INSTITUTE OF AERONAUTICAL ENGINEERING

(Autonomous)

Dundigal, Hyderabad -500 043

CIVIL ENGINEERING

COURSE LECTURE NOTES

Course Name	**DESIGN OF STEEL STRUCTURES AND DRAWING**
Course Code	ACE012
Programme	B.Tech
Semester	VI
Course Coordinator	Dr. Venu M, Professor
Course Faculty	Dr. Vamsi Mohan U
Lecture Numbers	1-60
Topic Covered	All

COURSE OBJECTIVES (COs):

	The course should enable the students to:
I	Discuss the concepts of structural steel design conforming to the IS 800 design code.
II	Identify various types of structural steel and its properties also define concepts of Limit State Design.
III	Analyse structures using plastic method of analysis and evaluate collapse load and plastic moment capacity.
IV	Design compression members, beams, connections and girders.

COURSE LEARNING OUTCOMES (CLOs):

Students, who complete the course, will have demonstrated the ability to do the following:

ACE012.01	Know the materials, making of iron and steel.
ACE012.02	Know the types of structural steel, mechanical properties of steel.
ACE012.03	Know the concepts of plasticity yield strength.
ACE012.04	Understand loads and combinations loading wind loads on roof trusses.
ACE012.05	Understand behavior of steel, local buckling.

ACE012.06	Concept of limit state design – different limit states as per IS 800:2007.
ACE012.07	Concept of design strengths deflection limits, serviceability.
ACE012.08	Evaluate the bolted connections.
ACE012.09	Evaluate welded connections, efficiency of joint.
ACE012.10	Analyze the prying action types of welded joints.
ACE012.11	Understand the design of tension members and design strength of members.
ACE012.12	Understand the design of compression members, buckling class, slenderness ratio.
ACE012.13	Understand the strength design, laced battened columns.
ACE012.14	Understand the design of column splice, column base, and slab base.
ACE012.15	Understand the design of beams, plastic moment.
ACE012.16	Analyze the bending and shear strength laterally supported beams.
ACE012.17	Understand the design, built up sections, large plates web buckling.
ACE012.18	Analyse the crippling and deflection of beams, design of purlin.
ACE012.19	Understand the design of eccentric connections with brackets.
ACE012.20	Analyse the beam end connections, web angle, unstiffened and stiffened seated connections, and design of truss joints.
ACE012.21	Understand the design of welded plate girders, optimum depth, and design of main section.
ACE012.22	Understand the design of end bearing stiffness and intermediate stiffness.
ACE012.23	Analyze the Connection between web and flange and design of flange splice and web splices.

I.   SYLLABUS

UNIT-I	INTRODUCTION ON MECHANICAL BEHAVIOUR OF STEEL
Materials, making of iron and steel, types of structural steel, mechanical properties of steel, concepts of plasticity yield strength, loads and combinations loading wind loads on roof trusses, behavior of steel, local buckling. Concept of limit state design – different limit states as per IS 800:2007. Design strengths deflection limits, serviceability, bolted connections, welded connections, efficiency of joint, prying action types of welded joints, design of tension members, design strength of members.	
UNIT -II	COMPRESSION MEMBERS
Design of compression members, buckling class, slenderness ratio, strength design, laced battened columns, column splice, column base, slab base.	
UNIT -III	BEAMS
Design of beams, plastic moment, and bending and shear strength laterally supported beams. Design of built up sections, large plates web buckling, crippling and deflection of beams, design of purlin.	
UNIT -IV	ECCENTRIC CONNECTIONS
Design of eccentric connections with brackets, beam end connections, web angle, unstiffened and stiffened seated connections (bolted and welded types), design of truss joints.	
UNIT -V	WELDED PLATE GIRDERS
Design of welded plate girders, optimum depth, design of main section, design of end bearing stiffness and intermediate stiffness. Connection between web and flange and design of flange splice and web splices.	
Text Books:	

1. S. K. Duggal, "Limit state design of steel structures", Tata McGraw-Hill, 3rd Edition, 2019. 2. N. Subramanian, "Design of steel structures", Oxford University Press, 2nd Edition,2018.

Reference Books:

1. K. S. Sai Ram, "Design of steel structures", Pearson Education, 2010.
2 Edwin H. Gaylord, Jr. Charles N. Gaylord and James Stallmeyer, "Design of steel structures", Tata McGraw-Hill Education private Limited, 3rd Edition, 2010.
3. Ramachandra, "Design of steel structures Volumes 1 and 2", Standard Publications, 2009.
4. S.S. Bhavikatti, "Design of steel structures", IK International Publication House, New Delhi, 2010.

Web References:

1. http://www.nptel.ac.in/downloads/105106112/
2. http://iitmweb.iitm.ac.in/phase2/courses/105103094/12
3. http://freevideolectures.com/Course/2679/Design-Of-Steel-Structures

E-Text Books:

1. http://www.freeengineeringbooks.com/Civil/Steel-Structure-Design-Books.php
2. https://books.google.co.in/books/about/Comprehensive_Design_of_Steel_Structures.html?id=pXekq3F

UNIT-I

INTRODUCTION ON MECHANICAL BEHAVIOR OF STEEL

When the need for a new structure arises, an individual or agency has to arrange the funds required for its construction. The individual or agency henceforth referred to as the owner then approaches an architect. The architect plans the layout so as to satisfy the functional requirements and also ensures that the structure is aesthetically pleasing and economically feasible. In this process, the architect often decides the material and type of construction as well. The plan is then given to a structural engineer who is expected to do locate the structural elements so as to cause least interference to the function and aesthetics of the structure. He then makes the strength calculations to ensure safety and serviceability of the structure. This process is known as structural design.

Finally, the structural elements are fabricated and erected by the contractor. If all the people work as a team then a safe, useful, aesthetic and economical structure is conceived. However in practice, many structures fulfill the requirements only partially because of inadequate coordination between the people involved and their lack of knowledge of the capabilities and limitations of their own and that of others. Since a structural engineer is central to this team, it is necessary for him to have adequate knowledge of the architects and contractors work. It is his responsibility to advise both the architect and the contractor about the possibilities of achieving good structures with economy.

Ever since steel began to be used in the construction of structures, it has made possible some of the grandest structures both in the past and also in the present day (The Hooghly cable stayed bridge,Jogighopa Road-cum-rail bridge across the river Brahmaputra). Inthe following paragraph, some of the aspects of steel structures, which every structural engineer should know, are briefly discussed.

Steel is by far the most useful material for building structures with strength of approximately ten times that of concrete, steel is the ideal material for modern construction. Due to its large strength to weight ratio, steel structures tend to be more economical than concrete structures for tall buildings and large span buildings and bridges. Steel structures can be constructed very fast and this enables the structure to be used early thereby leading to overall economy. Steel structures are ductile and robust and can withstand severe loadings such as earthquakes.

Steel structures can be easily repaired and retrofitted to carry higher loads. Steel is also a very eco-friendly material and steel structures can be easily dismantled and sold as scrap. Thus the lifecycle cost of steel structures, which includes the cost of construction, maintenance, repair and dismantling, can be less than that for concrete structures. Since steel is produced in the factory under better quality control, steel structures have higher reliability and safety. To get the

most benefit out of steel, steel structures should be designed and protected to resist corrosion and fire. They should be designed and detailed for easy fabrication and erection. Good quality control is essential to ensure proper fitting of the various structural elements. The effects of temperature should be considered in design. To prevent development of cracks under fatigue and earthquake loads the connections and in particular the welds should be designed and detailed properly. Special steels and protective measures for corrosion and fire are available and the designer should be familiar with the options available.

NOTES ON STEEL MATERIAL

Steel is a term given to alloys containing a high proportion of iron with some carbon. Other alloying elements may also be present in varying proportions. The properties of steel are highly dependent on the proportions of alloying elements, so that their levels are closely controlled during its manufacture. The properties of steel also depend on the heat treatment of the metal.

Steel is by far the most important metal, in tonnage terms, in the modern world, with the annual global production of over 700 million tonnes dwarfing the approximately 17 million tonnes of the next most prolific, aluminium. The low price and high strength of steel means that it is used structurally in many buildings and as sheet steel it is the major component of motor vehicles and domestic appliances. The major disadvantage of steel is that it will oxidize under moist conditions to form rust. Typical steel would have a density of about 7.7 g cm^{-3} and a melting point of about 1650°C.

MAKING OF IRON AND STEEL

Steel refers to any iron-carbon alloy, although steels usually contain other elements as well.

Iron occurs mainly as oxide ores, though it is also found in smaller quantities as its sulfide and carbonate. These other ores are usually first roasted to convert them into the oxide. On a world scale the most important ore is hematite (Fe_2O_3). The oxides are reduced with carbon from coal, through the intermediate production of carbon monoxide.

The carbon initially burns in air to give carbon dioxide and the heat, which is necessary for the process. The carbon dioxide then undergoes an endothermic reaction with more carbon to yield carbon monoxide:

$$C + O_2 \rightarrow CO_2 \qquad \Delta H = \text{-393 kJ mol}^{-1}$$
$$C + CO_2 \rightarrow 2CO \qquad \Delta H = \text{+171 kJ mol}^{-1}$$

The oxide ores are then principally reduced by the carbon monoxide produced in this reaction, the reactions involving very small enthalpy changes:

$$Fe_2O_3 + 3CO \rightarrow 2Fe + 3CO_2 \qquad \Delta H = \text{-22 kJ mol}^{-1}$$

$$Fe_3O_4 + 4CO \rightarrow 3Fe + 4CO_2 \qquad \Delta H = \text{-}10 \text{ kJ mol}^{-1}$$

In conventional iron making, this reduction occurs in a blast furnace. The iron produced in this way always contains high levels of impurities making it very brittle. Steel making is mainly concerned with the removal of these impurities. This is done by oxidizing the elements concerned by blowing pure oxygen through a lance inserted into the molten alloy. The oxides produced are either evolved as gases, or combine with limestone to form an immiscible slag which floats on the surface of the liquid metal and so is easily separated.

THE MANUFACTURING PROCESS

Iron ore is converted to steel via two main steps. The first involves the production of molten iron and the second is that of actual steel manufacture. The details of these steps are outlined below.

Step 1 - The production of molten iron

The Primary Concentrate is mixed with limestone and coal and heated. The iron oxides are reduced in the solid state to metallic iron, which then melts, and the impurities are removed either as slag or gas. The flow diagram for this process is shown in **Figure 1**.
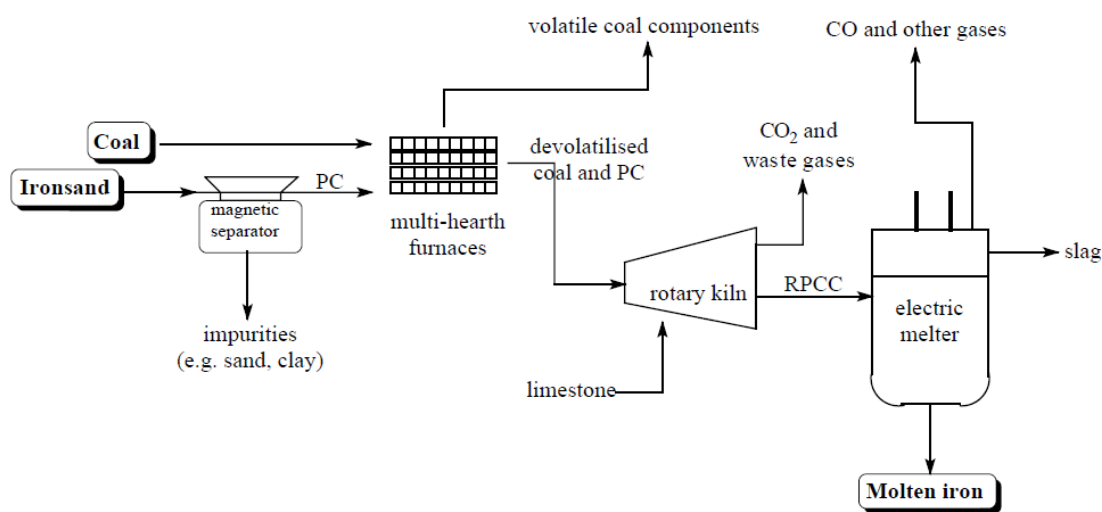
Figure 1 - The production of molten iron

The multi-hearth furnaces

There are four multi-hearth furnaces, each of which feeds a rotary kiln. The furnaces preheat the materials fed into the rotary kiln and reduce the amount of volatile matter present in the coal from about 44% to about 9%. This is important because the large volumes of gas produced during the emission of the volatile matter would otherwise interfere with the processes in the rotary kiln.

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There are 12 hearths in each furnace and the feedstock passes down through these. In the first three hearths, hot gases from the lower stages preheat the material in the absence of air to about $450^{\circ}C$. Air is introduced in hearths 4 to 9 to allow combustion of the volatile material, so as to increase the temperature to about 650oC. The supply of air is adjusted to control the percentage of residual volatiles and coal char in the product. In the final hearths (10 - 12) the char and the primary concentrate equilibrate and the final temperature is adjusted to 620oC. The total residence time in the multi-hearth furnace is 30 - 40 minutes.

The multi-hearth furnaces also have natural gas burners at various levels. These are used torestart the furnace after shutdown and to maintain the temperature if the supply of materials is interrupted.

The waste gas from the multi-hearth furnace contains water vapour and other volatile compounds from the coal (e.g. carbon dioxide, carbon monoxide and other combustion products) as well as suspended coal and primary concentrate dust particles. These solids are removed and returned to the furnace. This gas along with gas from the melter (mainly carbon monoxide) is mixed with air and burnt. The heat so produced is used to raise steam for the production of electricity. As well as providing a valuable source of energy, this combustion of the waste gases is necessary to meet emission controls.

The pre-heated coal char and primary concentrate from the furnaces is mixed with limestone and fed into the kiln. In the first third of the kiln, known as the pre-heating zone, the feed from the multi-hearth furnace is further heated to 900 - 1000oC. This increase in temperature is partly a result of the passage of hot gases from further along the kiln and partly a result of the combustion of the remaining volatile matter in the coal.

The last two-thirds of the kiln is known as the reduction zone, and is where the solid iron oxides are reduced to metallic iron. In this region the air reacts with the carbon from the coal to produce carbon dioxide and heat:

$$C + O_2 \rightarrow CO_2$$
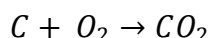
The carbon dioxide then reacts with more carbon to produce carbon monoxide, the principal reductant, in an exothermic reaction:

$$C + CO_2 \rightarrow 2CO$$
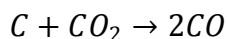
Some of the carbon monoxide burns with the oxygen to produce heat, whilst the remainder reduces the magnetite[1] to iron in a reaction that is almost thermo-chemically neutral.

$$2CO + O_2 \rightarrow 2CO_2$$
$$Fe_3O_4 + 4CO \rightarrow 3Fe + 4CO_2$$
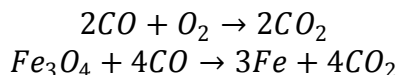
Note:[1]Magnetite can be regarded as 1:1 combination of wustite (FeO) and haematite (Fe2O3). The separate reduction processes from these two components are:
$$FeO + CO \rightarrow Fe + CO_2$$
$$Fe_2O_3 + 3CO \rightarrow 2Fe + 3CO_2$$
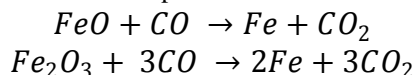
Step 2 - Steel making
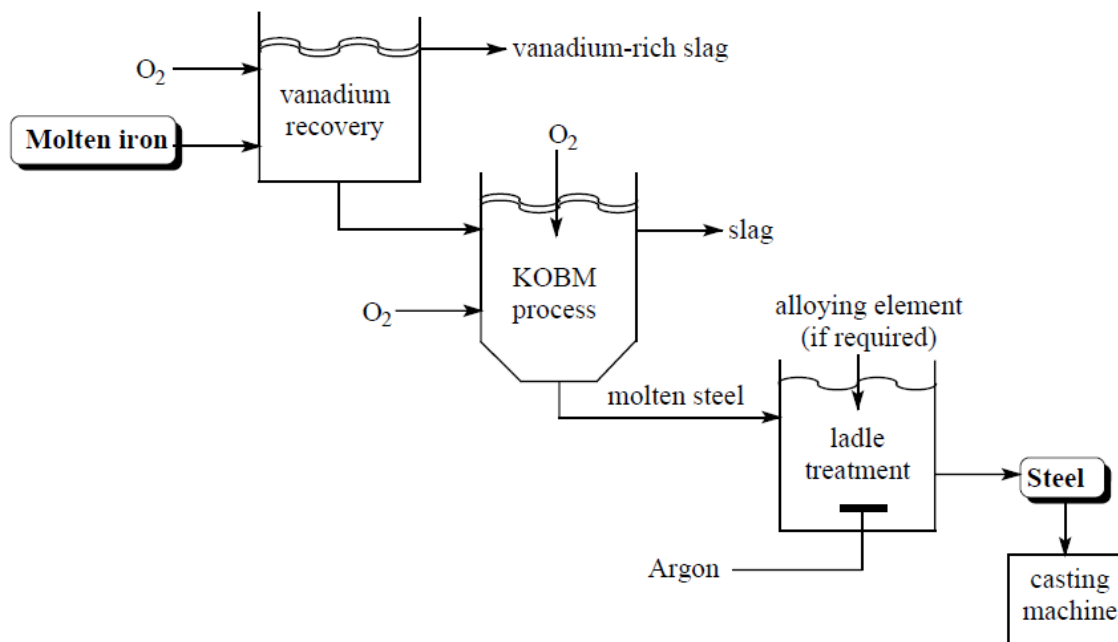The steel making process is shown in **Figure 2**.

Figure 2 - The steel making process

Vanadium recovery

Before conversion into steel, vanadium is recovered from the molten iron. This is done firstly because of the value of the vanadium rich slag produced (15% vanadium as V_2O_5) and secondly because a high vanadium content can make the steel too hard. In the vanadium recovery unit a ladle containing 75 tonnes of molten iron has oxygen blown over the surface, where it oxidizes silicon, titanium, manganese and vanadium to form a slag that floats on the surface. At the same time argon is blown through the molten metal to stir it. When the composition of the molten metal has reached the required vanadium specification, the slag is scraped off, cooled and crushed. Additional advantages of this pre-treatment are that it causes the molten metal to reheat, so permitting temperature control, and, if required, the procedure can be modified by the addition of lime to reduce sulfur levels.

The Klockner Oxygen Blown Maxhutte process(KOBM Process)

The KOBM steel making process, like most modern processes involves oxidizing dissolved impurities by blowing oxygen through the molten metal. The KOBM is unusual in that it blows oxygen through the bottom of the furnace as well as through a lance inserted from the top. This type of furnace was selected for Glenbrook because of its capacity to cope with high levels of

titanium and vanadium coupled with its very fast turn round time. The disadvantage of this type of furnace is that it is technically rather more complex than those that are blown only by a lance.

The KOBM is initially charged with about 6 tonnes of scrap steel. 70 tonnes of molten metal from the vanadium recovery unit is then added. Oxygen is then blown through six holes in the base of the furnace, at a total rate of about 1500 lts per second. Oxygen is also blown through a lance inserted from the top of the furnace at a rate of over 2500 lts per second.

The oxygen oxidizes the elements other than iron (including any free carbon) to their oxides. In this way contaminants are removed as the oxides form a slag which floats on the surface of the molten metal. Powdered lime is blown in to help with slag formation and this particularlyreduces the levels of sulfur and phosphorous by combining with their acidic oxides. Due to its low melting point, iron(II) sulfide (FeS) is particularly harmful to the high temperature properties of steel. So sulfur level must be reduced before further processing. Typical levels of the major elements in the metal fed into the furnace and in a typical steel are shown in **Table 1**.

