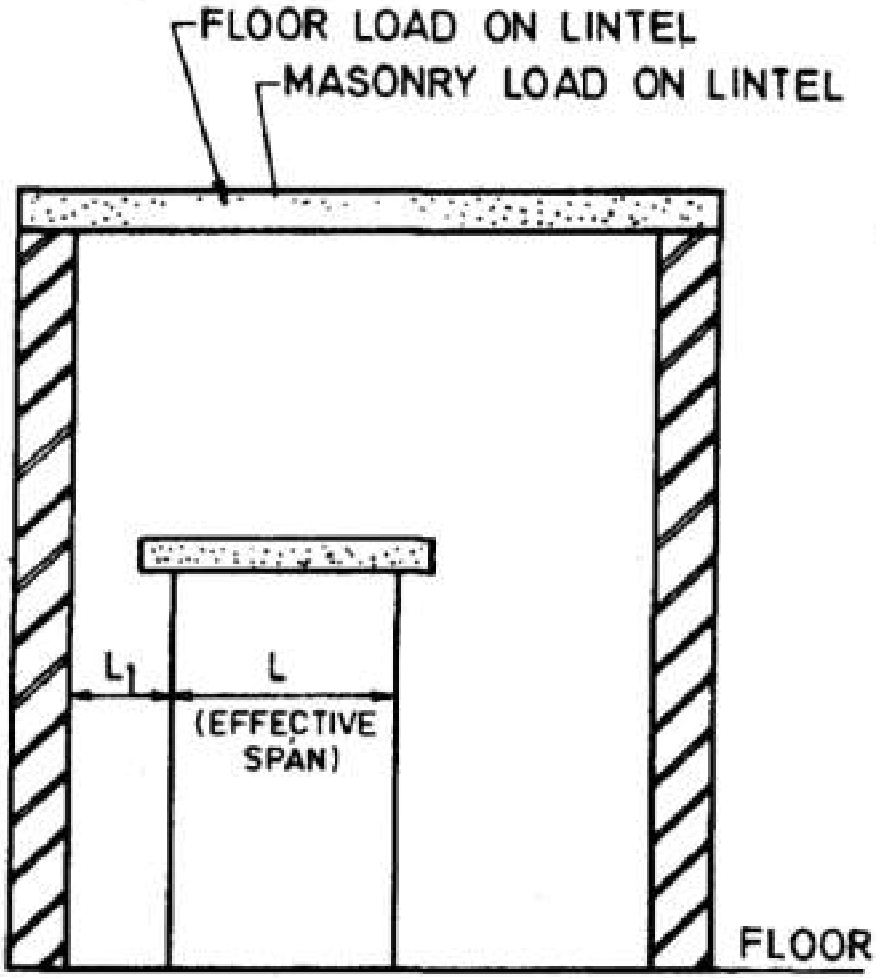


$x = L$ OR $\dfrac{L+H}{2}$, whichever is less.

ARCHING ACTION IN MASONRY

Lintels

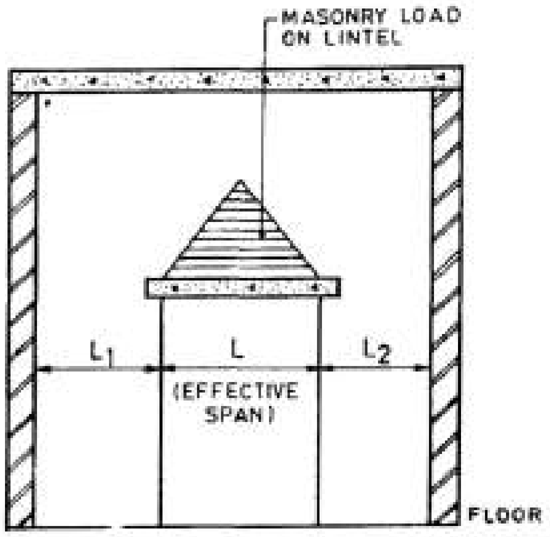
i) Lintels over openings are designed taking into consideration arching action in masonry where feasible as explained earlier. It is a common practice to assume that length of walls on both sides of an opening should be at least half the effective span of the opening for transfer of load to sides by arch action. In case it is less, lintel should be designed for full load over the opening regardless of the height of the floor slab



