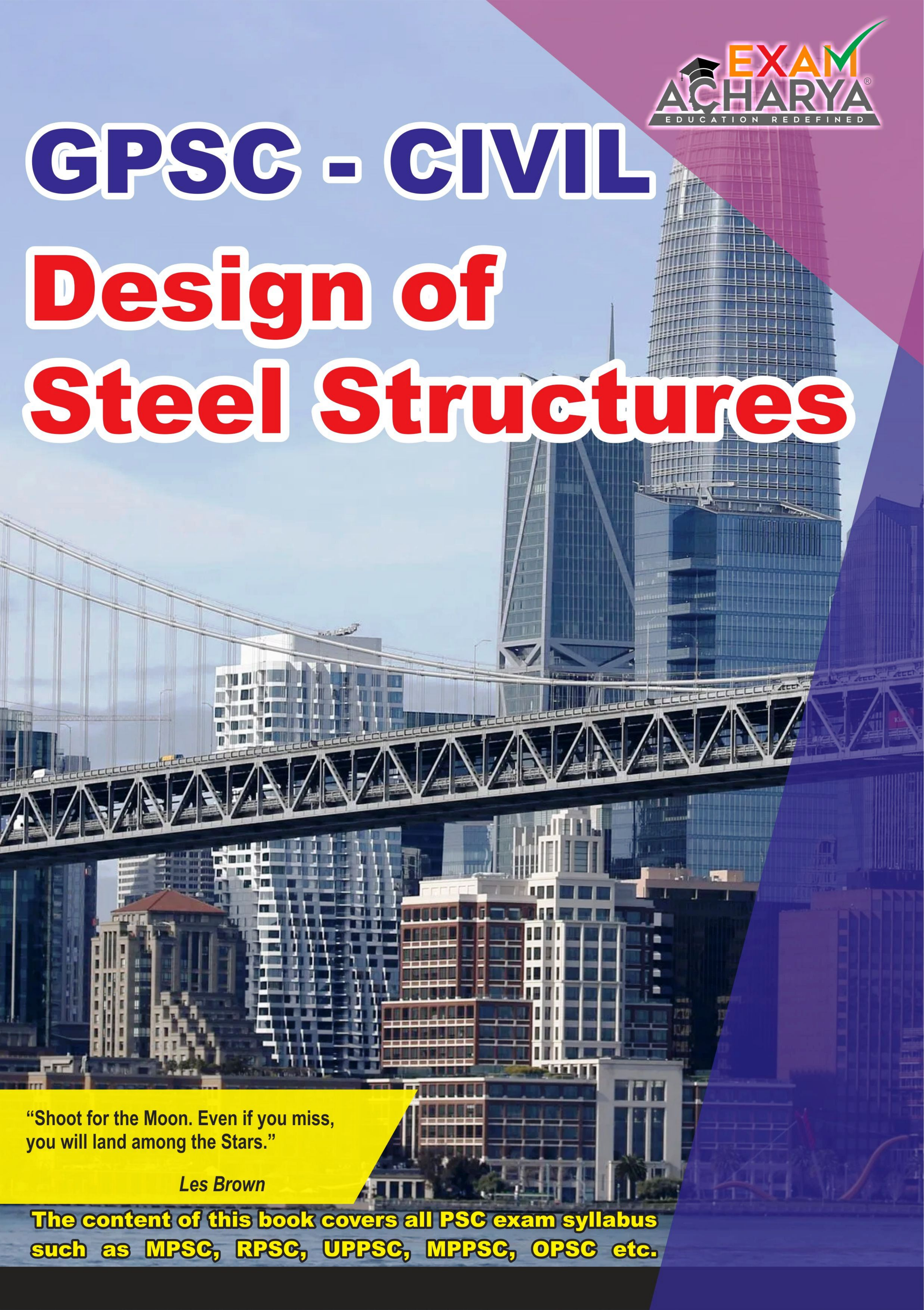
GPSC - CIVIL

Design of Steel Structures



"Shoot for the Moon. Even if you miss, you will land among the Stars."

Les Brown

The content of this book covers all PSC exam syllabus such as MPSC, RPSC, UPPSC, MPPSC, OPSC etc.

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Guidelines for the Aspirants

How to use this book?

- While preparing a subject, first cover all the theoretical topics of each chapter so that you will get a basic idea about particular topic.
- After covering the theoretical portion, solve the questions under “CLEAR YOUR CONCEPT” title.
- After covering the questions under “CLEAR YOUR CONCEPT” move towards the next set of questions under “TEST YOUR SELF” title.
- After finishing the theory and numerical portion of this book for each chapter, solve previous year GPSC questions which is provided in GPSC – CIVIL ENGINEERING book.
- After solving the previous year GPSC questions, for getting best results give the weekly, mid subject and full-length test prepared by Exam Acharya.

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CHAPTER – 1

INTRODUCTION

ADVANTAGES OF STEEL AS A STRUCTURAL MATERIAL

(a) As compared to other structural materials steel has high strength to weight ratio. It implies, steel possess very high strength and results in smaller sections as compared to other structural materials. Thus steel is particularly useful for carrying heavy loads with relatively small sections.

(b) Being light steel can be handled conveniently and thus it offers ease in transportation.

(c) Another important property of steel is that it is ductile. Due to this very useful property it does not fail abruptly but gives ample warning by yielding before actual collapse of the structure.

(d) Steel possess very high strength also due to which it does not undergo fracture due to large deformation and erection stresses.

(e) Steel has a very long life when maintained properly.

(f) Retrofitting of steel structures is quite easy as compared to other materials like RCC, timber, mortar etc.

(g) The resale value of steel is also very high amongst all building materials. Moreover, steel can be reused also.

DISADVANTAGES OF STEEL AS A STRUCTURAL MATERIAL

(a) Steel when placed in exposed conditions is prone to corrosion. Thus steel structures require proper protection measures to be adopted right from its manufacture.

(b) Steel is prone to fire and its strength reduces considerably due to high temperatures and thus steel structures require separate fire-proof treatment which ultimately adds to the cost of the structure.

(c) Steel structures when subjected to cyclic loading (like turbo-generators of power plants etc.) and reversal of stresses undergo fatigue. This fatigue results in reduction of the strength of the steel.

(d) Under certain conditions particularly at locations of stress concentrations, steel may lose its ductility which gets enhanced at low temperatures and under fatigue loading.

(e) In steel construction, the designer is not having too many options as in RCC as for as size of the section is concerned. The designer is compelled to use the available standard rolled sections.

BIS classifies structural steel on the basis of its ultimate strength or the yield strength. The chemical Composition, rolling methods, heat treatment and stress history etc. of steel determines its mechanical properties.

Some of the mechanical properties of steel are:

(a) Modulus of elasticity or Young's modulus (E)	2×10^5 N/mm^2 = 200 KN/mm^2
(b) Poisson's ratio (μ)	Elastic range 0.3
	Plastic range 0.5
(c) Shear modulus (G)	0.77×10^5 N/mm^2 = 200 kN/mm^2
(d) Mass density (ρ)	7850 Kg/m^3
(e) Coefficient of thermal expansion (α)	12×10^{-6} $^oC^{-1}$

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 into required shape. Steel sections of standard shapes, sizes and length are rolled in steel mills and marketed. User has to cut them to the required length and use required sections for the steel framework. Many steel sections are readily available in the market and are in frequent demand such steel sections are known as Regular Steel Sections. Some steel sections are not in use commonly, but the steel mills can roll them if orders are placed. Such steel section are known as Special Sections.

Various types of rolled steel sections manufactured are listed below:

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

(ii) Rolled steel Channel sections

(iii) Rolled steel Angle sections

(iv) Rolled steel Tee sections

(v) Rolled steel Bars

(vi) Rolled steel Tubes

(vii) Rolled steel Plates

(viii) Rolled steel Flats

(ix) Rolled steel Sheets and Strips.

ISI Hand Book for Structural Engineers-I gives nominal dimensions, weight per metre length and geometric properties of various rolled steel sections.

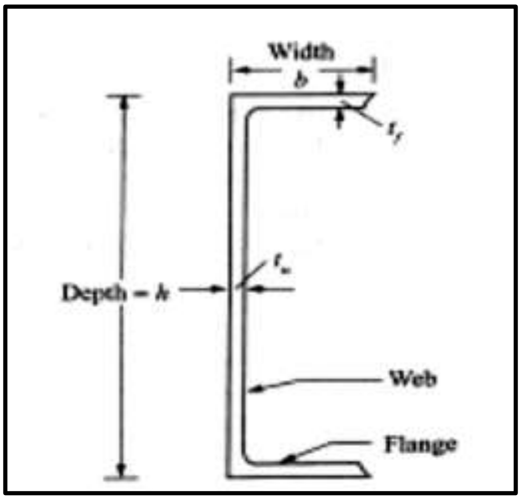
Rolled Steel I-section

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

Figure shows a typical channel section



Rolled steel channel section.

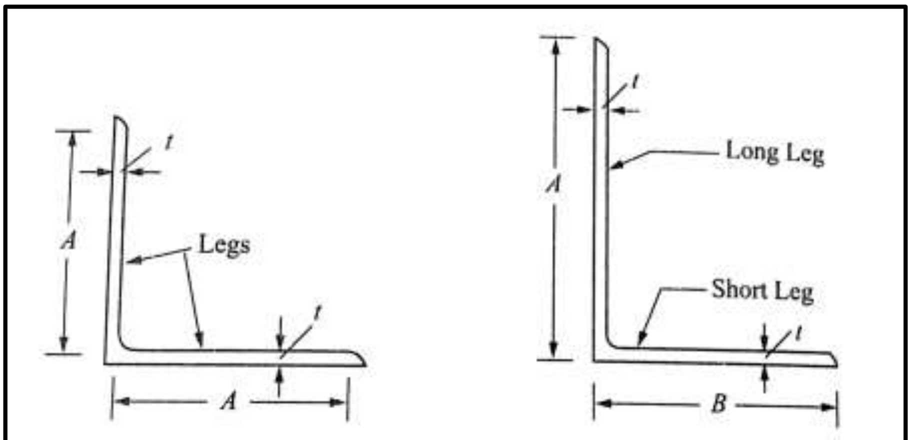
Rolled steel channel sections are designated by the series to which they belong, followed by depth (in mm) and weight (in kN/m). e.g. ISMC 300 @ 0.351 kN/m.

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.

Rolled Steel Bars

Rolled steel bars are classified into the following two series:

(a) Indian Standard Round bars - ISRO

(b) Indian Standard Square bars - ISSQ.

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

ISRO 16

ISSQ 20

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.

LOAD COMBINATIONS

A judicious combination of the loads is necessary to ensure the required safety and economy in the design keeping in view the probability of

(a) Their acting together
(b) Their disposition in relation to other loads and severity of stresses or deformation caused by the combination of various loads.

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Building Material and Construction

Dream is not that which you see while sleeping it is something that does not let you sleep.

A.P.J. Abdul Kalam

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

(i) Working Stress Method (WSM)

(ii) Ultimate Load Design (ULD) and

(iii)Limit State Design (LSD).

Working Stress Method

This is the oldest systematic analytical design method. Though IS: 800-2007 insists for the limit state design, permits use of this method wherever LSD cannot be conveniently adopted.

In this method stress strain relation is considered linear till the yield stress. To take care of uncertainties in the design, permissible stress is kept as a fraction of yield stress, the ratio of yield stress to working stress itself known as factor of safety. The members are sized so as to keep the stresses within the permissible value. Thus

$$\textbf{permissible stress} = \frac{\textbf{yield stress}}{\textbf{factor of safety}}$$

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

Stress due to DL + LL ≤ permissible stress

Stress due to DL + WL ≤ permissible stress

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

The Limitations of WSM

The limitations of working stress method are:

1. It gives the impression that factor of safety times the working load is the failure load, which is not true. Actually it is much more, because a material

Advantages of ULD

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

Disadvantages of ULD

It does not guarantee serviceability performance. To account for this IS: 800-1984 suggested limitations on deflection. However, it did not guarantee other serviceability limits like instability and fatigues etc. Finally, it was felt to suggest more comprehensive method to take care of design requirements from strength and serviceability criteria.

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

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):

e. Brittle fracture.

Limit States of Serviceability are limit states beyond which specified service criteria are no longer met. These include the following:

a. Deformation and deflections, which may adversely affect the appearance or effective use of the structure, or may cause improper functioning of equipment or services, or may cause damages to finishes and non-structural members.
b. Vibrations in the structure or any of its components causing discomfort to people, damages to the structure, or its contents which may limit its functional effectiveness. Special consideration should be given to floor systems susceptible to vibration, such as large open-floor areas free of partitions to ensure that such vibrations are acceptable for the intended use and occupancy.
c. Repairable damage due to fatigue (due to wind-induced oscillations).
d. Corrosion and durability.
e. Ponding of structures.

Partial Safety Factors for Loads, γ_f

Combination	**DL**	**Limit State of Strength LL**		**WL/EL**	**AL**	**DL**	**Limit State of Serviceability LL**		**WL/EL**
		Leading	**Accompanying (CL, SL etc.)**				Leading	Accompanying (CL etc.)	
DL+LL+CL	1.5	1.5	1.05	-	-	1.0	1.0	1.0	-
DL+LL+CL	1.2	1.2	1.05	0.6					
WL/EL	1.2	1.2	0.53	1.2	-	1.0	0.8	0.8	0.8
DL+WL/EL	1.5	-	-	1.5	-	1.0	-	-	1.0
	(0.9)*								
DL+ER	1.2	1.2	-	-	-	-	-	-	-
	(0.9)								
DL+LL+AL	1.0	0.35	0.35	-	1.0	-	-	-	-

* This value is to be considered when stability against overturning or stress reversal is critical

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Factored dead load = 1.5 × 48 = 72 kN

Factored live load = 1.5 × 24 = 36 kN

Factored design load = 72 + 36 = 108 kN

Example: The loads on a floor beam of a commercial building are as below.

Roof loads:

Dead load = 6 kN/m^2

Live load = 4 kN/m^2

Roof finish = 1.5 kN/m^2

Determine the design load for:

(a) Limit state of strength.

(b) Limit state of serviceability.

Solution: For Limit state of strength

Partial safety factors for:

Dead load = 1.5

Live load = 1.5

Factored dead load = 1.5 × (6 + 1.5)

= 11.25 kN/m^2

Factored live load = 1.5 × 4 = 6 kN/m^2

The design factored load = 6 + 11.25

= 17.25 kN/m^2

For Limit state of serviceability:

Partial safety factors for:

Dead load = 1.0

Live load = 1.0

CLEAR YOUR CONCEPT

Qu1 Elastic Modulus of Steel is __________

a) 1.5×10^9 N/mm^2

b) 2.0×10^5 N/mm^2

c) 2.0×10^5 N/m^2

d) 1.5×10^9 N/m^2

Qu2 Unit mass of Steel = ________

a) 785 kg/m^3

b) 450 kg/m^3

c) 450 kg/cm^3

d) 7850 kg/m^3

Qu3 Which of the following is a disadvantage of Steel?

a) High strength per unit mass

b) High durability

c) Fire and corrosion resistance

d) Reusable

Qu4 Poisson's ratio of steel is ________

a) 0.1

b) 1.0

c) 0.3

d) 2.0

CHAPTER – 2

RIVETED, BOLTED AND PINNED CONNECTIONS

CONNECTIONS

- Connections are the weakest point of failure in a structure and thus need to be property analyzed and designed.
- The various types of fasteners available for making connections are rivets, bolts, pins and the welds. Bolting has become so much popular that high strength bolts has almost replaced rivets now.

RIVETED CONNECTIONS

Although riveted construction is obsolete but an understanding of its behaviour and design is essential for the strength evaluation and rehabilitation of older structures. Just for the sake of making the reader familiar with riveted connection design a brief description about rivets and their patterns are presented. Since the analysis and design of riveted connections is same as that for ordinary bolts, the design and details may be done similar to ordinary bolts.

A rivet is made up of a round ductile steel bar piece (mild or high tensile) called shank with a head at one end. The head can be of different shapes as shown in -Figs. The usual form of rivet head employed in structural steel construction is the snap head.

- Rivets when heated before driving are called as hot driven field rivets (when placed in field) or hot driven shop rivets (when placed in workshop)
- The diameter of hot rivet is equal to the rivet hole diameter and is called as gross diameter.
- Hot rivet is plastic, expands and fills the hole completely while forming head at the other end of the rivet shank. But on cooling the rivet Shrinks both in diameter and length. Due to this shrinkage in rivet length, the connected parts get stressed resulting in residual tension of unknown amount in the shank and some compression in the plates to be connected.
- Cold driven rivets i.e. rivets driven at room temperature require high pressure for head formation at room temperature and thus its use is limited.

Note

➢ The strength of cold driven rivet is more than hot driven rivet but their clamping force is less as the cold driven rivets do not shrink like hot driven rivets. Rivets heads for small diameter rivets can be formed manually with an ordinary harmer and are referred to as hand driven rivets

Material of The Rivet

As per Cl. 2.3.2 IS: 800-2007, rivets should conform to IS:1929-1982 and IS 2155:1982.

CI. 2.3.3 of IS:800-2007 states that high tensile steel rivets should be made from steel conforming to IS:1149-1982.

Note

➢ As per CI. 17.4.4 of IS:800-2007, all loose, burnt and defective rivets must be Cut out and get replaced well before the structure is loaded.

PATTERNS OF RIVETED JOINTS

The rivets may be placed in a variety of patterns, depending upon the space available for connection and the shape of members to be connected. The most common types of rivet patterns are chain riveting (Fig. (a)) and diamond riveting (Fig. (b)). Staggered patterns can also be provided as shown in (Figs. (c, d)). Staggered chain riveting (Fig.

1. The erection of the structure can be speeded up.
2. Less skilled persons are required.
3. The overall cost of bolted construction is cheaper than that of riveted construction because of reduced labour and equipment costs and the smaller number of bolts required resisting the same load.

The general objections to the use of bolts are the following:

1. Cost of material is high, about double that of rivets.
2. The tensile strength of the bolt is reduced because of area reduction at the root of the thread and also due to stress concentration.
3. Normally, these are of a loose fit excepting turned bolts and hence their strength is reduced.
4. When subjected to vibrations or shocks, bolts may get loose.

The holes made for placing the bolts in the joints may either be drilled or punched. Fabricators prefer punched holes because punching is simple, time saving and cost effective but ductility and toughness is reduced causing brittle fracture.

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Classification of Bolted Connections

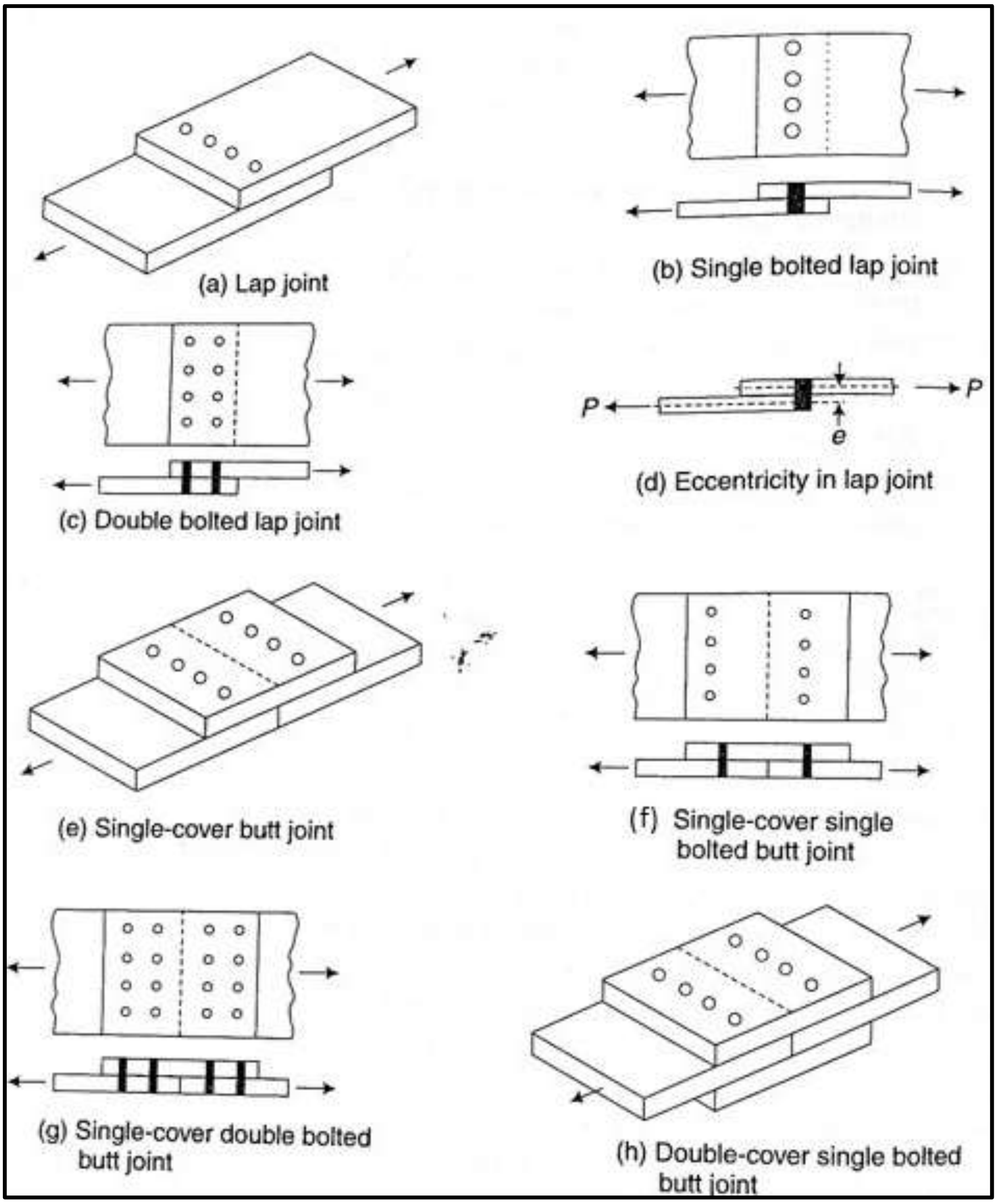
(a) Classification based on line of action of resultant force transferred

(i) Concentric connection: Here the load line passes through the CG of the section. e.g. Axially loaded tension or compression member.

(ii) Eccentric connection: Here the load line is away from the CG of the connection. e.g. Bracket connection. moment resisting connection, seat connection etc.

(b) Classification based on the type of force

(i) Tension connection: Here the load gets transferred through tension on bolts. e.g. Hanger connection provided on one side as in Figs. (e, f and g), it is called a single cover butt joint but if the cover plates are provided on both sides of the main plates, it is called a double cover butt joint as shown in Figs. (h, i and j). Figures (k and l) show transfer of forces in a lap joint and double-cover butt joint. It is more desirable to provide a butt joint rather than a lap joint for snug-tight bolts for two main reasons:



(a) Lap joint
(b) Single bolted lap joint
(c) Double bolted lap joint
(d) Eccentricity in lap joint
(e) Single-cover butt joint
(f) Single-cover single bolted butt joint
(g) Single-cover double bolted butt joint
(h) Double-cover single bolted butt joint

New Batches are going to start.....

Contact:

7622050066

201, Siddhi Vinayak Complex, besides Bank of India, Near Panchratna furniture,
Ellorapark, Subhanpura, Vadodara – 390023
Contact: 7622050066 Website: www.acumenhr.in

Test Series Available..

Total weekly test	: 35
Total mid subject test	: 16
Total full length test	: 13
Mock test	: 16
Total test	: 80

Shear Failure of Bolts

Shear stresses are generated when the plates slip due to applied forces. The maximum factored shear force in the bolt may exceed the nominal shear capacity of the bolt. Shear failure of the bolt takes place at the bolt shear plane (interface). The bolt may fail in single shear or double shear as shown in Fig.(a).

Bearing Failure of Bolts

The bolt is crushed around half circumference. The plate may be strong in bearing and the heaviest stressed plate may press the bolt shank (Fig. (b)). Bearing failure of bolts generally does not occur in practice except when plates are made of high strength steel.

Bearing Failure of Plates

When an ordinary bolt is subjected to shear forces, the slip takes place and bolt comes in contact with the plates. The plate may get crushed, if the plate material is weaker than the bolt material as shown in Fig. (b). The bearing problem can be complicated by the presence of a nearby bolt or the proximity of an edge in the direction of the load. The bolt spacing and end-distance will influence the bearing strength. A possible failure mode resulting from excessive bearing is shear tear-out at the end of connected member as shown in Fig. (c).

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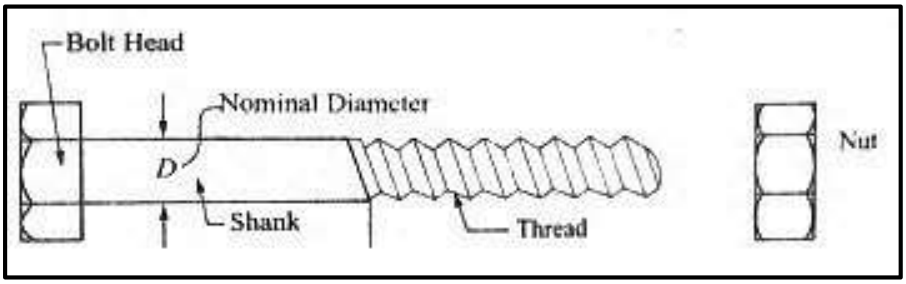
Tension Failure of Bolts

Bolts subjected to tension may fail at the stress area. In case if any of the connecting plates is sufficiently flexible additional prying forces induced in the bolts must also be considered.

Tension or Tearing Failure of Plates

Tearing failure occurs when the bolts are stronger than the plates. Tension on both the gross area and net effective area must be considered. The tension failure of a plate is shown in Fig. (d).



Bolt and nut

Unfinished / Black Bolts

These bolts are made from mild steel rods with square or hexagonal head. The shank is left unfinished i.e. rough as rolled. Though the black bolts of nominal diameter (diameter of shank) of sizes 12, 16, 20, 22, 24, 27. 30 and 36 mm are available, commonly used bolt diameters arc 16, 20, 24, 30 and 36 mm. These bolts are designated as M16, M20, M24, etc. IS 1364 (part 1) gives specifications for such bolts. In structural elements to be connected holes are made larger than nominal diameter of bolts. As shanks of black bolts are unfinished, the bolt may not establish contact with structural member at entire zone of contact surface. joints remain quite loose resulting into large deflections. The yield strength of commonly used black bolts is 240 N/mm^2 and ultimate strength 400 N/mm^2. These bolts are used for light structures under static loads such as trusses, bracings and also for temporary connections required during erections.

Finished / Turned Bolts

These bolts are also made from mild steel, but they are formed from hexagonal rods, which are finished by turning to a circular shape. Actual dimension of these bolts are kept 1.2 mm to 1.3 mm larger than the nominal diameter. As usual the bolt hole is kept 1.5 mm larger than the nominal diameter. Hence tolerance available for fitting is quite small. It needs special methods to align bolt holes before bolting. As connection is more tight, it results into much better bearing contact between the bolts and holes. These bolts are used in special jobs like connecting machine parts subjected to dynamic loadings. IS: 3640 covers specifications for such bolts.

5. Smaller number of bolts result into smaller sizes of gusset plates.

Disadvantages of HSFG Bolts

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

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

ADVANTAGES AND DISADVANTAGES OF BOLTED CONNECTIONS

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

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

The disadvantages of unfinished (black) bolt connections are listed here. However it may be noted that most of these disadvantages are overcome by using HSFG bolts.

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.

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(i) To prevent bearing failure of members between the two bolts.

(ii) To ease in installation of bolts i.e. sufficient space must be ensured to tighten the bolts, prevent overlapping of the washers and provide adequate resistance to tear-out of the bolts.

- The center to center distance between the holes should not be less than 2.5 times the nominal diameter of bolt. When bolts are placed at a distance lesser than this then very little clearance is left between the bolts and installation of bolts become difficult.

(b) Maximum pitch

- Maximum pitch is ensured for the following reasons:

(i) To reduce the length of joint and of gusset plate.

(ii) To have uniform stress in the bolts. It is assumed that load on the joint is equally distributed among all the bolts. In case of short length joints, a redistribution of forces in the bolts occurs due to plastic action and thus the bolts will share the load equally. However, this is true when there are only a few bolts in a line.

- In case of long joints (> 15 times the bolt diameter), the shear stress distribution is not uniform and bolts at the ends are stressed more as shown in Fig.