Table 1 - Compositions of steel and the metal from which it is made

	C	S	P	Ti	Si	V
Hot metal	3%	0.04%	0.07%	0.8%	0.08%	0.3%
Steel	0.03%	0.02%	0.01%	nil	nil	0.005%

The molten iron is analyzed just before being added to the furnace and the temperature taken. This determines the length of the oxygen blow and it also to a certain extent affects the amount and composition of the scrap added. The length of the oxygen blow required is also judged by monitoring the CO:CO2 ration in the gases from the furnace. Blow times vary, but 15 minutes would be typical. During the oxygen blow the temperature would typically rise from 1500oC to 1700°C owing to the exothermic reactions that are occurring.

The slag is firstly tipped off and, after cooling, it is broken up so that the iron trapped in it can be recovered magnetically. The slag, which contains sulfur and phosphorous and has a high lime content, is then sold for agricultural use. Aluminium, which removes excess dissolved oxygen, and alloying materials, such as ferro-silicon and ferro-manganese (which increase the hardness of the steel) are added at this point so that they are well mixed as the molten metal is tipped into a ladle. The whole cycle in the KOBM takes about 30 minutes.

The Glenbrook site also has an electric arc furnace for steel making, the feed for this being mainly scrap steel. The cycle time for the final charge of 75 tonne is about 3½ hours, so that is only responsible for a small fraction of the total steel production. It is, however, a very flexible process and it may be economically used to produce small batches of specialized steel.

Ladle treatment

The final stage of steel making is the ladle treatment. This is when fine adjustments are made to bring the composition of the molten steel, from either furnace, into line with the required

composition. The bulk of the alloying elements are added in the furnace and, after blowing argon through the molten metal to ensure homogeneity, the temperature is measured and a sample removed for analysis after stirring. The analysis by optical emission spectrometry, which measures the levels of 15 elements, takes about five minutes. Alloying materials are added to adjust the composition. If the metal requires cooling, scrap steel is added. If the temperature is too low, aluminium is added and oxygen blown through. When all adjustments are complete argon is blown through once again to ensure mixing and the ladle taken to the continuous casting machine. Here it is cast into slabs of 210 mm thickness and a width of between 800 and 1550 mm. This slab is cut into lengths of from 4.5 m to 10 m and sent for further processing. Most of the production is converted to steel coil.

ENVIRONMENTAL IMPLICATIONS

Due to the nature of the steel making process, large amounts of solid, liquid and gaseous wastes are generated in the steel plant. Careful planning is necessary to ensure that these do not have a negative impact on the environment.

The steel mill requires 1.2 to 1.4 million tonnes of ironsand each year, which means that up to 10 million tonnes of pure sand must be mined. The non-magnetic sand is returned to the area from which it was mined, and marram grass and radiata pines planted to stabilise the deposits.

Wet scrubbers and bag houses are the principal means of controlling air pollution. The wet scrubbers (see oil refining article) wash the dust out of the hot process waste gases which result from iron and steel making while the cloth bags inside a bag house filter dust out of the gas. The dust collection system is shared by the steel production and steel processing sections, and collects a total of between five and ten tonnes of dust every hour.

Extensive water recycling is used in the plant to minimise the quantity of waste water produced, and all waste water and storm water is treated in settling ponds on site before being discharged into the Waiuku Estuary.
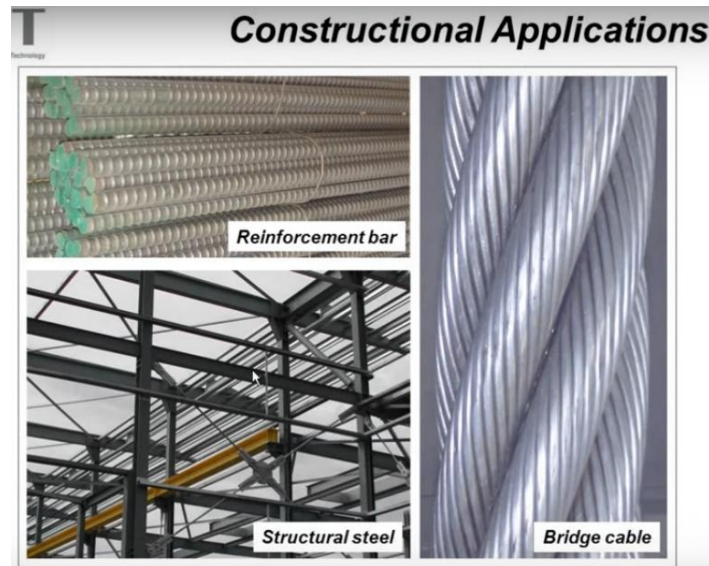
ADVANTAGES OF STEEL DESIGN
1. Better quality control
2. Lighter
3. Faster to erect
4. Reduced site time – Fast track construction
5. Large column free space and amenable for alteration
6. Less material Handling at site
7. Less percentage of floor area occupied by structural elements
8. Has better ductility and hence superior lateral load behavior, better earthquake performance

DISADVANTAGES OF STEEL DESIGN
1. Skilled labor is required
2. Higher cost of construction
3. Maintenance cost is high (Due to corrosion)

4. Poor fire proofing as at 1000°F (538°C) 65% and at 1600°F (871°C) 15% of strength remains
5. Electricity may be required (to hold joints, etc.)

Constructional Applications

Reinforcement bar

Structural steel

Bridge cable

Constructional Applications

Roof tile panels

Roof

Ventilation ducts

Applications: Packaging, Furniture

Packaging

Office furniture

Consumer Appliances

Microwave oven

Air conditioner

Washing machine

Automotive Applications
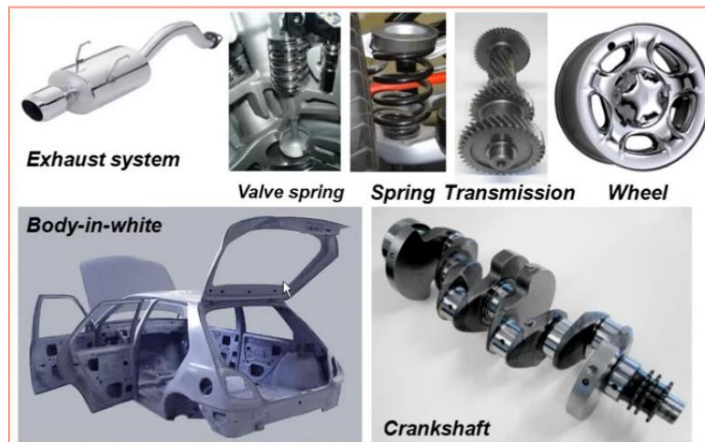
Exhaust system

Valve spring

Spring Transmission

Wheel

Body-in-white

Crankshaft

Oil Industry, Shipbuilding

Ship building

Offshore construction

Production

Bar steel

Forming processes
Surface treatments
Thermal treatments

Crankshaft

Wire steel

High Strength Bolt
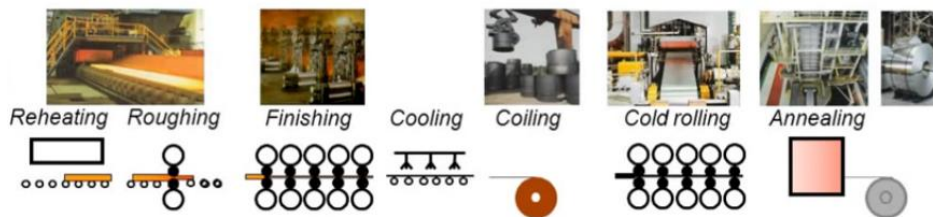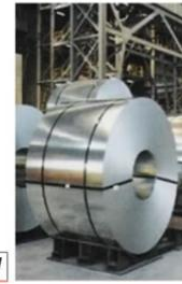
ANATOMY OF A STEEL STRUCTURE

Q. What is the anatomy of a steel structure?

Ans.

Beams

Columns

Floors

Bracing systems-- which is very important for higher rise cases

Foundation

Connections

So these are the anatomy of a steel building.

(Anatomy means usually the study or an examination of what something is like, the way it works or why it works)

TYPES OF STRUCTURAL STEEL

Now let us discuss some rolled steel sections

ROLLED STEEL SECTIONS

The steel sections manufactured in rolling mills and used as structural members are known as rolled structural steel sections. The steel sections are named according to their cross sectional shapes. The shapes of sections selected depend on the types of members which are fabricated and to some extent on the process of erection. Many steel sections are readily available in the market and have frequent demand. Such steel sections are known as regular steel sections. Some steel sections are rarely used. Such sections are produced on special requisition and are known as special sections. 'SP 6-1 (1964) ISI Handbook for Structural Engineers -Part- 1 Structural Steel Sections gives nominal dimensions, weight and geometrical properties of various rolled structural steel sections.

For Example:
TYPES OF ROLLED STRUCTURAL STEEL SECTIONS

The various types of rolled structural steel sections manufactured and used as structural members are as follows:

1. Rolled Steel I-sections (Beam sections).

2. Rolled Steel Channel Sections.

3. Rolled Steel Tee Sections.

4. Rolled Steel Angles Sections.

5. Rolled Steel Bars.

6. Rolled Steel Tubes.

7. Rolled Steel Flats.

8. Rolled Steel Sheets and Strips.

9. Rolled Steel Plates.

2.3 ROLLED STEEL BEAM SECTIONS

The rolled steel beams are classified into following four series as per BIS : (IS : 808-1989)

1. Indian Standard Joist/junior Beams   ISJB

2. Indian Standard Light Beams   ISLB

3. Indian Standard Medium Weight Beams   ISMB

4. Indian Standard Wide Flange Beams   ISWB

The rolled steel columns/heavy weight beams are classified into the following two series as per BIS (IS: 808-1989)

1. Indian Standard Column Sections   ISSC

2. Indian Standard Heavy Weight Beams   ISHB

The cross section of a rolled steel beam is shown in Figure below.
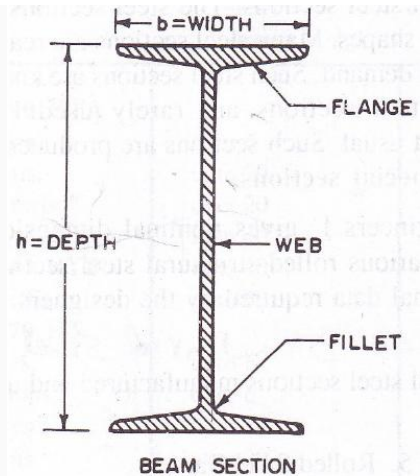
BEAM SECTION

The beam section consists of web and two flanges. The junction between the flange and the web is known as fillet. These hot rolled steel beam sections have sloping flanges. The outer and inner faces are inclined to each other and they intersect at an angle varying from 1½ to 8° depending on the section and rolling mill practice. The angle of intersection of ISMB section is 8°. Abbreviated reference symbols (JB, LB, MB, WB, SC and HB) have been used in designating the Indian Standard Sections as per BIS (IS 808-1989)

The rolled steel beams are designated by the series to which beam sections belong (abbreviated reference symbols), followed by depth in mm of the section and weight in kN per metre length of the beam, e.g., MB 225 @ 0.312 kN/m. H beam sections of equal depths have different weights per metre length and also different properties e.g., WB 600 @ 1.340 kN/m, WB 600 @ 1.450 kN/m, HB 350 @0.674 kN/m, HB 350 @0.724 kN/m.

I-sections are used as beams and columns. It is best suited to resist bending moment and shearing force. In an I-section about 80 % of the bending moment is resisted by the flanges and the rest of the bending moment is resisted by the web. Similarly about 95% of the shear force is resisted by the web and the rest of the shear force is resisted by the flanges. Sometimes I-sections with cover plates are used to resist a large bending moment. Two I-sections in combination may be used as a column.

ROLLED STEEL CHANNEL SECTIONS

The rolled steel Channel sections are classified into four categories as per ISI, namely,

1. Indian Standard Joist/Junior Channels   ISJC

2. Indian Standard Light Channels   ISLC

3. Indian Standard Medium Weight Channels   ISMC

4. Indian Standard Medium Weight Parallel Flange Channels   ISMCP

The cross section of rolled steel channel section is shown in Figure below.
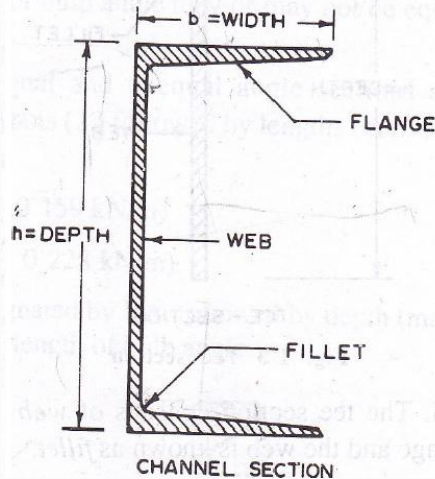
CHANNEL SECTION

The channel section consists of a web and two flanges. The junction between the flange and the web is known as fillet. The rolled steel channels are designated by the series to which channel section belong (abbreviated reference symbols), followed by depth in mm of the section and weight in kN per metre length of the channel, e.g., MC 225 @ 0.261 kN/m

Channels are used as beams and columns. Because of its shape a channel member affords connection of an angle to its web. Built up channels are very convenient for columns. Double channel members are often used in bridge truss. The channels are employed as elements to resist bending e.g., as purlins in industrial buildings. It is to note that they are subjected to twisting or torsion because of absence of symmetry of the section with regards to the axis parallel to the web, i.e., yy-axis. Therefore, it is subjected to additional stresses. The channel sections are commonly used as members subjected to axial compression in the shape of built-up sections of two channels connected by lattices or batten plates or perforated cover plates. The built-up channel sections are also used to resist axial tension in the form of chords of truss girders.

As per IS : 808-1989, following channel sections have also been additionally adopted as Indian Standard Channel Secions
1. Indian Standard Light Channels with parallel flanges   ISLC(P)
2. Medium weight channels   MC
3. Medium weight channels with parallel flanges   MCP
4. Indian Standard Gate Channels   ISPG

In MC and MCP channel sections, some heavier sections have been developed for their intended use in wagon building industry. The method of designating MC and MCP channels is also same as that for IS channels.

ROLLED STEEL TEE SECTIONS
The rolled steel tee sections are classified into the following five series as per ISI:

1. Indian Standard Normal Tee Bars   ISNT
2. Indian Standard Wide flange Tee Bars   ISHT
3. Indian Standard Long Legged Tee Bars   ISST
4. Indian Standard Light Tee Bars   ISLT
5. Indian Standard Junior Tee Bars   ISJT

The cross section of a rolled steel tee section has been shown in Figure below.
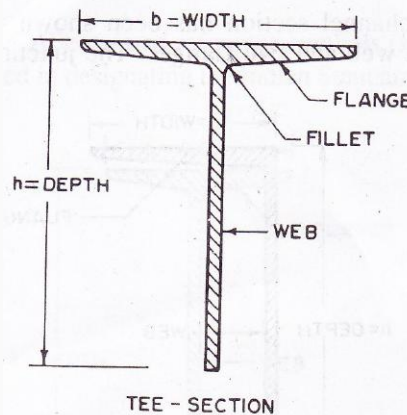
The tee section consists of a web and a flange. The junction between the flange and the web is known as fillet. The rolled steel tee sections are designated by the series to which the sections belong (abbreviated reference symbols) followed by depth in mm of the section and weight in kN per metre length of the Tee, e.g., HT 125 @ 0.274 kN/m. The tee sections are used to transmit bracket loads to the columns. These are also used with flat strips to connect plates in the steel rectangular tanks.

A per IS: 808-1984, following T-sections have also been additionally adopted as Indian Standard T-sections.

1. Indian Standard deep legged Tee bars   ISDT

2. Indian Standard Slit medium weight Tee bars   ISMT

3. Indian Standard Slit Tee bars from I-sections   ISHT

It is to note that as per IS 808 (part II) 1978, H beam sections have been deleted.

ROLLED STEEL ANGLE SECTIONS
The rolled steel angle sections are classified in to the following three series.

1. Indian Standard Equal Angles   ISA

2. Indian Standard Unequal Angles   ISA

3. Indian Standard Bulb Angles   ISBA

Angles are available as equal angles and unequal angles. The legs of equal angle sections are equal and in case of unequal angle section, length of one leg is longer than the other. Thickness of legs of equal and unequal angle sections are equal. The cross section of rolled equal angle section, unequal angle section and that of bulb angle section is shown in Fig. 2.4. The bulb angle consists of a web a flange and a bulb projecting from end of web.

L = LENGTH OF LEG
(A) EQUAL ANGLE

L = LENGTH OF LEG
(B) UNEQUAL ANGLE

(C) BULB ANGLE
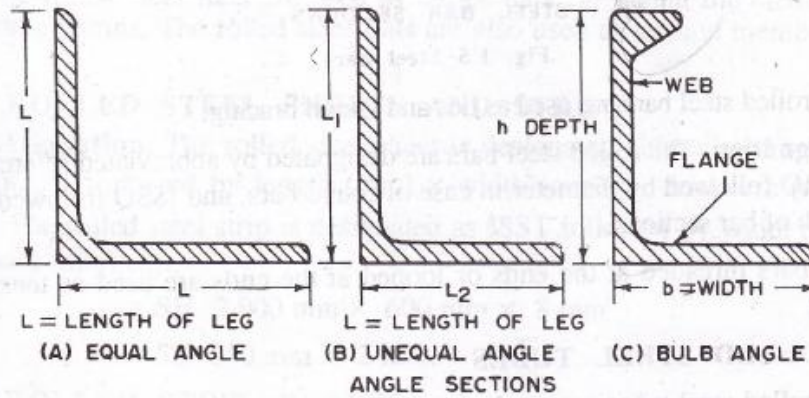
ANGLE SECTIONS

The rolled steel equal and unequal angle sections are designated by abbreviated reference symbols ∟ followed by length of legs in mm and thickness of leg, e.g.,

∟ 130 x 130 x 8 mm (∟ 130x 130 @ 0.159 kN/m)

∟ 200 x 100 x 10 mm (∟ 200 x 100 @ 0.228 kN/m)

The rolled steel bulb angles are designated by BA, followed by depth in mm of the section and weight in kN per metre length of bulb angle.

Angles have great applications in the fabrications. The angle sections are used as independent sections consisting of one or two or four angles designed for resisting axial forces (tension and compression) and transverse forces as purlins. Angles may be used as connecting elements to connect structural elements like sheets or plates or to form a built up section. The angle sections are also used as construction elements for connecting beams to the columns and purlins to the chords of trusses in the capacity of beam seats, stiffening ribs and cleat angles. The bulb angles are used in the ship buildings. The bulb helps to stiffen the outstanding leg when the angle is under compression.

As per IS : 808-1984, some supplementary angle sections have also additionally adopted as Indian Standard angle sections. However prefix ISA has been dropped. These sections are designated by the size of legs followed by thickness e.g., ∟ 200 150 x 15.

ROLLED STEEL BARS
The rolled steel bars are classified in to the following two series:

1. Indian Standard Round Bars   ISRO

2. Indian Standard Square Bars   ISSQ

The rolled steel bars are used as ties and lateral bracing. The cross sections of rolled steel bars are shown in Figure below. The rolled steel bars are designated by abbreviated reference symbol RO followed by diameter in case of round bars and ISSQ followed by side width of bar sections. The bars threaded at the ends or looped at the ends are used as tension members.