Effective Load when

$$L_1 < \frac{L}{2}$$

ii) When location and size of opening is such that arching action can take place, lintel is designed for the load of masonry included in the equilateral triangle over the lintel . In case floor or roof slab falls within a part of the triangle in question or the triangle is within the influence of a concentrated load or some other opening occurs within a part of the triangle, loading on the lintel



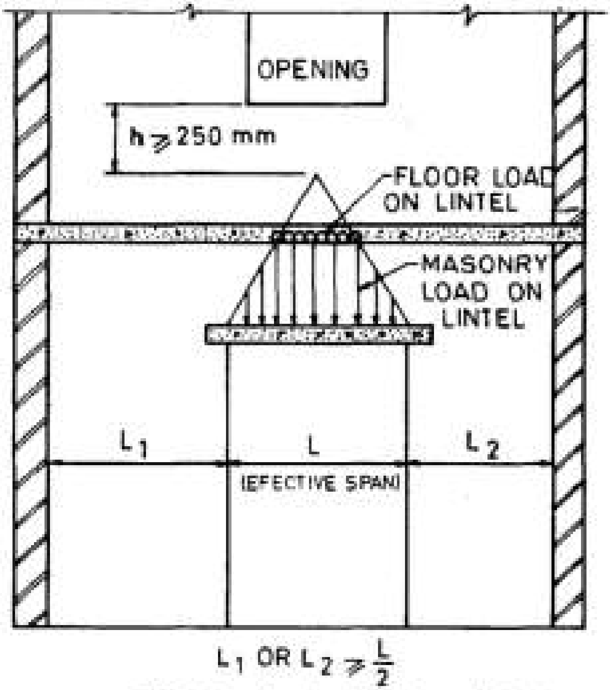
Effective Load when L_1 and $L_2 \geqslant L/2$ and Floor/Roof Slab does not Intercept the Equilateral Triangle Over the Lintel

iii) When stretches of wall on sides are equal to or greater than $L/2$ and equilateral triangle above the lintel is intercepted by the floor/roof slab, the lintel is designed for load of masonry contained in the equilateral triangle plus load from the floor falling within the triangle

iv) When stretches of wall on the sides of the opening are equal to or greater than $L/2$ with the equilateral triangle over the lintel intercepted by floor slab and another opening comes within the horizontal plane 25 cm above the apex of the triangle, lintel is to be designed for loads

v) When any other load is coming between the lintel and horizontal plane 25 cm above the apex of the equilateral triangle over the lintel, the latter is designed for the loads

vi) It may be clarified that in fact load coming on a lintel is indeterminate and the above suggestions for the design of lintels are based on empirical rules derived from



Effective Load when L_1 and $L_2 \geqslant L/2$, and Equilateral Triangle Over the Lintel is Intercepted by Floor Slab Above with no Other Opening to Intercept Arch Action

experience and general principles of engineering.

vii) Economy in the design of lintels may be effected by taking advantage of composite action between lintel and the masonry above it. For this purpose centering of the lintel should not be removed till both masonry (up to 25 cm above the apex of equilateral triangle above the lintel) and RCC of the lintel have gained sufficient strength so as to be able to bear stresses in the composite beam having masonry in compressive zone and RCC lintel in the tensile zone.

single brickwidth walls for vertical loads are stronger than multiple brick width walls as can be readily seen from the test results reproduced below

Wall Construction	*Wall Thickness* cm (in)	*Relative Strength*
(1)	(2)	(3)
Single brick-width	12.7 (5)	1.00
-do-	15.2 (6)	0.89
-do-	17.8-25.4 (7-10)	0.80

(1)	(2)	(3)
Multiple brick-width	25.4-38.1 (10-15)	0.68

Theoretical explanation for the above behaviour of masonry is that presence of vertical joints, which have a much lower lateral tensile strength, reduces the compressive stress of masonry under axial loading. Thus greater is the frequency of vertical joints, lesser is the compiessive strength of masonry. Thus a 20/23 cm thick brick wall (one brick-length) is weaker than a 10/11.5 cm brick wall of single brick-width because of presence of vertical joints in both the directions in the former.