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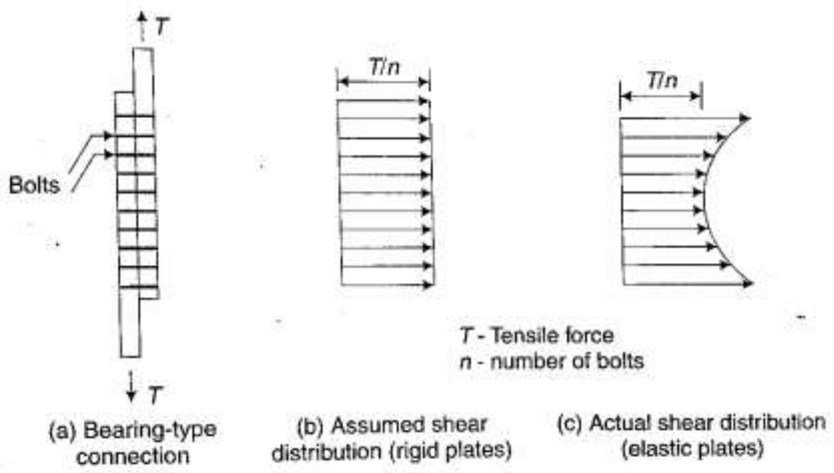

Shear distribution in a long bolted connection

(a) If value of gauge length exceeds after providing design fasteners at maximum edge distances tacking rivets should be provided

(i) At 32 t or 300 mm, whichever is less, if plates are not exposed to weather

(ii) At 16 t or 200 mm, whichever is less, if plates are exposed to weather.

In case of a member made up of two flats, or angles or tees or channels, tacking rivets are to be provided along the length to connect its components as specified below:

(a) Not exceeding 1000 mm, if it is tension member

(b) Not exceeding 600 mm, if it is compression member

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.

ASSUMPTIONS IN DESIGN OF BEARING BOLTS

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

1. The friction between the plates is negligible
2. The shear is uniform over the cross-section of the bolt
3. The distribution of stress on the plates between the bolt holes is uniform
4. Bolts in a group subjected to direct loads share the load equally
5. Bending stresses developed in the bolts is neglected.

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

SHEARING STRENGTH OF BOLTS

The bolt shank shears along the plane of the slip. i.e. the interface. The number of planes along which the bolts can be sheared indicates the number of shears. i.e. single or double shear. The resistance of a bolt to shear is called the nominal capacity of a bolt in shear and is denoted by V_{nsb}. It depends upon the ultimate tensile strength of bolt f_{ub} the number of shear Planes n, the nominal shank area A_{sb} and the net tensile stress area A_{nb}, of the holt in each shear plane. The nominal shear capacity of the bolt is given by

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb}) \qquad (1)$$

The nominal shear capacity of a bolt for long joints will be lesser and is modified and expressed as

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}} (n_n A_{nb} + n_s A_{sb})\, \beta_{lj}\, \beta_{lg}\, \beta_{pkg} \qquad (2)$$

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or $$V_{sb} = 184.75\,(n_n A_{nb} + n_s A_{sb})\beta_{lj}\,\beta_{lg}\,\beta_{pkg} \quad (5)$$

$$= 184.75\, A_e\, \beta_{lj}\,\beta_{lg}\,\beta_{pkg} \quad \text{(for bolts in single shear)}$$

and

$$= 2 \times 184.75 \times A_e\, \beta_{lj}\,\beta_{lg}\,\beta_{pkg} \quad \text{(for bolts in double shear)}$$

where A_e = effective area

Notes

1. Since threads of the bolts can occur in the shear plane, the effective area A_e for resisting shear should normally be taken as the net tensile stress area A_{nb} of the bolts. For bolts where net tensile stress area is not defined, A_{nb} should be taken as the area at the root of the threads. However, if the threads do not occur in shear plane A_e may be taken as the cross-sectional area of the shank A_{sb}.
2. If normal bolts and member sizes are used, the threads will always be excluded from the shear planes. Assuming threads to be in the shear plane will be extremely conservative.
3. The shearing strength of a bolt can be maximum in double shear.

The reduction factors β_{lj} and β_{lg} and β_{pkg} are described as follows and are to be considered depending upon the case.

REDUCTION FACTOR FOR LONG JOINTS (β_{lj})

When the length of the joint exceeds 15d (d is the diameter of bolt) in the direction of load the shear capacity of the joint is reduced. The reduction factor is given by

$$\beta_{lj} = 1.075 - \frac{l_j}{200d} \text{ for } 0.75 \leq \beta_{lj} \leq 1.0 \quad (6)$$

Where l_j is the length of the joint and is taken as the distance between the first and last row of bolts in a joint measured in the direction of the load transfer.

Note:

- The reduction factor is taken as 1.0 for joints having uniform shear over entire length. for example connection of web and flange of a plate girder.

REDUCTION FACTOR FOR LARGE GRIP LENGTHS (β_{lg})

When the grip length of a bolt increases, the bolt is subjected to a greater bending moment due to shear forces acting on its shank. Therefore, if the total thickness of shear tear out at the end of a connected element. This tear-out can take place at the edge of a connected part or between two holes in the direction of the bearing load. To prevent excessive elongation of hole an upper limit is placed on the nominal bearing strength of the bolt. This upper limit is proportional to the projected bearing area times the ultimate tensile stress.

The nominal bearing strength of the bolt is given by

$$\mathbf{V_{npb} = 2.5k_b d t f_u} \qquad (9)$$

Where,

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

d_0 = Diameter of the hole

e, p = End and pitch distances of the fastener along bearing direction

f_{ub} = Ultimate tensile stress of the bolt

f_u = Ultimate tensile stress of the plate in Mpa

d = Nominal diameter of the bolt in mm, and

t = Aggregate thickness of the connected plates experiencing bearing stress in the same direction. In case the bolts are countersunk, then it should be taken as the thickness of the plate minus half of the depth of countersunking.

For the safety of the joint in bearing, the bearing strength of the bolt

$$\mathbf{V_{pb} \leq \frac{V_{npb}}{\gamma_{mb}}} \qquad (10)$$

Where, γ_{mb} = the partial safety factor for material of bolt

= 1.25

$$\mathbf{V_{pb} = 2.5k_b d t \frac{f_u}{\gamma_{mb}}} \qquad (11)$$

Net area $A_n = (B - nd_h)t$ for chain bolting (Fig. (a)) (15)

$$A_n = \left[B - nd_h + \sum_{i=1}^{m} \frac{p_{si}^2}{4g_i}\right]t$$ for staggered bolting (Fig. (b)) (16)

Notes	
1.	The details of staggered pitch and gauge are shown in Fig. (b, c)
2.	For more details reference may be made to Section 6.3.

The tensile strength of the plate is given by

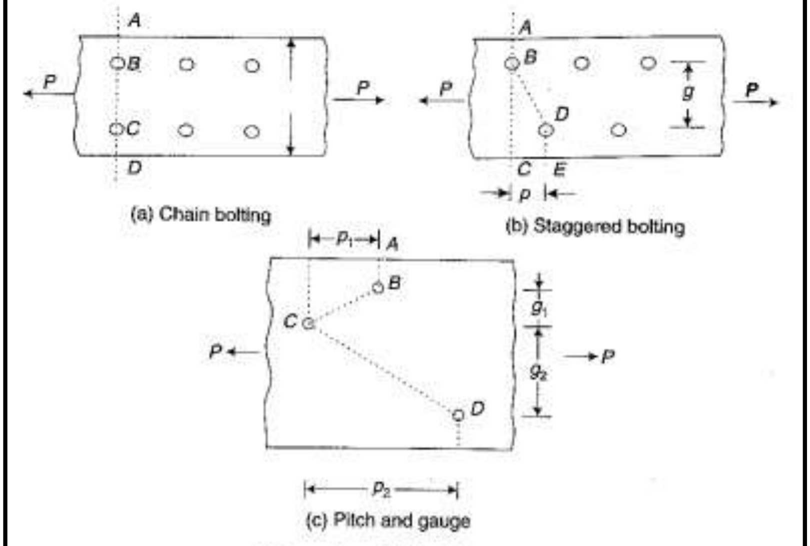
$$T_{nd} = 0.9\, A_n \frac{f_u}{\gamma_{m1}}$$

Where,

f_u = the ultimate stress of material in MPa

A_n = the net effective area in mm^2

γ_{m1} = partial safety factor = 1.25



Net-section of bolted plate

STRENGTH AND EFFICIENCY OF THE JOINT

Strength of a bolted joint is the minimum strength based on strength of bolts, in shear and bearing, in the joint and the strength of the main member at the net section. Efficiency of a bolted joint (η) also called the percentage strength of the joint is ratio of

Shear Strength of HSFG Bolts

As discussed above, the resistance to slip will be function of the product of the coefficient of static friction and the normal force between the connected parts. The relationship is reflected in the provisions of IS: 800-2007. The design slip resistance or nominal shear capacity of a bolt is given by

$$\mathbf{V_{nsf} = \mu_f n_e K_h F_o} \qquad (19)$$

The shear strength of the bolt, $V_{df} = V_{nsf} / \gamma_{mf}$

Where

μ_f = Slip factor

n_e = Number of interfaces offering frictional resistance to slip

K_h = 1.0 for fasteners in clearance holes

= 0.85 for fasteners in oversized and short slotted holes and for fasteners in long slotted holes loaded perpendicular to slot

= 0.7 for fasteners in long slotted holes loaded parallel to slot

F_o = Minimum bolt tension (proof load) at installation

= $A_{nb}\ f_o$

A_{nb} = Net area of bolt at the threads

f_o = Proof stress = 0. 7 f_{ub}

f_{ub} = Ultimate tensile stress of bolt

For the joint to be safe the factored design shear force,

$$\mathbf{V_f \leq \frac{V_{nsf}}{\gamma_{mf}}} \qquad (20)$$

Tensile Strength of HSFG Bolts

The nominal tensile strength of HSFG bolts subjected to a factored tensile force is determined in a way similar to that of the black bolts.

$$T_{nf} = 0.9\, f_{ub}\, A_{nb} \leq f_{yb}\, A_{sb}\, \frac{\gamma_{m1}}{\gamma_{m0}} \qquad (21)$$

The factored design tensile force, $T_f \leq \frac{T_{nf}}{\gamma_{mf}}$ (22)

The tensile strength of bolt, $T_{df} = T_{nf} / \gamma_{mf}$

Where,

γ_{m1} = Partial safety factor for material resistance governed by ultimate stress

= 1.15

γ_{m0} = Partial safety factor for material resistance governed by yield = 1.10

γ_{mf} = Partial safety factor for the material of bolts = 1.25

A_{nb} = Net tensile stress area

A_{sb} = Shank area of the bolt

f_{ub} = Ultimate tensile stress of bolt

PRYING FORCES

In the design of HSFG bolts subjected to tensile forces, an additional force, called as prying force Q is to be considered. These additional forces are mainly due to flexibility of connected plates. Consider the connection of a T-section to a plate as shown in Fig., subject to tensile force $2T_e$.

f_0 = Proof stress in consistent units

t = Thickness of end plate.

Example

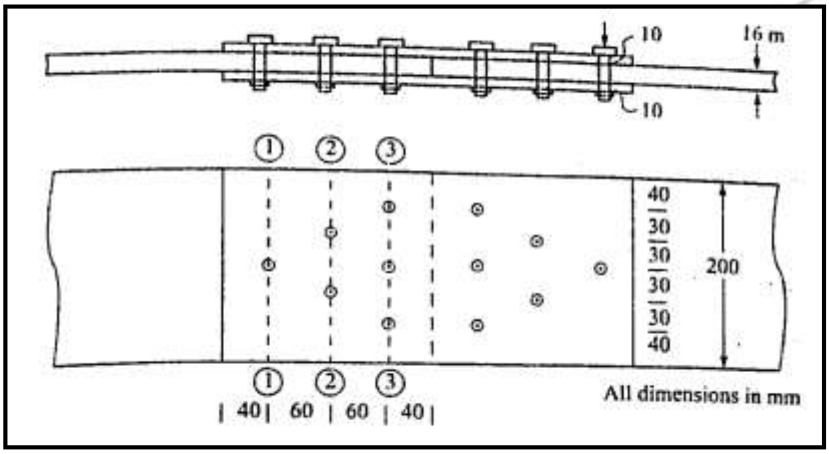
Find the maximum force which can be transferred through the double covered butt joint shown in Fig. Find the efficiency of the joint also. Given M20 bolts of grade 4.6 and Fe 410 steel plates are used.

Solution:

For M20 bolts of Grade 4.16,

d = 20 mm d_0 = 22 mm f_{ub} = 400 N/mm^2.

For grade Fe 410 plates, f_{ub} = 410 N/mm^2.



∴ Nominal strength of one bolt in shear (double shear)

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

$$= \frac{400}{\sqrt{3}}(1.78) \times \frac{\pi}{4} 20^2$$

$$= 129143 \text{ N}$$

∴ Design strength of one bolt in double shear

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

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

$$f_u = 410 \text{ N/mm}^2$$

(a) At section (1) - (1)

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

$$= 840730 \text{ N}$$

(b) At section (2) - (2)

When this section fails, bolt in section (1) - (1) also has to fail. Hence strength of plate at section (2) - (2)

$$T_{dn_2} = \frac{0.9 \times 410\,(200-2 \times 22) \times 16}{1.25} + 103314$$

$$= 840133 \text{ N}$$

At section (3) - (3)

T_{dn_3} = Plate strength + strength of 3 bolts

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

$$= 942851 \text{ N}$$

∴ Strength of plate in the joint = 840133 N

= 840.133 kN

∴ Strength of joint = 619.886 kN

∴ Maximum design force that can be transferred safely = 645.715 kN.

∴ Permissible force at working condition = $\frac{645.715}{1.5}$ = 430.477 kN Answer

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

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= 116228 N

Design shear strength of one bolt in shear

$$= \frac{116228}{1.25} = 92982.6 \text{ N}$$

∴ Design shear strength of 6 bolts in the joint

= 6 × 92982.6 = 557896 N

= 557.896 kN

Strength of bolts in bearing:

K_b is the minimum of

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

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

∴ $K_b = 0.6061$.

∴ Nominal strength of one bolt in bearing = 2.5 K_b dt f_u

= 2.5 × 0.6061 × 20 × 10 × 410

= 124250.5 N

Note ➢ Thickness of thinner plate t = l0 mm

∴ Design strength of a bolt = $\frac{124250.5}{1.25}$ = 99400 N

Design strength of 6 bolts = 6 × 99400

= 596400 N.

= 596.4 kN > 557.896 kN

To avoid failure of cover plates, the total thickness of cover plates should be more than the thickness of main plates. Provide cover plates of 8 mm thicknesses.

Design strength of plate per pitch width

$$= \frac{0.9 \times 410\ (p-18)\times 12}{1.25}$$

$$= 3542.4\ (p - 18) \qquad \text{(b)}$$

Equating (a) to (b) to get maximum efficiency, we get,

$$3542.5\ (p — 18) = 66121$$

∴ p = 36.67 mm.

Minimum pitch = 2.5 × 16 = 40 mm.

∴ Provide bolts at p = 40 mm.

Check for strength of bolt in bearing:

K_b is the minimum of $\frac{e}{3d_0}, \frac{p}{3d_0} - 0.25, \frac{f_{ub}}{f_u}, 1.0$

Assuming sufficient 'e' will be provided

$$K_b = 0.4907$$

∴ Design strength of bolt in bearing

$$= \frac{2.5\times 0.4907\times 16\times 12\times 400}{1.25} = 75372\text{ N} > 66121\text{N}$$

Hence, the assumption that bearing strength is more than design shear is correct.

Since pitch provided is slightly more than required from strength consideration of the plate, the strength of plate is more than the strength of the bolt.

∴ Design strength of joint per 40 mm width = 66121 N.

CLEAR YOUR CONCEPT

Qu1 What is riveting?

a) The process of making holes in the structure

b) Process of making mould for structure

c) Method of joining together pieces of metal by inserting ductile metal pins called rivets

d) Method of joining together pieces of metal by inserting non ductile metal pins called rivets

Qu2 Size of rivet hole is _________ size of rivet

a) More than

b) Less than

c) Equal to

d) Not compared with

Qu3 What is the yield strength of bolt of class 4.6?

a) 400 N/mm^2

b) 240 N/mm^2

c) 250 N/mm^2

d) 500 N/mm^2

Qu4 High strength bolt is used for ____________

a) Shear connection

b) Slip resistant connection only

c) Bearing type connection only

d) Both slip resistant and bearing type connection

CHAPTER – 3

WELDED CONNECTION

INTRODUCTION

- In welded connections, the two structural members are joined by a weld.
- It is the compactness and greater rigidity of the welded joints that offer design assumptions to be realized more precisely in practice.

Advantages of Welded Joints Over Other Joints

(a) Welded joints offer more efficient use of materials and it is due to welding only that we are able to have one piece construction.

(b) Welding helps in speeding up the erection and construction process thereby compressing the production schedules.

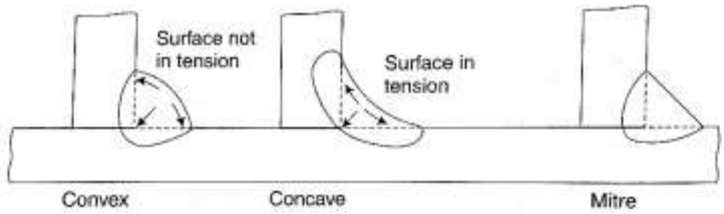
(c) Welding offers light weight construction and thus cuts costs of construction. Connecting steel plates are reduced or eliminated thereby reducing the self-weight of the structure.

(d) In welding no deductions for holes are made and thus whole of the gross section is effective in carrying the load.

(e) Welded joints perform better in case of fatigue loads, impact loads and vibrations.

(f) Welding offers complete freedom to architects and the engineers for their designs.

Slot and plug welds are used to supplement the fillet welds, when required length of fillet weld cannot be provided. Since, the penetration of these welds cannot be ascertained and since these are difficult to inspect, slot and plug are avoided.

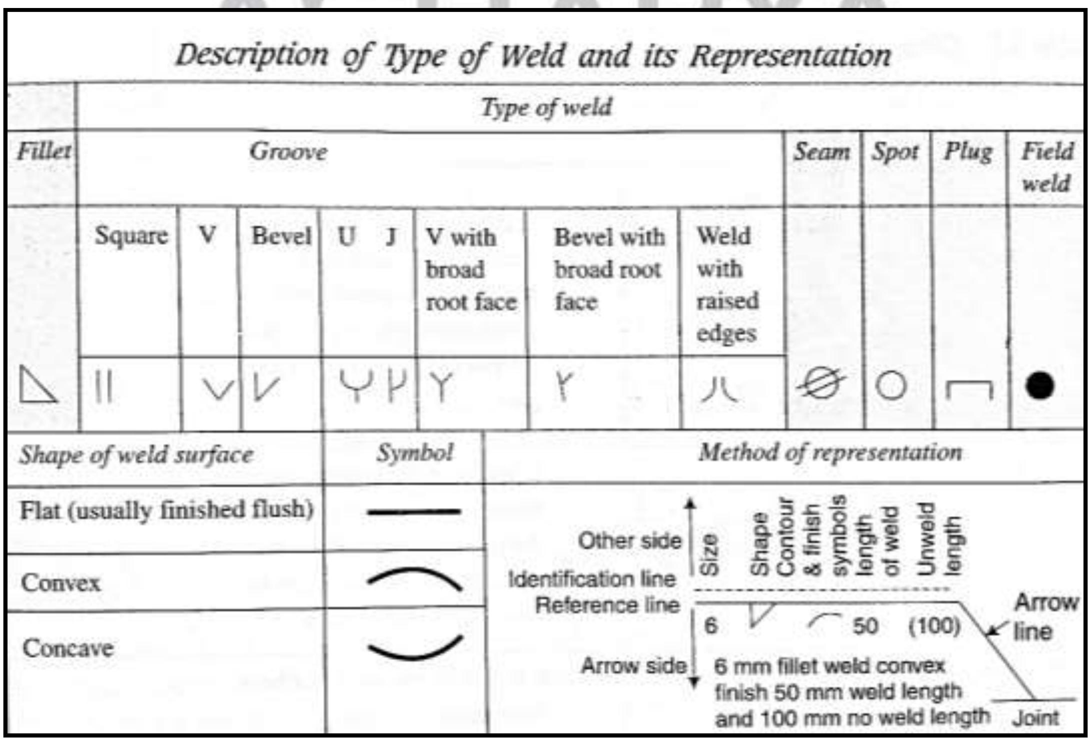


Types of fillet weld

SYMBOLS

A knowledge of welding symbols is essential for a site engineer to be able to read the drawings. Symbols save a lot of space as descriptive notes can be omitted.

Table depicts the symbols and the method of their representation on the drawings. The drawings need to indicate the side of welding, size, contour and finish, spacing and whether it is field or shop weld.



Description of Type of Weld and its Representation

Type of weld											
Fillet	Groove							Seam	Spot	Plug	Field weld
	Square	V	Bevel	U J	V with broad root face	Bevel with broad root face	Weld with raised edges				

Shape of weld surface	Symbol	Method of representation
Flat (usually finished flush)		
Convex		
Concave		

WELD DEFECTS

Good welding techniques, standard electrodes and proper joint preparation are the basic tools to achieve a sound weld. However, defects are inevitable and a knowledge of these is essential to minimize them. Some of the common defects in the welds are discussed below and are shown in Fig.

Incomplete Fusion is the failure of the base metal to get completely fused with the weld metal. It is caused by rapid welding and also because of foreign materials on the surfaces to be welded.

Incomplete Penetration is the failure of the weld metal to penetrate the complete depth of joint. It is normally found with single vee and bevel joints and also because of large size electrodes.

Porosity occurs due to voids or gas pockets entrapped in the weld while cooling. It results in stress concentration and reduced ductility of the metal. Normally porosity is not a problem because each void is spherical and not a notch. Even with a slight loss in the section because of the voids, their spherical shape may be considered to allow a smooth flow of stress around the void without any measurable loss in strength. Mainly these are caused because of careless use of back-up plates, presence of moisture in the electrodes, hydrogen in the steel and excessive current.

Slag Inclusions are metallic oxides and other solid compounds which are sometimes found as elongated or globular inclusions. Being lighter than the molten material these float and rise to the weld surface from where these are removed after cooling of the weld. However, excessive rapid cooling of the weld may cause them to be trapped inside the weld. These present a problem in vertical and overhead welding.

Cracks are divided as hot and cold. Hot cracks occur due to the presence of sulphur, carbon, silicon and hydrogen in the weld metal. Phosphorus and hydrogen trapped in the hollow spaces of the metal structure give rise to the formation of cold cracks. Preheating of the metal to be welded eliminates the formation of cracks.

Dye Penetration Method

The depth of a crack can be estimated by this method. A dye is applied over the weld surface and then the surplus is removed. A dye absorber is placed over the weld which oozes the dye giving an idea of the depth of the crack.

Ultrasonic Method

In this method, ultrasonic sound waves are sent through the weld. Defects like flaws, blow holes, etc., affect the time interval of sound transmission identifying the defect.

Radiography

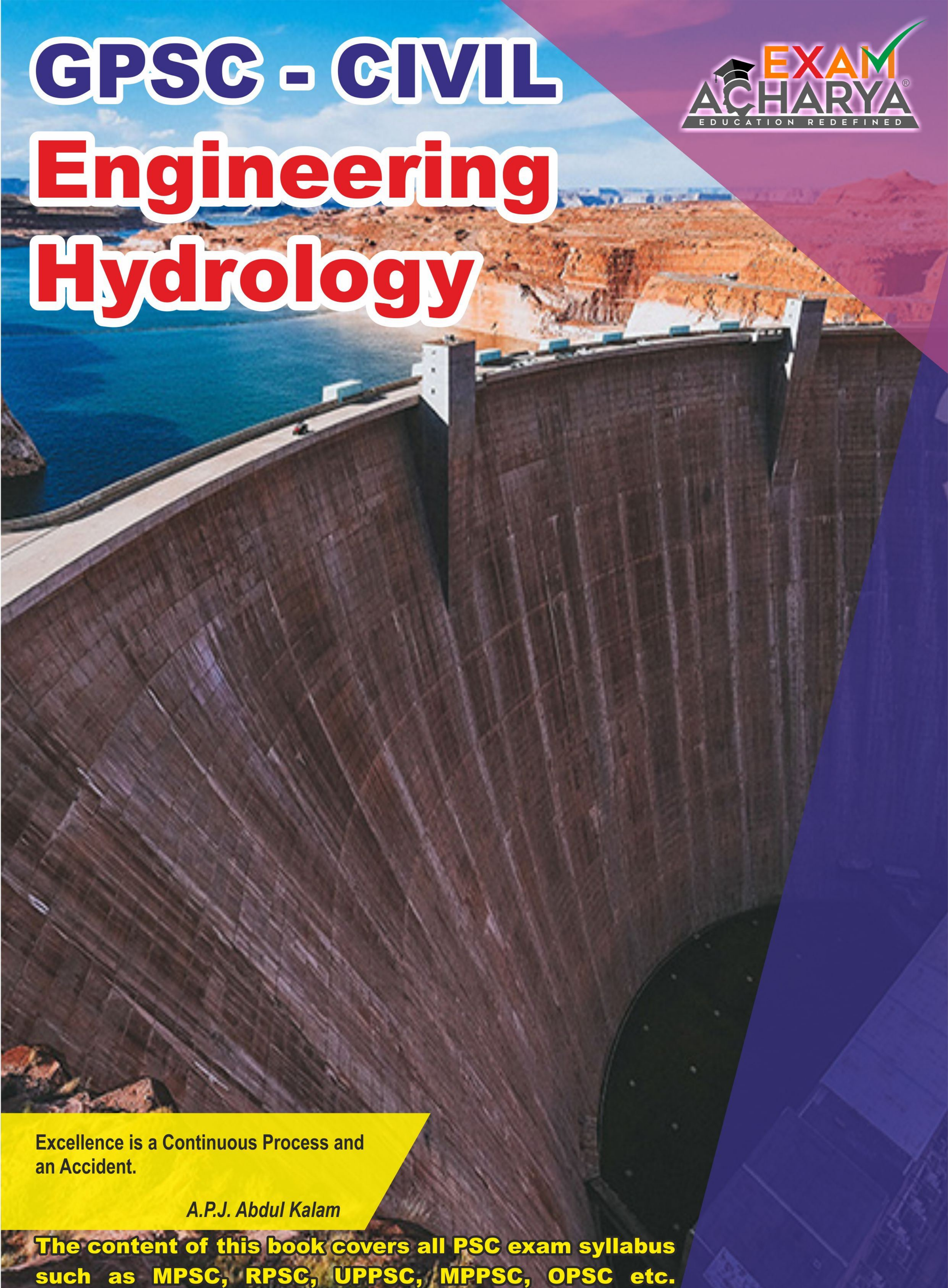
X-rays or gamma rays are used to locate defects. This method is used in groove welds only. It cannot be used in fillet welds because the parent material will also form part of the projected picture.

FILLET WELD

- Fillet weld is provided when two metal surfaces to be joined are in different planes. Fillet welds are more common that butt welds.
- Fillet welds are easy to make, require less material preparation and are easier to fit than the butt welds.
- However, for a given amount of weld material, they are not strong and cause greater concentration of stress.
- In lightly loaded structures where stiffness rather than strength is the governing design factor and fatigue or brittle fracture is not a problem, there fillet welds are more economical.

BUTT WELD

- Butt welds are better in highly stressed structures where smooth flow of stress is a necessity. If butt joint has the same characteristics as that of the parent metal, is flushed smooth on both sides with the parent metals and has complete



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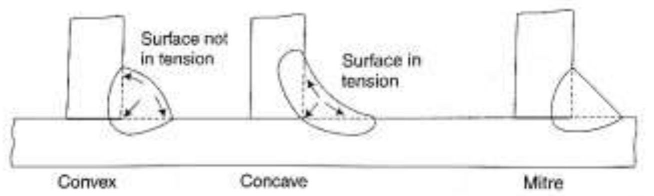
(b) Effective Area: The effective area of a butt weld is the product of effective throat thickness and the effective length of the butt weld.

(c) Reinforcement: It is difficult for a welder to make a weld flush with the parent metal and thus extra metal gets deposited.

- This extra weld metal makes the throat thickness at least 10% greater than the thickness of the welded material.
- Reinforcement increases the efficiency of the butt weld and it also ensures that the weld depth is at least equal to the plate thickness.
- Any reinforcement on the weld is not taken into account for strength calculation purpose.
- Reinforcement makes the butt weld stronger for the static loads. However in case of vibrating and dynamic loads, stress concentration occurs in the reinforcement leading to early failure of the weld. Thus at such locations, reinforcement is undesirable and weld is made flush with the parent metal.
- In any case, reinforcement should not exceed 3 mm.