STEEL BAR SECTIONS
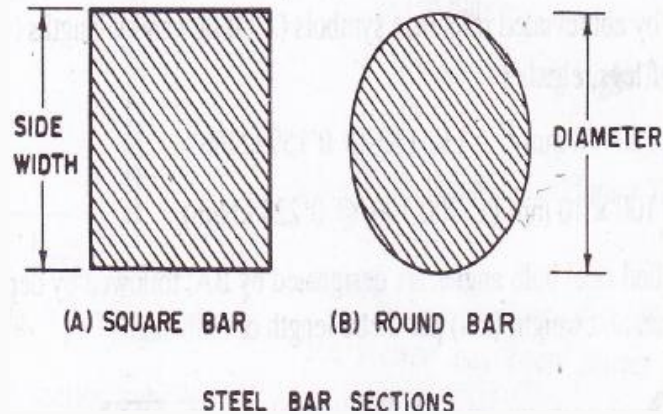
ROLLED STEEL TUBES

The rolled steel tubes are used as columns and compression members and tension members in tubular trusses. The rolled steel tubes are efficient structural sections to be used as compression members. The steel tube sections have equal radius of gyration in all directions. The cross section of rolled steel tube is shown in Figure below.

STEEL TUBES SECTION
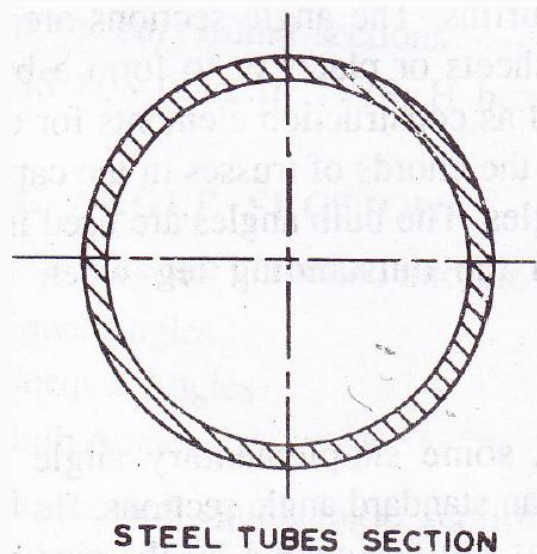
ROLLED STEEL FLATS

The rolled steel flats are used for lacing of elements in built up members, such as columns and are also used as ties. The cross section of rolled steel flat is shown in Figure below. the rolled steel flats are designated by width in mm of the section followed by letters (abbreviated reference symbol) F and thickness in mm, e.g., 50 F 8. This means a flat of width 50 mm and thickness 8 mm. The rolled steel flats are used as lattice bars for lacing the elements of built up columns. The rolled steel flats are also used as tension members and stays.

STEEL FLATS SECTION
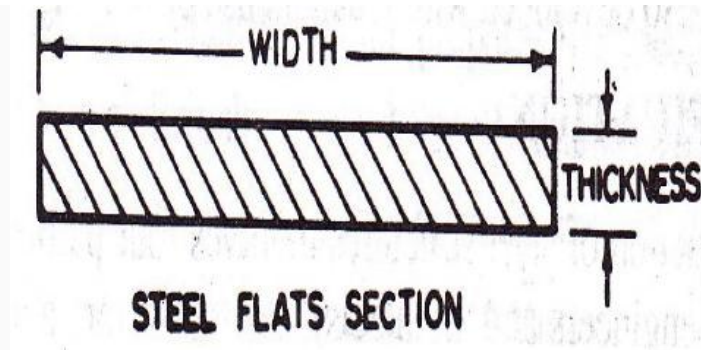
ROLLED STEEL SHEETS AND STRIPS

The rolled steel sheet is designated by abbreviated reference symbol SH followed by length in mm x width in mm x thickness in mm of the sheet. The rolled steel strip is designated as ISST followed by width in mm x thickness in mm, e.g., SH 2000 x 600 x 8 and ISST 250 x 2.

ROLLED STEEL PLATES

The rolled steel plates are designated by abbreviated reference symbol PL followed be length in mm x width in mm x thickness in mm of the plates, e.g., PL 2000 x 1000 x 6.

The rolled steel sheets and plates are widely used in construction. Any sections of the required dimensions, thickness and configuration may be produced by riveting or welding the separate plates. The rolled plates are used in the web and flanges of plate girders, plated beams and chord members and web members of the truss bridge girders. The rolled steel plates are used in special plate structures, e.g., shells, rectangular and circular steel tanks and steel chimneys.

RECENT DEVELOPMENTS IN SECTIONS

The rolled steel beam sections with parallel faces of flanges are recently developed. These beam sections are called as parallel flange sections. These sections have increased moment of inertia, section modulus and radius of gyration about the weak axis. Such sections used as beams and columns have more stability. Theses sections possess ease of connections to other sections as no packing is needed as in beams of slopping flanges. The parallel flange beam sections are not yet rolled in our country.

New welded sections using plates and other steel sections are developed because of welding. The development of beams with tapered flanges and tapered depths is also due to welding. The open web sections and the castellated beams were also developed with the rapid use of welding.

MECHANICAL PROPERTIES OF STEEL

Stress – strain behavior: tensile test

The stress-strain curve for steel is generally obtained from tensile test on standard specimens as shown in Figure below. The details of the specimen and the method of testing is elaborated in IS:

1608 (1995). The important parameters are the gauge length 'Lc' and the initial cross section area So. The loads are applied through the threaded or shouldered ends. The initial gauge length is taken as 5.65 (So) 1/2 in the case of rectangular specimen and it is five times the diameter in the case of circular specimen. A typical stress-strain curve of the tensile test coupon is shown in Fig.1.5 in which a sharp change in yield point followed by plastic strain is observed. After a certain amount of the plastic deformation of the material, due to reorientation of the crystal structure anincrease in load is observed with increase in strain. This range is called the strain hardening range. After a little increase in load, the specimen eventually fractures. After the failure it is seen that the fractured surface of the two pieces form a cup and cone arrangement. This cup and cone fracture is considered to be an indication of ductile fracture. It is seen from Fig.1.5 that the elastic strain is up to e_y followed by a yield plateau between strains e_y and e_{sh} and a strain hardening range start at e_{sh} and the specimen fail at e_{ult} where e_y, e_{sh} and e_{ult} are the strains at onset of yielding, strain hardening and failure respectively.
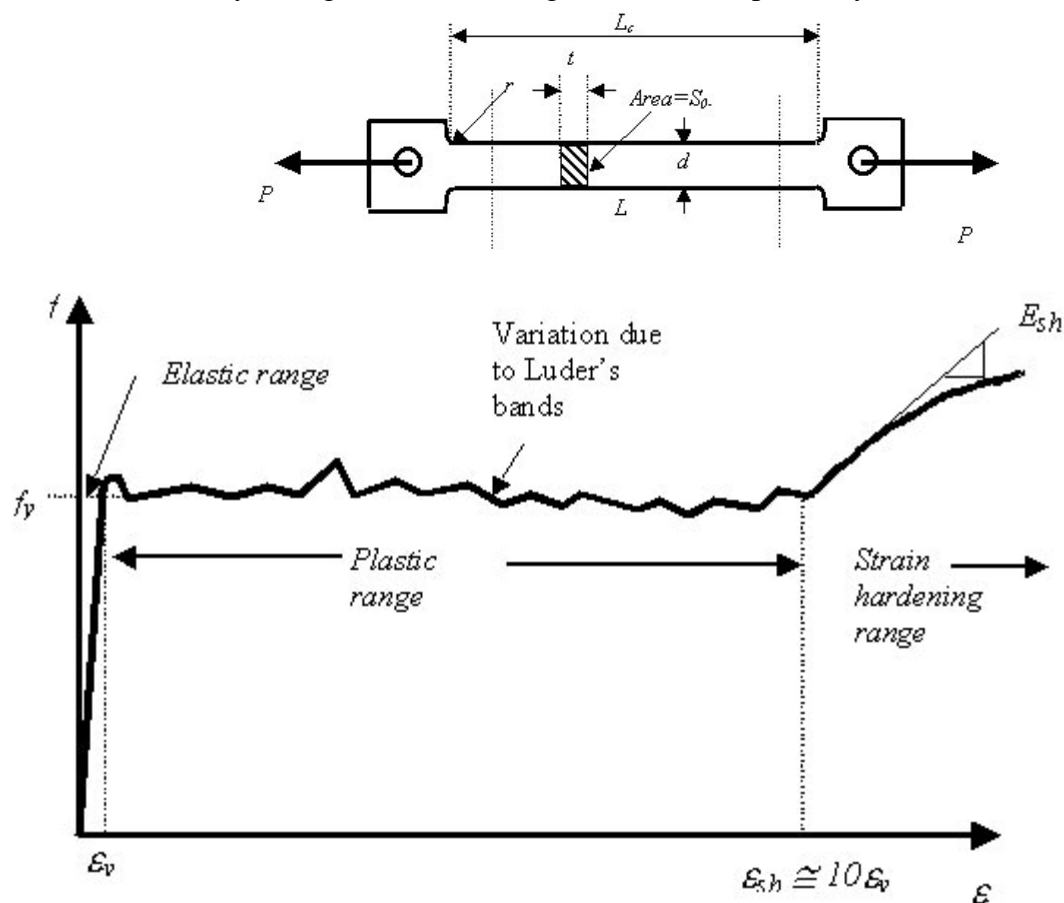

Fig 1.5. Stress strain curve for mild steel

Depending on the steel used, ε_{sh} generally varies between 5 and 15 ε_y, with an average value of 10 εy typically used in many applications. **For all structural steels, the modulus of elasticity can be taken as 205,000 MPa and the tangent modus at the onsetof strain hardening is roughly 1/30th of that value or approximately 6700 MPa.** High strength steels, due to their

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specific microstructure, do not show a sharp yield point but rather they yield continuously as shown in Fig. 1.6. For such steels the yield stress is always taken as the stress at which a line at 0.2% strain, parallel to the elastic portion, intercepts the stress strain curve. This is shown in Fig. 1.6.

**Stress strain curve for high strength steel**
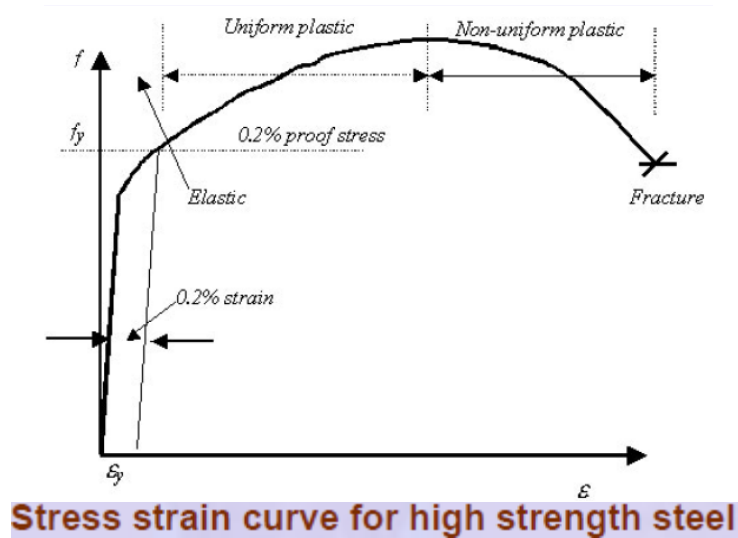
The nominal stress or the engineering stress is given by the load divided by the original area. Similarly, the engineering strain is taken as the ratio of the change in length to original length.

MECHANICAL PROPERTIES OF STEEL
1. Yield stress of steel (f_y) = range from 220 to 540 Mpa
2. Ultimate tensile strength = 1.2 f_y
3. Modulus of Elasticity (E_s) = 2 x 10^5 N / mm^2
4. Shear Modulus of steel = 0.4 E
5. Poissons Ratio
    (i)   Elastic Range = 0.3
    (ii) Plastic Range = 0.5

STRESS STRAIN CURVE FOR MILD STEEL

Stress-Strain Curve for Mild Steel

23
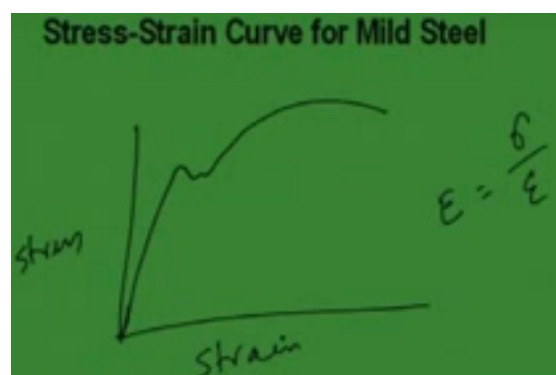
1.3.2 Hardness
Hardness is regarded as the resistance of a material to indentations and scratching. This is generally determined by forcing an indentor on to the surface. The resultant deformation in steel is both elastic and plastic. There are several methods using which the hardness of a metal could be found out. They basically differ in the form of the indentor, which is used on to the surface. They are presented in Table 1.2. In all the above cases, hardness number is related to the ratio of the applied load to the surface area of the indentation formed. The testing procedure involves forcing the indentor on to the surface at a particular road. On removal, the size of indentation is measured using a microscope. Based on the size of the indentation, hardness is worked out. For example, Brinell hardness (BHN) is given by the ratio of the applied load and spherical area of the indentation i.e.

Table 1.2 Hardness testing methods and their indentors

Hardness Testing Method		Indentor
(a)	Brinell hardness	Steel ball
(b)	Vickers hardness	Square based diamond pyramids of 135° included angle
(c)	Rockwell hardness	Diamond core with 120° included angle

Note: Rockwell hardness testing is not normally used for structural steels.

$$BHN = \frac{P}{\pi(d/2)\left[D - \sqrt{D^2 - d^2}\right]} \qquad (1.2)$$

Where P is the load, D is the ball diameter, d is the indent diameter. The Vickers test gives a similar hardness value (VHN) as given by

$$VHN = \frac{1.854P}{L^2} \qquad (1.3)$$

Where L is the diagonal length of the indent.
Both the BHN and VHN for steel range from 150 to 190.

1.3.3 Notch-toughness
There is always a possibility of microscopic cracks in a material or the material may develop such cracks as a result of several cycles of loading. Such cracks may grow rapidly without detection and lead to sudden collapse of the structure. To ensure that this does not happen,

materials in which the cracks grow slowly are preferred. Such steels are known as notch-tough steels and the amount of energy they absorb is measured by impacting a notched specimen with a heavy pendulum as in Izod orCharpy tests. A typical test set up for this is shown in Fig. 1.7 and the specimen used is shown in Fig. 1.8.

The important mechanical properties of steel produced in India are summarized in Table 1.3. In Table 1.3, the UTS represent the minimum guaranteed Ultimate Tensile Strength at which the corresponding steel would fail.

Table 1.3 Mechanical properties of some typical structural steels

Type of steel	Designation	UTS (MPa)	Yield strength(MPa) Thickness (mm)			Elongation Gauge $5.65\sqrt{S_0}$	Charpy V - notch values Joules (min)
			<20	20-40	>40		
Standard structural steel	Fe410A	410	250	240	230	23	27
	Fe410B	410	250	240	230	23	27
	Fe410C	410	250	240	230	23	27
<16 16-40 41-63							
Micro alloyed high strength steel	Fe440	440	300	290	280	22	-
	Fe540	540	410	390	380	20	-
	Fe590	590	450	430	420	20	-

Channel Section with Lacing

This is a channel section face to face lacings are provided. These lacings are provided in a zig-zag way in order to strengthen the column. These lacings are rectangular flats which are attached to fix the column in order to strengthen it making more stable for carrying upcoming load. Safely without displacing the column from its position. This built-up section is commonly used in Huge industries, heavy trusses, and railway stations. This is used as column and made up of steel members.

MECHANICAL PROPERTIES OF MATERIALS

Some are commonly or mostly preferred properties are

1. Stiffness
2. Elasticity
3. Plasticity
4. Ductility
5. Brittleness
6. Malleability
7. Toughness
8. Hardness
9. Creep
10. Fatigue

i) **STIFFNESS:**

It is the ability of materials to resist deformations under the action of loads. That is a material should not change its shape when the load is applied.

*Its unit is N/mm or kN/mm. It is load applied to produce per unit deflection. i.e., in order to produce deflection of 1mm, how much load should be applied i.e., in terms of Newtons.

* It is mostly considered in the design of springs.

Stiffness is given by the formula:

$$K = \frac{Load\ (W)}{Deflection\ \ (\delta)}$$

(ii) **ELASTICITY:**

It is a property by which a material changes its shape when load is applied and will regain its original shape when load is removed. So this the definition of Elasticity.

For example when we apply load over rubber band, then its shape will change. The moment I removed load over this, the rubberband come back to its original position.

Elasticity is measured by a term called as 'Youngs Modulus' or 'Modulus of Elasticity'

*Its Unit is 'N/mm^2' (or) 'kN/mm^2'

*Young's modulus decides how much elastic the material is.

For example, if 'Young's Modulus' or 'Elastic Modulus' is very high, it means the material is very elastic. If the value of Elastic Modulus is less, it means that material will behave in a less elastic manner.

(iii)**PLASTICITY:** It is the property by which a material is not able to regain it's original shape when the load is removed.

It means here in case of plasticity, the material will not regain it's original shape.

For example, if I have a pen and load is applied on this. It will change its shape. On the removal of load, the pen will not regain its original shape.

It is a permanent deformation. Materials used in machines are never allowed to behave in a plastic manner. See this is very important consideration like whatever the materials we are using for the machine design, that materials they should not operate in a region where the deformation would be plastic. i.e., they are not allowed to behave in a plastic manner. That means whatever the machines we are seeing in the machines, they are designed on the concept of elasticity. That is they should regain their original shape when the load is removed. They should not behave in a plastic manner.

(iv)**DUCTILITY:**

It is a property by which materials can be drawn into wires. A very important property that if a material is having ductility it means it can be drawn into wires. Now whatever wires we are seeing like incase of electrical connections those wires the material with the help of material which they are made that material it is called as ductile material. **So here I can say that**

- Ductile materials have the ability to flow. To flow means when the load is applied the material will elongate. It will change its shape.
  Example: Copper wires used in electric cables.
  Aluminum which is soft material is also a ductile material.

(v)**BRITTLENESS:** It is ability of a material by which it can break or develop cracks when loaded. Brittle materials are those in which when we are applying load either they can break suddenly or cracks would be developing in that material.

Example: Wood, Concrete, cast-iron (Contains more amount of carbon, the more amount of carbon we add, the more brittle it becomes)

- Brittle materials can break without any prior warning or they can develop cracks

(vi)**MALLEABILITY**: It is the property by which materials are able to be beaten or converted into thin sheets. See whatever the sheets of metal which we are seeing that metal it has the property of malleability that is why it is conerted into sheets.
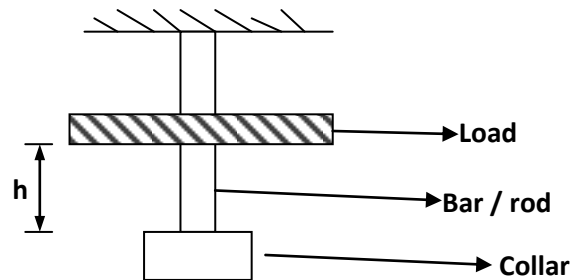
Materials which are elastic are better in malleability
Examples: steel, copper, Aluminum, brass, bronze, Zinc

(vii)**TOUGHNESS:**
It is the property by which a material is able to resist shocks or impact loading. Impact loading refers to load falling from a height.

Like if I can give the example by drawing the diagram.