Compressive Strength of Masonry, which is based on British Standard[2], may be presumed to hold good for one brick-length or thicker walls and thus in case of half-brick load bearing walls some increase in Basic stress may be permitted

. **Permissible Stresses**

.1 *Permissible Compressive Stress*

Stress reduction factor

When a wall or column is subjected to an axial plus an eccentric load resultant eccentricity of loading ($\bar{e}$) may be worked out as follows:

$$W = W_1 + W_2$$

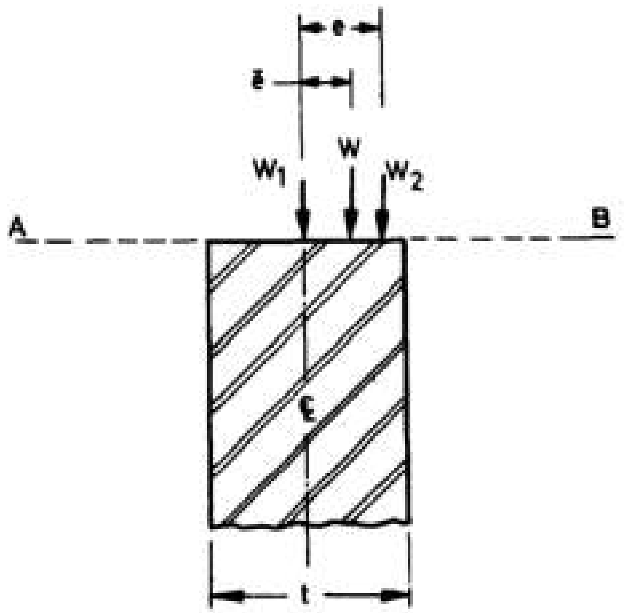
Taking moments about AB,

$$W\bar{e} = W_1 \times O + W_2 e$$

$$\therefore = \frac{W_2 e}{W_1 + W_2}$$

5.4.1.2 *Area reduction factor*

i) Provision of Area reduction factor in this Code was originally similar to that in 1970 version of British Standard Code CP III[2]. When the Code was revised in 1980, upper limit of 'small area' was reduced from 0.3 to 0.2 m² based on the provision in BS 5628 Part 1 : 1978[9].



W_1 = axial load.
W_2 = eccentric load at distance 'e' from centre line.
W = resultant load at distance '$\bar{e}$' from centre line.
$\bar{e}$ = resultant eccentricity.

RESULTANT ECCENTRICITY

ii) Area reduction factor due to 'small area' of a member is based on the concept that there is statistically greater probability of failure of a small section due to sub-standard units as compared to a large element.

The reason for this seems to be that factor of safety/load factors inherent in a Code should be enough to cover the contingency mentioned above for this provision.

limits for smallness of area in this context are taken 0.13 and 0.10 m^2, respectively.

Shape reduction factor

Shape modification factor is based on the general principle that lesser the number of horizontal joints in masonry, greater its strength or load carrying capacity. It has, however, been found from experimental studies that for units stronger than 15 N/mm^2, extent of joints in masonry does not have any significant effect on strength of masonry because of use of the comparatively high strength mortar that normally goes with high-strength units.

. . . Increase in permissible compressive stresses allowed for eccentric vertical and/or lateral loads under certain conditions

i) Eccentric vertical load (vertical load plus lateral load in case of free standing walls) on masonry causes bending stress in addition to axial stress. It has been found that masonry can take 25 percent greater compressive stress, when it is due to bending than when it is due to pure axial load, because maximum stress in case of bending occurs at the extreme fibres and then it gets reduced linearly while in axial compression, stress is more or less uniform throughout the section. For similar reasons permissible compressive stress in concrete for beams is greater than that in columns subjected to vertical loads.

ii) When loading on a masonry element has some eccentricity, the Code lays down the design approach for various ranges of eccentricity ratios namely (a) eccentricity ratio of $\frac{1}{24}$ or less; (b) eccentricity ratio exceeding $\frac{1}{24}$ but not exceeding $\frac{1}{6}$, and (c) eccentricity ratio exceeding $\frac{1}{6}$. Basis of the design approach is explained below (*see also* Fig. E-28).

a) Eccentricity ratio of $\frac{1}{24}$ or less—

Refering to Fig. E-28B, W is total vertical load per unit of wall with resultant eccentricity $\bar{e}$, t is thickness of wall, f_1 and f_2 are the stresses at the two faces of the wall and f_m is Permissible compressive stress for axial loading.