FILLET WELD



Types of fillet weld

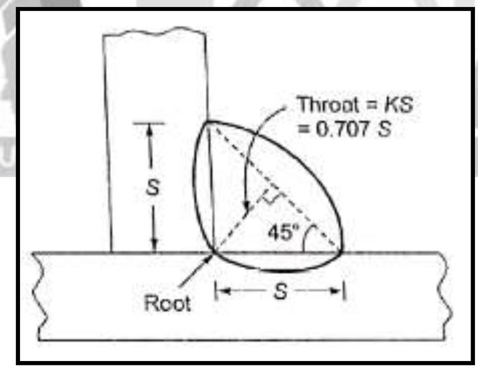
- This weld is required where members overlap each other or the connecting members are in different Planes. In such cases butt weld cannot be provided.
- Fillet weld is predominantly Subjected to shear stresses.
- Fillet weld can either be convex or concave as per requirement
- From appearance point of view, concave fillet weld appears to be larger in length than convex fillet weld but actually concave fillet weld has less penetration and smaller throat thickness. Thus convex fillet weld is stronger

- When fillet weld is applied to sections with rounded toes, there the maximum size of the fillet weld should not exceed 75% (or 3/4) of the thickness of the section at the toe.
- The maximum size of the fillet weld is specified by IS: 800. These are the minimum sizes for the first run in order to avoid cracking.

Minimum size of fillet weld

Thickness of thicker member		Minimum size (mm)
Over (mm)	**Up to and including (mm)**	
0	10	3
10	20	5
20	32	6
32	50	8 for first run, 10

(b) Effective throat thickness: it is the shortest distance from the root of fillet weld to the hypotenuse of the inscribed right triangle within the weld (Fig.)



Typical cross-sectional dimensions of a fillet weld

- In any case, the effective throat thickness should not be less than 3 mm and should not exceed 0.7t or 1.0t under certain situations, where 't' is the thickness of the thinner member jointed.
- Thus, Effective throat thickness = K × size of fillet weld = KS

Where,

l_j = length of the welded joint in the direction of force transfer.

t_t = Size of throat of the weld.

(d) Effective area:

Effective area of fillet weld = effective length × effective throat thickness

(e) Overlap: The overlap of the plates to be joined by fillet weld should not be less than four times the thickness of thinner member to be jointed or 40 mm whichever is greater.

Design Strength of Fillet Weld

- The design stress in a fillet weld is given by,

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

Where, f_{wn} = Nominal strength of fillet weld = $\frac{f_u}{\sqrt{3}}$

- The design strength of a fillet weld is a function of throat area and is given by

$$\mathbf{P_{dw} = l_w\, t_t \frac{f_u}{\sqrt{3}.\gamma_{mw}} = l_w\, KS \frac{f_u}{\sqrt{3}.\gamma_{mw}}}$$

Where,

l_w = effective length of the fillet weld (in mm)

t_t = throat thickness of fillet weld (in mm)

S = Size of the fillet weld (in mm)

f_u = lesser of ultimate strength of weld or the parent metal (in N/mm^2)

P_{dw} = Design strength of weld (in N)

γ_{mw} = Partial factor of safety for weld.

= 1.25 for Shop welding

= 1.5 for site/field welding.

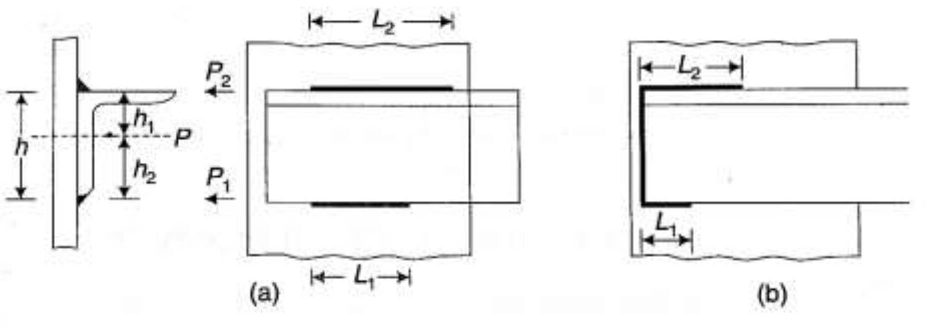
- Where the specified size of the fillet weld is such that the parent metal will not project beyond the weld, then no melting of outer covers is allowed to occur to such an extent so as to reduce the throat thickness.

FILLET WELD FOR TRUSS MEMBERS

Truss members are composed of single-angle or double-angle sections. The following point should be borne in mind while designing the weld.

1. The calculated weld length is placed as longitudinal fillet weld either on the two sides parallel to the axis of the load (fig (a)), or on three sides as shown in (fig.(b)), i.e., transverse welds along with longitudinal welds. A longitudinal fillet weld length should never be placed on one side only as there will be possibility of rotation.

2. The centre of gravity of the weld should coincide with the centroid of the section used as truss member. If the member is symmetrical, the welds will be placed symmetrically but if the member is unsymmetrical (angle, channel) as is usually the case of a truss member, the length of longitudinal fillet weld are kept different on the two sides, as shown in fig. (a), to achieve the above condition.



Fillet Welds for Truss Member

Taking moment about the line passing through L_2,

$$P_1h + P_3\frac{h}{2} - Ph_1 = 0$$

Since P_3 is known; the above Eqs. can be solved for P_1 and P_2. Once the factored design forces P_1 and P_2 are known the fillet weld lengths L_1 and L_2 can be designed.

PLUG AND SLOT WELDS

Plug and slot welds are used most often to tie to parts together and, in particular to reduce the unsupported dimensions of cover plates in compression, which increases the critical stress. They may also be used for shear transmission. Their use is generally reserved for locations where it is impractical to make a fillet weld yet possible to provide a plug or slot. The critical section for either a plug weld or a slot weld is the faying surface between the connected parts. The unit shearing resistance on this section is essentially the same as that of a fillet weld. Plug and slot welds should not be used to transmit tension that is, a force normal to the faying surface. Tensile resistance depends largely upon the degree of penetration of the weld, which is apt to be rather than uncertain in either a plug or a slot weld. Some engineers distrust plug and slot welds because of the difficulty of inspection. It is rather ease to make a plug or a slot weld which appears excellent on the surface yet contains voids at the critical section.

The following specifications should be adhered to while designing plug and slot welds.

1. Width or diameter should be ≥ 3t and also ≥ 25 mm.
2. Corner radius in slotted hole should be ≥ 1.5t and also ≥ 12 mm.
3. Clear distance between holes should be ≥ 2t and also ≥ 25 mm where t is the thickness of plate having a hole or slot.

Note

- A combination of plug weld and types of welds is permissible and the strength of the joint is the sum of the individual capacities of the welds.

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Butt Welds

For butt welds check for the combination of stresses need not be done if

1. Butt welds are axially loaded, and
2. In single and double bevel welds the sum of normal and shear stresses does not exceed the design normal stresses, and the shear stress does not exceed 50 per cent of the design shear stress.

Combined Bearing, Bending and Shear

Where bearing stress, f_{br} is combined with bending (tensile or compressive) and shear stresses under the most unfavorable conditions of loading, the equivalent stress f_e is obtained from the following formula:

$$f_e = \sqrt{f_b^2 + f_{br}^2 + f_b f_{br} + 3q^2}$$

Where,

f_e = equivalent stress

f_b = calculated stress due to bending in N/mm²

f_{br} = calculated stress due to bearing in N/mm²

q = shear stress in N/mm²

The equivalent stress f_e as calculated from Eq. (12) should not exceed the values allowed for the parent metal.

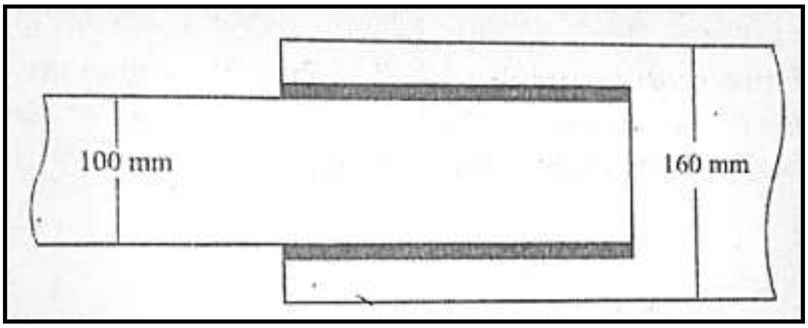
WELDED JOINTS VS BOLTED AND RIVETED JOINTS

1. Welded joints are economical. This is because splice plates and bolt / rivet materials are eliminated. Also. the gusset plates required are of a smaller size because of the reduced connection length. Labour cost is also less as only one person is required to do the welding whereas at least two persons are required for bolting and four for riveting.

15. A more skilled person is required to make a welded joint as compared to a bolted/riveted joint.

Example

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



Solution

Minimum size to be used = 5 mm

Maximum size = 12 - 1.5 = 10.5 mm

Uses = 10 mm fillet weld

$F_u = 410$ N/mm^2, $\gamma_{mw} = 1.25$, thickness of plate = 12 mm,

breadth of plate =100 mm

∴ Full design strength of smaller plate= $\frac{A_g f_y}{\gamma_{mo}}$

$f_y = 250$ Mpa, $\gamma_{m0} = 1.1$

∴ Full design strength= $12 \times 100 \times \frac{250}{1.1} = 272727$ N

Let effective length of welds be L_w

Assuming normal weld, throat thickness

$t = 0.7 \times 10 = 7$mm

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

Let L_w be the total length of the weld required.

Assuming normal weld, t = 0.7 × 6 mm

∴ Design strength of the weld = $L_w t \frac{f_u}{\sqrt{3}} \times \frac{1}{1.25}$

$$= L_w \times 0.7 \times 6 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25}$$

Equating it to the factored load, we get

$$L_w \times 0.7 \times 6 \times \frac{410}{\sqrt{3}} \times \frac{1}{1.25} = 225 \times 10^3$$

$$\therefore L_w = 283 \text{ mm.}$$

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

Let L_1 be the length of top weld and L_2 be the length of lower weld. To make centre of gravity of weld to coincide with that of angle,

$$L_1 \times 3l = L_2 (100 - 31)$$

$$\therefore L_1 = \frac{69}{31} L_2$$

$$L_1 + L_2 = 283$$

$$\text{i.e. } L_2 \left(\frac{69}{31} + 1\right) = 283$$

$$\text{or } L_2 = 87 \text{ mm.}$$

$$\therefore L_1 = 195 \text{ mm.}$$

Provide 6 mm weld of L_1 = 195 mm and L_2 = 87 mm as shown in the Fig.

CLEAR YOUR CONCEPT

Qu1 Which of the following type of weld is most suitable for lap and T-joints?

a) Fillet weld

b) Groove weld

c) Slot weld

d) Plug weld

Qu2 The minimum size of fillet weld should _______

a) Not be less than 3mm

b) Be less than 3mm

c) Be less than 2mm

d) Greater than thickness of thinner part joined

Qu3 The maximum size of fillet weld is obtained by _______

a) Adding 1.5mm to thickness of thinner member to be jointed

b) Adding 3mm to thickness of thinner member to be jointed

c) Subtracting 3mm from thickness of thinner member to be jointed

d) Subtracting 1.5mm from thickness of thinner member to be jointed

Qu4 What is the minimum specified length of fillet weld?

a) Two times the size of weld

b) Four times the size of weld

c) Six times the size of weld

d) Half the size of weld

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CHAPTER – 4

DESIGN OF TENSION MEMBER

INTRODUCTION

A structural member subjected to two pulling (tensile) forces applied at its ends is called a tension member. The members and connections are so arranged that eccentricity in the connection and bending stresses on the member are not developed. Also, the bending moment/stresses due to self-weight of the member, being insignificant, are neglected. However, if some eccentricity exists due to either member not being perfectly straight or eccentricity in connections, then either bending stresses are considered in the design or net area is modified as per specifications. The strength/efficiency of a tension member may be seriously affected because of the end connections (bolt holes), reversal of loads (causing buckling) and, bending moments (due to eccentricity in the end connections or lateral loads on the member).

DESIGN STRENGTH OF A TENSION MEMBER

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

1. Design strength due to yielding of gross section T_{dg}.
2. Rupture strength of critical section, T_{dn} and
3. The block shear T_{dh}.

Design Strength Due to Yielding of Gross Section

This strength is given by

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

Where,

f_y = yield stress of the material

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

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

Where,

A_{nc} = Net area of the connected leg

A_{go} = Gross area of the outstanding leg

$$\beta = 1.4 - 0.0076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\frac{b_s}{L_c} \le \frac{f_u \gamma_{mo}}{f_y \gamma_{ml}} \ge 0.7$$

Where,

w = Outstanding leg width.

b_s = Shear leg width

L_c= Length of the end connection, that is, the distance between outermost bolt in the end joint measured along the load direction or length of the weld along the load direction.

t = Thickness of leg.

For preliminary design IS code recommends the following formula:

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

Where,

α = 0.6 for one or two bolts

= 0.7 for three bolts

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

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

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

Where,

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

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

Note

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

NET SECTIONAL AREA

The net sectional area of a tension member is the gross sectional area of the member minus the sectional area of the maximum number of holes. The reason for considering the net section in the calculation of stresses is the failure of sections with holes. The unit stress in a tension member is increased due to the presence of a hole even if the hole is occupied by a bolt. This is because the area of steel to which load is distributed is reduced and some concentration of stress occurs along the edges of the hole.

Plates

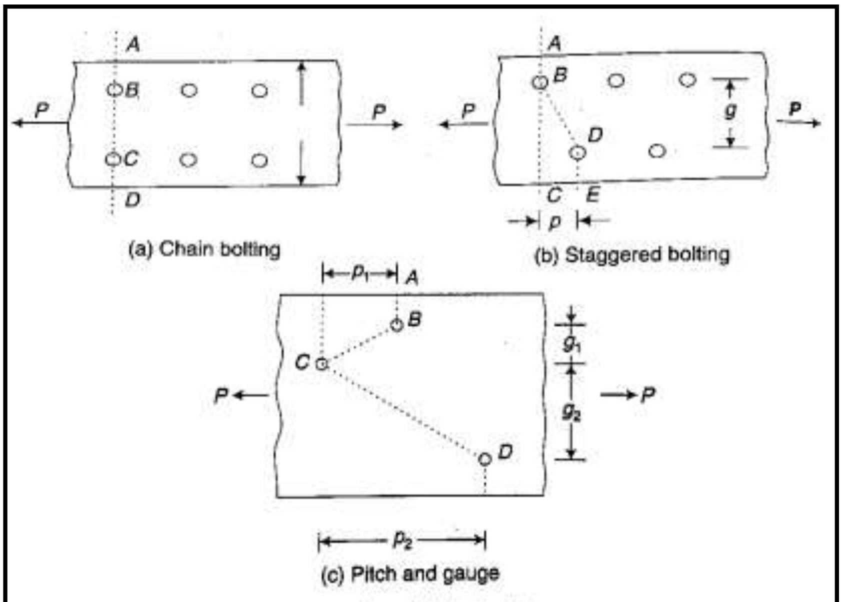
Generally, a tension member without bolt holes can resist loads up to the ultimate load without failure. The presence of holes reduces the strength of the tension member. The net sectional area of the plate members is obtained by deducting the area of bolt holes from the gross sectional area of plates as follows.

1. Refer to the plate shown in Fig. (a) subjected to a pull T and provided with chain bolting. The possibility of failure can be along the section ABCD. Net area at the section is equal to the gross area minus the area of bolt holes B and C.

$$A_n = A_g - \text{sectional areas of holes}$$

$$A_n = bt - n\,(d_h t)$$

$$A_n = (b - nd_h)t$$



Net-section of bolted plate

Deduction = sectional area of holes - $\left(\frac{P_1^2 t}{4\,g_1} + \frac{P_2^2 t}{4\,g_2}\right)$

$A_n = A_g$ - $\left[\text{sectional area of holes} - \left(\frac{P_1^2 t}{4\,g_1} + \frac{P_2^2 t}{4\,g_2}\right)\right]$

$$= bt - nd_h t + \frac{P_1^2 t}{4\,g_1} + \frac{P_2^2 t}{4\,g_2}$$

$$= \left[b - nd_h + \frac{P_1^2}{4\,g_1} + \frac{P_2^2}{4\,g}\right]t$$

If $p_1 = p_2$ and $g_1 = g_2$

$$A_n = \left[b - nd_h + 2\,\frac{p^2}{4g}\right]t$$

$$\text{In general, } A_n = \left[b - nd_h + 2\,\frac{n'p^2}{4g}\right]t$$

Where,

p = staggered pitch

g = gauge distance

n′ = numbers of staggered pitches, and

n = number of holes in the zig-zag line.

SLENDERNESS RATIO (λ)

The slenderness ratio of a tension member is the ratio of its unsupported length L to its least radius of gyration r. Theoretically, there is no limitation on the slenderness ratio of tension member since stability is of little concern. However, they may be subjected to load reversals during transportation, shipping, erection etc. Also, in case the axial load in a slender tension member is removed and small transverse load are applied, undesirable vibrations or deflections might occur. For example, this condition can occur in a bracing rod subjected to wind loads.

In order to provide adequate rigidity to prevent undesirable lateral movement or excessive vibrations, design specifications usually limit slenderness ratio for tension member; IS: 800-2007 limits it to the values as listed in Table.

Maximum Slenderness Ratios

S. No.	Member	Maximum Slenderness Ratio
1	A tension member in which reversal of direct stress due to loads other than wind or seismic forces occur.	180
2	A member normally acting as a tie in roof truss or a bracing system but subjected to possible reversal of stresses resulting from the action of the wind or earthquake forces.	350
3	Tension members (other than pretensioned Members).	400

NET EFFECTIVE AREA OF TENSION MEMBER (A_{ne})

Factors that affect efficiency of the tension member are:

(a) Ductility of the member (b) Shear lag effect

(c) Geometric configuration (d) Method of fabrication

Among all the above factors, shear lag effect is the most prominent one. The net effective area of a section is defined as:

$$A_{ne} = k_1 k_2 k_3 k_4 A_n$$

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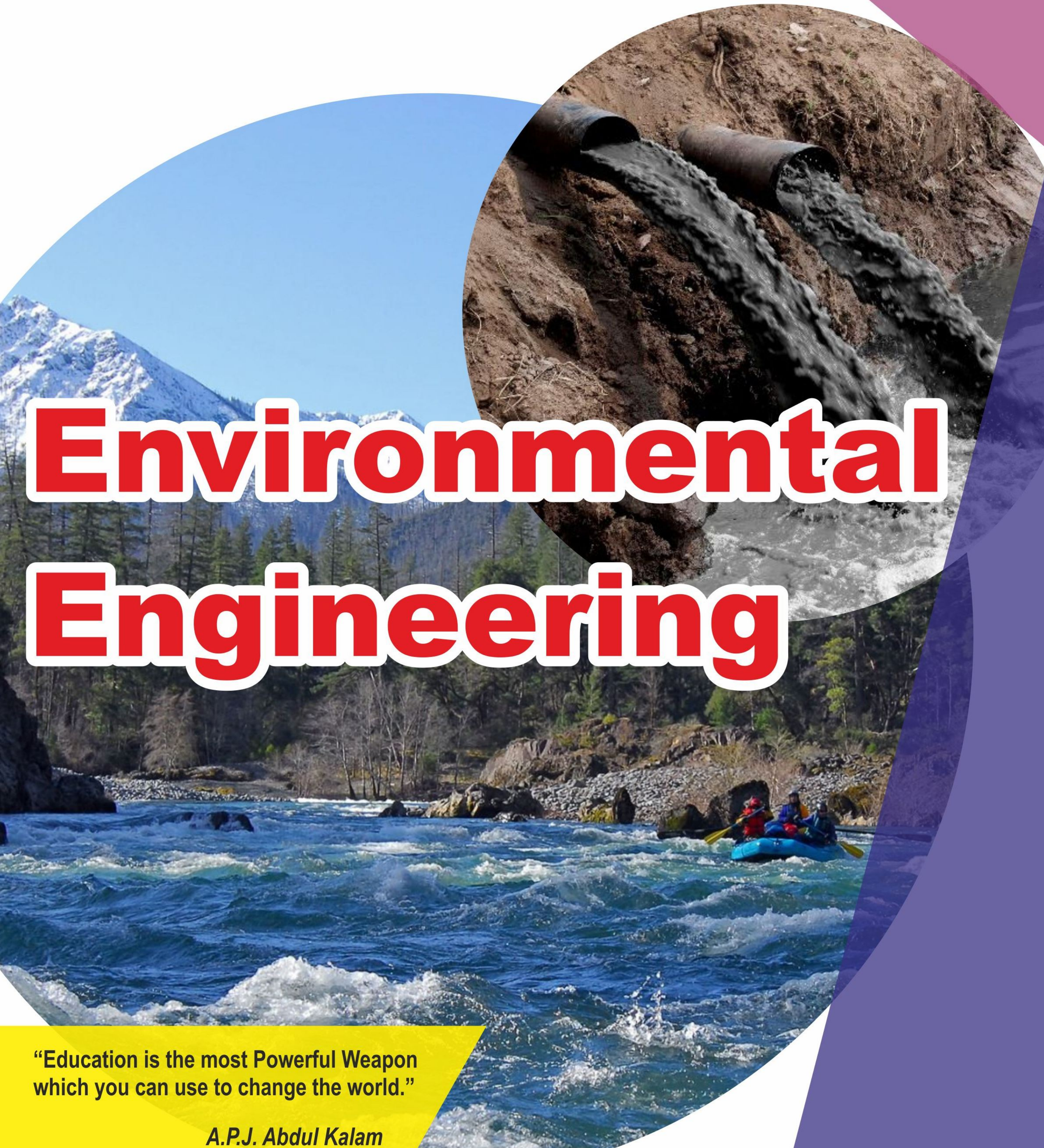

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than the claimed tensile strength of the steel except in that case where all the elements of a tension member are so arranged that uniform stress distribution occurs over the entire section.

The phenomenon of shear lag occurs when some elements of the member cross section are not connected. In Fig. (a), an angle section is acting as a tension member connected with one leg only. Thus, the connected leg will get over stressed and the unconnected leg will not be stressed fully. At connections, large amount of load is carried by the connected leg and it requires some distance called as transition distance to spread the stress uniformly across the whole angle section (Fig. (b)).

Now clubbing all the above four factors viz. k_1, k_2, k_3 and k_4, Eq. reduces to,

$$A_{ne} = k_4 A_n$$

($\because k_1 = k_3 = 1$, k_2 is taken care of in area of bolt holes)

IS: 800-2007 defines the factor k_4 as α

$$\mathbf{A_{ne} = \alpha A_n}$$

Where

α = 0.6 when number of bolts ≤ 2

= 0.7 when number of bolts = 3

= 0.8 when number of bolts ≥ 4

= 0.8 for welds

PROCEDURE FOR THE DESIGN OF TENSION MEMBER

Step-1. Determine the required gross area to Carry the factored tensile load (T_u) considering the strength in yielding as:

$$A_g = \frac{T_u}{f_y/\gamma_{m0}} = \frac{1.1\,T_u}{f_y}$$

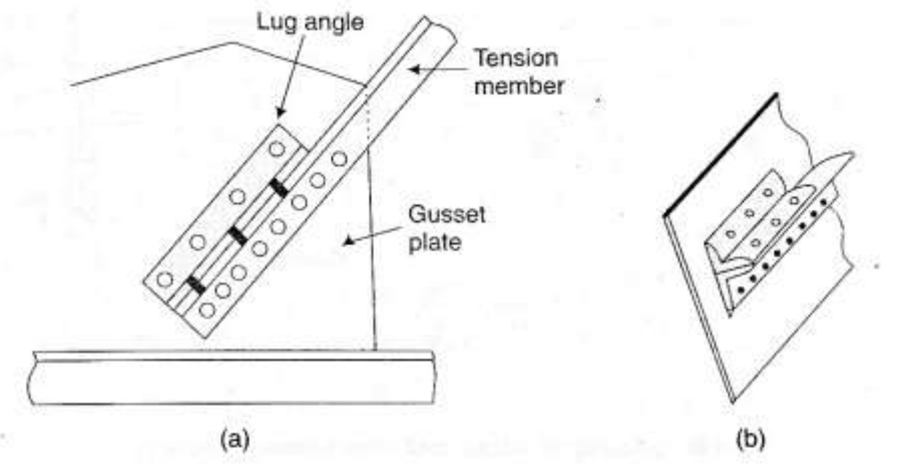
- As per CI. 10.3.3.3 of IS: 800-2007, the design shear strength of bolts carrying shear through a packing plate of thickness in excess of 6 mm shall be reduced by a factor given by,

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

Where, t_{pk} = Thickness of thicker packing plate (in mm)

LUG ANGLES

- For a heavily loaded tension member, the length of the connection comes out to be too large to be accommodated on a gusset plate.
- This length of connection can be reduced by using lug angles as shown in Fig.
- Use of lug angles saves the cost of gusset plate but this saving is offset by additional fasteners and angles required.



Lug angle

CI. 10. 12 of IS: 800-2007 specifies certain requirements for lug angles which are as given below:

(a) The effective connection of the lug angles shall be as far as possible, terminate at the end of the member.

(b) The connection of the lug angles to the main member shall preferably start in advance of the member to the gusset plate.

Solution

Strength of the plate is the least of

(a) Yielding of gross section

(b) Rupture of critical section

(c) The block shear strength

(a) From consideration of yielding:

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

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

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

(b) From the consideration of rupture along the critical section:

Critical section is having two holes.

Diameter of holes = 16 + 2 = 18 mm.

$$\therefore A_n = (130 - 2 \times 18) \times 12 = 1128\ mm^2$$

Strength of member from the consideration of rupture

(c) Block shear strength:

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

$A_{tg} = 60 \times 12 = 720\ mm^2$

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

$A_{tn} = (60 - 18) \times 12 = 504\ mm^2$

Solution

(a) 90 mm leg is connected to gusset:

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

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

$$Ag = 865\ mm^2 \text{ (from table)}$$

$$\therefore T_{dg} = \frac{864 \times 250}{1.1} = 196364\ N = 196.364\ kN.$$

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

$$A_{nc} = \left(90 - \frac{6}{2}\right) \times 6 = 522\ mm^2$$

$$A_{go} = \left(60 - \frac{6}{2}\right) \times 6 = 342\ mm^2$$

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

w = Length of outstanding leg = 60 mm

w_i = 50 mm

b_s = 60 + 50 – 6 = 104 mm

L_c = 4 × 50 = 200 mm

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

$$= 1.159 \leq \frac{f_u}{f_y} \times \frac{\gamma_{mo}}{\gamma_{ml}} \geq 0.7$$

Hence β = 1.159

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

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

$$A_{nc} = \left(60 - \frac{6}{2}\right) \times 6 = 342 \text{ mm}^2$$

$$A_{go} = \left(90 - \frac{6}{2}\right) \times 6 = 522 \text{ mm}^2$$

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

w = 90 mm w_i = 30 mm

b_s = 90 + 30 – 6 = 114 mm

L_c = 50 × 4 = 200 mm

$$\therefore \beta = 1.4 - 0.076 \times \frac{90}{6} \times \frac{250}{410} \times \frac{114}{200}$$

$$= 1.004 < \frac{f_u}{f_y}\frac{\gamma_{mo}}{\gamma_{ml}} > 0.7$$

$$\therefore \beta = 1.004$$

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

$$= \frac{0.9 \times 342 \times 410}{1.25} + \frac{1.004 \times 522 \times 250}{1.1}$$

$$= 220069 \text{ N} = 220.069 \text{ kN}$$

(iii) Block shear strength:

Tearing length in tension = 60 – 30 = 30 mm

A_{vg} = 230 × 6 = 1380 mm^2 A_{vn} = (230 - 4.5 × 18) × 6 = 894 mm^2

A_{tg} = 30 × 6 = 180 mm^2 A_{tn} = (30 - 18) × 6 = 72 mm^2

Block shear strength is the smaller of the following two values:

(a)

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

$$= \frac{1380 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 72 \times 410}{1.25}$$

$$= 202332 \text{ N} = 202.332 \text{ kN}$$

Strength in bearing:

$e_{min} = 1.5 \times 20 = 30$ mm $p_{min} = 2.5 \times 20 = 50$ mm

Let e = 30 mm   p = 50 mm

Then K_b is smaller of $\frac{30}{3\times22}, \frac{50}{2\times22} - 0.25, \frac{400}{410}, 1.0$

$$\therefore K_b = 0.4545$$

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

$\therefore$ Bolt value = 45272 N.

Note
➢ In case of single shear bolt value is usually governed by value in single shear.