Here I have height 'h' through which the load is falling on to this collar. So now toughness means the ability of this collar to resist this load like for example this load is falling from a height 'h' then this collar should not deform much. By toughness we mean that on the application of this load, this collar should be able to resist this load. It should not go or it should not deflect suddenly. The deflection should be minimum.

If I can draw the diagram, that after deflection it will look something like this. So this much is the amount of deflection 'δL'

This property is very useful in the design of springs. Like previously we have seen the property which was stiffness. Stifness is required in the design of springs. At the same toughness is also considered in the design of springs.

(viii) **HARDNESS:**
Hardness is the opposite of Toughness. It is the property of a material by which it can resist scratches, marks, or wear and tear. Hardness is the independent of the weight of a material. This property is mostly preferred while designing components which slide over one another.
Brittle materials are more hard. It means brittle materials are able to resist scratches more. Like for example, Cast iron, Concrete, Glass, Diamond.

(ix) **CREEP:** It is the ability of materials to resist high or extremely high temperatures. So as from the definition I think it is very much clear that any material which is able to resist high amount of temperatures that material we will say that it is **CREEP RESISTANT**. And if the material changes its shape when the temperature is high then it would be called as the material is not able to resist the high temperature, then it would be **LESS IN CREEP.**

Because of Creep, high amount of temperature stresses are developed. As we know when we have a metal when we heating that metal we know that it will expand. Because of that expansion there is some stress which is stored in the material. That stress would be called as temperature stress.

Examples: I.C. Engines (Internal Combustion Engines), Boilers, Steam-turbines, and Furnaces require creep resistant materials.

What is Rolling Process?

In metalworking, rolling is a metal forming process in which metal stock is passed through one or more pairs of rolls to reduce the thickness and to make the thickness uniform. The concept is similar to the rolling of dough. Rolling is classified according to the temperature of the metal rolled.

The process of plastically deforming metal by passing it between rolls. Rolling is the most widely used process which provides high production and close control of final product. The metal is subjected to high compressive stresses as a result of the friction between the rolls and the metal surface.

Terminology

An ingot is a piece of relatively pure material, usually metal, that is cast into a shape suitable for further processing. In steelmaking, it is the first step among semi-finished casting products.

Bloom is the product of first breakdown of ingot (Cross sectional area greater than 100 cm^2)

Billet is the product obtained from a further reduction by hot rolling (Cross sectional area greater than 40 x 40 mm^2)

Slab is the hot rolled ingot (cross sectional area greater than 100 cm^2 and with a width greater than or equal to 2 x thickness)

Plate is the product with a thickness greater than 6 mm.

Sheet is the product with a thickness less than 6 mm and width greater than 600 mm.

Strip is the product with a thickness greater than 6 mm and width less than 600 mm.

Recrystallization: the formation of new strain-free grain structure from that existing in cold worked metal.

- **Recrystallization:** the formation of new, strain-free grain structure from that existing in cold worked metal.
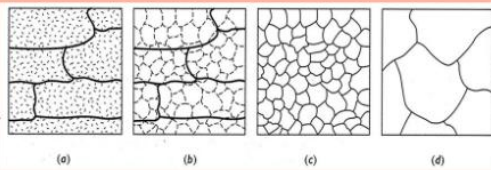
The effects of annealing on metals: (a) cold work, (b) after recovery, (c) after recrystallization, and (d) after grain growth.

Annealing:

There are different types of heat treatments. Annealing is one of the heat treatments given to metals. Main aim of annealing is to increase the ductility of the metal. Annealing is a heat treatment in which the metal is heated to a temperature above its re-crystallization temperature, kept at that temperature some time for homogenization of temperature followed by very slow cooling to develop equilibrium structure in the metal or alloy. The steel is heated 30 to 50°C above Ae_3 temperature in case of hypo-eutectoid steels and 30 to 50°C above A_{cm} temperature in case of hyper-eutectoid temperature.

The cooling is done in the furnace itself. In case of annealing of steels, the steel is heated to different temperatures depending upon the aim of annealing followed by furnace cooling.

Annealing is a heat treatment designed to eliminate the effects of cold working. The properties of a metal may revert back to the precold-work states by annealing, through recovery, recrystallization and grain growth.

HOT ROLLING

It is a metal working process that occurs above the re-crystallization temperature of the material.Hot rolled metals generally little directionality in their mechanical properties and deformationinduced residual stresses. However, I certain instances non-metallic inclusions will impart some directionality.

Non-uniformed cooling will induce a lot of residual stresses which usually occurs in shapes that have a non-uniform cross-section, such as I – Beams and H-beams.

Application
Hot rolling is used mainly to produce sheet metal or simple cross sections such as rail tracks.

COLD ROLLING
Cold rolling occurs with the metal below its re-crystallization temperature. (usually at room temperature). It also improves the surface finish and holds tighter tolerances. Due to smaller size of the workpieces and their greater strength than hot rolled stock , four-high or cluster mills are used .commonly cold-rolled products include sheets, strips and rods; products being smaller than the same products that are hot rolled.

CONCEPT OF LIMIT STATE DESIGN OF BEAM COLUMNS

Steel structures are important in a variety of land-based applications, including industrial (such as factory sheds, box girder cranes, process plants, power and chemical plants etc.), infrastructural (Lattice girder bridges, box girder bridges, flyovers, institutional buildings, shopping mall etc.) and residential sector. The basic strength members in steel structures include support members (such as rolled steel sections, hollow circular tubes, square and rectangular hollow sections, built-up sections, plate girders etc.), plates, stiffened panels/grillages and box girders. During their lifetime, the structures constructed using these members are subjected to various types of loading which is for the most part operational, but may in some cases be extreme or even accidental.

Steel-plated structures are likely to be subjected to various types of loads and deformations arising from service requirements that may range from the routine to the extreme or accidental. The mission of the structural designer is to design a structure that can withstand such demands throughout its expected lifetime.

The structural design criteria used for the *Serviceability Limit State Design* (hereafter termed as SLS) design of steel-plated structures are normally based on the limits of deflections or vibration for normal use. In reality, excessive deformation of a structure may also be indicative of excessive vibration or noise, and so, certain interrelationships may exist among the design criteria being defined and used separately for convenience.

The SLS criteria are normally defined by the operator of a structure, or by establishedpractice, the primary aim being efficient and economical in-service performance without excessive routine maintenance or down-time. The acceptable limits necessarily depend on the type, mission and arrangement of structures. Further, in defining such limits, other disciplines such as machinery designers must also be consulted.

The structural design criteria to prevent the *Ultimate Limit State Design* (hereafter termed as ULS) are based on plastic collapse or ultimate strength. The simplified ULS design of many types of structures has in the past tended to rely on estimates of the buckling strength of components, usually from their elastic buckling strength adjusted by
a simple plasticity correction. This is represented by point A in Figure 7.1. In such a design scheme based on strength at point A, the structural designer does not use detailed information on the post-buckling behavior of component members and their interactions. The true ultimate strength represented by point B in Figure 7.1 may be higher although one can never be sure of this since the actual ultimate strength is not being directly evaluated.

In any event, as long as the strength level associated with point B remains unknown (as it is with traditional allowable stress design or linear elastic design methods), it is difficult to determine the real safety margin. Hence, more recently, the design of structures such as offshore platforms and land-based structures such as steel bridges has tended to be based on the ultimate strength.

The safety margin of structures can be evaluated by a comparison of ultimate strength with the extreme applied loads (load effects) as depicted in Figure 7.1. To obtain a safe and economic

structure, the ultimate load-carrying capacity as well as the design load must be assessed accurately. The structural designer may even desire to estimate the ultimate strength not only for the intact structure, but also for structures with existing or premised damage, in order to assess their damage tolerance and survivability.
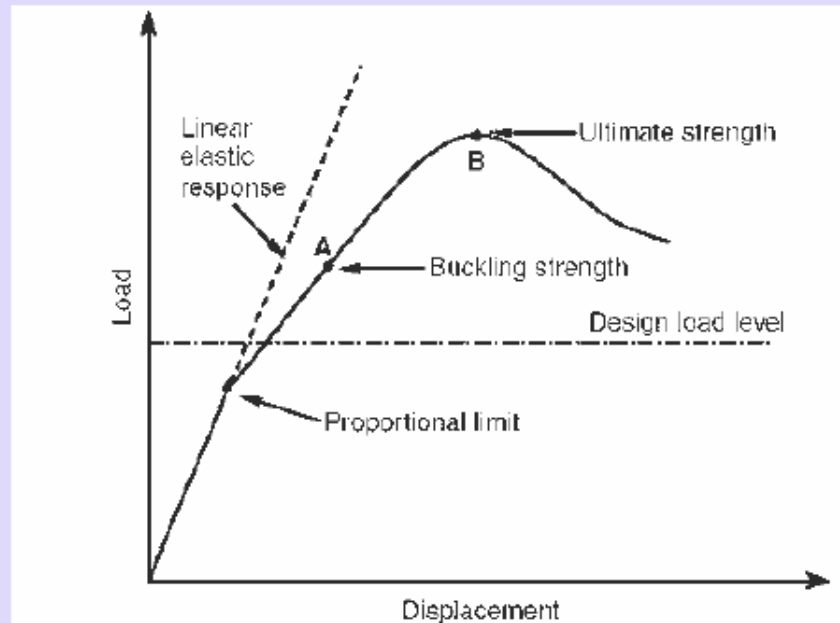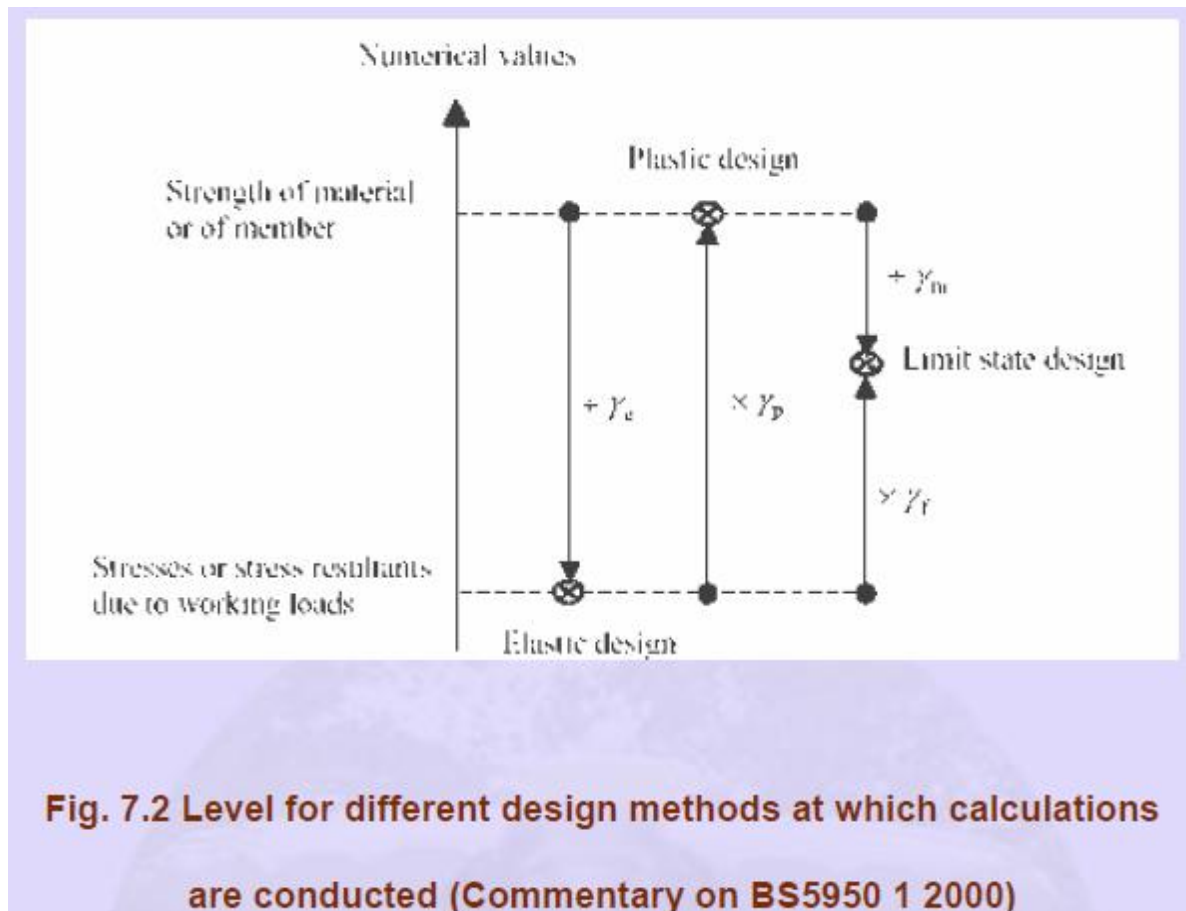

Fig 7.1 Structural design considerations based on the ultimate limit state

In the structural design process, **"analysis"** usually means the determination of the stress resultants, which the individual structural members must be capable to resist. **"Design"** can mean the development of the structural layout, or arrangement of members, but it usually means the selection of sizes of members to resist the imposed forces and bending moments. Three methods of analysis are available, i.e. elastic analysis, plastic analysis and advanced analysis. Limit state design is a design method in which the performance of a structure is checked against various limiting conditions at appropriate load levels. The limiting conditions to be checked in structural steel design are ultimate limit state and serviceability limit state.Limit state theory includes principles from the elastic and plastic theories and incorporates other relevant factors to give as realistic a basis for design as possible.

Fig. 7.2 Level for different design methods at which calculations are conducted (Commentary on BS5950 1 2000)

Ultimate Limit State Design of Steel Structures reviews and describes both fundamentals and practical design procedures in this field. Designs should ensure that the structure does not become unfit / unserviceable for the use for which it is intended to. The state at which the unfitness occurs is called a limit state.

Figure 7.2 shows how limit-state design employs separate factors γ_f, which reflects the combination of variability of loading γ_l, material strength γ_m and structural performance γ_p. In the elastic design approach, the design stress is achieved by scaling down the strength of material or member using a factor of safety γ_e as indicated in Figure 7.2, while the plastic design compares actual structural member stresses with the effects of factored-up loading by using a load factor of γ_p.

Special features of limit state design method are:
• Serviceability and the ultimate limit state design of steel structural systems and
  their components.
• Due importance has been provided to all probable and possible design conditions
  that could cause failure or make the structure unfit for its intended use.

- The basis for design is entirely dependent on actual behavior of materials in structures and the performance of real structures, established by tests and long-term observations
- The main intention is to adopt probability theory and related statistical methods in the design.
- It is possible to take into account a number of limit states depending upon the particular instance.
- This method is more general in comparison to the working stress method. In this method, different safety factors can be applied to different limit states, which is more rational and practical than applying one common factor (load factor) as in the plastic design method.
- This concept of design is appropriate for the design of structures since any development in the knowledge base for the structural behavior, loading and materials can be readily implemented.

CONNECTIONS

Today we are going to introduce new topic name is 'connections'. When we are going to design a steel structure completely, first we have to know the elementary design. Elementary design means design of Beam member (flexural member), Design of a compression member (Column member),Design of a tension member, Base plate, the foundations and similarly the 'Connections'.

The utility of the connection is that to withstand the load and to transfer the load from one member to another member. Like suppose beam and column. Now the load from beam to column is going to pass through that joint. If joint is not sufficiently strong then chances of failure will be there. In general we see we use to give much importance on design of different types of elements. But often we forget to design the connections properly. We must give due importance to the connection aspectsbecause steel structure may fail, if their connections are improper. So the beam member or the column member may be strong enough to send the load. If their joint is weak, then as rule the structure will fail. So we have to consider the connections as important so that failure doesn't occur at the joint level.

Now Connections means different type of members are connected at a joint. Different type of members means like say:
- Beam & column
- Beam & Beam
- Beam & cross beam
- Column &column
- Column &brackets
- Column & caps
- Base plate of trusses
- Truss member connections through gussets
- Purlins &rafter
- Wind braces and columns
- Rails & columns

I am just showing one picture. Say suppose one column is there and another beam is here. So how to connect it. So this is a column and this is a beam. Now connection is to be made. So what we used to do? That connections can be made either temporarily or permanent in nature. Say suppose we are providing some bolted or riveted connection to with stand the load coming from the beam to column.
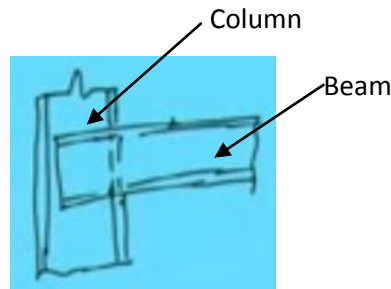
Now connections means different types of connections are there. As per the requirement in the field, we need to choose the connections like riveted connections, Bolted connections, Pin connections and welding connections. In general Bolted Connection and Pin connections are temporary in nature that means we can use as a temporary basis. Rivetted connection and welded connection are permanent in nature. Other types of connections made like stiffeners in plate girders, Diaphragms in plate girders, Flange and web connections in plate girders, and stiffener plates in column joints.

As I told that method of fabrications is
Riveted joints
Welded joints
Bolted joints (or Pin joints we can say)
The combination of two or three of the above (means any of the combinations also can be made)

Now these two (Riveted joints, Welded joints) joints are permanent in nature and these two joints (Bolted joints, The combination of two or three of the above) are temporary.

Now when we are going to talk about the connections, we must know "what are the requirements for the good connections?". Means what are the points we have to remember to make a good connection.

i.e., 1. it (Connection) should be rigid enough to avoid fluctuating stresses which may cause fatigue failure.

   2. It should be such that there is the least possible weakening of the parts to be joined.
   3. It should be such that it can be easily installed, inspected, & maintained.

Now connections we have told. There are three types of connections basically, one is riveting, another is welding and another is bolted connection.
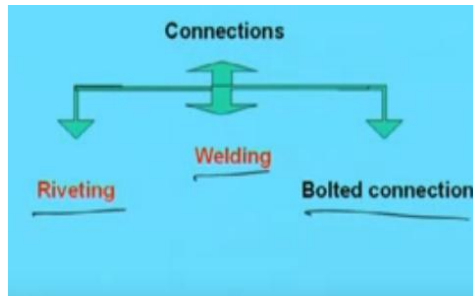
Now we will discuss in details about the **Riveted connections.**

In case of rivet connections analysis is in general difficult. Certain assumptions have been made to make the analysis simple. What are those assumptions? Assumptions like

1. Friction between the plates is neglected.
2. The shear stress is uniform on the crosssection of the rivet
3. The distribution of direct stress on the portion of the plates between the rivet holes is uniform.
4. Rivets in group subjected to direct loads share the load equally. (that means if the 'n' number of rivets are there and then total load is connect by as 'P', then the load shared by each rivet become **'P/n'**
5. Bending stress in the rivet is neglected.
6. Rivets fill completely the holes in which they are driven.
7. Bearing stress distribution is uniform and contact area is dx t. where 'd' is the diameter of the rivet and 't' is the thickness of the plate.

So with these assumptions the analysis would be done. Analysis means, what is the strength of the rivet, how many rivets should be required to connect the particular joint. All these things will be decided under these assumptions.

Now how the rivet look like?
As you know,

Rivet
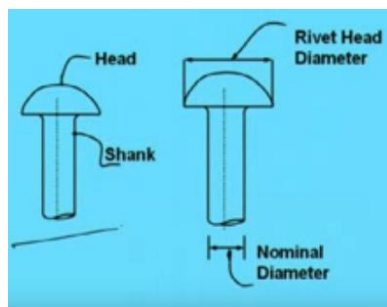
It has a head and another part is shank. So a rivet consist of shank and head. The length of shank is depends or decided based on thickness of the plate. How much shank length is required accordingly shank will be decided.