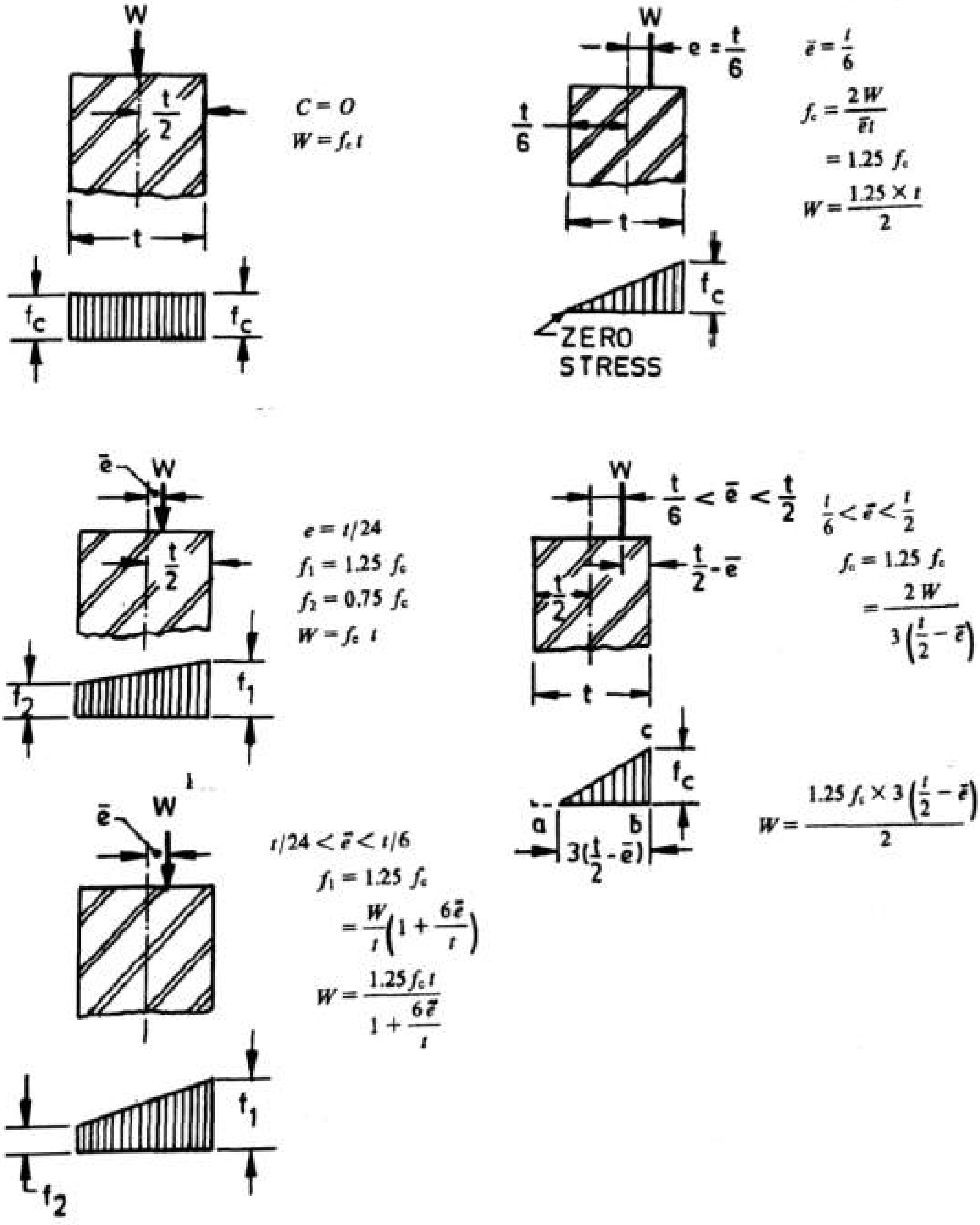
$$f_1 = \frac{W}{A} + \frac{M}{Z}$$

$$f_2 = \frac{W}{A} - \frac{M}{Z}$$

Substituting values of A, M and Z

$$f_1 = \frac{W}{t} + \frac{W\bar{e} \times 6}{t^2} = \frac{W}{t}\left(1 + \frac{6\bar{e}}{t}\right)$$

$$f_2 = \frac{W}{t} - \frac{W\bar{e} \times 6}{t^2} = \frac{W}{t}\left(1 - \frac{6\bar{e}}{t}\right)$$



W = permissible load per unit length of wall.
f_c = permissible compressive stress of masonry.
$\bar{e}$ = resultant eccentricity of loading.
t = thickness of wall.