(a) Connection without lug angle:

Number of bolts required = $\frac{430000}{45270}$ = 9.5

Provide 10 bolts.

Length of connection, $L_c = 9 \times 50 = 450$ mm

15 d =15 × 20 = 300 mm.

$\therefore L_c > 15$ d. It is long connection.

$\therefore \beta_{lj} = 1.075 - 0.005 \frac{450}{20} = 0.9625$

Shear strength of bolt (after reducing for long connection)

= 0.9625 × 45272 = 435743 N

$\therefore$ No. of bolts required = $\frac{430000}{43574.3}$ = 9.87

Lug angle is to be designed to take a load of = 1.2 × 215 = 258 kN

Gross area of lug angle required = $\frac{258\times1000}{250/1.1} = 1135 \text{ mm}^2$

Provide ISA 100 X 100, 6 mm.

$\therefore$ A_g provided = 1167 mm^2

The strength of lug angle in rupture = $\frac{0.9\times(100+100-10-22)6\times410}{1.25}$

= 297562 N > 258000 N O.K.

Bolt value:

In single shear = 45272 N

In bearing = $\frac{2.5\times0.4545\times20\times6\times400}{1.25}$ = 43632

$\therefore$ Bolt value = 43630 N

Number of bolts required = $\frac{258\times1000}{43630} = 5.91$

Provide 6 bolts.

Design force for connected leg = 1.4 × 215 kN

$\therefore$ Number of bolts required to connect lug angle with main angle = $\frac{1.4\times215\times1000}{43630}$ = 6.89

Provide 7 bolts.

Connection of main angle to gusset plates:

Force to be transferred = 215 kN

Bolt value for this is 45272 N.

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

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CLEAR YOUR CONCEPT

Qu1 Bars and rods are not used as:

a) Tension members in bracing systems

b) Friction resistant members

c) Sag rods to support purlin

d) To support girts in industrial buildings

Qu2 What are steel tension members?

a) Structural elements that are subjected to direct compressive loads

b) Structural elements that are subjected to direct tensile loads

c) Structural elements that are subjected to indirect compressive loads

d) Structural elements that are subjected to indirect tensile loads

Qu3 What is the maximum effective slenderness ratio for a tension member in which stress reversal occurs?

a) 180

b) 200

c) 280

d) 300

Qu4 What is the maximum effective slenderness ratio for a member subjected to compressive forces resulting only from combination of wind/earthquake actions?

a) 180

b) 200

c) 340

d) 250

Qu8 The design tensile strength of tensile member is

a) Minimum of strength due to gross yielding, net section rupture, block shear

b) Maximum of strength due to gross yielding, net section rupture, block shear

c) Strength due to gross yielding

d) Strength due to block shear

Answer

1-(b), 2-(b), 3-(a), 4-(d), 5-(b), 6-(c), 7-(d), 8-(a)

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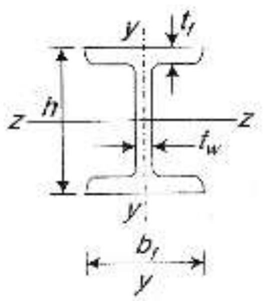
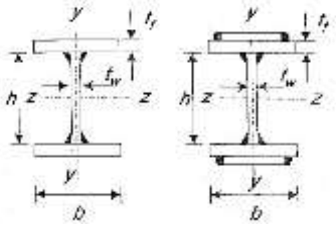
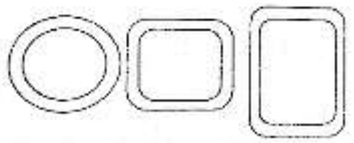
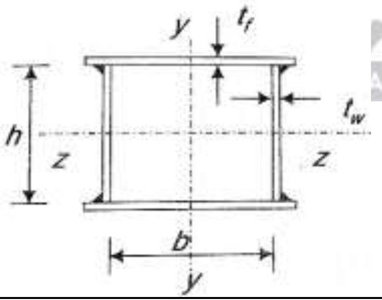
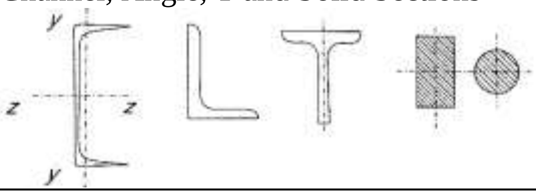
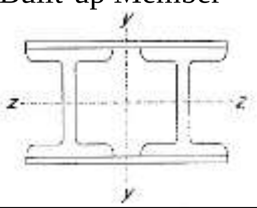
Bulking class of cross-sections [Refer Table 10 in IS: 800]

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

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(c) In frames:

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

(d) In case of stepped columns:

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

Appropriate Radius of Gyration

Appropriate radius of gyration means the radius of gyration of compression member about the axis of buckling. For example, in case of column shown in Fig. when length of the column is taken 6 m, the radius of gyration about z-z axis should be considered. For buckling about y-y axis, the length of column is 3 m and radius of gyration about y-y axis is to be considered. The maximum slenderness ratio governs the design strength. If the length of the column to be considered is the same for buckling about any axis, naturally the governing slenderness ratio is $\frac{KL}{r_{min}}$.

DIFFERENT TYPES OF COMPRESSION MEMBER

- Column, stanchion or post: This type of compression member supports floors or girders of a building and usually carries very heavy loads.
- Strut: It is lightly loaded compression member used in a truss and is usually of small span. A typical strut may be continuous or discontinuous. A continuous strut passes over more than one joint (apart from end joints) like top chord member of a truss bridge, principal rafter of roof truss etc. A discontinuous strut spans between two end joints only like the vertical or the inclined compression member of a truss.
- Boom: It the compression member of a crane.

S.No.	Type of column	Failure feature
1.	Long	• Due to elastic buckling. • Stress in the column will not exceed proportional limit and is much lower than this limit.
2.	Intermediate	• Due to inelastic buckling (yielding + buckling) and flexural rigidity EI changes continuously. • Extreme fiber reaches the yield point while stress in all other fibres remains elastic.
3.	Short	• Due to yielding (crushing) • Axial shortening of column occurs till it gets crushed.

S. No.	Type of member	λ
1.	Tension member prone to reversal of stresses due to the loads other than wind or earthquake.	180
2.	Member carrying compressive loads due to dead and imposed (live) loads.	180
3.	Member carrying compressive force due to the combination of wind and earthquake only provided deformation of such members does not adversely affect the stress in any part of the structure.	250
4.	Compression flange of a beam restrained against lateral torsional buckling.	300
5.	A member normally acting as a tie in a roof truss or a bracing system not considered effective when subjected to possible reversal of stresses due to wind or earthquake forces.	350

- Thin walled members with open cross-sectional shapes are weak in torsion and hence undergo buckling by twisting rather than bending.
- Torsional buckling occurs when torsional rigidity of the member is very small as compared to flexural rigidity.
- Failure occurs due to rotation about longitudinal axis (xx-axis) of the member.
- This occurs only with doubly symmetric sections with very slender cross sectional elements.
- Standard hot rolled sections are not prone to torsional buckling but a member made of thin plate elements must be checked for possible torsional buckling.
- Torsional buckling is quite complex and thus it is always avoided. This is done by proper arrangement of the members and by providing bracing to prevent lateral movement and twisting.

Flexural Torsional Buckling

- This type of buckling failure is caused by combination of flexural buckling and torsional buckling.
- Herein the member bends and twists simultaneously.
- This type of failure occurs with unsymmetrical sections including both with one axis of symmetry like channels, T-sections, double angle sections etc. and with no axis of symmetry at all.
- Usually an analysis of torsional or flexural-torsional buckling only is made as and when it seems to be appropriate.
- This type of buckling can either be elastic or inelastic in nature.

Where,

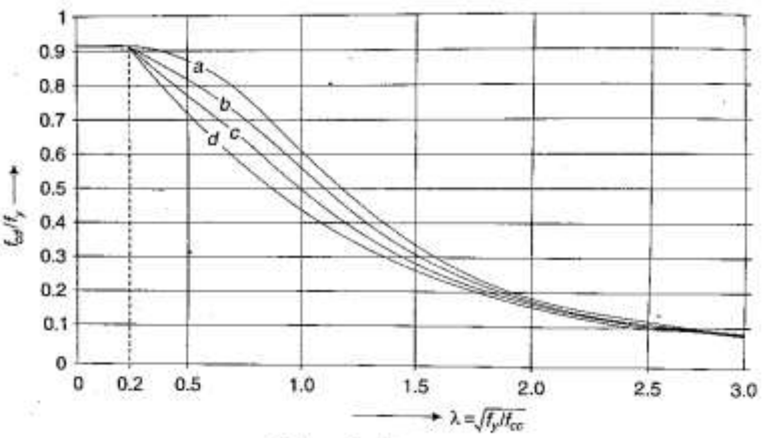
f_y = Yield stress in N/mm^2

f_{cc} = Elastic critical stress in compression = $\pi^2 \frac{E}{\lambda^2}$

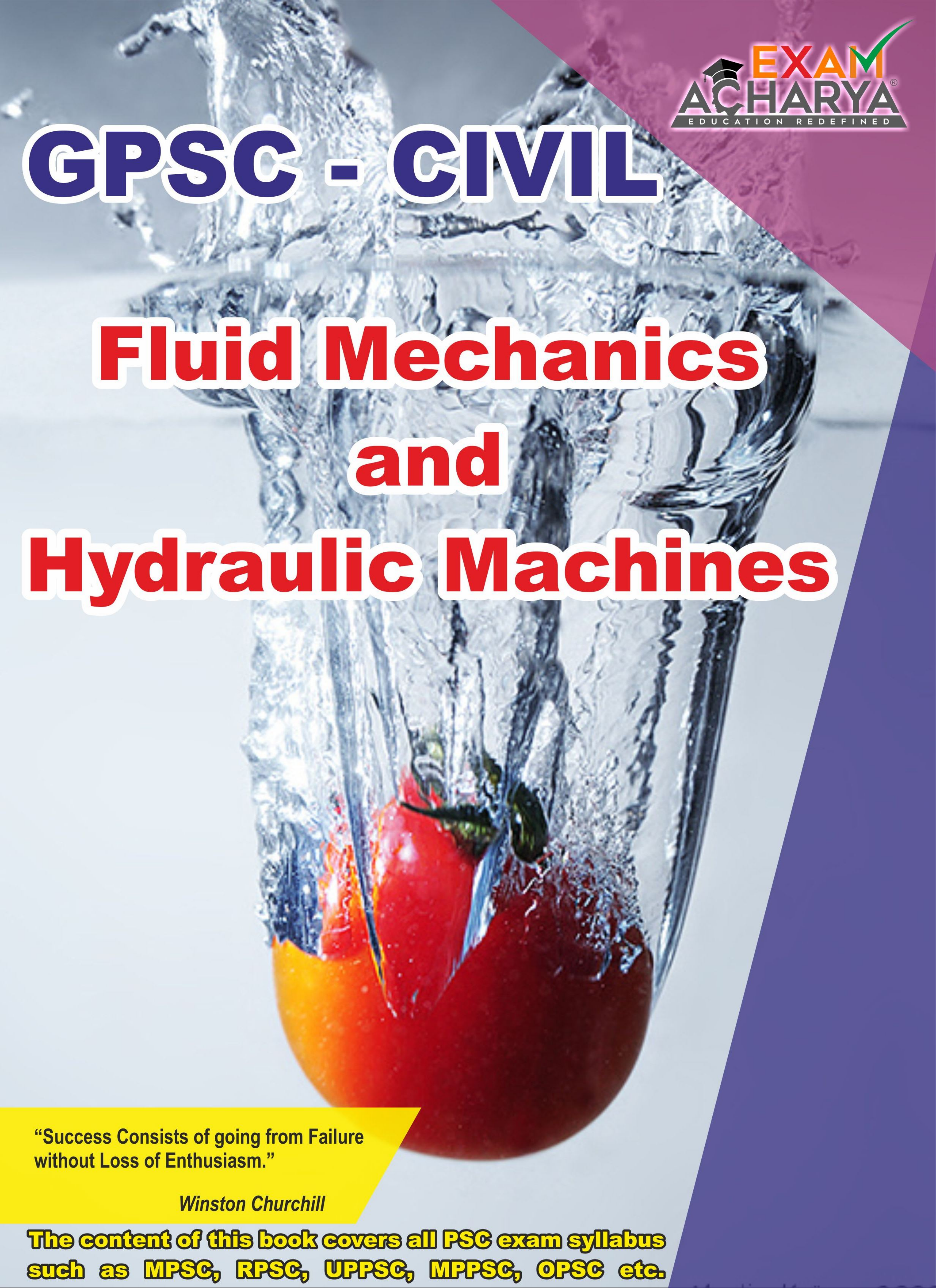
λ = Slenderness ratio

n = A factor which ranges from 1 to 3 and usually taken as 1.4

- From Merchant Rankine formula, it follows that for small values of slenderness ratio (λ), the failure stress tends to become yield stress (f_y) and for large values of slenderness ratio, it becomes the elastic critical stress (f_e).
- IS: 800-2007 recommends different column curves viz. a, b, c and d in non-dimensional form based on Perry Robertson approach based on cross-section classification.
- Due to various imperfections that exist in a real column, this yield stress need to be reduced but it is quite difficult to quantify the amount (or fraction) of reduction in this yield stress.
- Thus based on statistical test results, curves a, b, c and d were proposed which takes into account all these imperfections.
- The buckling curves (a, b, c and to give the value of reduction factor x of the resistance of the column as a function of non-dimensional effective slenderness ratio called as reference slenderness for various types of cross sectional shaped.



Column buckling curves



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Fluid Mechanics and Hydraulic Machines

"Success Consists of going from Failure without Loss of Enthusiasm."

Winston Churchill

The content of this book covers all PSC exam syllabus such as MPSC, RPSC, UPPSC, MPPSC, OPSC etc.

Table aids in selection of appropriate buckling curve as a function of type of cross section and axis about which the buckling is taking place.

- Cl. 7.1.2.1 of IS: 800-2007 recommends the following formula for estimating the design compressive stress f_{cd} of an axially loaded compression member as:

$$f_{cd} = \frac{f_y/\gamma_{mo}}{\emptyset + \sqrt{\emptyset^2 - \lambda^2}} = x\,\frac{f_y}{\gamma_{mo}} \le \frac{f_y}{\gamma_{mo}}$$

Where,

$\emptyset = 0.5[1 + \alpha(-0.2) + \lambda^2]$, λ = Non dimensional effective slenderness ratio

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

F_{cc} = Euler's bulking stress, α = Imperfection factor,

γ_{mo} = Partial factor of safety for material (steel) = 1.1

Stress reduction factor $x = \frac{1}{\emptyset + \sqrt{\emptyset^2 - \lambda^2}}$

- The design compressive strength (P_d) of the compression member is then given by,

  $P_d = A_e f_{cd}$

Where,

A_e =Effective sectional area = Gross sectional area if bolt holes are filled with bolts

PROCEDURE FOR THE DESIGN OF AXIALLY LOADED COMPRESSION MEMBER

Assumptions

(a) The ideal column is assumed to be absolutely straight with no initial crookedness which is in fact a purely hypothetical situation and never occurs in practice.

(b) The modulus of elasticity is assumed to be constant in a built up column.

(c) Certain types of secondary stresses which may even be of the order of 25% to 40% of primary stresses are completely ignored i.e. not taken into account.

- There are two unknowns and it is required to assume one and calculate the other. Obviously the section has further to be checked for safety.

Design Procedure

Step-1. For an average column of height 3 m to 5 m. the slenderness ratio lies between 40 and 60. For slender column, a little higher value than 60 is assumed. For column with heavy factored load, a smaller value of slenderness ratio is assumed.

For the assumed value of slenderness ratio, the design compressive stress for that particular value is determined from Table for the curve that is relevant to the buckling class of cross section as given in Table.

Alternatively, assume a design compressive stress in the compression member.

Where, k_1, k_2, k_3 are the constants depending on end conditions as given in Table.

$$l_{vv} = \frac{\left(\frac{l}{r_{\omega}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}}$$

$$\lambda_{\emptyset} = \frac{(b_1 + b_2)}{2t\varepsilon\sqrt{\frac{\pi^2}{250}}}$$

Constants k_1, k_2, k_3

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

l = Center to center length of the supporting member

r_v = Radius of gyration about the minor axis

b_1, b_2 = Width of the two legs of the angle section

t = Thickness of the leg

ε = Yield stress ratio = $\sqrt{250/f_y}$

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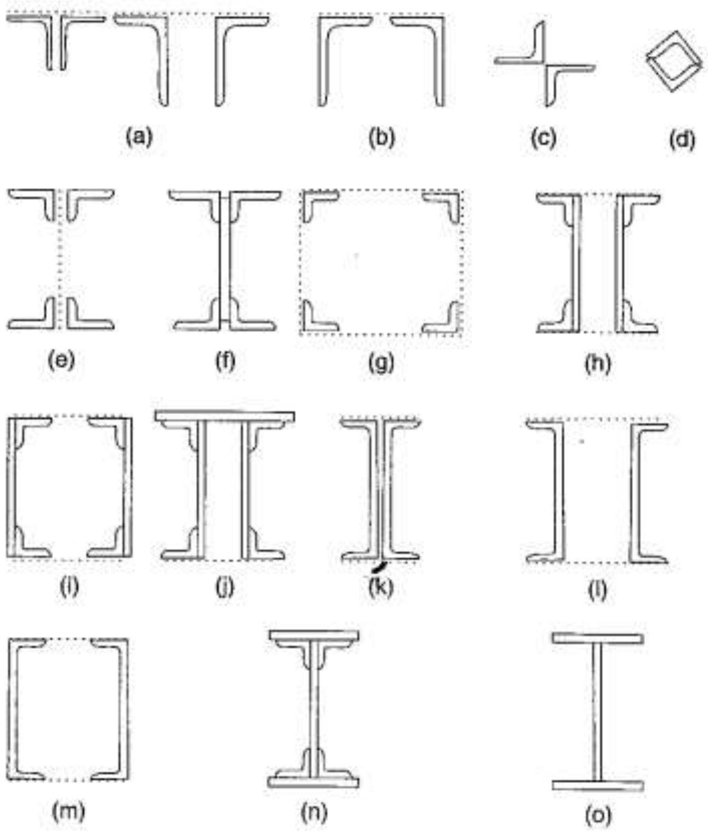
Step-7. The design compressive strength (P_d) of the member is computed by multiplying the design compressive stress (f_{cd}) with the effective cross sectional area (A_e). This design compressive strength must be more than the factored compressive load i.e.

$$P_d > P_u$$

Revise the section if P_d calculated differs considerably from the factored axial compressive load P_u.



Shapes of compression members

Example

In a truss a strut 3 m long consists of two angles ISA 100 X 100, 6 mm. Find the factored strength of the member if the angles are connected on both sides of 12 mm gusset by.

(i) One bolt

(ii) Two bolts

(iii) Welding, which makes the joint rigid.

Solution

From steel table for a ISA 100 X 100, 6 mm,

area = 1167 mm²; $C_{zz} = C_{yy}$ = 26.7 mm.

$r_{zz} = r_{yy}$ = 30.9 mm.

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

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

$$\therefore P_d = A_e f_{cd}$$

$$\rightarrow = 2 \times 1167 \times 111.2 = 259541 \text{ N}.$$

i.e. $P_d = 259.541$ kN Answer

Case (ii): When two bolts are used

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

$$\therefore KL = 0.85 \times 3000 = 2550 \text{ mm}.$$

Hence in this case $\frac{KL}{r} = \frac{2550}{30.9} = 82.5$

For steel with $f_y = 250$ N/mm²,

f_{cd} for = $\frac{KL}{r} = 80$ is 136 N/mm²

for $\frac{KL}{r} = 90$ is 121 N/mm²

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

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

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

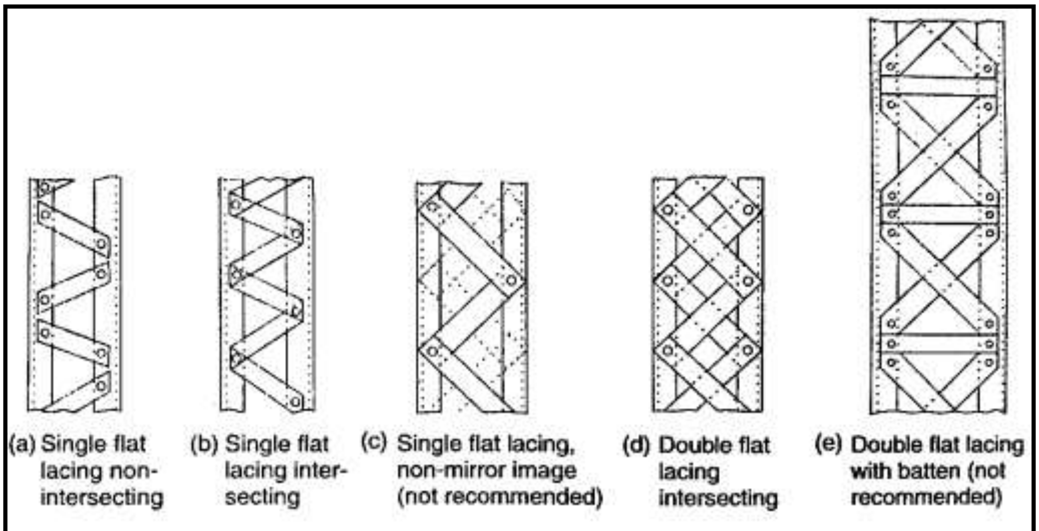
$$\therefore P_d = 2 \times 1167 \times 132.25$$

$\rightarrow$ = 308672 N = 308.672 kN Answer

Case (iii): Rigid joint by welding

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

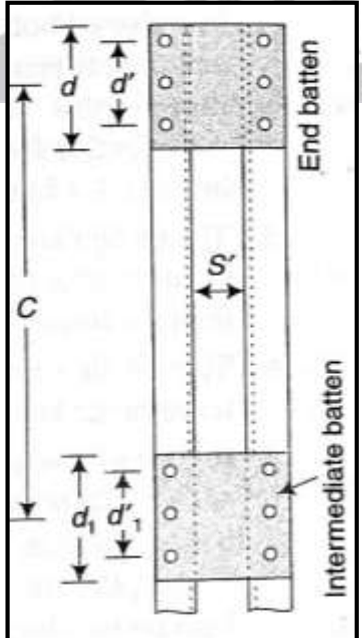
The object of providing lateral system is to keep the main members of the column away from principal ones. In doing so, the lacings are subjected to shear forces due to horizontal forces on columns.



Lacing systems

Battens

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



Battened column

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

DESIGN OF BATTENED COLUMNS

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

1. Batten plates should be provided symmetrically.
2. At both ends batten plates should be provided. They should be provided at points where the member is stayed in its length.
3. The number of battens should be such that the member is divided into not less than three bays. As far as possible they should be spaced and proportioned uniformly throughout.
4. Battens shall be of plates, angles, channels, or I-sections and at their ends shall be riveted, bolted or welded.
5. By providing battens distance between the members of columns is so maintained that radius of gyration about the axis perpendicular to the plane of battens is not less than the radius of gyration about the axis parallel to the plane of the batten ($r_{yy} > r_{xx}$).
6. The effective slenderness ratio of battened columns shall be taken as 1.1 times the maximum actual slenderness ratio of the column, to account for shear deformation.
7. The vertical spacing of battens, measured as centre to centre of its end fastening, shall be such be such that the slenderness ratio of any component of column over that distance shall be neither greater than 50 nor greater than 0.7 times the slenderness ratio of the member as a whole about its z-z axis.
8. Battens shall be designed to carry the bending moments and shear forces arising from transverse shear force V_t equal to 2.5% of the total axial force.
9. In case columns are subjected to moments also, the resulting shear force should be found and then the design shear is sum of this shear and 2.5% of axial load.

New Batches are going to start.....

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Total test	**: 80**

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(i) Those having ends cut by ordinary method.

(ii) Those having the ends cut and milled.

If the ends are not milled, the splice plates and their connections to the column are designed to transmit all forces. The columns having milled ends, the ends are placed firmly in contact with each other and hence considerable load is transferred by bearing. The connections and splice plates are designed for only 50 percent of axial load.

The various types of column splices used are shown in Fig. The situations in which they are used are

(a) When the columns are of the same size, milled ends are provided.

(b) When columns are of slightly different sizes, filler plates are used. Load is transferred partially by bearing.

(c) When the columns are of considerably different sizes, bearing plates are used.

DESIGN OF COLUMN SPLICES

The following procedure may be used in the design of column splices.

(1) Column splice plates may be assumed to act as short columns of zero slenderness ratio i.e. assume $f_{cd} = \frac{f_y}{1.1}$ and calculate required area.

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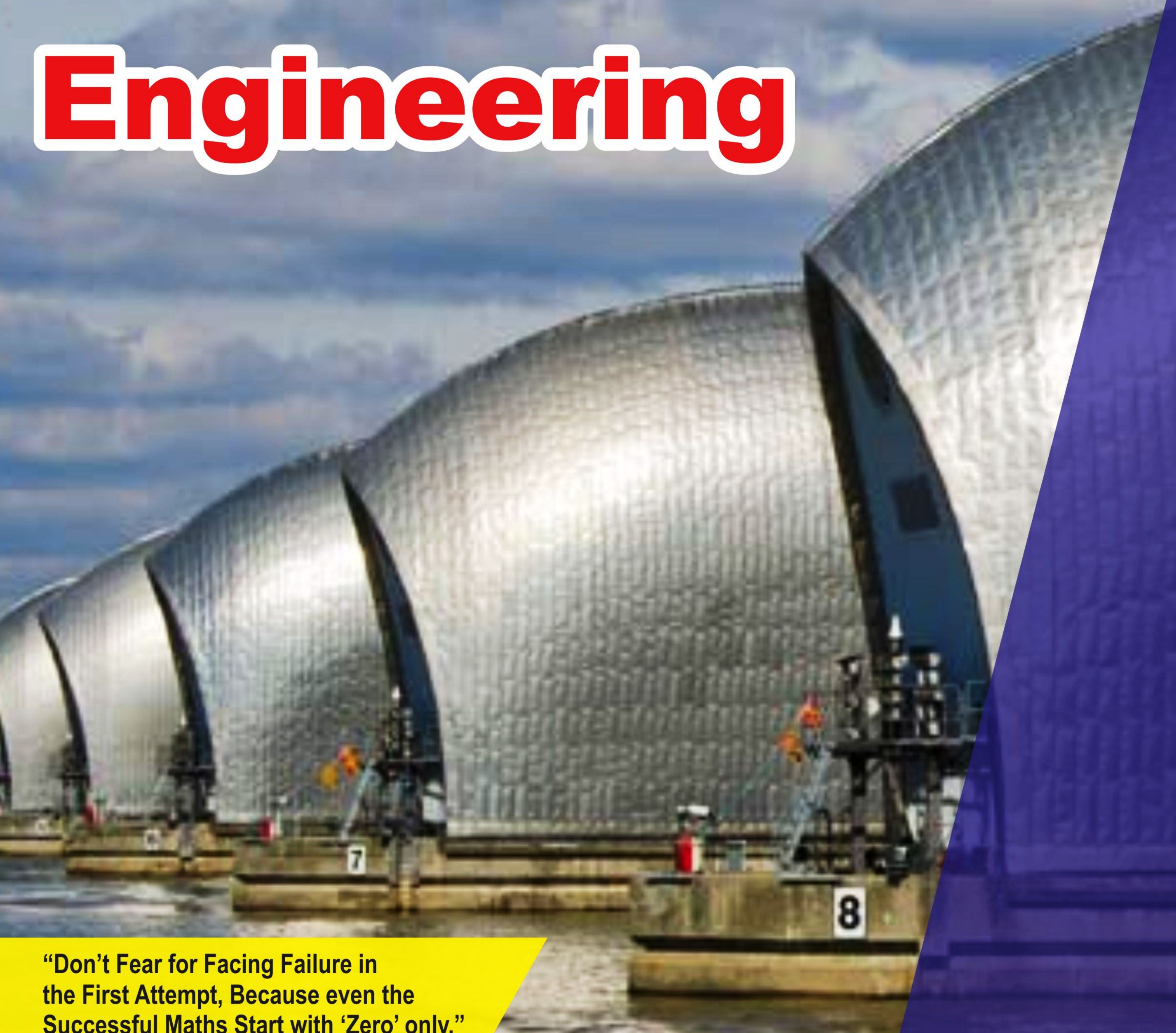
Water Resource Engineering



"Don't Fear for Facing Failure in the First Attempt, Because even the Successful Maths Start with 'Zero' only."

A.P.J. Abdul Kalam

The content of this book covers all PSC exam syllabus such as MPSC, RPSC, UPPSC, MPPSC, OPSC etc.