Now Nominal diameter is the diameter of the shank. Now here another term we will get is the "**Gross diameter**" which is basically the some clearance that means hole diameter. Hole diameter is made nominal diameter + some additional clearance.

As per the codal provisions, if the nominal diameter is below the 25 mm, then 1.5 mm extra clearance has been taken for calculating the gross diameter andif the nominal diameter is more than 25 mm, then 2.0 mm extra clearance has been taken for calculating the gross diameter.

That means gross diameter will be say 'd_g' = d_n + 1.5 (d_n is below 25 mm)
                                    = d_n + 2.0 (d_n is more than 25 mm)

So rivet looks like this.
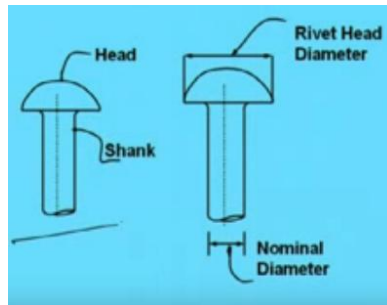
Now Rivet can be divided into two categories
One is power driven riveting or Hot rivet.
Another is Hand driven rivet or Cold rivet.

Now Power driven rivet is of two types
One is Power driven shop rivet (PDS) and
another is Power driven field rivet (PDF). In short we use 'PDS' or 'PDF'.

Similarly in case Hand driven riveting, we use to categorize as 'Hand driven shop rivet (HDS) and Hand driven field rivet (HDF).

<u>Now we will show some commonly used rivet head</u>.

1.) One is 'Snap Head'. This is the most commonly used rivet in practice we make and the standard dimension is like this. If the nominal diameter of the rivet is 'd', the diameter of head will become '1.6d' and the height of head will become '0.7d'.
This is called length *'l'* whatever is required to fit the connections, to fit the thickness of the plate. So snap head is looking like this.
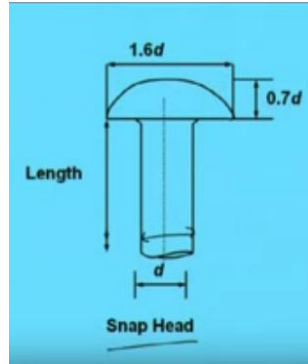
Snap Head

2. **Another type of rivet is called 'PAN Head'**

In this Pan Head, the head will look like this. Here if the diameter is 'd'then the height of head will be '0.7d' like the previous one (like the height of head of 'Snap Head'). The maximum width of head will be '1.6d'. so the specialty of PAN head is that the head will be '0.7d' and the maximum width will become '1.6d' where the width at the top will become 'd'.


Pan Head
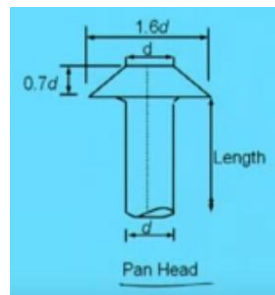
3. **Another Common head is called 'Mushroom Head'.** Mushroom Head means here the maximum diameter of head will be '2.25d' and the height of head will become '0.5d'. The centre of the curve will be at the '1.516d'. So the specialty of mushroom head is Height of head will be '0.5d' and width of head will become '2.25d', where the centre of circle will be at '1.516d'.

Mushroom Head
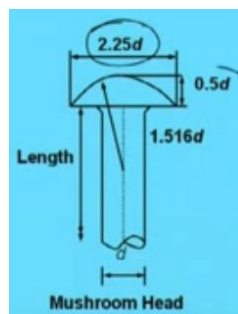
4. Another is **Countersunk Head 120⁰**. Here if the diameter is 'd', then the width of the head will become '2d', and the height of head will become '0.29d', and the slope will be

made in such a way that this will become 120°. The angle between this two slopes will become 120°.

Countersunk Head 120°
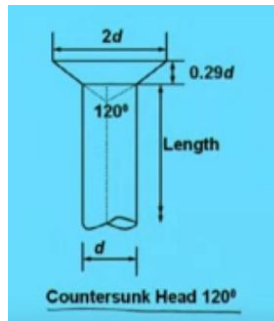
5. Another type of rivet is '**FlatCountersunk 90°**. Here if the diameter is 'd', then the width of the head will become '2d', and the height of head will become '0.5d', and the slope will be made in such a way that this will become 90°. The angle between this two slopes will become 90°.

Flat counter sunk 90°
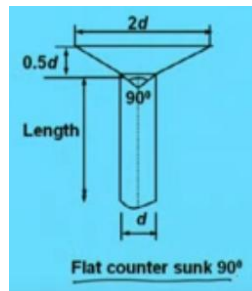
6. Another type of rivet is '**FlatCountersunk head 60°**'. Here if the diameter is 'd', then the width of the head will become '1.5d', and the height of head will become '0.433d', and the slope will be made in such a way that the angle will become 60°. The angle between this two will be 60°.

Flat counter sunk head 60°
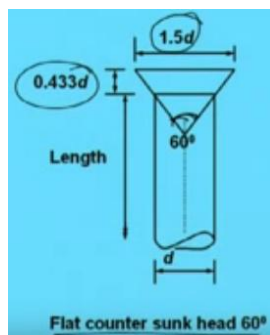
7. **Another type of rivet is round counter sunk head 60°.**

Here the anagle between these two will become 60°. This radius will become '1.5d'. so the width will become 1.5d of the circle and height of head will become '0.433d'.

Round counter sunk head 60°
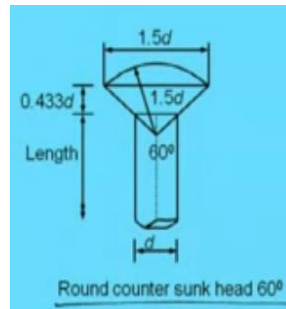
8. **Another type of rivet is Flat head**. In case of flat head, the height of the head will become '0.25d' where d is the nominal diameter of the sunk, and the width of head will become '2d'.

Flat head
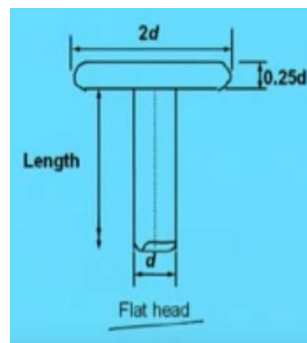
NOW WE WILL DISCUSS ABOUT THE **DIFFERENT TYPES OF CONVENTIONAL SYMBOLS**

These conventional symbols has to be known for knowing the drawing and the engineer executing the construction at the site, must know how to read the drawing. That means in a drawing some symbols are given for the connections so he should know, what is the meaning and what type of connections the design engineer has made and accordingly he has to make. So for the sake of simplicity certain convention symbols has been used which are given below for our learning purpose.
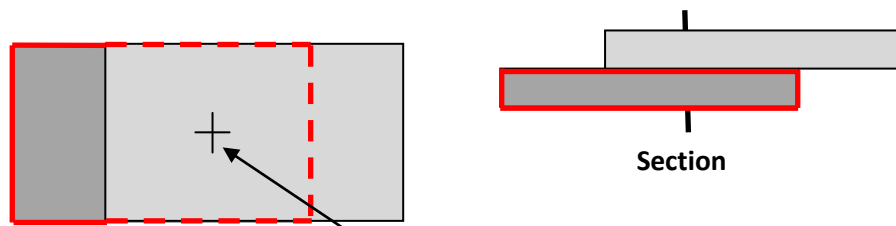
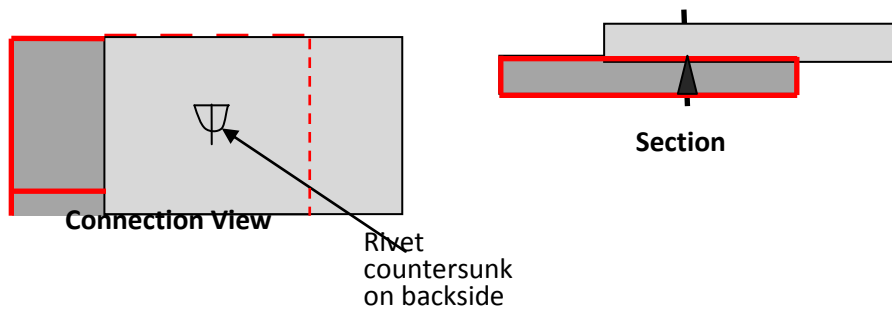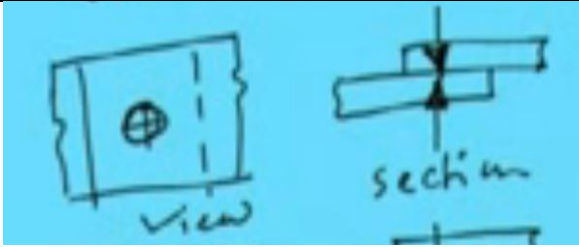1. One is generally rivet. One plate is this. Another plate is
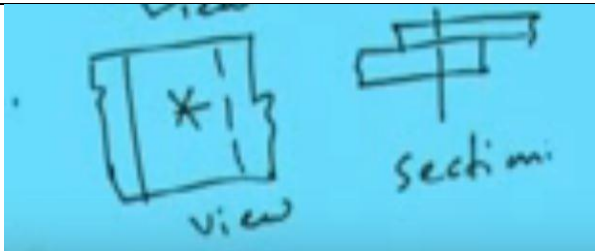

Plate-1

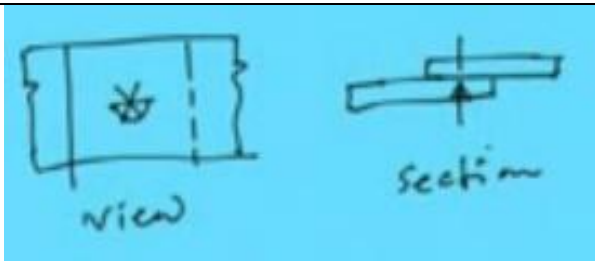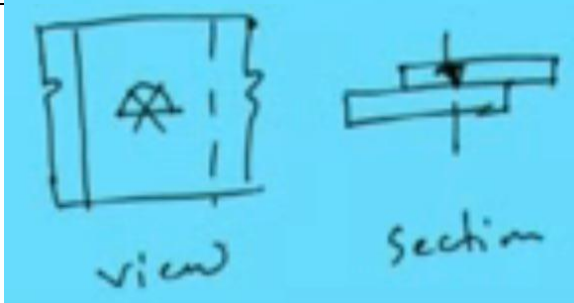**Plate-2**

Connection View

Rivet General

Section

2. Suppose **Rivet countersunk on backside**. To mean this, the drawing will be something like this. So to represent the Rivet countersunk on backside, the view and section will be looking like shown below.

Connection View

Rivet
countersunk
on backside

Section

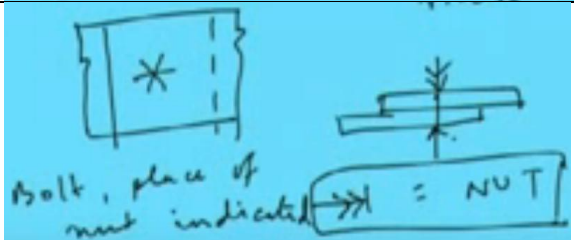3. Suppose **Rivet countersunk on front side**. To mean this, the drawing will be something like this. So to represent the Rivet countersunk on front side, the view and section will be looking like shown below.

Connection View

Rivet
countersunk
on backside

Section
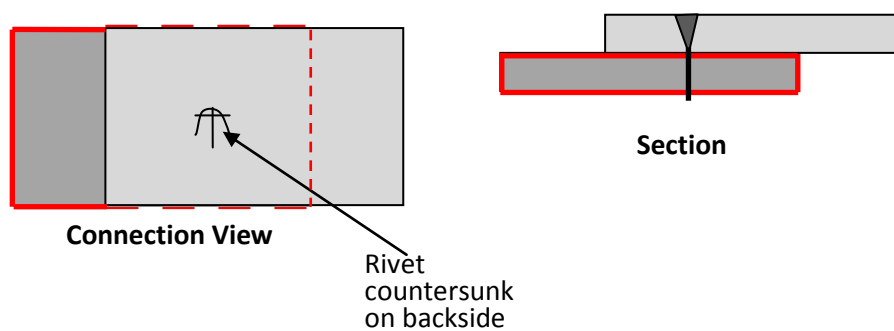
4.		**Rivet Counter sunk on front and back side**
5.		**Bolt General**
6.		**Bolt head counter sunk on back side**
7.		**Bolt head counter sunk on front side**
8.		**Bolt to distinguished from the rivet**
9.		**Bolt, place of Nut indicated**

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Terminology

What is pitch distance?
For connecting two members, we need to know what will be the Pitch distance, Edge distance and other things.

Pitch distance means the distance between two rivets in a plane in a particular direction and **Edge distance** means the distance from the outer most rivet to the edge.

q. What should be the minimum edge distance, What should be maximum pitch, minimum pitch, Min edge distance
Max edge distance
All these things has been given in the code IS 800:1984. In that(this) code, all the details has been given and according to the codal provisions we have to follow and design accordingly. So before designing, before going to analyse the details of the rivet joints, we must know what are the codal provisions and we should know some of the terminology, so that we can know all these things before going to the analysis.

Say this is one plate another plate is here overlapped. Now say rivets are there like this. Now the load is acting in this direction. Then the Pitch will be along the action of load, the distance between two rivets.(Distance between two rivets is called Pitch) and this is called 'Lap' means Lap length. That means overlapping of two plates. This is one plate and this is another plate. This plate is continue upto this and this plate is continue upto this. So Overlapping is from this to this which is called 'Lap' and Edge distance is this one.
The distance between two pitch perpendicular to the action of load is called gauge. So this gauge. So we should not mix up with Pitch and Gauge.

This 'p' basically stands for Pitch. So what are the terminology we got from here.
One is Edge, 'e', another is Pitch, 'p', and another is gauge. 'g'. so these three terms will be required frequently for analysis of the rivet joints.

Pitch, p➤ Pitch is the centre to centre distance of adjacent rivets or bolt holes measured in the direction of stress
Gauge, g ➤A row of rivets which is parallel to the direction of stress is called gauge line. The normal distance between two adjacent gauge lines is called gauge.

Edge distance, e➤ The distance between the edge of a member or cover plate from the centre of the nearest rivet hole.

So this three terminology is important and the codal provision has been given that what is the minimum edge distance and gauge distance and pitch distance should be maintained. Those things has been told in Codal provisions we will come to them later through which we have to design the details of the joint.

Nominal diameter, d➜ It is the diameter of the shank of the rivet. For bolts the diameter of the unthreaded portion of the shank is called its nominal diameter.
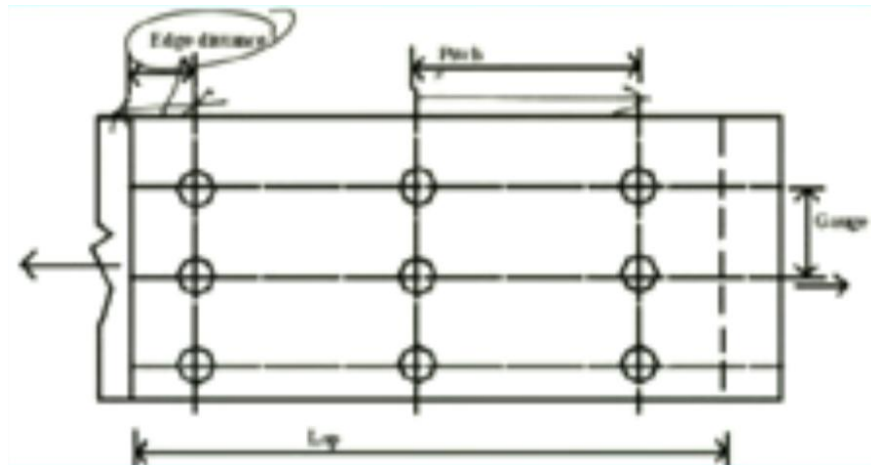
Gross diameter, D➜ The diameter of the rivet hole or bolt hole is called its gross diameter. This gross diameter can be calculated from the given formula.

As per clause 3.6.1.1 of IS 800:
$$D = d + 1.5 \text{ mm for } d < 25 \text{ mm}$$
$$= d + 2 \text{ mm for } d >= 25 \text{ mm}$$

So in short if I repeat once again using Figure given below.

This is called edge distance, This is the 'Pitch', and this is the 'gauge' and this is the 'Lap'. In short we have to say, if the load, 'P' is acting in this direction. Remember, see first the load in which direction it is acting, accordingly we have to decide which one will be the pitch and which one will the gauge. Pitch distance is the distance along the action of the load and the gauge distance will be the perpenidcular to the action of the load.

TYPES OF RIVET JOINTS
Now the rivet joints can be classified into three category.

1. Depending upon arrangement of rivets and plates (That means how the plates and rivets have been arranged. on that basis the classification can be made.)
2. Depending upon mode of load transmission (How that load is transmitted on that basis the rivet joint can be classified)
3. Depending upon nature and location of load (that means where is the load and what type of load like whether the load is concentric or eccentric or only tensile coming into picture or only compression force on that basis rivet joint can be classified.)

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1. Depending upon arrangement of rivets and plates (This first case it can be divided into two category. Depending upon arrangement of rivets and plates, the rivet joint can be classified as 'Lap Joint' and 'Butt Joint'.)

   Lap Joint (so Lap joint is again classified into three category)
   (a) Single riveting
   (b) Chain riveting
   (c) Staggered or Zig-Zag riveting

   Butt Joint
   (a) Single riveting
   (b) Chain riveting
   (c) Staggered or Zig-Zag riveting

2. Depending upon the mode of load transmission, the rivet joints can be classified into four category:
   (a) Single shear
   (b) Double shear
   (c) Multiple shear
   (d) Bearing

That means in how the load is getting transmitted on that basis the rivet joints has been classified whether it is single shear or double shear or Multiple shear or bearing

3. Depending upon the nature and location of load
   (a) Direct shear connection
   (b) Eccentric connection
   (c) Pure moment connection
   (d) Moment shear connection

[So all this types will be discussed now. What type of joints should be made in case of different type of rivet thing arrangement. Those things will be discussed now] say first let us consider now say

SINGLE RIVETED LAP JOINT

Chain riveting lap JOINT

STAGGERED OR ZIG ZAG RIVETTING LAP JOINT

CONCEPTS OF PLASTICITY

There are five basic concepts in the theory of plasticity:
1. Yield condition
2. Hardening curve
3. Incompressibility
4. Flow rule
5. Loading/unloading criterion

All of the above concept will _rst be explained in the 1-D case and then extended to the general 3-D case.

Hardening Curve and Yield Curve

If we go to the lab and perform a standard tensile test on a round specimen or a at dog-bone specimenmade of steel or aluminum, most probably the engineering stress-strain curvewill look like the one shown in Fig. (12.1a). The following features can be distinguished:

Point A - proportionality limit

Point B - 0.02% yield

Point C - arbitrary point on the hardening curve showing different trajectories on loading/unloading

Point D - fully unloaded specimen

For most of material the initial portion of the stress-strain curve is straight up to the proportionality limit, point A. From this stage on the stress-strain curve becomes slightly curved but there is no distinct yield point with a sudden change of slope. There is in international standard the yield stress is mapped by taking elastic slope with 0.02% strain ($\varepsilon = 0{:}0002$) offset strain. Upon loading, the material hardens and the stress is increasing with diminishing slop until the testing machine (either force or displacement controlled) is stopped. There are two possibilities. On unloading, meaning reversing the load or displacement of the cross-load of the testing machine, the unloading trajectory is straight. This is the elastic unloading where the slop of the stress-strain curve is equal to the initial slope, given by the Young's modulus. At point D the stress is zero but there is a residual plastic strain of the magnitude OD. The experiment on loading/unloading tell us that the total strain _total can be considered as the sum of the plastic strain _plastic and elastic strain _elastic. Thus

$$\epsilon^{\text{total}} = \epsilon^{\text{plastic}} + \epsilon^{\text{elastic}}$$

The elastic component is not constant but depends on the current stress

$$\epsilon^{\text{elastic}} = \frac{\sigma}{E}$$

The plastic strain depends on how far a given specimen is loaded, and thus there is a di_erence between the total (measured) strain and known elastic strain. Various empirical formulas were

suggested in the literature to _t the measured relation between the stress and the plastic strain. The most common is the swift hardening law

$$\sigma = A(\epsilon^{\text{plastic}} + \epsilon_o)^n$$

where A is the stress amplitude, n is the hardening exponent and _o is the strain shift parameter.