Variation in Stress Distribution with Change in Eccentricity of Loading for eccentricity ratio $\frac{e}{t} = \frac{1}{24}$, and since $\frac{W}{t}$ is equal to axial compressive stres f_c.

$$f_1 = \frac{W}{t}\left(1 + \frac{1}{4}\right) = 1.25\ f_c$$

$$f_2 = \frac{W}{t}\left(1 - \frac{1}{4}\right) = 0.75\ f_c$$

As we allow 25 percent additional compressive stress in case of eccentric loading, it follows that maximum compressive stress (f_1) for eccentricity ratio up to $\frac{1}{24}$ does not exceed axial compressive stress by more than 25 percent which is permitted by the code. Therefore for eccentricity ratio of $\frac{1}{24}$ or less, it is not necessary to compute and add bending stress to the axial stress. The designer is expected to work out only axial compressive stress for the purpose of design and see that it does not exceed Permissible compressive stress for axial load.

$\therefore$ Design load, $W = f_c\ t$

b) Eccentricity ratio exceeding $\frac{1}{24}$ but not exceeding $\frac{1}{6}$ (*see* Fig. E-28C and E-28D)

Bending stress $= \frac{We \times 6}{t_2}$

for eccentricity ratio $\frac{1}{6}$ (substituting in the above equations),

$$f_1 = \frac{W}{t} + \frac{W}{t} = \frac{2W}{t}$$

$$f_2 = \frac{W}{t} - \frac{W}{t} = 0$$

Thus on one face compressive stress gets doubled and on the other face it is fully nullified by tensile stress and there is no tension in the cross section. For loading with eccentricity ratio between $\frac{1}{24}$ and $\frac{1}{6}$, we have to limit the maximum stress f_1 to 1.25 f_c

$$f_1 = \frac{W}{t}\left(1 + \frac{6e}{t}\right) = 1.25\ f_c$$

$\therefore$ Design Load,

$$W = \frac{1.25\ f_c t}{1 + \frac{6e}{t}}$$

c) *Eccentricity ratio exceeding* $\frac{1}{6}$ (*see* Fig. E-28E)—We had seen from (b) above that when eccentricity ratio reaches the value $\frac{1}{6}$, stress is zero on one face; when this ratio exceeds $\frac{1}{6}$ there will be tension on one face rendering ineffective a part of the section of the masonry and stress distribution in this case would thus be as shown in Fig. E-28E. Average compressive stress:

$$f_a = \frac{f_1 + 0}{2} = \frac{f_1}{2}$$

Since f_1 has to be limited to 1.25 f_c

$$f_a = \frac{1.25\ f_c}{2}$$

The design load W in this case will be equal to average compressive stress multiplied by length *ab* of the stress triangle *abc*. Since for equlibrium, the load must pass through the centroid of the stress triangle *abc* and the load is at a distance of $\frac{t}{2} - \bar{e}$ from the compressive face, we get

$$\frac{ab}{3} = \frac{t}{2} - \bar{e}$$

and $ab = 3\left(\frac{t}{2} - \bar{e}\right)$

Thus design load, W = average stress $\times$ *ab*

$$= \frac{1.25 \times f_c}{2} \times 3\ (t - \bar{e})$$

From the above equation we can see that theoretically design load W is zero when $\bar{e} = t/2$. However from practical considerations $\bar{e}$ should be limited to $t/3$.

iii)

. It seems necessary to add that in case some tension is likely to develop in masonry because of eccentricity of concentrated loads, the bed blocks should be suitably reinforced and these should be long enough so as to prevent tensile cracks in masonry due to eccentricity of loading.

Permissible Tensile Stress

tensile stress up to 0.1 N/mm² and 0.07 N/mm² in the masonry of boundry/compound walls is permitted when mortar used in masonry is of M1 and M2 grade respectively or better. This relaxation has been made to effect economy in the design of the boundry/compound walls since there is not much risk to life and property in the event of failure of such walls.

Permissible Shear Stress

In 1969 version of the Code, provision for Permissible value of shear stress was 0.15 N/mm² (1.5 kg/cm²) for walls built in mortar not leaner than 1 : 1 : 6 cement : lime : sand mortar.

, value of Permissible shear stress was suitably modified and was related to amount of preloading, subject to a maximum of 0.5 N/mm² and minimum of 0.1 N/mm².

If there is tension in any part of a section of masonry, that part is likely to be cracked and thus cannot be depended upon for resisting any shear force. The clause is based on this consideration. This situation is likely to occur in masonry elements subjected to bending.