(i) Bearing plate may be assumed as short beam to transmit the axial load to the lower column.

(ii) Axial load of the column is assumed to be taken by flanges only. Thus the load transfer is as shown in Fig.

Hence maximum moment in bearing plate = $\frac{P}{2}$ a

∴ The thickness of bearing plate required 't' is given by $\frac{1}{6}$ bt^2 f_{bs} = M or t = $\sqrt{\frac{6M}{bf_{bs}}}$

Where, f_{bs} = design bending stress = $\frac{f_y}{\gamma_{mo}} = \frac{250}{1.1}$ = 227.27 N/mm^2

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Qu5 What is the effective length when both ends of compression member are hinged?

a) 0.65L

b) 0.8L

c) L

d) 2L

Qu6 What is slenderness ratio of compression member?

a) Ratio of effective length to radius of gyration

b) Ratio of radius of gyration to effective length

c) Difference of radius of gyration and effective length

d) Product of radius of gyration and effective length

Qu7 Lacing shall be designed to resist a total transverse shear equal to ____ of axial force in member

a) 5%

b) 1%

c) 4.3%

d) 2.5%

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Test Your Self

Qu8 Slenderness ratio of lacing is limited to

a) 200

b) 145

c) 500

d) 380

CHAPTER – 6

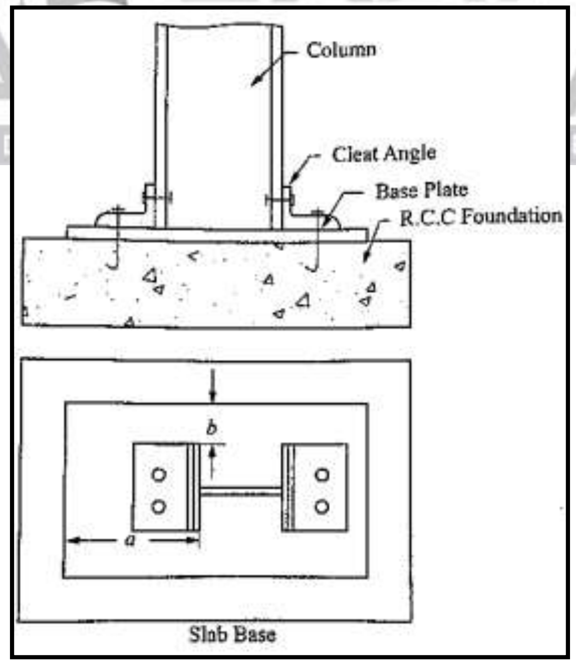
COLUMN BASE AND FOUNDATION

Column bases transmit the column load to the concrete or masonry foundation blocks. The column base spreads the load on wider area so that the intensity of bearing pressure on the foundation block is within the bearing strength. There are two types of column bases commonly used in practice:

1. Slab Base
2. Gusseted Base.

Slab Base

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



Slab base.

Thickness of Base Plate:

(1) Find the intensity of pressure

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

(2) Minimum thickness required is given by

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

Where,

t_s = thickness of base plate

and t_f = thickness of flange.

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

Connections:

(1) Connect base plate to foundation concrete using four 20 mm diameter and 300 mm long anchor bolts.

(2) If bolted connection is to be used for connecting column to base plate, use 2 ISA 65 X 65, 6 mm thick angles with 20 mm bolts.

(3) If weld is to be used for connecting column to base check the weld length of fillet welds.

DESIGN OF GUSSETED BASE

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

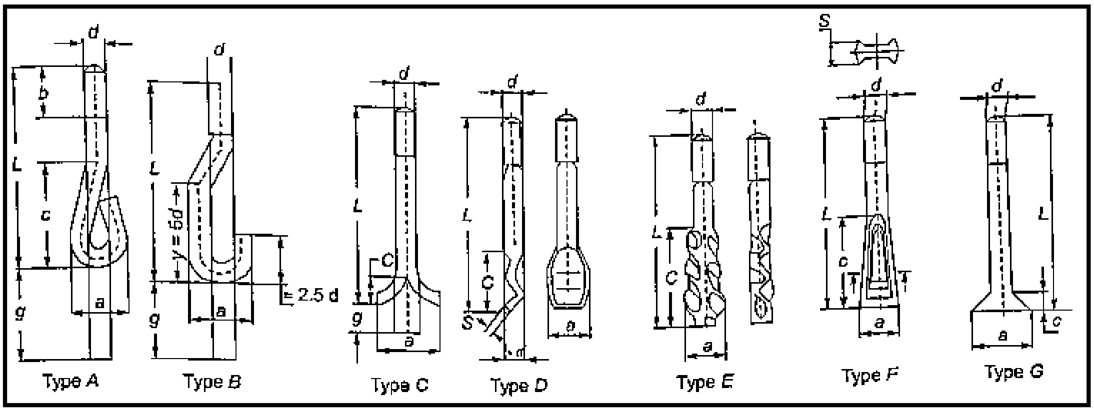


Fig. Some typical anchor bolts

- The transfer of tensile load occurs due to bond between anchor bolt and concrete (in case of cast-in-place anchor bolts) or epoxy/grout filling (in case of post-installed anchor bolts).
- For very heavy uplift forces, a plate may be bolted at the lower end of the bolt.
- Cast-in-place anchor bolts generally fail due to yielding and fracture of anchor shank and breakout of concrete.
- A minimum of four bolts, one each at the corners of the base plate are provided which are in fact sufficient also.
- Larger number of bolts may also be provided it required but it should not exceed eight (08) from considerations of construction aspects.
- A minimum of two anchor bolts are provided even if column is subjected to only axial loads.
- In presence of bending moments, foundation bolts must be of adequate size so as to hold down the column to the concrete pedestal.
- The capacity of these bolts to resist tension depends on their lengths available to deform elastically. Their capacity can be increased by pre-tensioning them.

Referring to Fig., let the column is subjected to an axial load P and a moment M. This column will try to overturn either about 1-1 or 2-2 passing through the bolts depending on whether the moment is clockwise or anti-clockwise. The stabilizing moment (=Px/2) is provided by the axial load P.

In case of grouted anchors, failure may occur by bond failure at the grout - concrete interface. The bond strength is given by,

$$N_b = \tau_o \pi d_o l$$

Where,

τ_o = Bond strength of grout - concrete

d_o = Diameter of bolt hole

l = Length of embedment

PROCEDURE FOR THE DESIGN OF FOUNDATION BOLTS

Step-1. The uplift force F_b on the bolts is determined.

Step-2. The diameter of the bolts is assumed beforehand and stress area is referred from the Table.

Tensile stress area of bolt

Bolt size, d(mm)	12	16	20	22	24	27	30	36
Tensile stress area (mm^2)	84.3	157	245	303	353	459	561	817

The tensile strength of the bolt is,

$$T_{db} = \frac{T_{nb}}{\gamma_{mb}} 0.9\, f_{ub} A_{nb}$$

$$< \frac{f_{yb} A_{sb} \gamma_{mb}}{\gamma_{m0}}$$

Where,

A_{nb} = Area of bolt at the root of the thread i.e. stress area, in mm^2

f_{ub} = Ultimate stress of the bolt

γ_{m0} = Partial factor of safety = 1.25

Step-3. The number of bolts required to resist the uplift force are given by, Try 2 ISMC 350 @ 413 N/m.

Area provided = 2 × 5366 = 10732 mm²

r_{zz} = 136.6 mm

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

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

Since it is a laced column

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

From Table

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

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

Load carrying capacity = 10732 × 141.0

= 1513.297×10^3

= 1513.297 kN > 1400 kN O.K.

Spacing Between the Channels

Let it be a clear distance 'd',

Now: $I_{xx} = 2 \times 10008 \times 10^4 = 20016 \times 10^4$ mm^4

$$I_{yy} = 2\left[430.6 \times 10^4 + 5366\left(\frac{d}{2} + 24.4\right)^2\right]$$

Equating I_{yy} to I_{xx}, we get

$$2\left[430.6 \times 10^4 + 5366\left(\frac{d}{2} + 24.4\right)^2\right] = 20016 \times 10^4$$

Shear to be resisted by each lacing systems = $\frac{35000}{2}$ = 17500 N.

Length of lacing = (220 + 60 + 60) $\frac{1}{\cos 45}$ = 480.83 mm.

Minimum thickness of lacing = $\frac{1}{40}$ × 480.83

= 12.02 mm.

Use 14 mm flats

Minimum width of lacing, if 20 mm bolts are used = 3 × 20 = 60 mm.

Use 60 ISF 14

Sectional area = 60 × 14 = 840 mm².

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

$$\therefore \frac{\mathbf{KL}}{\mathbf{r}} = \frac{\mathbf{480.83}}{\mathbf{4.041}} = \mathbf{118.97 < 145} \qquad \text{O.K.}$$

Strength of 20 mm shop bolt

(a) in single shear = 0.78 × $\frac{\pi}{4}$ × 20² × $\frac{400}{\sqrt{3}\times 1.25}$ = 45272 N

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

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

(b) Strength in bearing = $\frac{2.5K_b d\, t\, f_u}{1.25}$

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

= 101808 N

∴ Bolt value = 45272.

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Test Series Available..

Total weekly test	: 35
Total mid subject test	: 16
Total full length test	: 13
Mock test	: 16
Total test	: 80

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Example

A column section ISHB 300 @ 577 N/m is carrying a factored axial load of 600 kN, a factored moment of 30 kN-m and a factored shear force of 60 kN. Design a suitable column splice. Assume ends are milled.

Solution

Since the ends are milled, 50% of axial load is transferred through bearing and splice plates transfer the remaining 50% of the load.

∴ Load to be transferred by splice plate = 300 kN

i.e., load to be transferred by each splice plate = 150 kN

Assuming the thickness of splice plate 6 mm, for the calculation of lever arm,

a = 300 + 6 = 306 mm.

$$\therefore \textbf{Force in each plate due to moment} = \frac{30\times10^3}{306} = 98.04 \text{ kN}$$

∴ Total load in each splice plate = 150 + 98.04 = 248.04 kN

For rolled steel section, f_y = 250 N/mm².

Area required = $\frac{248.04\times10^3}{250/1.1}$ =1091.376 mm²

Width of splice plate = width of flange = 250 mm.

$$\therefore \textbf{Thickness required} = \frac{1091.376}{250} = 4.365 \text{ mm}$$

Provide 6 mm plates.

Using 20 mm bolts of grade 4.6,

Strength in single shear = $0.78 \times \frac{\pi}{4} \times 20^2 \times \frac{400}{\sqrt{3}\times1.25}$ = 45272 N

CLEAR YOUR CONCEPT

Qu1 In the case of an axially loaded column machined for full bearing, the fastenings connecting the column to the base plate in gusseted base are designed for

a) 100% of the column load

b) 50% of the column load

c) 25% of the column load

d) Erection loads only

Qu2 Anchor bolts are provided in column bases to

1. Resist the tension forces
2. Fix column in place during erection
3. Serve as reinforcement in concrete pedestal below the base plate

a) 1, 2 are correct

b) 2, 3 are correct

c) 3 and 1 are correct

d) All are correct

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Qu3 A column base is subjected to moment. If the intensity of bearing pressure due to axial load is equal is to stress due to the moment, then the bearing pressure between the base and the concrete is

a) Uniform compression throughout

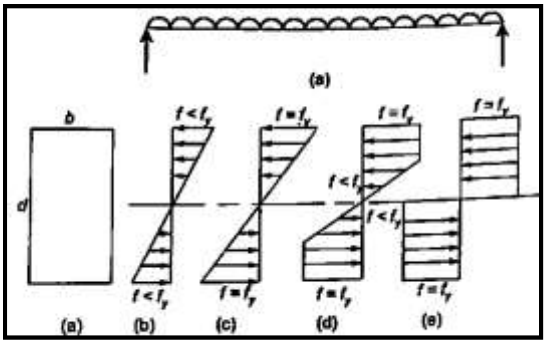
b) Zero at one end and compression at the other end

c) Tension at one end and compression at the other end

d) Uniform tension throughout

Answer

1-(b), 2-(a), 3-(b)



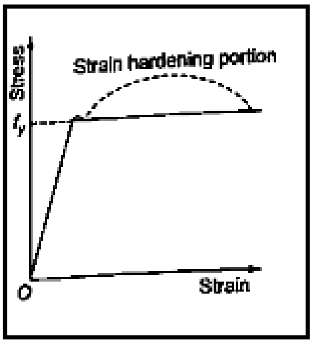
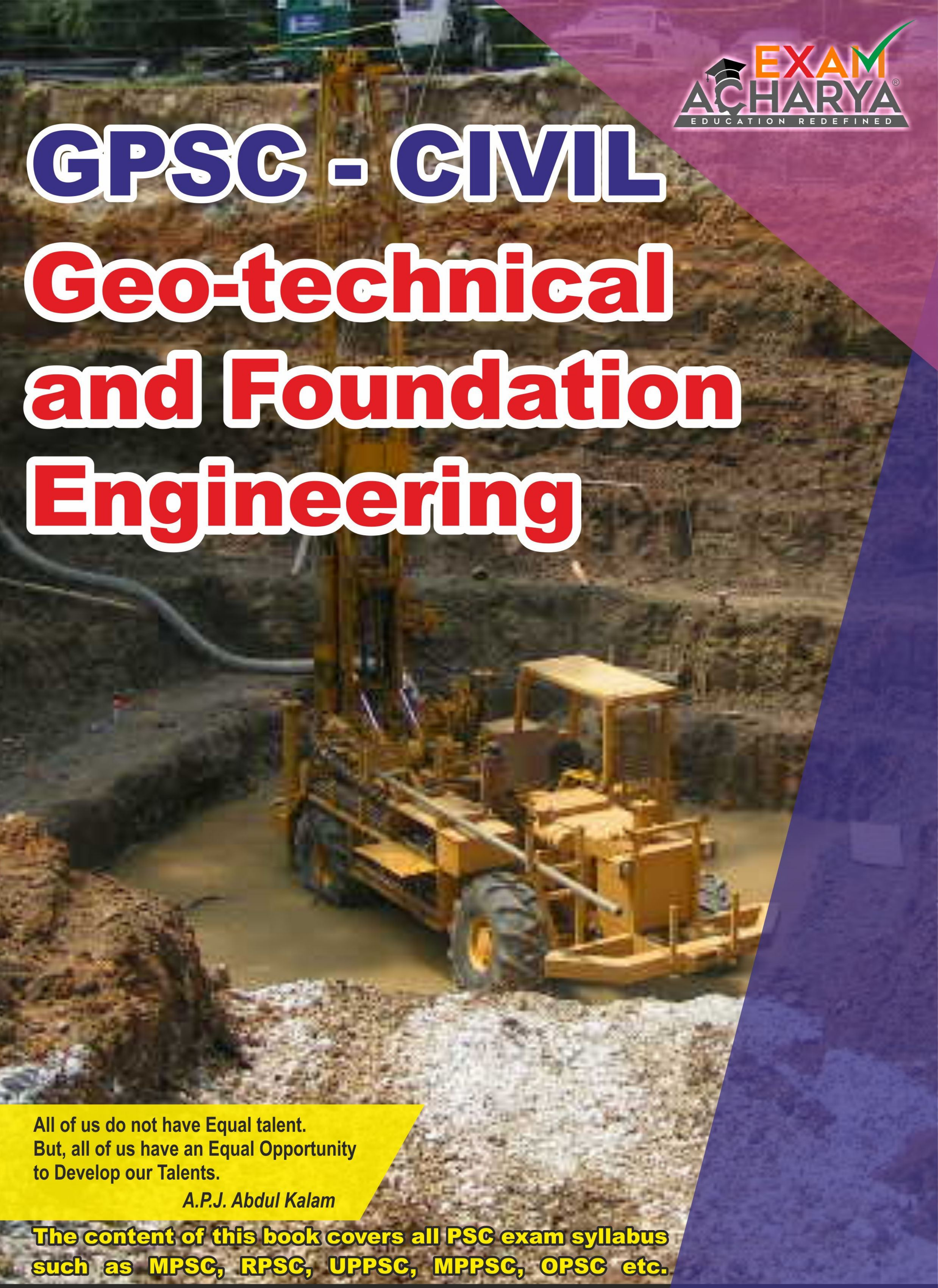


Fig. Plastic moment capacity of a beam section

Fig. Stress strain curve for steel

- Now the load is gradually increased which results in the increase in the stresses in the fibers with extreme fibers being highly stressed. The loading is increased till the extreme fibers reach yield stress (Fig. (c)). The stress-strain relationship for steel is assumed as shown in Fig. wherein the strain hardening part of the stress-strain curve is ignored and it is assumed that once the yield point is reached, fibers go on yielding without resisting any additional load.
- As per the theory of plastic analysis, the highly stressed fiber once yields is not capable of resisting any load (or moment). But since the inner fibers has not yielded yet, and thus additional load is resisted by the un-yielded portion of the beam section.
- As the loading is increased further gradually, the inner fibers will also yield and will cease to take any additional load [Fig. (d)].
- With still further increase of load, resistance to load is offered by inner fibers which are not yielded yet. This resistance to load increases till all the fibers get yielded (Fig.(e)).
- Once all the fibers get yielded, no more resistance to the load is offered by the beam section. This condition i.e. when all the fibers have yielded at a section is called as formation of plastic hinge.
- Once plastic hinge condition has reached then, infinite rotation can take place at constant load and without resisting any additional load. This moment carrying capacity of the section (at this ultimate load) is called as plastic moment capacity of the section (M_p).



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But, all of us have an Equal Opportunity
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it can be deduced that,

$$M_p = f_y Z_p$$

Where,

f_y = Yield stress of the material of the beam section

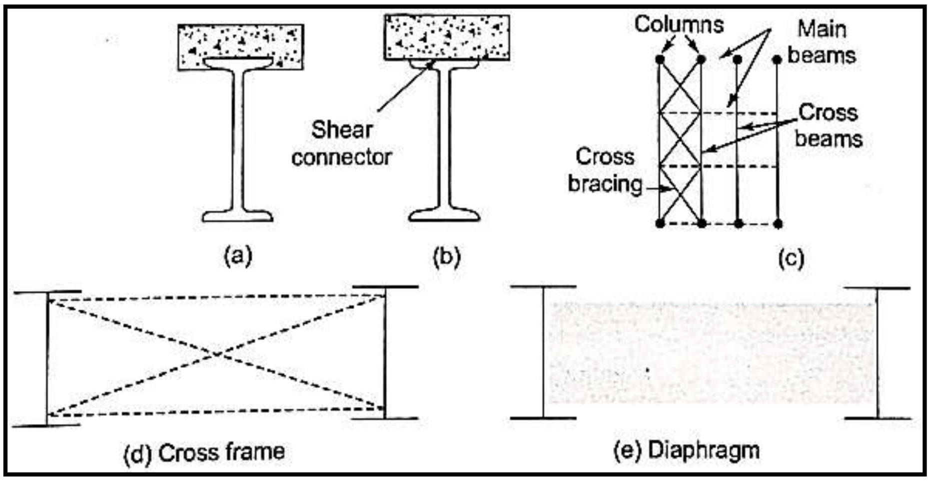
Z_p = Plastic section modulus of the beam section

- For standard rolled sections, Z_p is about 1.125 to 1.14 times Z_e for I-sections and about 1.7 to 1.8 times for channel sections.

1. Indian rolled steel sections consist of sloping flanges from inside, fillets at the junctions and rounded edges.
2. For determinate structures, formation of a single hinge causes collapse of the whole structure since it goes on rotating without resisting any additional load but this is not so for indeterminate structures.

LATERAL STABILITY OF BEAMS

- When a beam is loaded, one of the flanges of the beam comes in compression and other in tension. For economy in beam design, I_z is made considerably larger than I_y.
- Such beam sections are quite weak in bending in the plane normal to the web and thus compression flange of the beam is liable to buckle in the direction in which it is free to move i.e. in the horizontal direction.
- This buckling tendency increases as the ratio I_z/I_y increases. However, the bottom flange of the beam remains in tension and thus remains straight. But the bottom flange, web and the compression flange acts a unit and thus the whole section rotates as shown in Fig.



Lateral support in beams

The compression flange, full lateral support should be assumed at the connections. It may happen that the cross beams are connected to the tension flange of the main beam. In such cases, cross bracing as shown in Fig. (c) is done to achieve the lateral restraint. Lateral bracing, which prevents lateral translation, should be applied as close to the compression flange as possible. In practice, however, bracing is often proportioned solely by judgment. Since bracing requirements are usually quite modest, this is generally satisfactory.

The torsional bracing in the form of cross frames or diaphragms as shown in Fig. (d, e) prevents twist directly.

There are many other methods also to achieve lateral support but the degree of restraint provided by them is questionable and, therefore, if a doubt arises, it is safer to design the beam as a laterally unsupported beam.

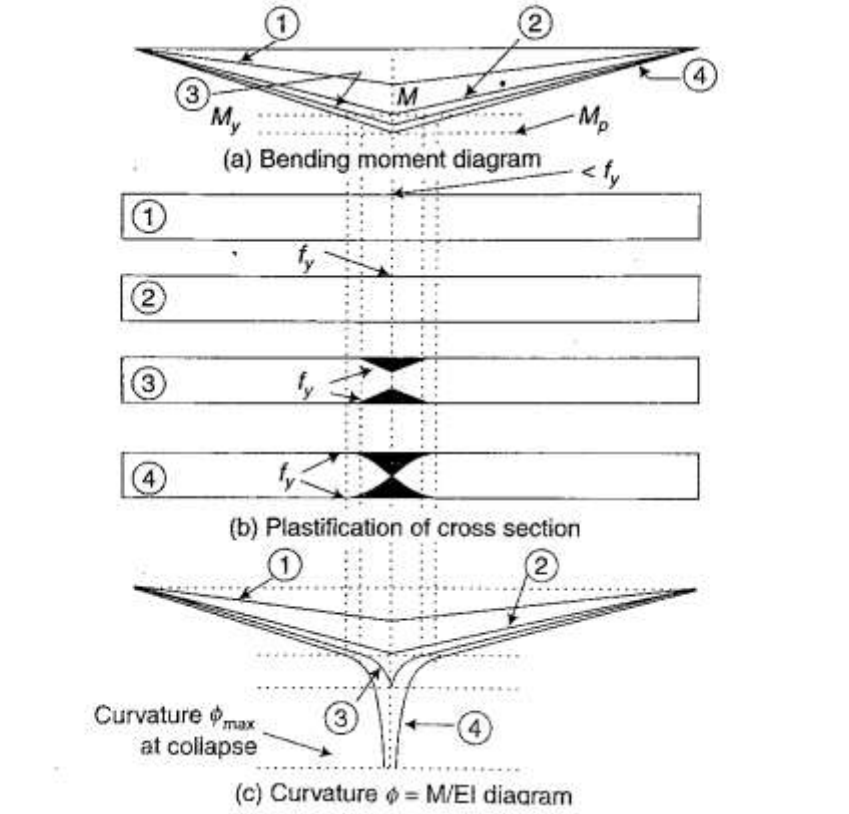
Note

1. In case of beams which support concrete slabs over compression flanges with fixed live loads on the slab, full lateral support may be assumed due to the friction between the compression flange and slab. On the contrary, if the loads are moving and vibrations exist, then the friction is reduced and therefore, full lateral support should not be assumed.
2. It is difficult to confirm whether a particular beam is laterally restrained or not. It is for the designer to judge if a particular beam has satisfactory lateral support. In case, a beam is continuous over the support, the compression flange will be the lower flange to some distance on the both sides of it and the lower flange will also have to be restrained near the supports.

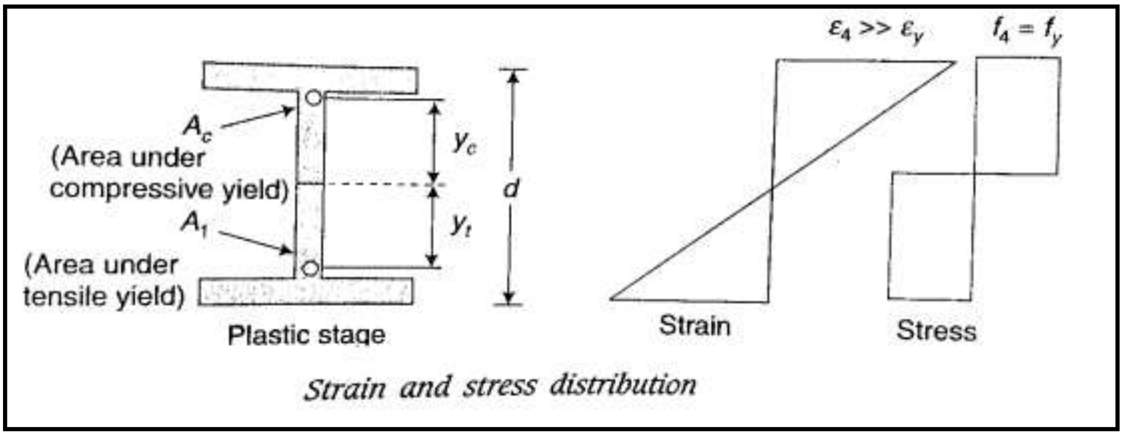
At point 2 marked on the curve, the stress f_2 becomes equal to f_y. The moment corresponding to this point is known as first yield moment and is given by

$$M_2 = M_y = f_2 Z_e = f_y Z_e$$

The extreme fibers after attaining yield stress do not take any more stress. On increasing the load further, the yield progresses inwards, i.e., the stresses are redistributed inwardly towards the neutral axis, for example say for point 3 of Fig. Thus the outer fibers are plastified while the inner fibers near the neutral axis are. The moment capacity at this point is sum of contributions from plastic and elastic portions.



BM diagram, spread of plasticity across the section and corresponding curvatures for the beam



Strain and stress distribution in section corresponding to point 4 of bending stress distribution curve

The M_p value at this stage is generally about 10-20% more than M_y. In reality the beam can carry a load higher than its plastic moment capacity due to strain hardening but this is neglected for design purposes. Furthermore, the residual stresses cause yielding to initiate at a lower load, but however, M_p values are unaffected. It is because residual stresses are self-equilibrating.

LATERAL - TORSIONAL BUCKLING

The compression flange of an I-beam acts like a column and will buckle sideways if the beam is not sufficiently stiff or the flange is not restrained laterally. The buckling of a beam loaded in the plane of its

BENDING STRENGTH OF BEAMS

In general, a beam may fail by lateral-torsional buckling, flange local buckling, or web local buckling. Any of these types of failure can be in either elastic range or in inelastic range. The strength corresponding to each of the three limit states must be computed and the smallest value will control. The important aspects which need consideration in a beam design are moments, shears, deflection, crippling, buckling and lateral support. The choice of a proper rolled section to be used for a beam is often based on its ability to carry the bending moment. The design bending strength discussed in this section is for plastic, compact. and semi-compact sections only. For a laterally supported beam,

$\leq 1.2\ Z_e\ f_y/\gamma_{m0}$ (for simply supported beams)

or $\leq 1.5\ Z_e\ f_y/\gamma_{m0}$ (for cantilever beams)

or

as appropriate.

Where,

β_b = 1.0 (for plastic and compact sections)

= Z_e/Z_p (for semi-compact sections)

Z_e, Z_p = Elastic and plastic section moduii of the cross section, respectively (Appendix XV)

f_y = Yield stress of the material

γ_{m0} = 1.1, the partial safety factor

The check that the design bending strength determined from Eq. (13) should be less than 1.2 $Z_e\ f_y/\gamma_{m0}$ for simply supported beams and 1.5 $Z_e\ f_y/\gamma_{m0}$ for cantilever beams is to ensure that the onset of plasticity under unfactored loads-dead loads, imposed loads, and wind load-is prevented. This implies that yield does not occur at working loads in I and channel sections bending about the zz-axis and buckling about the yy-axis.

For slender sections:

$$\mathbf{M_d = Z_e f'_y}$$

Where, f'_y = Reduced design strength for slender sections

Note

1. For most of the I-sections and channel sections, the ratio Z_{pz}/Z_{ez} is less than 1.2 and the plastic moment capacity governs the design.
2. For sections having $Z_{pz}/Z_{ez} > 1.2$, the constant 1.2 may be replaced by the ratio of factored load/service load, i.e. by γ_f. Thus, 1.2 $Z_e\ f_y$ in the check is purely notional and becomes in practice $\gamma_f Z_e\ f_y$.

bending theory and results in higher stresses near the junction of web to flange elements with the stress dropping as the distance from the beam web increases. The resultant stress distribution across the flange is therefore non-uniform. This phenomenon is known as shear lag. For rolled I-sections this effect is negligible while for flanges of built-up sections and for unusually wide flange beams the effect may be considerable and of concern. Shear lag effect depends upon width-to-span ratio, beam end restraints, and the type of load. Point load causes more shear lag than uniform load.