In many practical problems the magnitude of plastic strain is mud larger than the parameter _o, giving rise to a simpler power hardening law, extensively used in the literature.

$$\sigma = A\epsilon^n$$

For most metals the exponent n is the range of n = 0:1□0:3, and the amplitude can vary alot, depending on the grade of steel. **A description of the reverse loading and cycling plasticloading is beyond the scope of the present lecture notes.**

Various other approximation of the actual stress-strain curve of the material are incommon use and some of then are shown in Fig. (12.3).

A further simplification is obtained by considering the average value _o of the stress- strain curve, illustrated in Fig. (12.3b). This concept gave rise to the concept of the rigid-perfectly plastic material characteristic time, depicted in Fig. (12.4). The material model shown in Fig. (12.4) is adopted in the development of the limit analysis of structures. The extension of the concept of the hardening curve to the 3-D case will be presented later, after deriving the expression for the yield condition.

12.2 Loading/Unloading Condition
In the 1-D case the plastic ow rule is reduced to the following statement:

$$\dot{\epsilon}^P > 0 \qquad \sigma = \sigma_o$$
$$\dot{\epsilon}^P < 0 \qquad \sigma = -\sigma_o$$
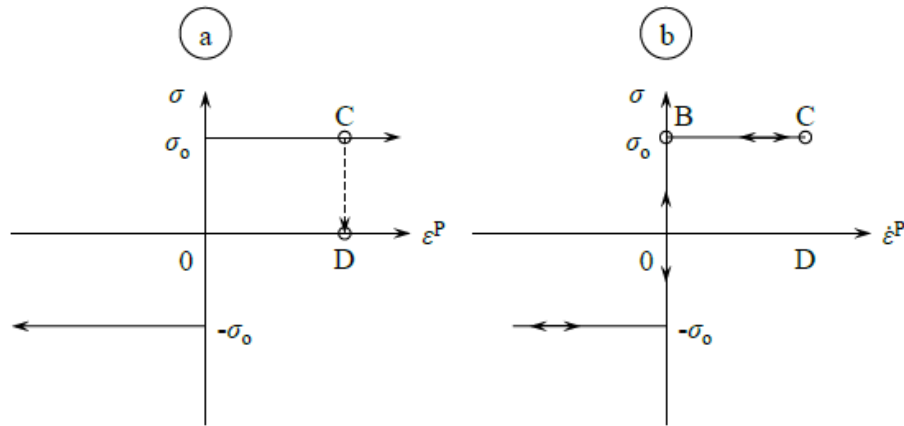$$\dot{\epsilon}^P = 0 \qquad \sigma_o < \sigma < -\sigma_o$$

Figure 12.4: The flow stress $|\sigma_o|$ is assumed to be identical in tension and compression in the rigid-perfectly plastic material model.

In the case of unloading, the stress follow the path CD on the _-_p graph. If the strain rate is an independent variable, the path of all unloading cases is the same CBO, as shown in Fig. (12.4).

12.3 Incompressibility

Numerous experiments performed over the past 100% have shown that metals are practically incompressible in the plastic range. Let's explore the consequences of this physical fact in the case of one-dimensional case. Denote the gauge length of the prismatic bar by l and its cross-sectional area by A. The current volume of the gauge section is V = Al. Incompressibility means that the volume must be unchanged or dV = 0.

$$dV = d(Al) = dAl + Adl = 0$$

From Eq. (12.5) we infer that the strain increment d_ can be calculated either by tracking down the gauge length or the cross-sectional area

$$d\epsilon = \frac{dl}{l} = -\frac{dA}{A}$$

Integrating the first part of Eq. (??)

$$\epsilon = \ln l + C_1$$

The integration constant is obtained by requiring that the strain vanishes when the length l is equal to the gauge initial, reference length lo, which gives C = □ln lo. Thus

$$\epsilon = \ln \frac{l}{l_o}$$

which is the logarithmic de_nition of strain, introduced in Lecture 2. Similarly, integrating the second part of Eq. (??) with the initial condition at A = Ao, _ = 0, one gets

$$\epsilon = \ln \frac{A_o}{A}$$

In tension l > lo or A < Ao, so both Eqs. (??) and (??) gives the positive strain. In compression the strain is negative. The same is true for strain increments d_ or strain rates

$$\dot{\epsilon} = \frac{\dot{l}}{l} \quad \text{or} \quad \dot{\epsilon} = -\frac{\dot{A}}{A}$$

From the above analysis follows a simple extension of the plastic incompressibility condition into the 3-D case. Consider an infinitesimal volume element V = x1x2x3, Fig. (12.5).

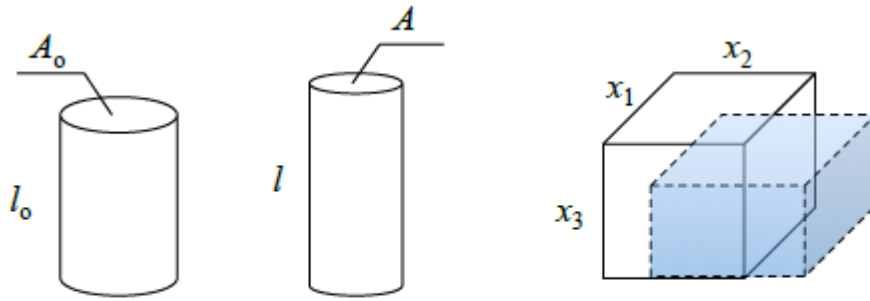Figure 12.5: Undeformed and deformed 1-D and 3-D volume elements.

The plastic incompressibility requires that

$$dV = d(x_1 x_2 x_3) = dx_1 (x_2 x_3) + x_1 d(x_2 x_3)$$
$$= dx_1 x_2 x_3 + x_1 dx_2 x_3 + x_1 x_2 dx_3$$

Dividing both sides of the above equation by the volume, one gets

$$\frac{dx_1}{x_1} + \frac{dx_2}{x_2} + \frac{dx_3}{x_3} = 0 \tag{12.12}$$

or

$$d\epsilon_{11} + d\epsilon_{22} + d\epsilon_{33} = 0, \quad d\epsilon_{kk} = 0$$

Noting that $d\epsilon_{11} = \frac{\partial \epsilon_{11}}{\partial t} dt = \dot{\epsilon}_{11} dt$, an alternative form of the incompressibility condition is

$$\dot{\epsilon}_{11} + \dot{\epsilon}_{22} + \dot{\epsilon}_{33} = 0, \quad \dot{\epsilon}_{kk} = 0 \tag{12.13}$$

The sum of the diagonal components of the strain rate tensor must vanish to ensure incompressibility. It follows from the ow rule (to be formulated later) that in uniaxial tension in

x1 direction the components $\dot{\epsilon}_{22} = \dot{\epsilon}_{33}$. Therefore $\dot{\epsilon}_{11} + 2\dot{\epsilon}_{22} = 0$ or $\dot{\epsilon}_{11} + 2\dot{\epsilon}_{33} = 0$. Finally we obtain

$$\dot{\epsilon}_{22} = -0.5\dot{\epsilon}_{11}, \quad \dot{\epsilon}_{33} = -0.5\dot{\epsilon}_{11}$$

The coefficient 0.5 can be interpreted as the Poisson ratio

$$\nu = -\frac{\dot{\epsilon}_{22}}{\dot{\epsilon}_{11}} = -\frac{\dot{\epsilon}_{33}}{\dot{\epsilon}_{11}} = 0.5$$

We can conclude that plastic incompressibility requires that the Poisson ratio be equal to 1=2, which is different from the elastic Poisson ratio, equal to 0:3 for metals. Many other materials such as rubber, polymers and water are incompressible.

12.6 Flow Rule

The simplest form of the associated flow rule for a rigid perfectly plastic material is given by

$$\dot{\epsilon}_{ij} = \dot{\lambda}\frac{\partial F(\sigma_{ij})}{\partial \sigma_{ij}} \tag{12.29}$$

where the function $F(\sigma_{ij})$ is defined by Eq. (12.12), and $\dot{\lambda}$ is the scalar multiplication factor. Equation (12.15) determines uniquely the direction of the strain rate vector, which is always directed normal to the yield surface at a given stress point. In the case of plane stress, the two components of the strain rate vector are

$$\dot{\epsilon}_1 = \dot{\lambda}(2\sigma_1 - \sigma_2) \tag{12.30a}$$
$$\dot{\epsilon}_2 = \dot{\lambda}(2\sigma_2 - \sigma_1) \tag{12.30b}$$

The magnitudes of the components $\dot{\epsilon}_1$ and $\dot{\epsilon}_2$ are undetermined, but the ratio, which defines the direction $\dot{\epsilon}/\epsilon_2$, is uniquely determined.

In particular, under the transverse plain strain $\dot{\epsilon}_2 = 0$, so $\sigma_1 = 2\sigma_2$ and $\sigma_1 = \frac{2}{\sqrt{3}}\sigma_y$.
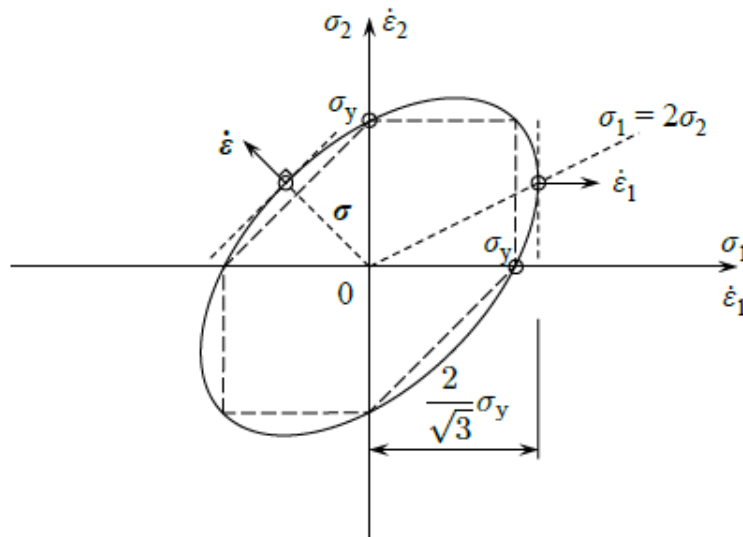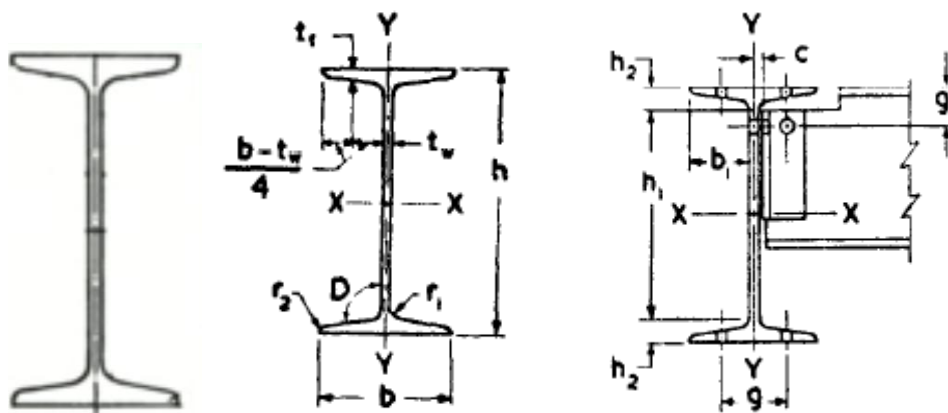
Figure 12.9: The strain rate vector is always normal to the yield surface.

First let us discuss about the I-sections. I section means how does it look and what are the things we have, let us discuss.

Let us draw theI- section how it looks. **(Draw it on Board)**

This total depth of the I-section is denoted in the code book [means SP: 6-(1): Hand Book for Structural Engineers] is denoted by *'h'.*

and the width of the flange is denoted by *'b'*

and this is called 'web'. Thickness of web is *'t$_w$'.*

Another thing is, thickness of flange is written as *'t$_f$'.*

This is the X-X direction and this is Y-Y direction.

and this 't$_f$'. As you see thickness of flange is varying from here to here.Right it is varying! so Where to measure this is a measure at a distance of (b - t$_w$) / 4. All this things details will get in SP: 6-(1). Right. where 'b' is the width of the flange and 't$_w$' is the thickness of the flange

So(b-t$_w$)/4, at a distance of (b-t$_w$)/4 from end of the flange,the thickness of the flange come is called 't$_f$' the thickness of flange. Right!.

and 'this is the slope of flange. This is termed as 'D'

this is called 'r$_1$' "radius at root".

this is called 'r$_2$' "radius at toe".

So these are the standard things which are given in the SP: 6-1 code. right So we have to know what are terms and accordingly the specifications has been given and value has been given. Right! now let us see how the terms are coming. Like

We use to say ISJB or JB what is this INDIAN STANDARD JUNIOR BEAM (ISJB) – JB
Indian Standard Junior Beam. It is denoted by ISJB or JB

Similarly, INDIAN STANDARD LIGHT BEAM (ISLB) that is denoted by LB
Indian Standard Light Beam, denoted by ISLB or LB

Similarly, INDIAN STANDARD **MEDIUM** WEIGHT **BEAM**(ISMB) – MB
Similarly Indian Stanadard Medium Weight Beam, denoted by ISMB or MB

Right! So different type of I- section beam means I section is available and mainly these are used for beams

INDIAN STANDARD **WIDE** FLANGE **BEAM** (ISWB) – WB
Similarly Indian Standard Wide Flange Beam, denoted by ISWB or WB

INDIAN STANDARD **HEAVY** WEIGHT **BEAM** (ISHB) – HB
Similarly Indian Standard Heavy Weight Beam, denoted by ISHB or HB

INDIAN STANDARD **COLUMN SECTION** (ISSC) – SC

Similarly Indian Standard Column Section, denoted by ISSC or SC

Now in Table-1 of SP: 6-1, there you see what the things are given in Table 1.
Supposefor ISMB 550, in Table 1 Corresponding to ISMB 550 we will get several parameter details.

First we will get 'w' which is weight per meter in Kg. so ISMB 550 when we are telling we know what is the weight of the beam.

Then you will get the 'a' 'a' is the section area. i.e., what is the cross-sectional area of ISMB 550 Section. That is also given there.

Then 'h' i.e., depth of section in mm
Then 'b' i.e., width of flange
Then 't_f' - thickness of flange
Then 't_w' - thickness of web

So in this way all the details will be given there. Apart from these what the other things are given. Along with these *'D', 'r₁', 'r₂'* are also given

So like I_{xx} Moments of Inertia about X-direction value, I_{yy} Moments of Inertia about Y-direction value, r_{xx} – radius of gyration about X direction, r_{yy} radius of gyration about Y-direction.
then section Modulus in X-direction, Z_{xx} ,
section modulus in Y-direction, Z_{yy}
r_1, radius at root,
r_2 radius at toe,
and D slope of flange
so these are things will be getting from HandBook.

Along with these,other things will get i.e., **connection details**.Because as it is a rolled section so some standard things are there which is given. so through that, we have to calculate. These are all standard. So for ISMB 550, the values has been given like '*h_1*', 'h_2', 'b_1', 'C', 'g (gauge distance)', 'g_1' and also maximum size of rivet. So all these you will get in the tabular form.
So if you go through the code you will understand in details.

Similarly for channel sectionhow does it look. For channel section it will be like this.

UNIT-II

COMPRESSION MEMBERS

Introduction Column, top chords of trusses, diagonals and bracing members are all examples of compression members. Columns are usually thought of as straight compression members whose lengths are considerably greater than their cross-sectional dimensions. An initially straight strut or column, compressed by gradually increasing equal and opposite axial forces at the ends is considered first. Columns and struts are termed "long" or "short" depending on their proneness to buckling. If the strut is "short", the applied forces will cause a compressive strain, which results in the shortening of the strut in the direction of the applied forces. Under incremental loading, this shortening continues until the column yields or "squashes". However, if the strut is "long", similar axial shortening is observed only at the initial stages of incremental loading. Thereafter, as the applied forces are increased in magnitude, the strut becomes "unstable" and develops a deformation in a direction normal to the loading axis and its axis is no longer straight. The strut is said to have "buckled".
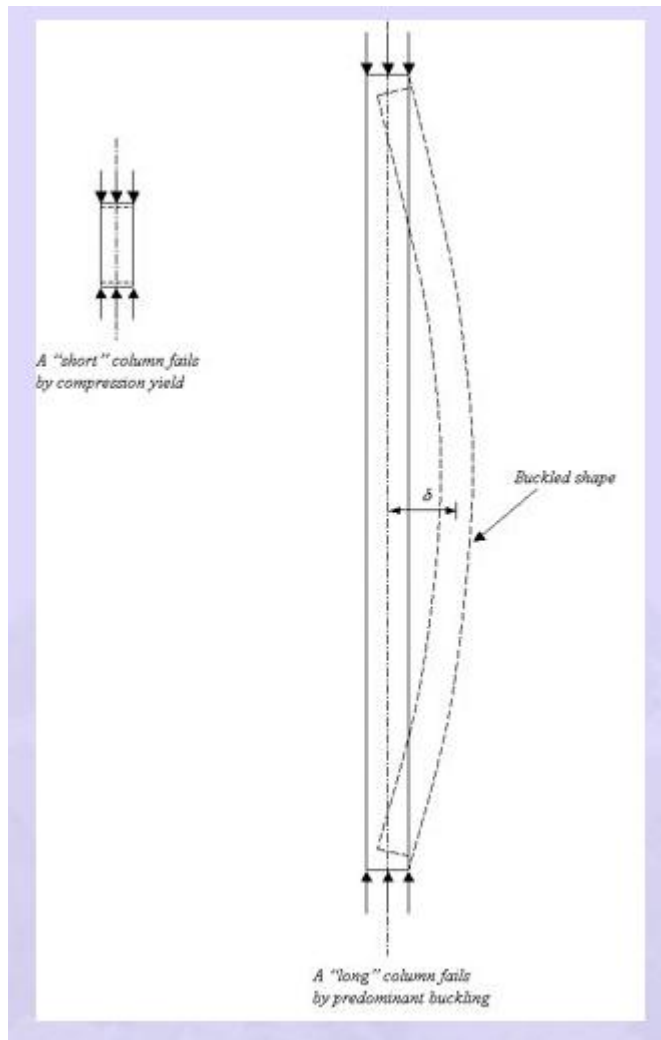
Short Columns vs Long Columns

Buckling behaviour is thus characterized by large deformations developed in a direction (or plane) normal to that of the loading that produces it. When the applied loading is increased, the buckling deformation also increases. Buckling occurs mainly in members subjected to compressive forces. If the member has high bending stiffness, its buckling resistance is high. Also, when the member length is increased, the buckling resistance is decreased. Thus the buckling resistance is high when the member is short or "stocky" (i.e. the member has a high bending stiffness and is short) conversely, the buckling resistance is low when the member is long or "slender".