Design Thickness/Cross Section

Walls and Columns Subjected to Vertical Load

1 *Solid walls*

Brick work is generally finished by either pointing or plastering and with that in view, it is necessary to rake the joints while the mortar is green, in case of plaster work raking is intended to provide key for bending the plaster with the background.

However in case ot design of masonry based on permissible tensile stress (as for example design of a free standing wall), if walls are plastered over (plaster of normal thickness i.e. 12 to 15 mm) with mortar of same grade as used in masonry or M2 grade—whichever is stronger or are flush pointed with mortar of M1 grade or stronger, raking may be ignored.

Walls and Columns Mainly Subjected to Lateral Loads

.2. *Free standing walls*

i) 1980 version of the Code provided for design of a free-standing wall as a gravity structure that is, without placing reliance on the flexural moment of resistance of the wall due to tensile strength of masonry. It was seen that this approach to design resulted in fairly thick walls and maximum height of an unplastered 23 cm thick wall (one-brick thick of conventional size) could be only about 0.86 m while it has been a common practice since long to build such walls to heights much greater than 0.86 m.

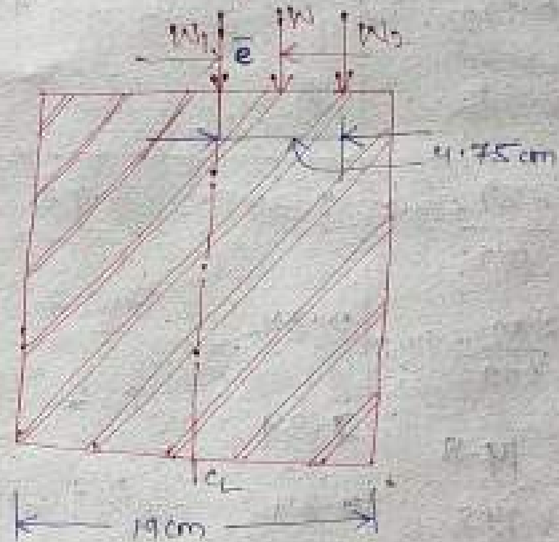
From a study of practices being followed in some other countries in this regard, it is evident that, for design of free-standing walls, it is appropriate to take into consideration flexural moment of resistance of masonry according to the grade of mortar used for the masonry.

ii) Method of working out thickness of free-standing walls by taking advantage of flexural moment of resistance of the wall It would be seen that self-weight of a free standing wall reduces tensile stress in masonry caused by lateral load that is, wind pressure. Thus heavier the masonry units, lesser is the design thickness of wall for a particular height. It is, therefore, advantageous to build compound walls in stone masonry in place of brick masonry when stone is readily available and thickness has to be greater than one brick. Also it should be kept in view that use of light-weight units such as hollow bricks/blocks in free-standing walls has obvious structural disadvantage.

iii) As a general rule, a straight compound wall of uniform thickness is not economical except for low heights or in areas of low wind pressure. Therefore, when either height is appreciable or wind pressure is high, economy in the cost of the wall could be achieved by staggering, zig-zagging or by providing diaphragm walls.

It can be seen that for wind pressure of 750 N/m², maximum height of a 23 cm thick brick wall using grade M1 mortar can be 1.5 m for a straight wall, 3.2 m, for a staggered wall and 4.0 m for a diaphragm wall.

Q.2 A masonry wall, 20cm thick carries an axial load 27 kN/m from the wall above and an eccentric load 16 kN/m from RCC floor acting at a distance 4.75cm from the centre line of the wall. Determine the resultant eccentricity of loading and eccentricity ratio.



$W_1 = 27$ kN/m

$W_2 = 16$ kN/m

$W = W_1 + W_2 = 27 + 16$

$= 43$ kN/m.

Solution :- Let 'W' be the total vertical load and 'e' the resultant eccentricity of all loads.

Taking moments about the centre line of wall.

$W\bar{e} = W_1 \times 0 + W_2 \times 4.75$ (Pg-20, cl-5.4.1.1, SP-20)

$(27+16) \times 10^3 \times \bar{e} = 27 \times 10^3 \times 0 + 16 \times 10^3 \times 4.75$

$\bar{e} = \dfrac{16 \times 10^3 \times 4.75}{43 \times 10^3}$

$\bar{e} = 1.77$ cm

Resultant eccentricity ratio,

$\dfrac{\bar{e}}{t} = \dfrac{1.77}{19} = 0.09$