As per the provisions of IS: 800-2007, the shear lag effects in flanges may be disregarded provided:

(a) $b_0 \leq \frac{L_0}{20}$ (for outstand elements)

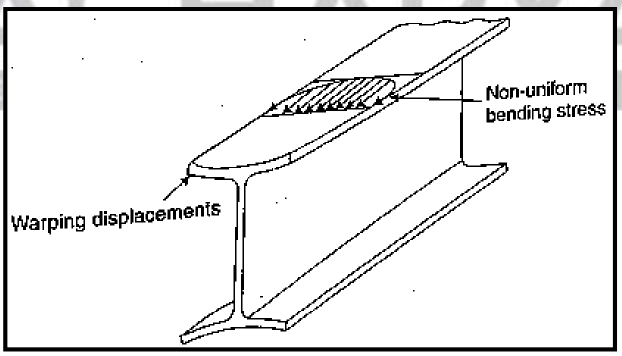
(b) $b_i \leq \frac{L_0}{10}$ (for internal elements)

Where,

L_0 = Length between points of zero moment (inflection) in the span

b_0 = Width of the flange outstand

b_1 = Width of an internal element



Shear Lag Effect

Laterally Unsupported Beams

Beams with major axis bending and compression flange not restrained against lateral bending (or inadequate lateral support) fail by lateral-torsional buckling before attaining their bending strength. The effect of lateral-torsional buckling need not be

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

Where

M_{cr} = Elastic critical moment corresponding to lateral-torsional buckling of beam

I_y = Moment of inertia about minor axis

I_w = Warping constant = $(1 - \beta_f)\ \beta_f\ I_y\ h^2{}_f$

I_t = St. Venant's constant

$= \Sigma \frac{b_i t_i^3}{3} = 2\frac{b_f t_f^3}{3} + \frac{b_f t_w^3}{3}$ for open section (e.g. I section)

(In the second term h_f, c/c distance between flanges = $h - t_f$, has been used in place of $h - 2t_f$ to account for added stiffness of web-to-flange junction and fillets.)

G = Shear modulus

Above equation can be simplified for prismatic members made of standard rolled I-sections and welded doubly symmetric I-sections and is as follows.

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

Where,

r_y = Radius of gyration about the weaker axis

L_{LT} = Effective length for lateral-torsional buckling (Tables)

B_f = Ratio of moment of inertia of compression flange to the sum of moments of inertia of compression and tension flanges

t_f = Thickness of the flange

$f_{cr,b}$ = Extreme fiber compressive elastic lateral buckling stress

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The shear stress distribution corresponding to Eq. (27) for an I-section beam in elastic range is shown in Fig. (a) and that in the plastic range in Fig. (b).

It can be seen that the flanges resist a very small portion of shear and a significant portion of the shear is resisted by web. Furthermore, the maximum and average shear for I-sections is almost same. Clearly, the web will completely yield long before the flanges begin to yield. Because of this, yielding of the web represents one of the shear limit states. For all practical purposes the average shear stress is determined by

$$\tau_{av} = \frac{V}{d\, t_w} = \frac{V}{A_v}$$

Where, d is the depth of web, t_w is the thickness of web and A_v is the shear area.

The nominal shear yield strength of the web based on Von-Mises yield criterion can be represented as

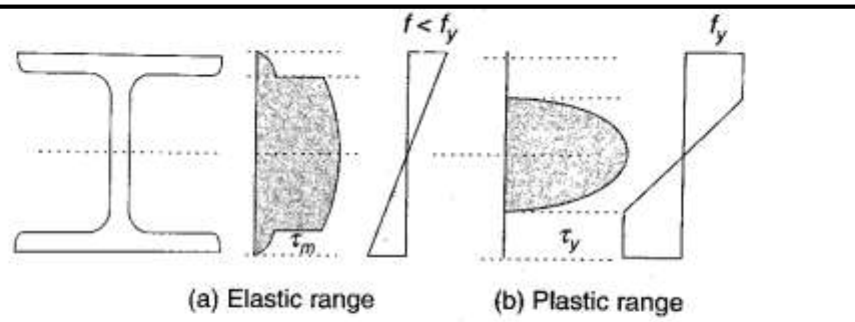
$$\tau_y = \frac{V_n}{A_v} = \frac{f_{yw}}{\sqrt{3}}$$

or

$$V_n = \frac{A_v f_{yw}}{\sqrt{3}}$$

Where, f_{yw} is the yield strength of the web, V_n is the nominal shear resistance.

There may be two cases: (a) the nominal shear resistance of a cross section governed by plastic shear resistance, or (b) strength of web governed by shear buckling.



Shear stress and bending stress distribution in elastic and plastic ranges

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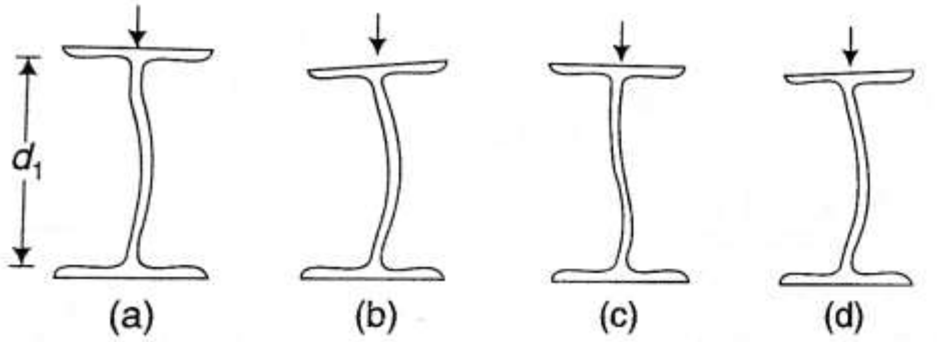
t_f = thickness of the flange

t_w = thickness of the web

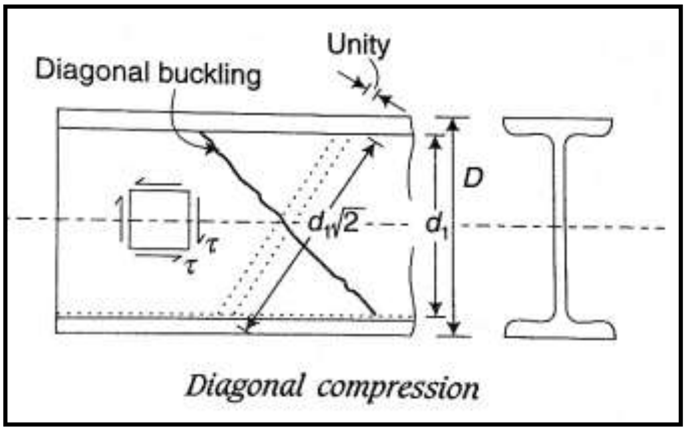
WEB BUCKLING

The web in a rolled steel section behaves like a column when placed under concentrated loads. The web is quite thin and is, therefore, subjected to buckling. Web buckling (vertical buckling) occurs when the intensity of vertical compressive stress near the center of section becomes greater than the critical buckling stress for the web acting as column. The buckling of the column web is very much influenced by the restraints provided for the flanges, Various types of buckling's that can occur are shown in Fig. In all cases, the bottom flange is assumed to be restrained against lateral deflection and rotation. For the top flanges, the end restraints and the effective depth of the web to be considered are as below. For the top flanges,

1. Restrained against lateral deflection and rotation, the effective depth = $d_1/2$ (Fig. (a))
2. Restrained against lateral deflection but not against rotation, the effective depth = (2/3) d_1 (Fig. (b))
3. Restrained against rotation but not against lateral deflection, effective depth = d_1 (Fig. (c))
4. Not restrained against rotation and lateral deflection, the effective depth = $2d_1$ (Fig. (d))



Buckling failure from vertical loading



Consider a column of unit thickness and of length equal to $d_1\sqrt{2}$ (as inclined at 45° NA). The effective length will be $(d_1\sqrt{2})/2$, if the ends are assumed to be fixed, whereas for pin connected ends, the effective length will be $d_1\sqrt{2}$. Assuming the ends to be 100% fixed or pin connected cannot be justified. Therefore, for an exact analysis, some constant will have to be introduced. For a simplified solution, ends are assumed to be fixed.

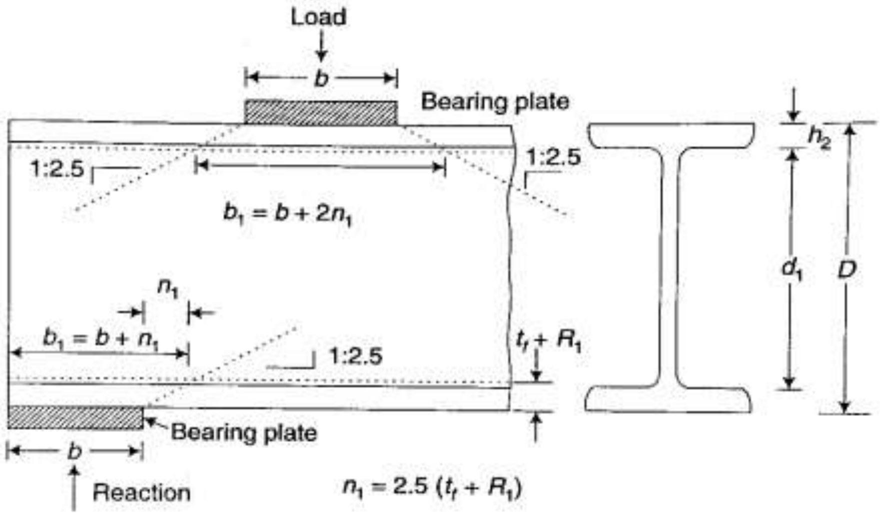
$$\text{Effective length} = \frac{d_1\sqrt{2}}{2} = \frac{d_1}{\sqrt{2}}$$

Minimum radius of gyration = $\frac{t}{\sqrt{12}}$

Slenderness ratio, $\lambda = \frac{d_1/\sqrt{2}}{t/\sqrt{12}} = \frac{d_1\sqrt{6}}{t} = 2.45\,\frac{d_1}{t}$

WEB CRIPPLING

Loads and reactions concentrated along a short length of flange of beam are resisted by compressive stresses in the web which vary with distance form the load. The webs of rolled steel sections are, therefore, subjected to a large amount of stresses just below the concentrated loads and above the reactions from the support. Stress concentration occurs at the junction of the web and the flange. As a result, large bearing stresses are developed below the concentrated loads. Consequently, the web near the portion of the stress concentration tends to fold over the flange. This type of local buckling phenomenon is called crippling or crimpling of the web (Fig.). Web crippling is therefore buckling of the web caused by the compressive force delivered through the at the load points and are connected to the web so as to transfer the force to it gradually, not abruptly. The other is to make the web thicker. Since web crippling is a problem at only a few sections of a beam, it is economical to provide bearing stiffeners at these sections than to increase the thickness of the entire web



Bearing length

Note

- Out of buckling and crippling of the web, it has been found that if the beam section is safe in crippling, it will be safe for buckling too.

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Qu5 Which of the following is true about sections with high shear case V>0.6V_d?

a) Web area is ineffective

b) Web area is fully effective

c) Flanges will not resist moment

d) Moment is not reduced

TEST YOUR SELF

Qu6 The design bending strength of laterally supported beams is governed by

a) Torsion

b) Bending

c) Lateral torsional buckling

d) Yield stress

Qu7 Shear lag effect depends on

a) Material of beam

b) Width of beam only

c) Width-to-span ratio

d) Cost

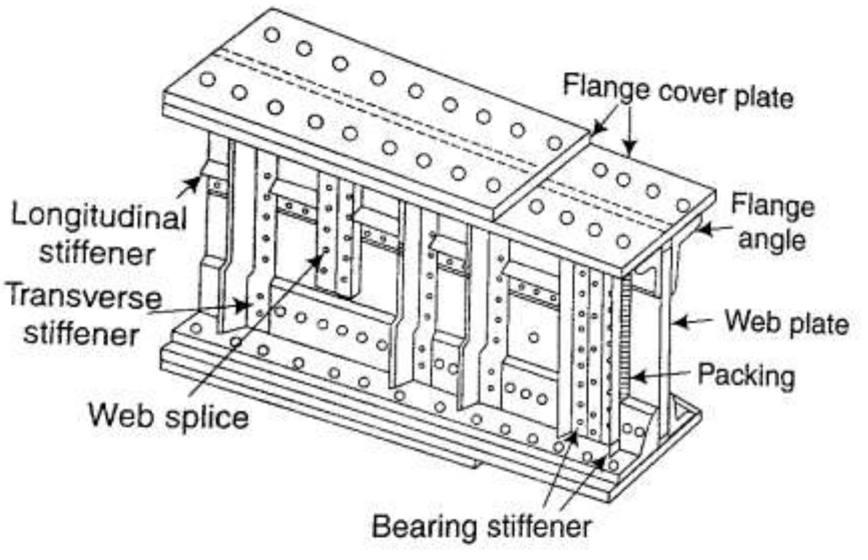
Answer

1-(a), 2-(c), 3-(c), 4-(b), 5-(a), 6-(d), 7-(c)

ELEMENTS OF PLATE GIRDERS

The following are the elements of a typical plate girder:

1. Web
2. Flanges
3. Stiffeners.



Bolted plate girder

Webs of required depth and thickness are provided to:

(a) Keep flange plates at required distances

(b) Resist the shear in the beam.

Flanges of required width and thickness are provided to resist bending moment acting on the beam by developing compressive force in one flange and tensile force in another flange.

Stiffeners are provided to safeguard the web against local buckling failure. The stiffeners provided may be classified as:

(a) Transverse (vertical) stiffeners and

(b) Longitudinal (horizontal) stiffeners.

Where, w - Factored self weight

and W - Total factored load on the girder.

Considering this value of self weight and the other applied loads, moment M and shear force F to be considered for the design is found.

ECONOMICAL DEPTH

Assuming that the moment M is carried out by flanges only, the economical depth 'd' of girder may be found as given below:

$$\mathbf{M = f_y\, b_f\, t_f\, d}$$

Where, b_f and t_f are the breadth and thickness of the flange.

$$\therefore \mathbf{b_f\, t_f = \frac{M}{f_y d}}$$

The gross sectional area of the girder

A is given by

$$\mathbf{A = 2\, b_f\, t_f + d\, t_w}$$

$$\mathbf{= \frac{2M}{f_y d} + d\, t_w}$$

Taking $\frac{d}{t_w}$ = k, where k is assumed constant, then,

$$\mathbf{A = \frac{2M}{f_y d} + \frac{1}{k} d^2}$$

For A to be minimum, the above expression is to be differential w.r.t. ‘d' and equated to zero. Hence

$$\mathbf{0 = -\frac{2M}{f_y d^2} + \frac{1}{k} 2d}$$

or $$\mathbf{d^3 = \frac{Mk}{f_y}}$$

(c) When transverse stiffeners and longitudinal stiffeners at one level only are provided (0.2 d from compression flange)

1. when 2.4 d ≥ c ≥ d

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

2. when 0.74 d ≤ c ≤ d

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

3. when c < 0.74 d

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

(d) When a second longitudinal stiffeners (located at neutral axis is provided)

$$\frac{d}{t_w} \leq 400\ \epsilon_w.$$

III. Minimum web thickness based on compression flange buckling requirement (clause 8.6.1.2 in IS: 800):

In order to avoid buckling of the compression flange into the web, the web thickness shall satisfy the following:

(a) When transverse stiffeners are not provided

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

Where, ϵ_f = yield stress ratio of flange

$$= \sqrt{\frac{250}{f_{yf}}}$$

(b) When transverse stiffeners are provided and

1. When c ≥ 1.5 d

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

Where, D = depth of girder (including flange thickness)

and L = equivalent span of the girder.

SIZE OF FLANGES

Assuming moment is resisted by flanges only, and using material partial safety factor for a plastic section,

$$\frac{A_f \times f_y \times d}{1.1} = M$$

Hence area of flange A_f may be found. Select $9.4\epsilon < t_f < 13.6\ b_f\ \epsilon$ so that bending strength can be found by the formula for semi compact section as per the clause 8.2.1.2 in IS: 800. Thus

$$b_f\, t_f = A_f$$

$$\text{i.e., } 13.6\ \epsilon\ t_f^2 = A_f$$

Hence t_f is found. Then $b_f = \frac{A_f}{t_f}$

SHEAR BUCKLING RESISTANCE OF WEB

For thin webs, it is necessary to check the shear resistance of web for buckling. IS 800-2007, clause 8.4.2 specify that this check is necessary when;

$$\frac{d}{t_w} > 67\ \epsilon \text{ for a web without stiffeners, and}$$

$$> 67\ \epsilon \sqrt{\frac{K_v}{5.35}} \text{ for a web with stiffeners}$$

Where, K_v = 5.35 when transverse stiffeners are provided at support

$$= 4.0 + \frac{5.35}{\left(\frac{c}{d}\right)^2} \text{ for } \frac{c}{d} < 1.0$$

$$= 5.35 + \frac{4.0}{\left(\frac{c}{d}\right)^2} \text{ for } \frac{c}{d} \geq 1.0$$

The Poisson's ratio μ for steel may be taken as 0.3.

DESIGN OF CONNECTION BETWEEN FLANGE AND WEB PLATES

If 'V' is the shear force acting on the section, then shear stress at the junction is,

$$q_w = \frac{V}{b\, I_z}(a\,\bar{y})$$

$$\therefore \textbf{Shear force per unit length} = \frac{V}{I_z}(a\,\bar{y})$$

If weld of throat thickness 't' is provided on both side, then strength of shop weld per unit length

$$= 2t\,\frac{fw}{\sqrt{3}} \times \frac{1}{1.25}$$

Equating the force to strength we get

$$\frac{V}{I_z}(a\,\bar{y}) = 2t\,\frac{fw}{\sqrt{3}} \times \frac{1}{1.25}$$

Hence throat thickness of weld 't' can be found, from which size of the weld is obtained as $s = \frac{t}{0.7}$.

In finding shear stress, moment of inertia of flange alone may be considered i.e.

$$I_z = \frac{b_f d^3}{12}$$

If weld size comes out too small intermittent welding may be adopted.

BUCKLING RESISTANCE OF STIFFENERS

For this purpose, the effective section is full area or core area of the stiffener together with an effective length of web on each side as shown in Fig. It may be noted that some time (in case of end stiffener) there may not be web on one side of stiffener or, it may

CHECK FOR TORSIONAL STIFFENERS

After erecting ends of plate girder may have lateral restraint. But during transportation and erection, it may not have lateral restraint. Hence it is necessary to check for torsional restraint of the plate girder. Clause 8.7.9 in IS code specifies that

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

where I_s = second moment of area of the stiffener section about central line of web.

$$\alpha_s = 0.006 \text{ for } \frac{L_{LT}}{r_y} \leq 0$$

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

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

L_{LT} = Effective length for lateral torsional buckling

D = Overall depth of beam at support

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

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

PROCEDURE OF DESIGN OF PLATE GIRDER

1. Assuming self weight is equal to $\frac{W}{200}$, where W is total factored load, determine the factored shear force and moment.

2. Decide whether to use or not to use transverse stiffeners, and assume the value of k i.e., $\frac{d}{t_w}$ Determine economical depth as

$$d = \left[\frac{Mk}{f_y}\right]^{1/3}$$

select available plate around this depth.

$$= 4233.6 \text{ kN-m.}$$

Maximum shear force = End reaction

$$V = \frac{wL}{2} = \frac{58.8 \times 24}{2}$$

$$= 705.6 \text{ kN}$$

2. Depth of web plate:

If stiffeners are to be avoided,

$$k = \frac{d}{t_w} \leq 67$$

∴ Economical depth of web

$$d = \sqrt[3]{\frac{Mk}{f_y}} = \left(\frac{4233.6 \times 10^6 \times 67}{250}\right)^{1/3}$$

$$= 1043 \text{ mm.}$$

Use 1000 mm plates.

$$t_w \geq \frac{1000}{67} \text{ i.e., } t_w \geq 14.92$$

Select t_w = 16 mm.

Thus web plate selected is 1000 mm × 16 mm.

3. Selection of Flange:

Neglecting the moment capacity of web, area of flange required is

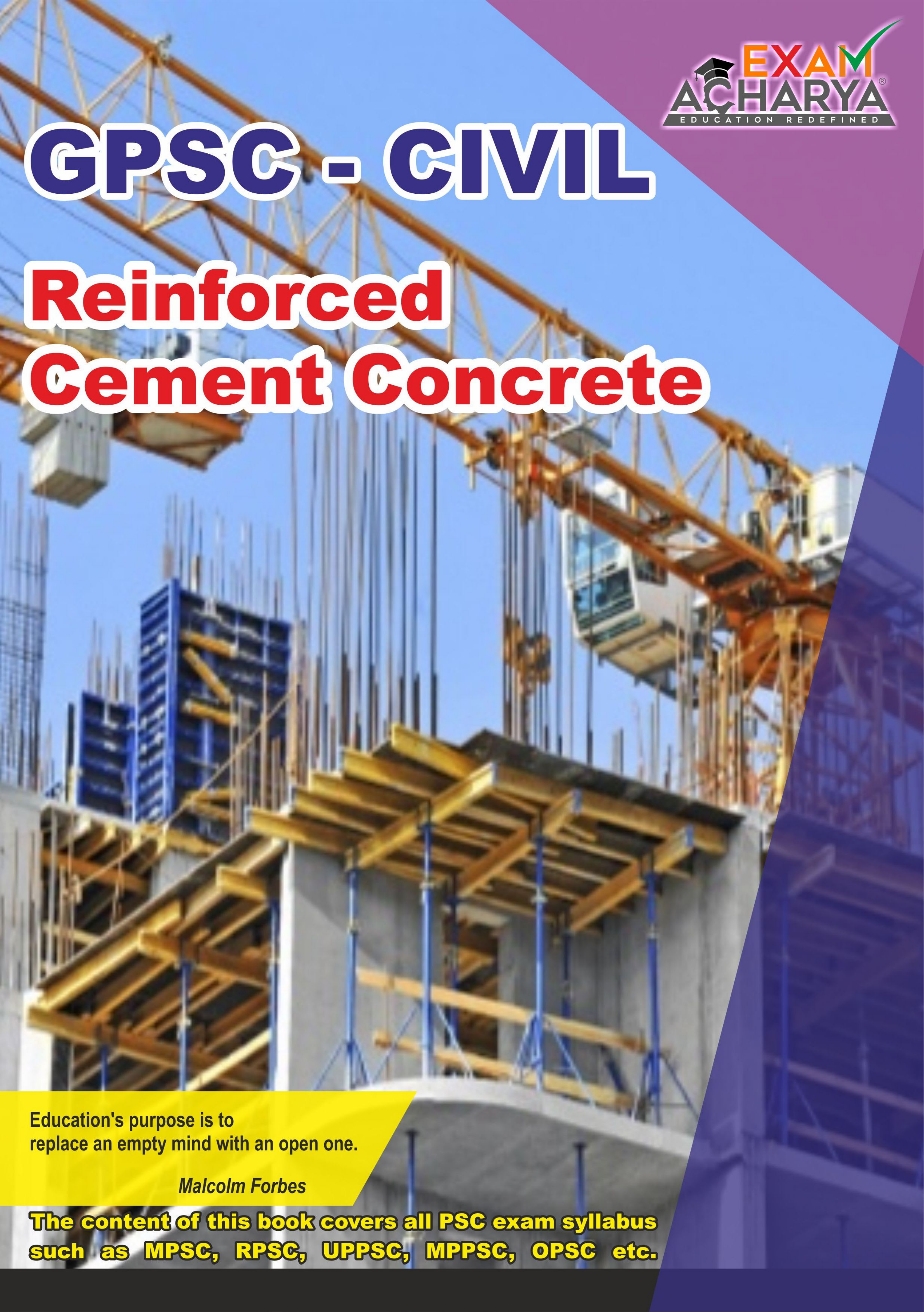
$$\frac{A_f f_y d}{1.1} \geq M$$

$$\frac{A_f \times f_y d \times 1000}{1.1} \geq 4233.6 \times 10^6$$

$$\therefore A_f = 18628 \text{ mm}^2$$

To keep the flange in plastic category $\frac{b}{t_f} \leq 8.4 \therefore \frac{b_f}{2t_f} \leq 8.4$

Assuming $t_f = 16.8\ A_f$



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Reinforced Cement Concrete

Education's purpose is to replace an empty mind with an open one.

Malcolm Forbes

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$$= 4714.77 \text{ kN-m} > M$$

Hence section is adequate.

5. Shear resistance of web

$$V_d = \frac{V_n}{\gamma_{m0}} = \frac{A_v f_{yw}}{\gamma_{m0}\sqrt{3}} = \frac{d\, t_w\, f_{yw}}{\gamma_{m0}\sqrt{3}}$$

$$\therefore V_d = \frac{1000 \times 16 \times 250}{1.1\sqrt{3}} = 2099.455 \times 10^3 \text{ N}$$

$$= 2099.455 > 705.6 \text{ kN}$$

Hence section is adequate.

No stiffeners are required.

6. Check for end bearing:

Bearing strength of web

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

Assuming that the width of support is 200 mm, minimum stiff bearing provided by support = 100 mm.

Dispersion length $n_2 = 2.5 \times 40 = 100$ mm

$$F_w = (100 + 100) \times 16 \times \frac{250}{1.1} = 727 \times 10^3 \text{ N} = 727 \text{ kN.}$$

$$> 705.6 \text{ kN}$$

Hence adequate.

End stiffener is also not required.

7. Design of weld connecting web plate and flange:

Maximum shear force = 705.6 kN.

Shear stress in flange at the level of junction of web and flange

$$q = \frac{F}{bI}(a\bar{y})$$

CLEAR YOUR CONCEPT

Qu1 A plate girder is used when

a) Span is large and loads are heavy

b) Span is small and loads are heavy

c) Span is small and loads are light

d) Span is large and loads are light

Qu2 Bending resistance of plate girders can be increased by

a) Decreasing distance between flanges

b) Increasing distance between flanges

c) Reducing distance between flanges to half

d) Bending resistance cannot be increased

Qu3 The modes of failure of plate girder are

a) By yielding of compression flange only

b) By buckling of tension flange only

c) By yielding of tension flange and buckling of compression flange

d) By yielding of compression flange and buckling of tension flange

Qu4 Which of the following causes web buckling in plate girder?

a) Diagonal tension

b) Diagonal compression

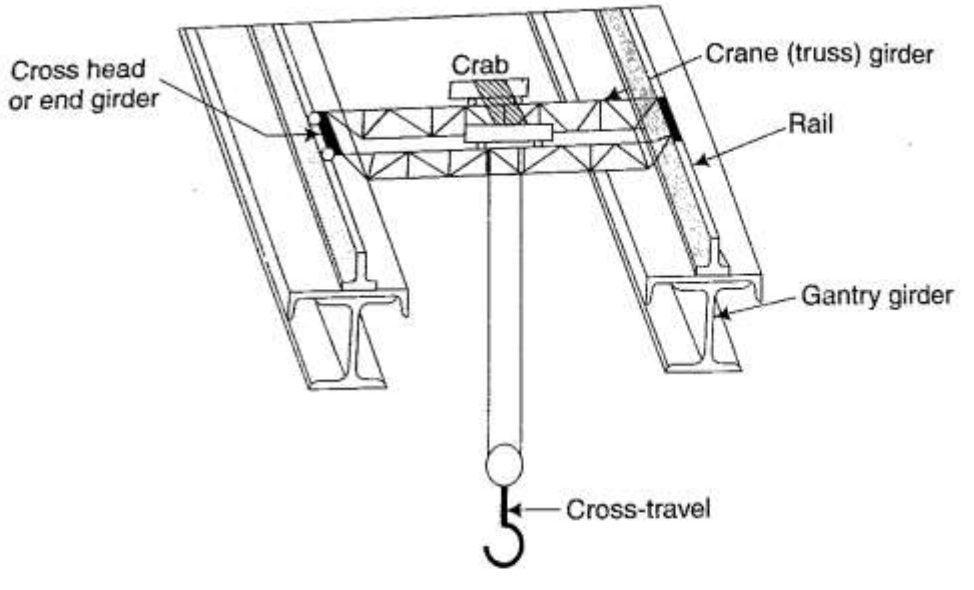
c) Diagonal tension and diagonal compression

d) Neither diagonal tension nor diagonal compression

CHAPTER – 9

GANTRY GIRDER

GANTRY GIRDER AND THEIR USE



Gantry and truss girder arrangement

- Gantry girders are overhead travelling cranes which transport or rather shift heavy machinery, equipment and other heavy jobs from one place to another in an industrial installation.
- These gantry cranes may either be manually operated overhead travelling crane (MOT) or the electrically operated overhead travelling crane (EOT). Fig. shows a typical gantry girder with its accessories.
- A crane consists of a bridge made up of two parallel trusses. This bridge is called as crane bridge, crane girder or the crab girder which spans across the bay of the shop/industrial installation and moves longitudinally.
- This longitudinal movement is assisted by the provision of wheels in the crane girder. These wheels roll on rails which are placed centrally on the gantry girder. These gantry girders are laterally unsupported and are thus designed as laterally unsupported beams.