Elastic buckling of an ideal column or strut with pinned end

The classical Euler analysis of the elastic behaviour of an idealized, pin-ended, uniform strut makes the following assumptions.
 • The material is homogeneous and linearly elastic (i.e. it obeys Hooke's Law).
 • The strut is perfectly straight and there are no imperfections.
 • The loading is applied at the centroid of the cross section at the ends

We will assume that the member is able to bend about one of the principal axes. (See Fig. 5.2). Initially, the strut will remain straight for all values of P, but at a particular value
 P = Pcr, it buckles. Let the buckling deformation at a section distant x from the end B be y.

**The bending moment at this section = Pcr.y**

 The differential equation governing the deformation can be obtained by considering moment equilibrium about point C as

Strength curve for all ideal strut We will assume that the stress-strain relationship of the material of the column is as shown in Fig. 5.5. Such a strut under compression can therefore resist only a maximum force given by fy.A, when plastic squashing failure would occur by the plastic yielding of the entire cross section; this means that the stress at failure of a column can never exceed fy
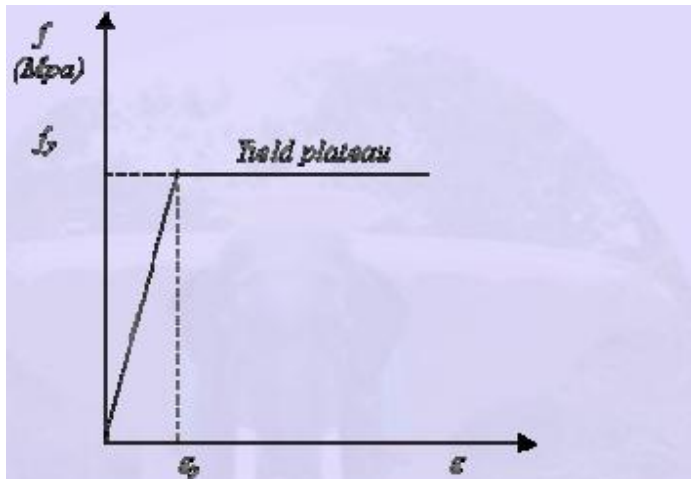
.5 Idealised elastic-plastic relationship for steel

it is obvious that the column would fail by buckling at a stress given by (π 2 EI / λ2)
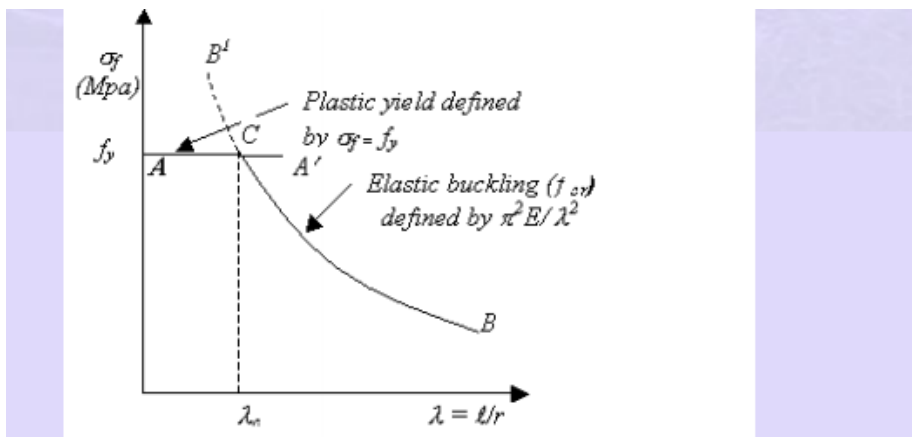
Strength curve for an axially loaded initially straight pin-ended column

Strength of compression members

 in practice The highly idealized straight form assumed for the struts considered so far cannot be achieved in practice. Members are never perfectly straight and they can never be loaded exactly at the centroid of the cross section. Deviations from the ideal elastic plastic behaviour defined by Fig. 5 are encountered due to strain hardening at high strains and the absence of clearly defined yield point in some steel. Moreover, residual stresses locked-in during the process of rolling also provide an added complexity. Thus the three components, which contribute to a reduction in the actual strength
of columns (compared with the predictions from the "ideal" column curve) are
(i) Initial imperfection or initial bow.
(ii) Eccentricity of application of loads.
(iii) Residual stresses locked into the cross section.

The concepts of effective lengths So far, the discussion in this chapter has been centred around pin-ended columns. The boundary conditions of a column may, however, be idealized in one the following ways
• Both the ends pin jointed (i.e. the case considered before)
 • Both ends fixed.
• One end fixed and the other end pinned.
• One end fixed and the other end free. By setting up the corresponding differential equations, expressions for the critical
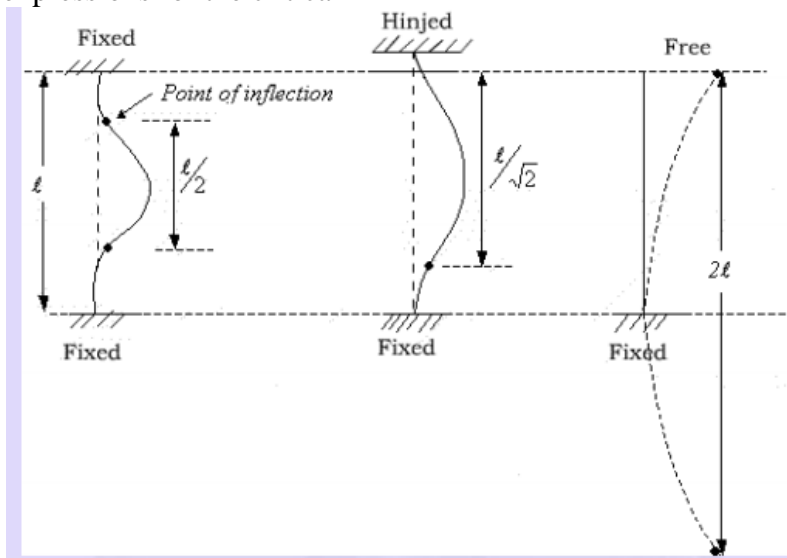
Buckled mode for different end connections
Using the column, pinned at both ends as the basis of comparison, the critical load in all the above cases can be obtained by employing the concept of "effective length", Le.

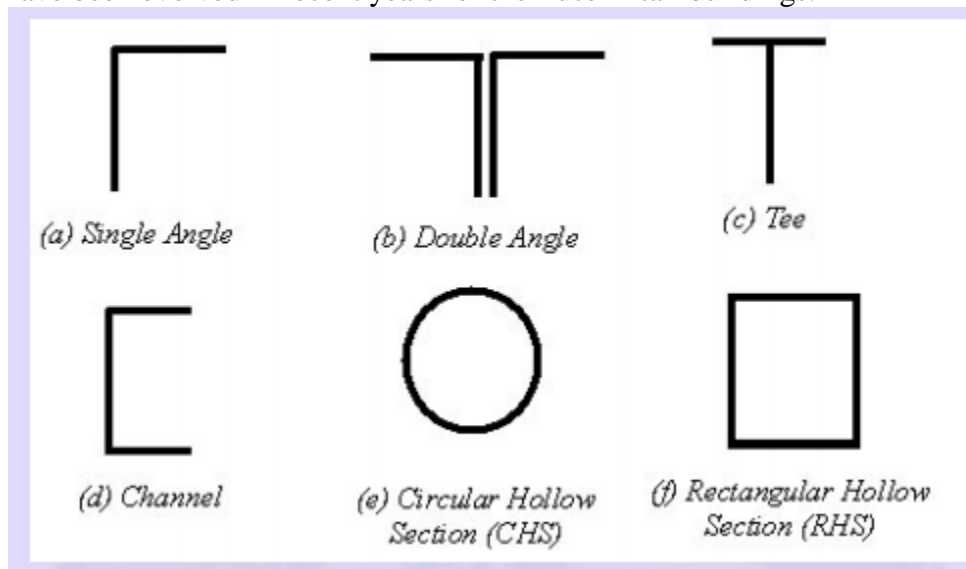**Effective length of compression members**

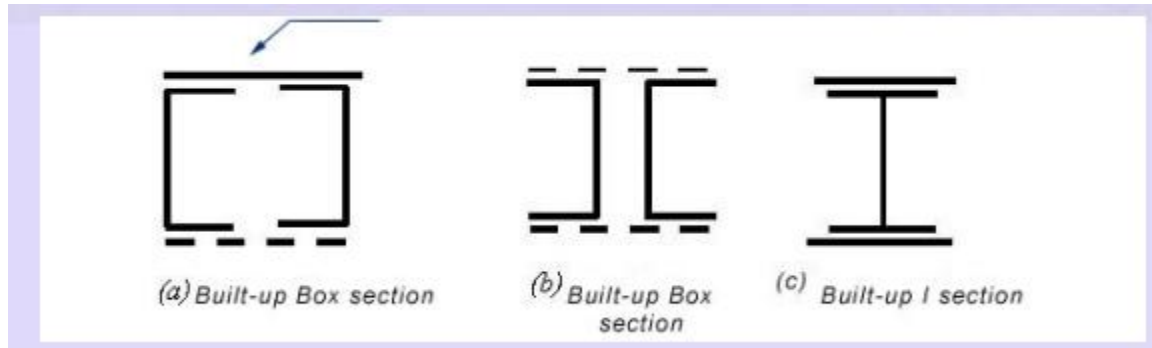| Boundary conditions	Theory	Code value (Cl.7.2.2)
Both ends pin ended	1.0L	1.0L
Both ends fixed	0.5L	0.65L
One end fixed and the other end pinned	0.707L	0.8L
One end fixed, and the other free to sway	1.2L	1.2L
One end fixed and the other end free	2.0L	2.0L

Types of column sections

Rolled Steel Sections
Some of the sections employed as compression members are shown in Fig.
Single angles are satisfactory for bracings and for light trusses. Top chord members of roof trusses are usually made up of double angles back-to-back. The pair of angles used, has to be connected together, so they will act as one unit. Welds may be used at intervals – with a spacer bar between the connecting legs. Alternately "stitch bolts", washers and "ring fills" are placed between the angles to keep them at the proper distance apart (e.g. to enable a gusset to be connected). Such connections are called tack connections and the terms tack welding or tacks bolting are used. When welded roof trusses are required, there is no need for gusset plates and T sections can be employed as compression members. Single channels or C-sections are generally not satisfactory for use in compression, because of the low value of radius of gyration in the weak direction. They can be used if they could be supported in a suitable way in the weak direction. Circular hollow sections are perhaps the most efficient as they have equal values of radius of gyration about every axis. But connecting them is difficult but satisfactory methods have been evolved in recent years for their use in tall buildings.

(a) Single Angle
(b) Double Angle
(c) Tee
(d) Channel
(e) Circular Hollow Section (CHS)
(f) Rectangular Hollow Section (RHS)

(a) Built-up Box section   (b) Built-up Box section   (c) Built-up I section

Design strength

Based on the studies of Ayrton & Perry (1886), the compressive strength of the column can be obtained from the following equation

$$\left(f_y - f_c\right)\left(f_e - f_c\right) = \eta.f_e.f_c$$

Where, f_y = yield stress, f_c = compressive strength, f_e = Euler buckling stress, λ = Slenderness ratio (l/r) and η = a parameter allowing for the effects of lack of straightness and eccentricity of loading and can be expressed as $a\lambda$ where α is a function of the shape of the cross section. Since Robertson evaluated the mean values of α for many sections, the design method is termed "Perry-Robertson method". Equation (5.8) will result in column strength values lower than f_y even in very low slenderness cases as indicated by the Robertson's curve in Fig. 5.19. By modifying the slenderness, λ to $(\lambda - \lambda o)$, a plateau to the design curve can be introduced for low slenderness values. This has the effect of shifting the curve to the right by a value equal to λo. The value of λo may be taken as $0.2(\pi\sqrt{E/fy})$. Thus, the elastic critical stress can be calculated as $fe = \pi 2 E/(\lambda - \lambda o) 2$. Note that calculations for fe is not needed when $\lambda \le \lambda e$ as the column would fail by squashing at f_y.
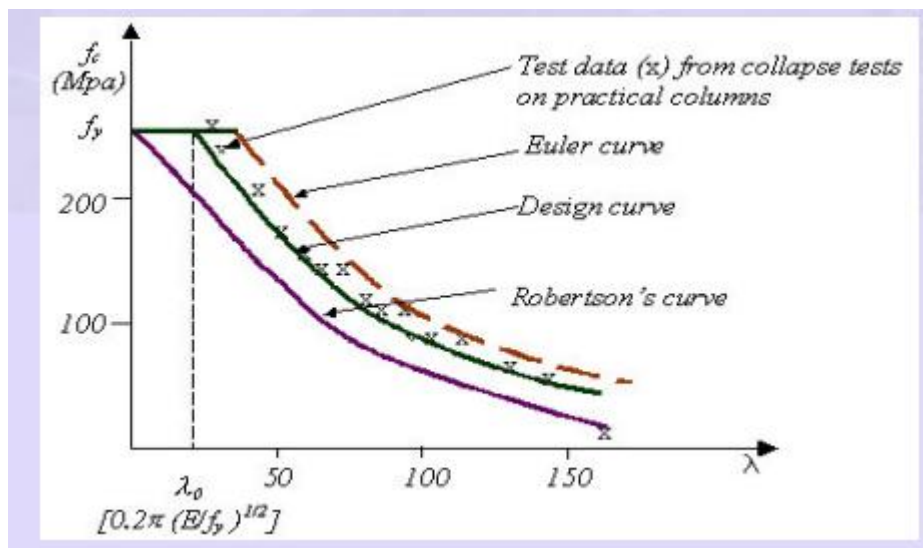
Column strength curves

Steps in the design of axially loaded columns

The procedure for the design of an axially compressed column is as follows:
(i) Assume a suitable trial section and classify the section in accordance with the classification in chapter. (ii) Arrive at the effective length of the column by suitably considering the end conditions.
(iii) Calculate the slenderness ratios (λ values) in both minor and major axes direction and also calculate λo using the formula given below:

$$\lambda_0 = 0.2\pi\sqrt{E\big/f_y}$$

(iv) Calculate fcd values along both major and minor axes from equation 12
(v) Compute the load that the compression member can resist (pd=Acfcd)
(vi) Calculate the factored applied load and check whether the column is safe against the given loading. The most economical but safe section can be arrived at by trial and error, i.e. repeating the above process.

DESIGN OF BATTENED COLUMNS:

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

1. Batten plates should be provided symmetrically.
2. At both ends batten plates should be provided. They should be provided at points where the member is stayed in its length.
3. The number of battens should be such that the member is divided into not less than three bays. As far as possible they should be spaced and proportioned uniformly throughout.
4. Battens shall be of plates, angles, channels, or I-sections and at their ends shall be riveted, bolted or welded.
5. By providing battens distance between the members of columns is so maintained that radius of gyration about the axis perpendicular to the plane of battens is not less than the radius of gyration about the axis parallel to the plane of the batten ($r_{yy} > r_{xx}$, in Fig. 6.6).
6. The effective slenderness ratio of battened columns shall be taken as 1.1 times the maximum actual slenderness ratio of the column, to account for shear deformation.
7. The vertical spacing of battens, measured as centre to centre of its end fastening, shall be such that the slenderness ratio of any component of column over that distance shall be neither greater than 50 nor greater than 0.7 times the slenderness ratio of the member as a whole about its z-z axis.
8. Battens shall be designed to carry the bending moments and shear forces arising from transverse shear force 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.
10. The design shear and moments for batten plates is given by

$$V_b = \frac{V_t C}{NS} \quad \text{and} \quad M = \frac{V_t C}{2N} \qquad \text{at each connection.}$$

where,

V_t = transverse shear force as defined in 8 and 9.

C = distance between centre to centre of battens longitudinally.

N = number of parallel planes.

S = minimum transverse distance between the centroid of the fasteners connecting batten to the main member.

11. The effective depth of end battens (longitudinally), shall not be less than the distance between the centroids of main members.
12. Effective depth of intermediate battens shall not be less than ¾th of above distance.
13. In no case the width of battens shall be less than twice the width of one member in the plane of the batten. It is to be noted that the effective depth of a batten shall be taken as the longitudinal distance between the outermost fastners.
14. The thickness of battens shall be not less than $\frac{1}{50}$th of the distance between the innermost connecting lines of rivets, bolts or welds.
15. The length of the weld connecting batten plate to the member shall not be less than half the depth of batten plate. At least one third of the weld shall be placed at each end of this edge.

Example:

Design the built up column of example 6.6 using battens instead of lacing system.

Solution:

The design of column is same as in the previous example i.e., use 2ISMC 350 @ 413 N/m

with clear spacing of 220 mm. $\frac{KL}{r} = 1.1 \times \frac{10000}{136.6} = 80.52$

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

Design of battens:

Let C be the spacing of battens, longitudinally.

Radius of gyration of one channel = 28.3 mm

$$\therefore \quad \frac{C}{28.3} < 50 \quad \text{i.e.,} \quad C < 1415.$$

It should also satisfy the condition,

$$\therefore \quad \frac{C}{28.3} < 0.7 \times 80.52 \quad \text{i.e.,} \quad C < 1595.$$

Let us select $C = 1200$ mm.

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

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

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

Size of battens:

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

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

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

Provide depth = 210 mm

Giving edge distance of 35 mm,

Overall depth = 210 + 2 × 35 = 280 mm

Thickness of battens $\nless \frac{1}{50} \times 340$

$\nless 6.8$

Use 8 mm thick plates.

Check for stresses in batten plates:

$$\text{Shear stress} = \frac{61765}{280 \times 8} = 27.57 \text{ N/mm}^2 < \frac{f_y}{\sqrt{3}} \times \frac{1}{1.1} \qquad \text{O.K.}$$

Shear stress < 0.6 × permissible stress

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

$$= 100.45 < 227.27 \text{ N/mm}^2 \qquad \text{O.K.}$$

Obviously end plate satisfies these requirements since it is deeper.

Connections:

It is to be designed to transmit both shear and bending moment.

Using 20 mm bolts,

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

Strength in bearing is much higher.

$\therefore$ Bolt value = 45272 N.

$$\text{Number of bolts required} = \frac{78125}{45272} = 1.72$$

Let us provide 3 bolts to take into account stresses due to bending also.