Fig. shows the various forces that act on a girder and are summarized below:

(a) Crane girder reaction acting vertically downwards.

(b) Longitudinal thrust due to starting and stopping of the crane acting in the longitudinal direction.

(c) Lateral thrust due to starting and stopping of the crab acting horizontally.

Vertical Loads

- Reaction from the crane girder composing of sell-weight of the crane, self weight of crab and the crane capacity constitute vertical load on the gantry girder.
- The wheels attached to the crane girder transfer this vertical load to the gantry girder.
- Thus design reaction is computed from the maximum crane wheel load and it occurs when the crane is nearest to the gantry girder. Self-weight of the rail also constitute the vertical load.

Lateral Loads/ Surge Loads

Lateral loads on crane girders are caused due to the following:

(a) Sudden stopping of the crab and the load when traversing the carb girder.

(b) Crab transferring weights across the shop floor.

The lateral load is assumed to act in the plane of center of gravity of the upper flange. Now this lateral force has a lever arm that produces torque which is quite small and can be neglected safely. It is assumed that tensile flange of the gantry girder does not offer any resistance to the lateral loads.

Longitudinal Loads/ Drag Loads

- Starting and stopping of the crane girders produce longitudinal forces on the gantry girders.

SPECIFICATIONS

1. A gantry girder section is subjected to vertical loads and horizontal thrust simultaneously. Therefore, the allowable stresses are increased by 10%. This increase in the allowable stress is not in addition to that allowed for erection loads with or without wind or seismic forces.
2. Either of the two horizontal forces should be considered to act along with the vertical loads at a time.
3. The vertical deflection of a gantry girder should not exceed the values specified below:

   (i) Where the cranes are manually operated — L/500

   (ii) Where the cranes are travelling overhead and operated electrically up to 500 kN — L/750

   (iii) Where the cranes are traveling overhead and operated electrically over 500 kN — L/1000

   (iv) Other moving loads, such as charging cars, etc. — L/600

   Where, L = span of gantry girder

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PROCEDURE FOR THE DESIGN OF GANTRY GIRDERS

- In case of rolled sections subjected to lateral loads, the suitable section is arrived at by trial and error procedure.
- However, in the design of gantry girders, the lateral load is assumed to be resisted entirely by the compression flange provided the lateral load is applied at the level of compression flange.

Step-5. Determination of required plastic section modulus.

After having determined the maximum bending moment, determine the plastic section modulus required as:

$$\mathbf{M_p = Z_p f_y}$$

Where $$\mathbf{Z_p = A\bar{y}}$$

Now since the gantry girders are laterally unsupported and thus a trial section can be taken as 40 to 50% an excess of M_u/f_y.

In gantry girders, generally an l-section with channel section at the top (i.e. on the compression flange) is provided. The economical depth of gantry girder is not less than 1/12th of the span and the width should be between 1/40 to 1/50 of the span in order to ensure that excessive lateral deflection is not there.

Based on above, select a suitable section from IS Handbook No. 1.

Step-6. Classification of section.

In general the girder section should be plastic i.e. both the flanges and the web should be plastic.

Step-7. Determination of moment capacity of the girder.

In the event when lateral support is provided throughout the compression flange level of the gantry girder by providing catwalk etc., the trial section must be checked for the moment capacity which must exceed the maximum bending moment coming on the girder. This check is made as per the following expression,

$$\mathbf{M_{dz} = Z_{pz}\frac{f_y}{\gamma_{m0}} \leq 1.2\,\frac{Z_{ez}f_y}{\gamma_{m0}}}$$

Apart from that, the top flange should also be checked for bending about both the axes by using the following interaction expression,

Buckling resistance = $(b_1 + 2n_1)t_w\, f_{cd}$ > Maximum wheel load

Where,

b_1 = Wheel diameter

n_1 = Dispersion length under the wheel with angle of dispersion being assumed as 45°

Step-11. Check for bearing stiffeners.

Check the girder section for bearing stiffeners and provide the same if required.

Step-12. Connection design.

Design the rivets/bolts or the welds that connects the channel section to the I-section.

Step-13. Deflection check.

Check the deflection of the gantry girder under service loads at a position where the center of gravity of wheel loads coincides with the mid-span. This deflection is given by.

$$\delta_{cal} = WL^3 \frac{\left(\frac{3a}{4L} - \frac{a^3}{L^3}\right)}{6EI}, \text{ where} \qquad L = \text{Span of the girder}$$

$$a = \frac{L-c}{2}, \qquad c = \text{Wheel base}$$

The calculated deflection must be less than the permissible deflection as given in Table.

Step-14. Check the girder for fatigue strength

Step-15. Design the suitable bracket if required.

Design the bracket and its connection with the column. For bolted connections, no slip bolts are preferred. A pair of bracket plates, one on each flange of the l-section column connected with a diaphragm is provided to make the seat tor the gantry girder.

Qu5 What is the maximum vertical deflection allowed for a gantry girder where the cranes are manually operated?

a) L/500

b) L/700

c) L/600

d) L/800

Answer

1-(b), 2-(d), 3-(a), 4-(c), 5-(a)

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Surveying

The best Brains of the Nation may be
found on the last Benches of the Classroom.

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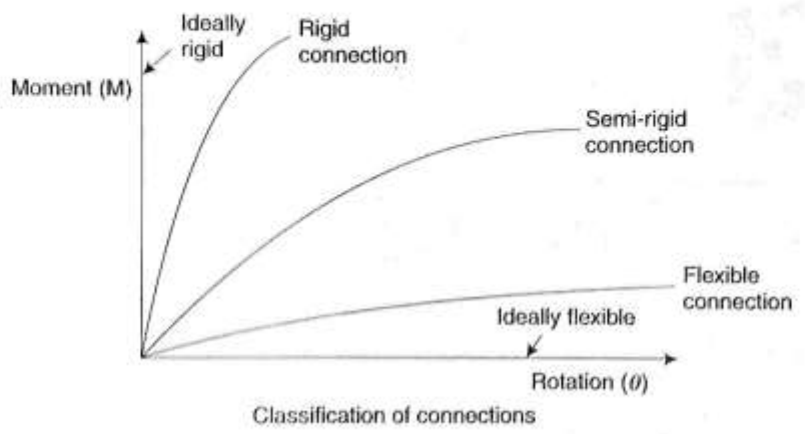
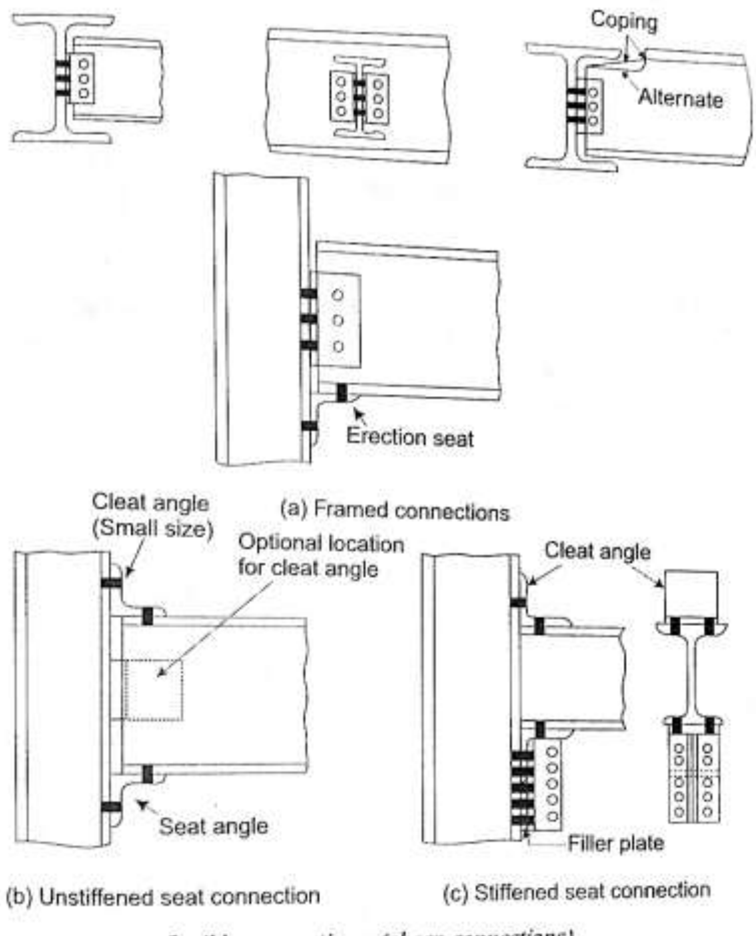


Theoretical connection characteristics



Some flexible connections (shear connections)

- The bolts in this type of connection are subjected to shear and torsion/bending due to eccentric shear.
- When the bolts are subjected to direct shear and torque due to shear then the connection is classed as bracket connection type-l (Fig. (a)).
- When the bolts are subjected to shear and tension then the connection is called as bracket connection type-II (Fig. (b)).
- There are two different approaches for analyzing the bracket connection type-l viz. the elastic method and the ultimate strength method.
- In the elastic method, the friction between the connected parts i.e. frictional resistance is altogether Ignored and the connected parts are assumed to be perfectly rigid with connectors being perfectly elastic. The elastic method of analysis gives very conservative results.
- The second approach viz. the ultimate strength method gives results that are more close to the realistic values but this method is quite difficult to apply in routine designs. IS: 800 does not specify any particular method for the analysis of bracket connection type - I and thus it depends on the discretion of the designer to use any of the analysis method.

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Bracket Connection-Type I

When twisting moment is in the plane of connection the connection may be termed as a bracket connection-type I (Fig. (a)). This situation may arise when the line of action of load is in the plane of the bolted connection and the centre of gravity of the connection (elastic method) or the instantaneous centre (ultimate method) is the centre of rotation. The bolt group is subjected to shear and torsion.

$$= \mathbf{K \Sigma r^2}$$

$$= \frac{\mathbf{F_2}}{\mathbf{r}} \mathbf{\Sigma r^2}$$

The resisting torque should be equal to the torque over the connection. Hence,

$$\mathbf{M} = \frac{\mathbf{F_2}}{\mathbf{r}} \mathbf{\Sigma r^2}$$

$$\text{or} \quad \mathbf{Pe_0} = \frac{\mathbf{F_2}}{\mathbf{r}} \mathbf{\Sigma r^2}$$

$$\text{or} \quad \mathbf{F_2} = \frac{\mathbf{Pe_0 r}}{\mathbf{\Sigma r^2}}$$

Force F_2 is maximum when distance r is maximum. Let the distance of the extreme bolt be r_n. Then,

$$\mathbf{F_2} = \frac{\mathbf{Pe_0 r_n}}{\mathbf{\Sigma r^2}}$$

The two forces F_1 and F_2 act at some angle on various bolts in the connection. Let θ be the angle between these forces on the critical bolt. Then the resultant force F on the critical bolt will be

$$\mathbf{F} = \sqrt{\mathbf{F_1^2 + F_1^2 + 2F_1F_2 \cos\theta}}$$

For the connection to be safe, this force F must be less than the strength of the bolt.

M = Torque in Nmm (caused by the eccentric load)

= Load × eccentricity

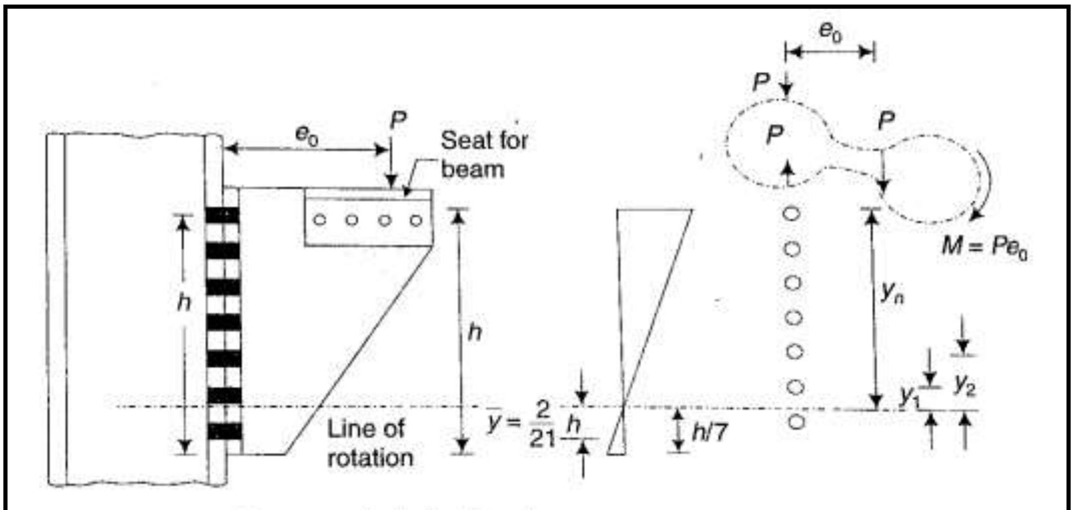
e_0 = The perpendicular distance measured from the centre of rotation of the bolt group to the line of action of load, called eccentricity

P = The eccentric load acting over the joint in N

$r_1, r_2, r_3, \ldots r_n$ = The distances of the bolts from the centre of rotation of the bolt group

Bolted Bracket Connection Type-II

- Here moment is in the plane normal to the plane of connection.
- The line of action of load does not lie in the plane of bolt group. Also the line of rotation does not pass through C.G. of the bolt group.



Forces on bolts in bracket connection-type II

The forces acting on bolts are

(i) Direct shear

(ii) Tension due to moment

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Now force is shared equally by all the bolts thus,

$$\mathbf{F = \frac{P}{n}}$$

Bolts above the line of rotation are in tension and bracket section below the line of rotation will be in compression. It is assumed that line of rotation lies at (h/7) from the bottom of bracket where,

h = Distance from bottom of bracket to top most bolt in the connection

Thus bolts above line of rotation will experience direct shear and tension due to moment. Bracket below the line of rotation provides necessary compression.

$\Rightarrow \qquad M = M' + \left(\frac{M'\Sigma y_i}{\Sigma y_i^2}\right)\bar{y}$

$\Rightarrow \qquad M = M'\left(1 + \frac{\Sigma y_i}{\Sigma y_i^2} \times \frac{2h}{2l}\right) \qquad \left(\because \bar{y} = \frac{2h}{2l}\right)$

$\Rightarrow \qquad M' = \frac{M}{1+\frac{2h}{2l}\times\frac{\Sigma y_i}{\Sigma y_i^2}}$

Where, $M = Pe_0$ (due to eccentric load P)

e_0 = eccentricity of load P from the plane of bolt group.

Procedure for the Design of Bolted Bracket Connection Type-II

Step-1. Assume bolt diameter, pitch and edge distance

Step-2. Compute strength of the bolt

Step-3. Compute number of bolts required

Srep-4. Compute shear force F in the extreme bolt (= P/r)

Step-5. Compute tensile force T in the extreme bolt

Step-6. Check the connection for collective effect of shear and tension as:

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

Where,

V_{sb} = Shear force in the bolt

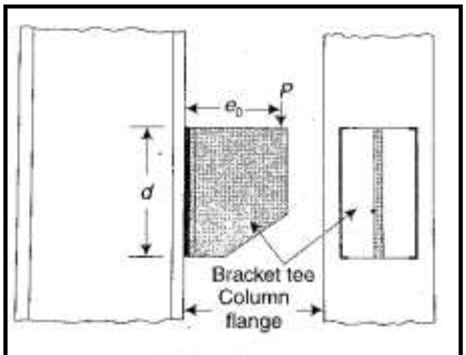
T_b = Tensile force in the bolt

V_{dsb} = Design shear strength of bolt

T_{db} = Design tensile strength of bolt

Procedure for the Design of Welded Bracket Connection Type-I

Step-1. Assume overlap of bracket plate on the column flange and also the weld size.



Welded bracket connection-type II

Step-2. Compute distance of centroid of weld group ($\bar{x}$)

Step-3. Compute polar moment of inertia of the weld group (I_p).

Step-4. Compute distance 'r' of the extreme weld from C.G. of weld group.

Step-5. Shear stress 'q_1' and 'q_2' are computed

Step-6. Resultant stress (q) is computed and $q \le \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$

Step-7. Compute weld size as t_t = KS where t_t is the throat thickness and S is the size of weld.

Welded Bracket Connection Type-II

- In this connection, the moment is in a plane normal to the plane of weld i.e. the CG of the weld group lies in a plane normal to the plane of line of action of load (external) and the weld group is acted upon by direct shear and bending as shown in Fig.
- Here the shear stresses and maximum bending moment occur at different locations in the joint. So combining these two stresses at a point is not possible instead these stresses are checked individually.

Combined stress in the weld

As per Cl. 10.5.10.1.1 of IS: 800-2007 for fillet weld, the equivalent stress (f_e) is given by

$$f_e = \sqrt{f_{a,cal}^2 + 3q_{cal}^2} \leq \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

Where, l_w = Length of the weld connecting the bracket plate with column flange generally equal to depth of the bracket plate

For butt weld, the equivalent stress is given by,

$$f_e = \sqrt{f_{b,cal}^2 + 3q_{cal}^2} \leq \frac{f_y}{\gamma_{m0}}$$

As per Cl. 10.5.10.1.2 of IS: 800-2007 check for the combination of stresses need not be done for:

(a) side fillet welds joining cover plates and flange plates, and

(b) fillet welds where sum of normal and shear stresses does not exceed f_{wd}.

Where, f_{wd} is strength of a fillet weld.

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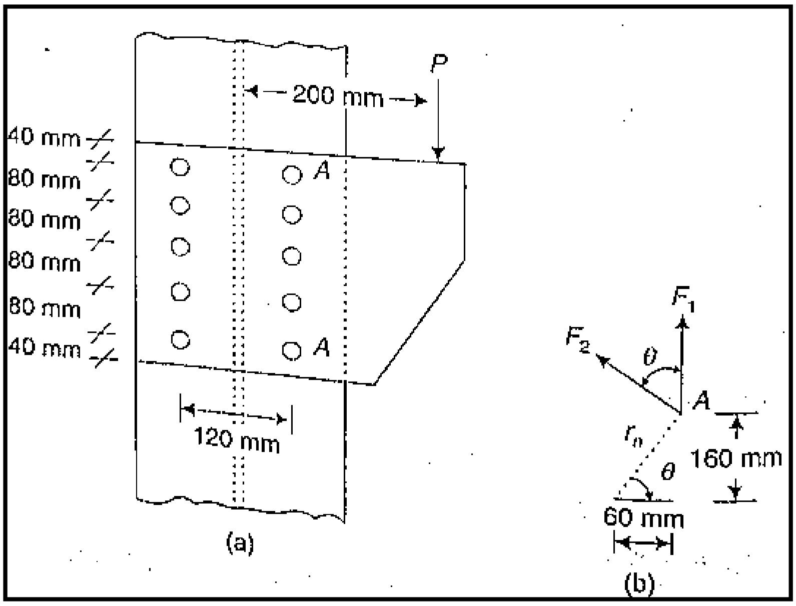
Procedure for the Design of Welded Bracket Connection Type-II

- In the design of bracket connection type-II, the size of the weld is assumed beforehand and length of the weld required is determined.
- If the weld length required is more than twice the depth of the bracket then the weld size assumed earlier needs to be revised.

Step-1 Assume a size of weld and compute throat thickness. Determine the design strength of fillet weld per unit length (f_{wd}). The depth of the bracket is determined as given below.



Solution

For Fe 410 grade of steel: f_u = 410 MPa

For bolts of grade 4.6: f_{ub} = 400 MPa

Partial safety factor for the material of bolt: γ_{mb} = 1.25

A_{nb} = stress area of 20 mm diameter bolt = 245 mm²

Given: diameter of bolt, d = 20 mm; pitch, p = 80 mm; edge distance, e = 40 mm

For d = 20 mm, d_0 = 20 + 2 = 22 mm

Strength of the bolt in single shear,

$$\mathbf{V_{sb} = A_{nb}\frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} = 245 \times \frac{400}{\sqrt{3}\times 1.25} \times 10^{-3} = 45.26\ kN}$$

Strength of the bolt in bearing,

$$\mathbf{V_{pb} = 2.5\ k_b\ dt\ \frac{f_u}{\gamma_{mb}} \qquad (f_u = \text{least of } f_u \text{ and } f_{ub})}$$

For 20 mm diameter bolt the diameter of bolt hole, d_0 = 22 mm

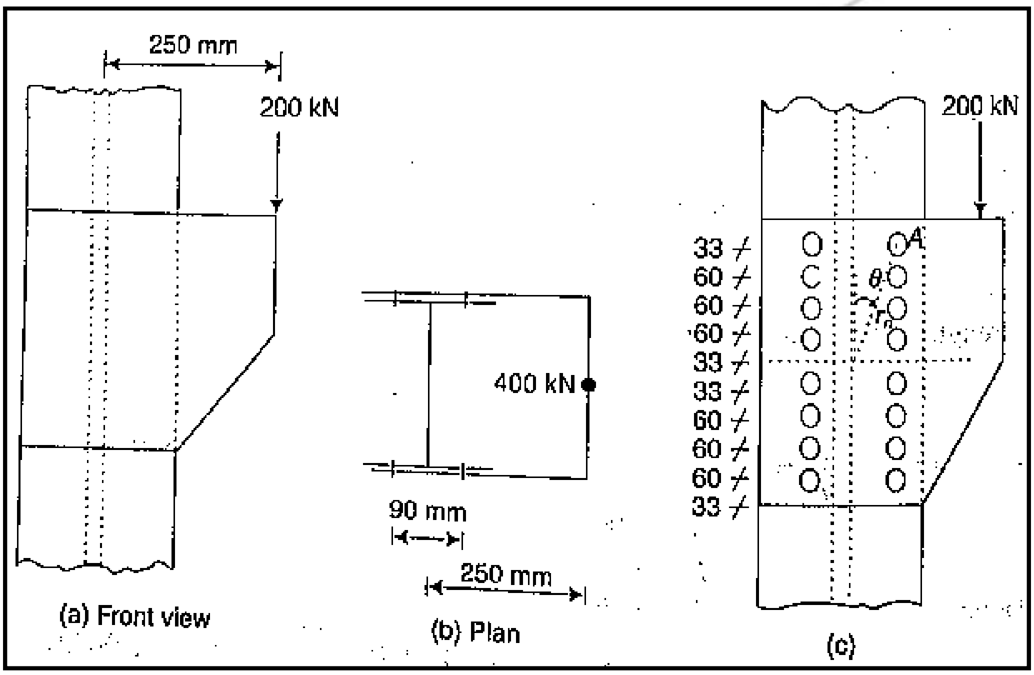
$$45.26 \leq \sqrt{\left(\frac{P_1}{10}\right)^2 + (0.20839\, P_1)^2 + 2 \times \frac{P_1}{10} \times 0.20839\, P_1 \times 0.3511}$$

$$\Rightarrow \quad \mathbf{P_1 = 173.49\ kN}$$

The service load, $P = \frac{P_1}{\text{load factor}} = \frac{173.49}{1.5} = 115.65$ kN.

Example

Design a bolted bracket connection to support an end reaction of 400 kN because of the factored loads supported by the beam. The eccentricity of the end reaction is as shown in Fig. The steel used is of grade Fe 410. Use bolts of grade 4.6. The thickness of bracket plate may be taken as 10 mm. The column section is ISHB 150 @ 300.19 N/m.



Solution

For Fe 410 grade of steel: $f_u = 410$ MPa

For bolts of grade 4.6: $f_{ub} = 400$ N/mm²

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$$V_{pb} = 2.5 \times 0.50 \times 20 \times 9.0 \times \frac{400}{1.25} \times 10^{-3} = 72.00 \text{ kN}$$

Hence, the strength of the bolt, V_{sd} = 45.26 kN

Let us provide bolts in two vertical rows. Number of bolts required in one row,

$$n = \sqrt{\frac{6M}{pn' V_{sd}}} = \sqrt{\frac{6 \times 200 \times 250}{60 \times 2 \times 45.26}} = 7.432 \simeq 8$$

Provide 16 bolts on each bracket plate with 8 bolts in each row.

The critical bolt will be the bolt A shown in Fig. (c).

Force on critical bolt A

The direct force, $F_1 = \frac{P}{n} = \frac{200}{16}$ = 12.5 kN

The force in the bolt due to torque, $F_2 = \frac{Pe_0 r_n}{\Sigma r^2}$

Eccentricity. e_0 = 250 mm

$$r_n = \sqrt{213^2 + 45^2} = 217.70 \text{ mm}$$

$$\Sigma r^2 = 4 \times [(213^2 + 45^2) + (153^2 + 45^2) + (93^2 + 45^2) + (33^2 + 45^2)]$$

$$= 346.464 \text{ mm}^2$$

$$F_2 = \frac{200 \times 250 \times 217.70}{346{,}464} = 31.417 \text{ kN}$$

$$\cos\theta = \frac{45}{\sqrt{213^2 + 45^2}} = 0.206$$

Resultant force on the critical rivet,

$$F = \sqrt{12.5^2 + 31.417^2 + 2 \times 12.5 \times 31.417 \times 0.206} = 36.12 \text{ kN}$$

$$< 45.26 \text{ kN}$$

$$= 1960.2 \times 10^4 \text{ mm}^4$$

$$I_y = \frac{200 \times 4.2^3}{12} + 2 \times \left[\frac{4.2 \times 200^3}{12} + 4.2 \times 200\,(100 - 66.66)^2\right] + 200 \times 4.2 \times 66.66^2$$

$$= 1120.12 \times 10^4 \text{ mm}^4$$

$$I_p = I_x + I_y$$

$$= 1960.2 \times 10^4 + 1120.12 \times 10^4 = 3080.32 \times 10^4 \text{ mm}^4$$

r = distance of extreme point of weld from the C.G. of the weld group

$$= \sqrt{100^2 + (200 - 66.66)^2} = 166.672 \text{ mm}$$

Direct shear stress, $q_1 = \frac{P_1}{(2\times200+200)\times4.2} = 0.0003968\ P_1 \text{ N/mm}^2$

Shear stress due to twisting moment,

$$q_2 = \frac{P_1\times(200-66.66+75)}{3080.32\times10^4} \times 166.672 = 0.0011273\ P_1 \text{ N/mm}^2$$

$$\cos\theta = \frac{200-66.66}{166.672} = 0.80$$

Resultant shear stress,

$$q_R = \sqrt{(0.0003968\ P_1)^2 + (.0011273\ P_1)^2 + 2 \times .0003968\ P_1 \times .0011273\ P_1 \times 0.8}$$

$$= 1.4642 \times 10^{-3}\ P_1$$

Now, the resultant shear stress q_R should be $\leq \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$

Let us assume shop welding, $\gamma_{mw} = 1.25$

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Solid Mechanics

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Solid Mechanics

CHAPTER – 11

PLASTIC ANALYSIS

INTRODUCTION

- Steel has a very unique property i.e. ductility because of which it is able to absorb large deformations beyond the elastic limit without getting fractured.
- Due to this unique property only, it has a reserve of strength beyond its yield point.
- In plastic method of design, the design philosophy is the ultimate strength and thus behavior of members beyond the yield stress in inelastic (or plastic) range is considered.
- The structure fails at a much higher load than working load and that is called as collapse load. The working loads are enhanced by specified factors known as load factors to get the ultimate loads at which the structure collapses. There after the maximum plastic moment is found. The plastic section modulus Z_{pz} is found by dividing the maximum plastic moment by the yield stress. i.e., $Z_{pz} = M_p/f_y$.
- Plastic: The term plastic implies that at failure, parts of the member will be under very large strains so that the member is put into plastic range. When the entire cross section becomes plastic, infinite rotation takes place at constant bending moment and a sort of plastic hinge is formed. After the formation of sufficient number of plastic hinges in the member at the maximum stressed locations, a collapse mechanism develops. Since the working loads are less than the collapse load by a factor of safety called as load factor, the members designed will be safe beyond doubt.

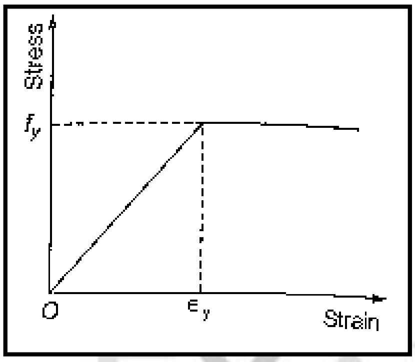
Note
1. The curve is not to scale
2. The curves shaped show actual behavior
3. The values of moments and curvature are typical values

Idealized moment-curvature relationship for an I-section beam

Theory of Plastic Bending

Assumptions:

(a) Plane section remain plane and normal to the axis of bending at all stages. Thus it is assumed that strains are proportional to the distance from the neutral axis.



Idealised Stress-Strain diagram

(b) The stress-strain relationship is idealized to consist of two straight lines as shown in Fig.

(c) The stress in any fiber can be found from its strain according to the stress-section curve idealized as above without reference to other fibers. The shearing strains are neglected.

(d) The deformations are assumed to be small, so the slope of the beam at any point may be assumed to be equal to its tangent.

(e) There is no axial load on the beam.

(f) Steel is ductile, able to deform plastically without fracture.

(g) The properties of steel in compression are assumed to be the same in tension.

(h) The influence of normal and shearing force are neglected.

(i) Strain energy stored due to elastic bending is ignored.

(j) The connections provide full continuity so that plastic moment can be transmitted through them.

Consider any section with one axis of symmetry. The neutral axis lies in the plane containing the symmetrical axis. Under fully plastic conditions, the stress at the section, both in tension and compression zone is assumed to be at yield point of the material as shown in Fig.

Equilibrium Condition-1: Axial thrust is zero at the section.

$$\int_A f_y \, dA = 0$$

$$\Rightarrow \quad f_y (A_2 - A_1) = 0 \Rightarrow A_2 = A_1$$

Hence the neutral axis under plastic condition divides the section into two equal areas.

Remember

- Neutral axis under plastic condition or plastic neutral axis divides the section into two equal areas.

Equilibrium Condition-2: Internal moment is equal to applied moment.

$$\int_A f_y \, y dA = M_p$$

$$\Rightarrow \quad f_y (A_1\bar{y}_1 + A_2\bar{y}_2) = M_p$$

It follows that the plastic moment of the section is given by the yield stress multiplied by the sum of moments of areas in tension and compression zones about the neutral axis.

Now the plastic moment M_p is given by $f_y Z_p$

Where, $Z_p = A_1\bar{y}_1 + A_2\bar{y}_2$

Z_p is termed as the plastic modulus. As plastic neutral axis divides whole area into two equal halfs, so

$$A_1 = A_2 = \frac{A}{2}$$

$$\therefore Z_p = \frac{A}{2} (\bar{y}_1 + \bar{y}_2)$$

RELATIONSHIP AMONG LOAD FACTOR, SHAPE FACTOR AND FACTOR OF SAFETY

$$\textbf{Load Factor} = \frac{\text{Ultimate load}}{\text{Working load}} = \frac{\text{Collapse load}}{\text{Working load}} = \frac{P_u}{P_w}$$

$$\textbf{Shape Factor, f} = \frac{\text{Plastic moment}}{\text{Yield moment}} = \frac{M_p}{M_y} = \frac{f_y Z_{pz}}{f_y Z_{ez}} = \frac{Z_{pz}}{Z_{ez}}$$

$$\textbf{Factor of Safety, (FOS)} = \frac{\text{Yield Stress}}{\text{Permissible or working stress}} = \frac{f_y}{f}$$

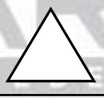
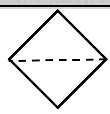
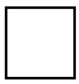
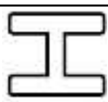
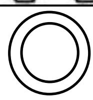
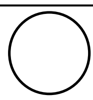
Now, $$\textbf{Load factor} = \frac{P_u}{P_w} = \frac{M_p}{M_y} = \left(\frac{f_y}{f}\right)\frac{Z_{Pz}}{Z_{ez}}$$

⇒ Load factor = Factor of safety × Shape factor

Margin of safety

Margin of safety = FOS – 1

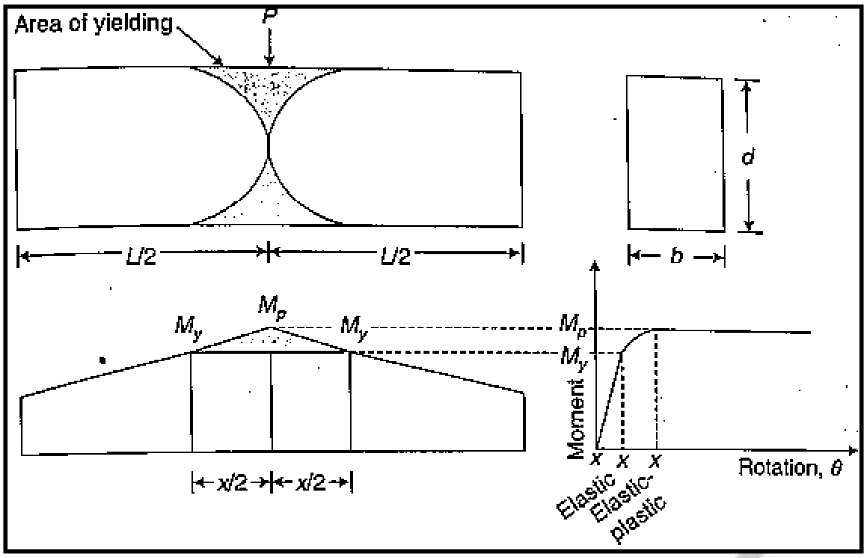
Shape factor for different sectional shape

Section			Shape Factor
1.	Triangular		2.34
2.	Diamond		2.0
3.	Rectangle/Square		1.5
4.	I-section		1.12 to 1.14
5.	H-section		1.5
6.	Circular hollow section		1.27
7.	Circular section		1.67

$$= \frac{2}{3} \times f_y \times Z_p$$

$$= \frac{2}{3} M_p$$

From the bending-moment diagram (Fig.),



Development of plastic hinge and hinge length

$$\frac{M_p}{\frac{L}{2}} = \frac{M_y}{\frac{L}{2} - \frac{x}{2}}$$

$$\frac{(L-x)}{2} M_p = \frac{L}{2} M_y$$

$$(L - x)\ M_p = LM_y$$

$$(L - x)\ M_p = L \times \frac{2}{3} \times M_p$$

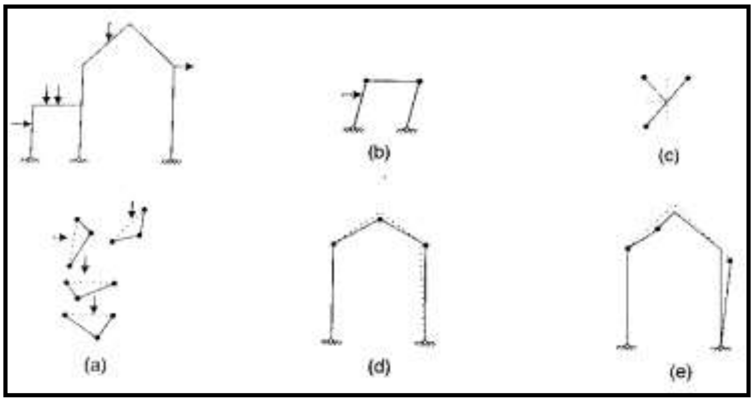
$$(L - x) = \frac{2}{3} L$$

$$x = \frac{L}{3}$$

Therefore the hinge length of the plasticity zone is equal to 1/3rd of the span.

Note

- The hinge length of the plasticity zone for a simple beam subjected to uniformly distributed load is $L/\sqrt{3}$.



(a) **Beam mechanisms-** All the loaded spans behave as beam mechanism.

(b) **Sway mechanism-** It is formed due to lateral loads.

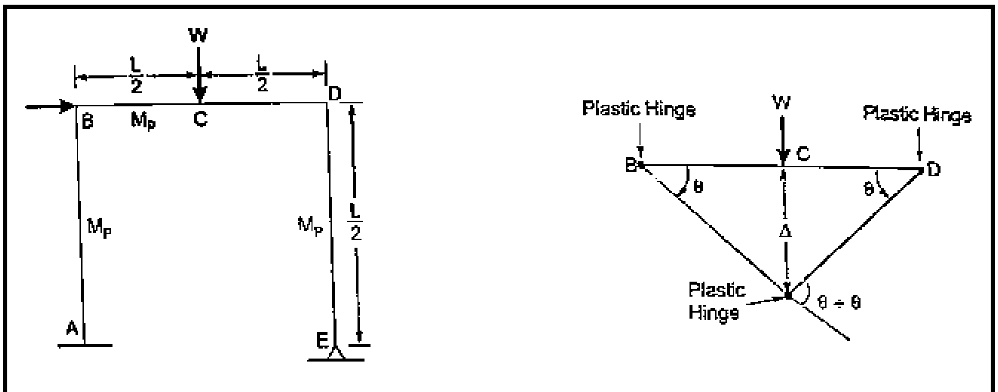
(c) **Joint mechanism-** It is formed due to action of moment. The number of members at the joint should be three or more.

(d) **Gable mechanism-** It is exhibited in gable frames. Columns spread more at the top than that at the base.

(e) **Composite mechanism-** Any of the two independent mechanisms may be combined to form composite mechanism. A beam and gable mechanism are joined to form a gable mechanism in this figure.

Beam Mechanism

All the loaded spans behave as beam mechanism. A portal frame is shown in Fig. The beam and column are of same cross-section.



Beam mechanism Explanation of beam mechanism

The beam mechanism for the portal frame ABCDE is as shown in Fig.

failure or it may be a "complete" combined mechanism in which, the frame is determinate at failure.

In order to obtain the lowest possible ultimate load P_u, combinations are selected in such a way that the external work is minimum or internal work is minimum. The procedure generally is to make combinations involving mechanism motions by as many loads as possible and the elimination of plastic hinges which reduces the internal work.

Number of Independent Mechanism

To avoid any possible omission of a combined mechanism it is essentials to have in advance a clear idea of the number of independent mechanism.

Let N = number of possible plastic hinges

r = number of redundancies

n = possible independent mechanism

Then $\mathbf{n = N - r}$

After getting the number of independent mechanisms all the possible combinations are made in such a way that the external work is minimum or internal work minimum. This is done to obtain the lowest possible load.

CONDITIONS IN PLASTIC ANALYSIS

In the elastic method of design the conditions to be satisfied are the equilibrium condition, the compatibility (or continuity) condition and the moment curvature relation (initial yield or limiting stress condition). The conditions to be satisfied for the plastic methods of analysis are as follows:

1. Equilibrium Condition: All the equilibrium conditions, i.e. summation of all the forces and moments should be equal to zero.
2. Mechanism Condition: The structure at collapse must be capable of deforming as a mechanism due to the formation of plastic hinges, i.e., the ultimate load is reached when a mechanism forms. This is also called the continuity condition.

both the cases. Thus, depending upon the method chosen to obtain a solution to the problem, one obtains either an 'upper limit' (upper bound) below which the correct answer must lie, or a 'lower limit' (lower bound) above which the correct answer lies.

Static or Lower Bound Theorem

For a given frame and loading if there exists any distribution of bending moments throughout the frame which is both safe and statically admissible with a set of loads P, then the value of the load P must be less than or equal to the collapse load P_u ($P \leq P_u$). The static theorem was first suggested by Kist and its proof was given by Gvozder, Greenberg and Prager, and also by Horne.

A load computed on the basis of an assumed equilibrium moment diagram in which the moments are not greater than M_p is less than or at best equal to the true ultimate load. Hence, the static method represents the lower limit to the true ultimate load and has a maximum factor of safety. The static theorem satisfies the equilibrium and yield conditions.

Kinematic or Upper Bound Theorem

For a given frame subjected to a set of loads P, the value of P which is found to correspond to any assumed mechanism, must be either greater than or equal to the collapse load P_u ($P \geq P_u$). A proof of this theorem was established by Gvozder and also by Greenberg and Prager.

METHODS OF ANALYSIS

With the principle of virtual work and the upper and lower bound theorems, a structure can be analysed for its ultimate load by any of the following methods:

1. Static method
2. Kinematic method

For complicated frames, the static method of analysis is more difficult, and finding the correct equilibrium equation becomes illusive. In these cases, the kinematic method is more practical.

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Structural Analysis

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But, all of us have an Equal Opportunity
to Develop our Talents."
A.P.J. Abdul Kalam

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Disadvantages

(a) The fabrication is to be done with ductile steel.

(b) The loads are carried by bending and the effects of axial load and shear force on a member are neglected. The theory of simple plastic analysis can not be applied to trusses, as the members are subjected to axial forces only.

(c) Strength is assumed to be the main criteria.

(d) It is very difficult to obtain the collapse mode it the structure is reasonably complicated.

(e) There is little savings in column design.

(f) It is difficult to design for fatigue.

(g) Lateral bracing requirements are more stringent than the elastic design.

SOME IMPORTANT ASPECTS OF PLASTIC DESIGN AND ELASTIC DESIGN

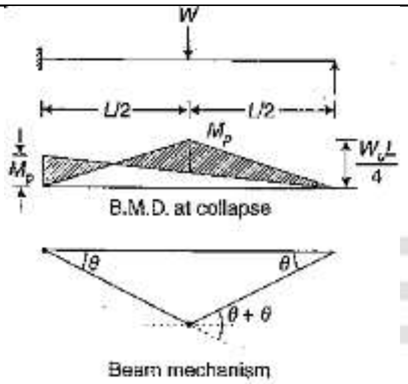
(a) The structure designed by the plastic method is economical. The saving in steel may be about 10-15%.

(b) For indeterminate and complicated structures the plastic method is more conveniently applicable and avoids analytical approximations.

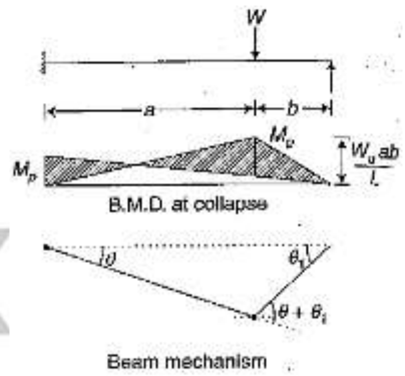
(c) In the elastic method of design, the design process is repeated several times to obtain an optimum section, which consumes more time. The plastic method of design produces a optimum section in a single attempt and thereby, saving the computational time.

(d) The factor of safety is same in both plastic and elastic design of indeterminate structure. The concept of this factor in the plastic design is more realistic.

(e) Stresses produced by settlement, erection etc., can be determined more correctly by the plastic method as the calculations are based on plastic deformations.

<table>
<tr>
<td>Kinematic method
$W_u a\theta = M_p(\theta + \theta_1)$
$= M_p\left(\theta + \frac{a}{b}\theta\right)$
or $W_u a\theta = M_p\,\theta\left(1 + \frac{a}{b}\right)$
$= M_p\,\theta\left(\frac{b+a}{b}\right)$
$W_u a\theta = M_p\,\theta\left(\frac{L}{b}\right)$
or $W_u = M_p\frac{L}{ab}$</td>
<td>Kinematic method
$W_u a\theta = M_p\theta + M_p(\theta + \theta_1) + M_p\theta_1$
Or $W_u a\theta = M_p\theta + M_p\left(\theta + \frac{a}{b}\theta\right) + M_p\frac{a}{b}\theta$
$= 2M_p\theta\left(\frac{a+b}{b}\right)$
$= 2M_p\theta\frac{L}{b}$
or $W_u = \frac{2L}{ab}M_p$
or $W_u = \frac{2M_pL}{ab}$</td>
</tr>
<tr>
<td colspan="2">Concentrated Load</td>
</tr>
<tr>
<td>Propped Cantilever</td>
<td>Propped Cantilever</td>
</tr>
<tr>
<td>

Static method
$\frac{W_uL}{4} = M_p + M_p\frac{L/2}{L}$
or $W_u = \frac{6M_p}{L}$

Kinematic method
$W_u\frac{L}{2}\theta = M_p\theta + M_p(\theta + \theta)$
or $W_u = \frac{6M_p}{L}$</td>
<td>

Static method
$\frac{W_u ab}{L} = M_p + M_p\frac{b}{L}$
or $W_u = \frac{L+b}{ab}M_p$

Kinematic method
$W_u a\theta = M_p\theta + M_p(\theta + \theta_1)$
or $= M_p\theta + M_p\left(\theta + \frac{a}{b}\theta\right)$
or $= \frac{b+(a+b)}{b}M_p\theta$
or $W_u = \frac{L+b}{ab}M_p$</td>
</tr>
</table>

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EXAM ACHARYA EDUCATION REDEFINED

$$I_z = 2 \times \frac{1}{12} b \left(\frac{h}{2}\right)^3 = \frac{bh^3}{48}$$

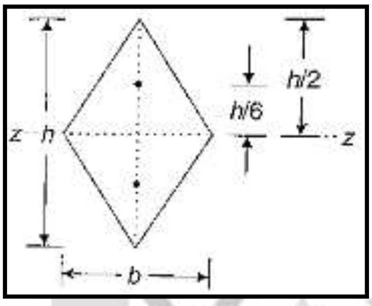
Elastic section modulus,

$$Z_e = \frac{bh^3/48}{h/2} = \frac{bh^2}{24}$$

Plastic section modulus,

$$Z_p = \frac{A}{2}(\bar{y}_1 + \bar{y}_2) = \frac{1}{2} \times \left(\frac{bh}{2}\right) \times \left(\frac{h}{6} + \frac{h}{6}\right) = \frac{bh^2}{12}$$

Shape factor = $\frac{bh^2/12}{bh^2/24}$ = 2

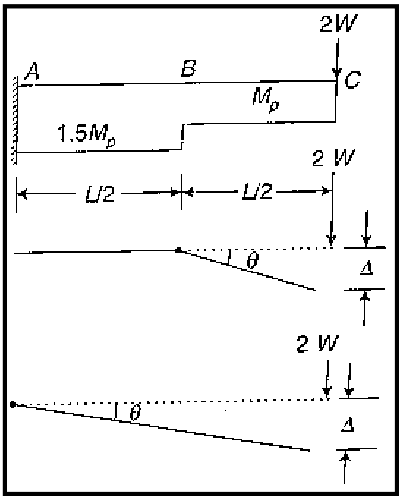


Example

Find out the collapse load for the cantilever shown in Fig.

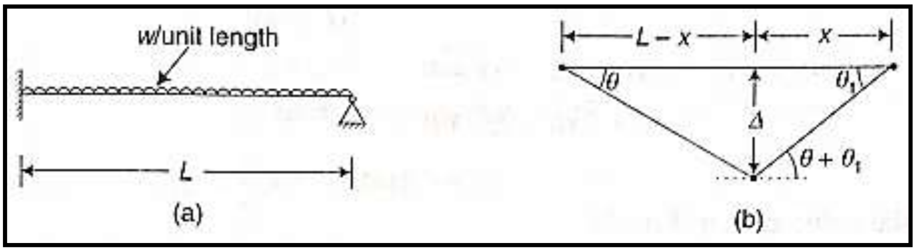
Solution

The beam shown in Fig is of a non-uniform cross section. A plastic hinge can form at the point where the section changes. A plastic hinge can also form at the fixed support.



Example

Find out the collapse load for a propped cantilever subjected to a uniformly distributed load w/unit length, as shown in Fig. Ex. (a).



Solution

For a propped cantilever subjected to uniformly distributed load the plastic hinge does not form at the centre of span as in the case of fixed ended beam, but lies off the centre and towards the propped end. Therefore, it becomes essential to first locate the position of the plastic hinge. Let us assume that it forms at a distance x from the propped end.

External work done in the case of uniformly distributed load is computed by multiplying the load and the average deflection or by multiplying the load/unit length and the area of the deflected shape (mechanism).

From the mechanism,

$$\mathbf{\Delta = (L - x)\,\theta = x\theta_1}$$

or $$\mathbf{\theta_1 = \frac{L-x}{x}\,\theta}$$

External work done = load × average deflection

$$= (w_uL) \times \frac{1}{2}(L - x)\theta$$

$$= \frac{1}{2}w_uL(L - x)\theta$$

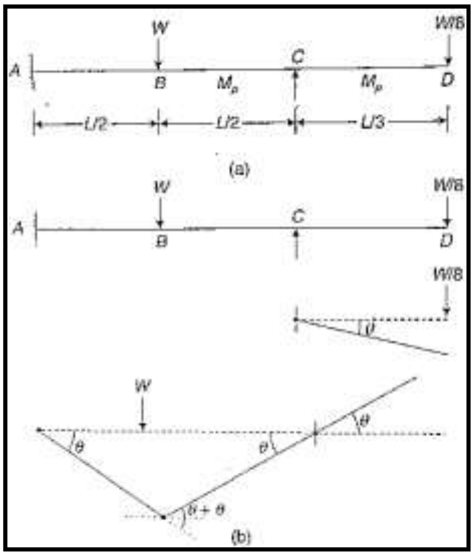
Internal work done = $M_p\theta + M_p\,(\theta + \theta_1)$

$$= M_p\theta + M_p\left(\theta + \frac{L-x}{x}\,\theta\right)$$

$$= M_p\left(1 + 1 + \frac{L-x}{x}\right)\theta$$

Example

A propped cantilever ABCD is loaded as shown in Fig Ex. Find the collapse load if the beam is of uniform cross section.



Solution

Span CD

End C is fixed and end D is free, so a plastic hinge will form at C only.

External work done = load × deflection

$$= \frac{W}{8} \times \frac{L}{3}\,\theta = \frac{WL}{24}\,\theta$$

Internal work done = $M_p\theta$

By the principal of virtual work,

External work done = Internal work done

$$\frac{WL}{24}\,\theta = M_p\theta$$

$$\Rightarrow \qquad W_u = 24\,\frac{M_p}{L}$$

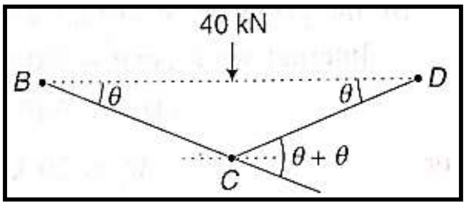
Span AC

Degree of redundancy, $r = 6 - 3 = 3$

Number of possible independent mechanisms, $n = N - r = 5 - 3 = 2$

The two independent mechanisms are:

1. Beam mechanism
2. Sway mechanism



These two independent mechanisms can form a combined mechanism.

Beam mechanism The ends B and D of the beam BD are fixed joints. Therefore, the beam BD acts as a fixed end beam. The possible locations of the plastic hinges are B, D and C (below the concentrated load) as shown in Fig. Ex. (b).

External work done = load × deflection

$$= 40 \times 2 \times \theta = 80\theta$$

Internal work done = moment × rotation

$$= M_p\theta + M_p(\theta + \theta) + M_p\theta = 4M_p\theta$$

By the principle of virtual work done,

Internal work done = External work done,

$$4\,M_p\theta = 80\theta$$

or $M_p = 20$ kNm

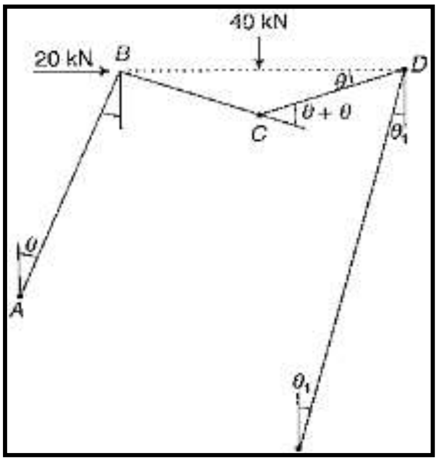
Sway mechanism- The columns AB and ED are of different lengths. They will deflect by the same amount. Therefore, the rotations at the ends of the columns will be different. From Fig. Ex. (c),

$$\Delta = 3\theta = 6\theta_1$$

or $\theta_1 = \theta/2$

or $M_p = \frac{140}{5} = 28$ kNm

The plastic moment for the frame is the maximum of the three plastic moments found. Therefore, the value of the plastic moment is 28 KNm.



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Qu5 The number of independent mechanisms is related to number of possible plastic hinge locations by ________

a) n = h * r

b) n = h / r

c) n = h + r

d) n = h – r

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Qu6 The shape factor in plastic method of analysis depends upon

a) Yield stress of the material

b) Hinge length

c) Geometry of the section

d) Redistribution of moments

Qu7 Which of the following sections has minimum value of shape factor?

a) Tube section

b) I section

c) Rectangular section

d) Circular section

Qu8 The collapse load for a propped cantilever of span l subjected to uniformly distributed load is

a) 0.414 M_p/l

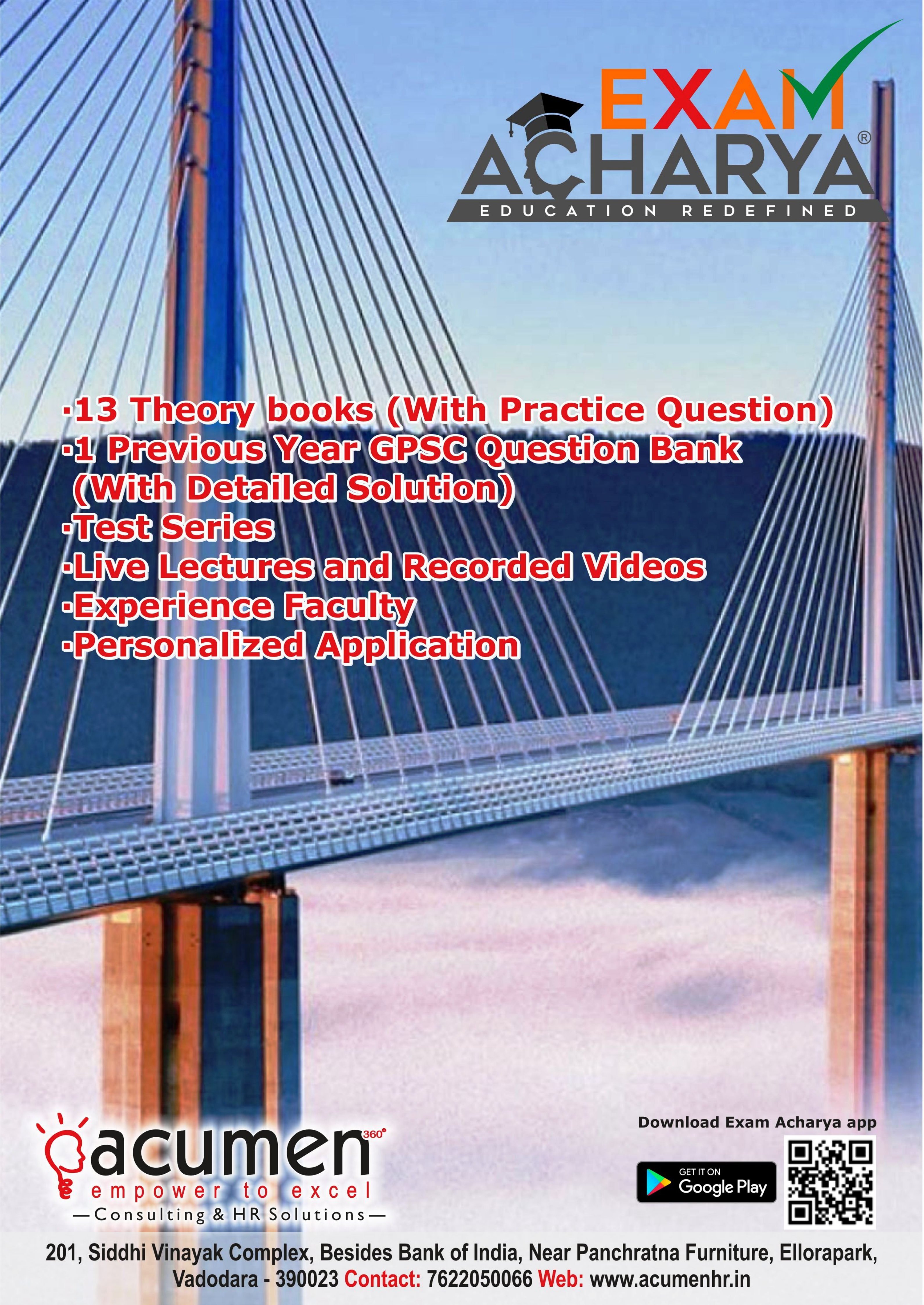
b) 0.586 M_p/l

c) 0.767 M_p/l

d) 11.656 M_p/l

Answer

1-(a), 2-(c), 3-(d), 4-(b), 5-(d), 6-(c), 7-(b), 8-(d)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