Check:

Force in each bolt due to shear $= \dfrac{78125}{3} = 26042$ N

Let the pitch be $\dfrac{210}{2} = 105$ mm

Force due to moment in extreme bolt $= \dfrac{Mr}{\Sigma r^2}$

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

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

Try 5 bolts as shown in Fig. 6.8.

i.e. Force in each bolt due to shear $= \dfrac{78125}{5} = 15625$ N

Force due to moment in extreme bolt $= \dfrac{Mr}{\Sigma r^2} = \dfrac{10500000 \times 100}{2(50^2 + 100^2)} = 42000$ N

$\therefore$ Resultant force $= \sqrt{15625^2 + 42000^2}$

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

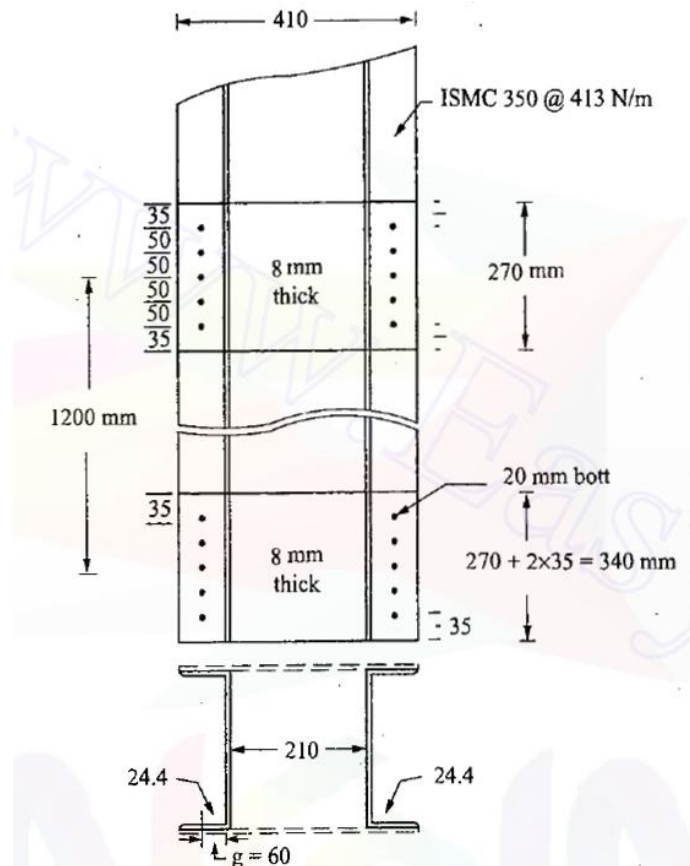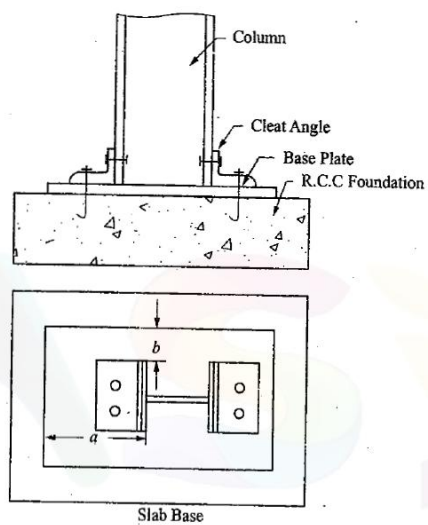Provide the bolts as shown in Fig. 6.8.

Figure 6.8

DESIGN OF SLAB BASE:

Slab Base

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

Size of Base plate:

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

(2) Therefore, area of base plate required $= \dfrac{P_u}{0.45 f_{ck}}$, where P_u is factored load.

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

Thickness of Base Plate:

(1) Find the intensity of pressure

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

(2) Minimum thickness required is given by

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

where

t_s = thickness of base plate

and t_f = thickness of flange.

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

Connections:

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

(2) If bolted connection is to be used for connecting column to base plate, use 2 ISA 6565, 6 mm thick angles with 20 mm bolts.

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

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

Solution:

Bearing strength of concrete $= 0.45\, f_{ck}$

$$= 0.45 \times 20 = 9 \text{ N/mm}^2$$

Factored load $P_u = 1000$ kN.

$$\therefore \quad \text{Area of base plate required} = \frac{1000 \times 10^3}{9}$$

$$= 111111 \text{ mm}^2$$

Provide 360×310 size plate.

Area provided $= 360 \times 310 = 111600 \text{ mm}^2$.

$$\text{Pressure} = \frac{1000 \times 10^3}{111600} = 8.96 \text{ N/mm}^2$$

Projections are

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

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

$$\therefore \quad t_s = \left[\frac{2.5 \times 8.96 \left(30^2 - 0.3 \times 30^2 \right) \times 1.1}{250} \right]^{0.5}$$

$$= 7.88 \text{ mm.}$$

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

Provide 12 mm thick plate.

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

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

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

Total length available for welding (Ref. Fig. 6.16)

$$= 2(250 + 250 - 7.6 + 300 - 2 \times 10.6) = 1542.4 \text{ mm.}$$

Strength of weld $= \dfrac{410}{\sqrt{3}} \times \dfrac{1}{1.25} = 189.37 \text{ N/mm}^2$

Let 's' be the size of weld. Then effective area of weld $= 0.7\, s\, L_e$

where L_e is effective length.

∴ The design condition is $0.7\, s\, L_e \times 189.37 = 1000 \times 10^3$

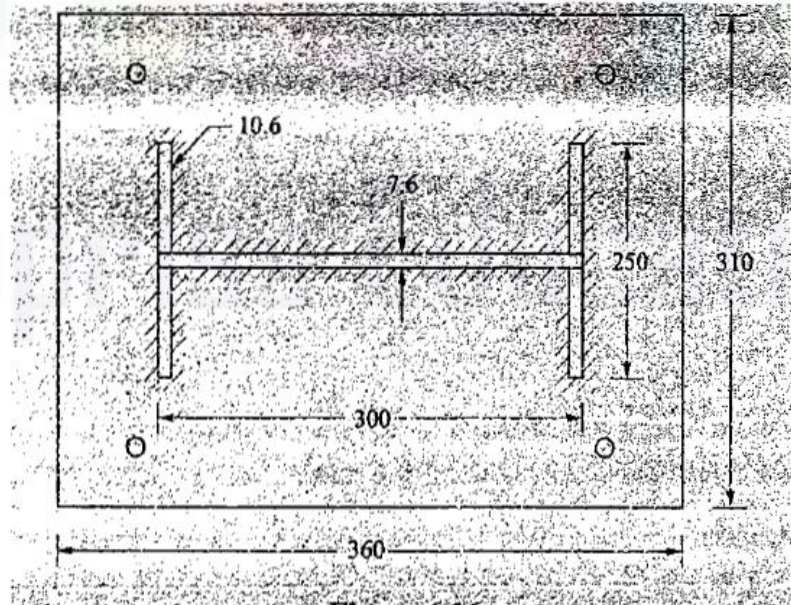$s\,L_e = 7543.8$

Using 6 mm weld, $L_e = 1257$ mm.

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

Available effective length $= 1542.4 - 2 \times 6 \times$ No. of returns (corners)
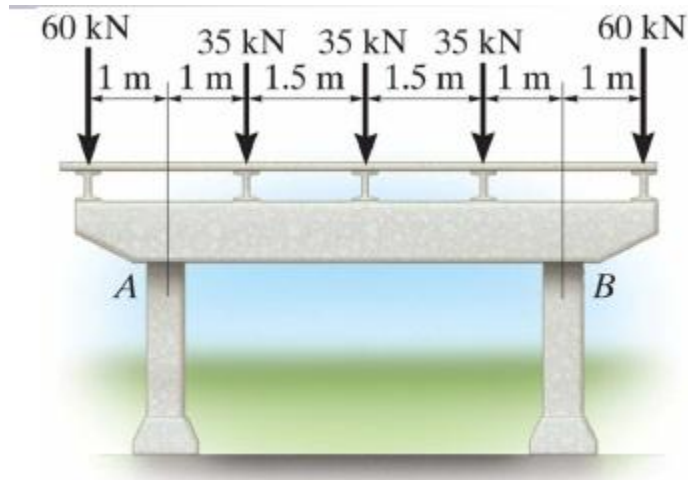
$$= 1542.4 - 2 \times 6 \times 12$$

$$= 1398.4 > 1257 \text{ mm.}$$

Hence 6 mm weld is adequate.

UNIT-III

BEAMS

A structural member subjected to transverse loads (Loads perpendicular to its longitudinal axis) is called a beam. See Figure ABOVE

Beams are most critical members in any structure. Their design should therefore not only be economical but also safe. The main considerations in the design of beams are:

1. They should be proportioned for strength in bending keeping in view the lateral and local stability of the compression flange and the capacity of the selected shape to develop the necessary strength in shear and local bearing.
2. They should be proportioned for stiffness, keeping in mind their deflections and deformations under service conditions.
3. They should be proportioned for economy, paying attention to the size and grade of steel to yield the most economical design.

Beam design consists merely of the provision of adequate bending and shear resistance. For optimum bending resistance, as much of the beams material as possible should be displaced as far as practicable from the neutral axis. The web area should be sufficient to resist shear.

Maximum moment and maximum shear usually occur at different sections. Though simple in design, the lateral buckling of beam as a whole, or of its compression flange or its web pose complications. Another problem is of proper depth – an increase in depth may be desirable for moment resistance, it may at the same impair resistance to lateral or web buckling (Figure).

TYPES OF BEAMS:

Beams are generally classified according to their geometry and the manner in which they are supported. They may be straight or curved.

Figure: Straight Beam

Girders usually the most important beams which are frequently at wide spacing.

Joists usually less important beams which are closely spaced, frequently with truss type webs.

Stringers- Longitudinal beams spanning between floor beams.

Purlins- Roof beams spanning between trusses

Girts- horizontal wall beams serving principally to resist bending due to wind on the side of an industrial building.

Lintels- Members supporting a wall over window or door openings.

Example:

Determine the design bending strength of ISLB 350 @ 486 N/m considering the beam to be

(a) laterally supported

(b) laterally unsupported

The design shear force V is less than the design shear strength. The unsupported length of the beam is 3.0 m. Assume steel of grade Fe 410.

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

Partial safety factor: $\gamma_{m0} = 1.10$

For rolled sections: $\alpha_{LT} = 0.21$

The relevant properties of ISLB 350 @ 486 N/m are:

Depth of section, $h = 350$ mm

Width of flange, $b_f = 165$ mm

Thickness of flange, $t_f = 11.4$ mm

Thickness of web, $t_w = 7.4$ mm

Radius at root, $R_1 = 16$ mm

Depth of web, $d = h - 2\,(t_f + R_1) = 350 - 2 \times (11.4 + 16) = 295.2$ mm

Moments of inertia: $I_z = 13158.3 \times 10^4$ mm^4, $I_y = 631.9 \times 10^4$ mm^4

Plastic section modulus: $Z_{pz} = 851.11 \times 10^3$ mm^3

Elastic section modulus: $Z_{ez} = 751.9 \times 10^3$ mm^3

Radius of gyration $r_y = 31.7$ mm

Section classification:

Section is classified as

Section is classified as	b/t_f	$\dfrac{d}{t_w}$
Plastic	$< 9.4\ \varepsilon$	$< 84\ \varepsilon$
Compact	$< 10.5\ \varepsilon$	$< 105\ \varepsilon$
Semi-compact	$< 15.7\ \varepsilon$	$< 126\ \varepsilon$

$$\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1.0$$

Outstand of flange, $b = \dfrac{b_f}{2} = \dfrac{165}{2} = 82.5$ mm

$$\frac{b}{t_f} = \frac{82.5}{11.4} = 7.23$$

$$< 9.4$$

$$\frac{d}{t_w} = \frac{295.2}{7.4} = 39.9$$

$$< 84$$

Hence, the section is plastic. Since section is plastic, $\beta_b = 1.0$.

Since
$$\frac{d}{t_w} = 39.9$$

$$< 67\,\varepsilon$$

Shear buckling check of web will not be required.

(a) *Laterally supported beam*

Design bending capacity,

$$M_d = \beta_b\, Z_{pz}\, \frac{f_y}{\gamma_{m0}} = 1.0 \times 851.11 \times 10^3 \times \frac{250}{1.1} \times 10^{-6} = 193.43 \text{ kNm}$$

$$< 1.2\, Z_e\, \frac{f_y}{\gamma_{m0}} = 1.2 \times 751.9 \times 10^3 \times \frac{250}{1.1} \times 10^{-6} = 205 \text{ kNm}$$

which is all right.

Hence, the design bending strength = 193.43 kNm.

(b) *Laterally unsupported beam*

Lateral-torsional buckling moment, $M_{cr} = \sqrt{\dfrac{\pi^2 EI_y}{L_{LT}^2} \left(GI_t + \dfrac{\pi^2 EI_w}{L_{LT}^2} \right)}$

Effective span, $\quad L_{LT} = KL = 3.0 \text{ m} = 3 \times 10^3 \text{ mm}$

Elastic modulus, $\quad E = 2 \times 10^5 \text{ N/mm}^2$

Shear modulus, $\quad G = \dfrac{E}{2(1 + \mu)} = \dfrac{2 \times 10^5}{2 \times (1 + 0.3)} = 76.92 \times 10^3 \text{ N/mm}^2$

St. Venant constant, $\quad I_t = \sum \dfrac{b_i t_i^3}{3} = 2 \times \dfrac{165 \times 11.4^3}{3} + \dfrac{(350 - 11.4) \times 7.4^3}{3}$

$$= 20.8 \times 10^4 \text{ mm}^4$$

Warping constant, $\quad I_w = (1 - \beta_f)\, \beta_f\, I_y\, h_f^2$

$$\beta_f = \frac{I_{fc}}{I_{fc} + I_{ft}} = 0.5 \qquad\qquad (\text{since } I_{fc} = I_{ft})$$

c/c distance between flanges, $h_f = 350 - t_f = 350 - 11.4 = 338.6$ mm

$$I_w = (1 - 0.5) \times 0.5 \times 631.9 \times 10^4 \times 338.6^2 = 1.81 \times 10^{11} \text{ mm}^6$$

$$M_{cr} = \sqrt{\frac{\pi^2 \times 2 \times 10^5 \times 631.9 \times 10^4}{3000^2} \times \left(76.923 \times 10^3 \times 20.8 \times 10^4 + \frac{\pi^2 \times 2 \times 10^5 \times 1.81 \times 10^{11}}{3000^2} \right)}$$

$$= 277.8 \times 10^6 \text{ Nmm}$$

$$= 277.8 \text{ kNm}$$

Example: Purlin Design

Design an I-section purlin, for an industrial building situated in the outskirts of Allahabad, to support a galvanised corrugated iron sheet roof for the following data:

Spacing of the truss c/c = 6.0 m
Span of truss = 12.0 m
Spacing of purlins c/c = 1.5 m
Intensity of wind pressure = 2 kN/m²
Weight of galvanised sheets = 130 N/m²
Grade of steel = Fe 410

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

Weight of galvanised corrugated iron sheets = 130 × 1.5 = 195 N/m
Assume dead load of purlin = 100 N/m
Total dead load = 195 + 100 = 295 N/m

The dead load acts vertically downwards.

The component of dead load parallel to roof = 295 sin 30° = 147.5 N/m
The component of dead load normal to roof = 295 cos 30° = 255.477 N/m

Wind load is assumed to act normal to the roof.

Wind load = 2 × 10³ × 1.5 × 1 = 3000 N/m

Total load on purlin normal to the roof
= 3000 + 255.477 = 3255.477 N/m

Factored load normal to roof,
P = 1.5 × 3255.477 = 4883.20 N/m (Table 3.1, γ_f = 1.5)

Factored load parallel to roof,
H = 1.5 × 147.5 = 221.25 N/m   (Table 3.1, γ_f = 1.5)

Maximum moment, $M_{uu} = M_z = \dfrac{PL}{10} = \dfrac{(4883.20 \times 6) \times 6 \times 10^{-3}}{10} = 17.579 \text{ kNm}$

Maximum moment, $M_{vv} = M_y = \dfrac{HL}{10} = \dfrac{(221.25 \times 6) \times 6 \times 10^{-3}}{10} = 0.796 \text{ kNm}$

Let us try a section with flange width $b_f = 75$ mm and depth, $d = 150$ mm

Plastic section modulus required, $Z_{pz,\, req} = M_z \dfrac{\gamma_{m0}}{f_y} + 2.5 \left(\dfrac{d}{b}\right)\left(M_y \dfrac{\gamma_{m0}}{f_y}\right)$

$Z_{pz,\, req} = 17.579 \times 10^6 \times \dfrac{1.1}{250} + 2.5 \times \dfrac{150}{75} \times 0.796 \times 10^6 \times \dfrac{1.1}{250} = 94.85 \times 10^3 \text{mm}^3$

From Appendix XV, ISMB 150 @ N/m furnishing plastic section modulus was tried ($Z_{pz} = 110.48 \times 10^3$ mm^3), but it failed in deflection. Hence, let us provide ISWB150@ 166.77 N/m with higher I_z and Z_{pz} values.

The sectional properties of ISWB 150 @ 166.77 N/m are:

Area,	$A = 2167 \text{ mm}^2$
Depth of section,	$h = 150$ mm
Width of flange,	$b_f = 100$ mm
Thickness of flange,	$t_f = 7.0$ mm
Thickness of web,	$t_w = 5.4$ mm
Radius at root,	$R_1 = 8.0$ mm
Depth of web,	$d = h - 2\,(t_f + R_1) = 150 - 2\,(7 + 8) = 120$ mm
Moments of inertia:	$I_z = 839.1 \times 10^4 \text{ mm}^4, I_y = 94.8 \times 10^4 \text{ mm}^4$

Plastic section modulus, $Z_{pz} = 126.86 \times 10^3$ mm^3

Elastic section modulus, $Z_{ez} = 111.9 \times 10^3$ mm^3, $Z_{ey} = 19.0 \times 10^3$ mm^3

Section classification

$$\varepsilon = \sqrt{\dfrac{250}{f_y}} = \sqrt{\dfrac{250}{250}} = 1$$

The outstand of flange, $b = \dfrac{b_f}{2} = \dfrac{100}{2} = 50$ mm

$$\dfrac{b}{t_f} = \dfrac{50}{7} = 7.14$$
$$< 9.4$$

$$\dfrac{d}{t_w} = \dfrac{120}{5.4} = 22.22$$
$$< 84$$

Hence, the section is plastic.

Check for design capacity of the section

$$M_{dz} = Z_{pz} \frac{f_y}{\gamma_{m0}} = 126.86 \times 10^3 \times \frac{250}{1.1} \times 10^{-6} = 28.83 \text{ kNm}$$

$$\leq 1.2 \, Z_{ez} \frac{f_y}{\gamma_{m0}} = 1.2 \times 111.9 \times 10^3 \times \frac{250}{1.1} \times 10^{-6} = 30.51 \text{ kNm}$$

which is all right.

Also, $M_{dz} = 28.83$ kNm

> 17.579 kNm

which is as should be.

$$M_{dy} = Z_{py} \frac{f_y}{\gamma_{m0}} \leq \gamma_f Z_{ey} \frac{f_y}{\gamma_{m0}}$$

$$Z_{py} = 2 \times \left(150 \times \frac{5.4}{2} \times \frac{5.4}{4} \right) + 2 \times \frac{100 - 5.4}{2} \times \left(\frac{100 - 5.4}{2} + \frac{5.4}{2} \right) \times 7.0$$

$$= 34.203 \times 10^3 \text{ mm}^3$$

$$M_{dy} = 34.203 \times 10^3 \times \frac{250}{1.1} \times 10^{-6} = 7.773 \text{ kNm}$$

$$\leq 1.5 \times 19.0 \times 10^3 \times \frac{250}{1.1} \times 10^{-6}$$

$$= 6.477 \text{ kNm} \qquad \left(1.2 \text{ is replaced by } \gamma_f \text{ since } \frac{Z_{py}}{Z_e} > 1.2 \right)$$

Hence, $M_{dy} = 6.475$ kNm > 0.796 kNm
which is all right.

Check for overall member strength (local capacity):

$$\frac{M_z}{M_{dz}} + \frac{M_y}{M_{dy}} \leq 1$$

$$\frac{17.579}{28.83} + \frac{0.796}{6.477} = 0.733 < 1$$

which is all right.

Check for deflection   Deflection will be checked at service loads.

$$\delta_{all} = \frac{l}{180} = \frac{6000}{180} = 33.33 \text{ mm}$$

$$\delta = \frac{5}{384} \times \frac{wl^4}{EI} = \frac{5}{384} \times \frac{3255.477 \times 10^{-3} \times (6 \times 10^3)^4}{2 \times 10^5 \times 839.1 \times 10^4} = 32.73 \text{ mm}$$
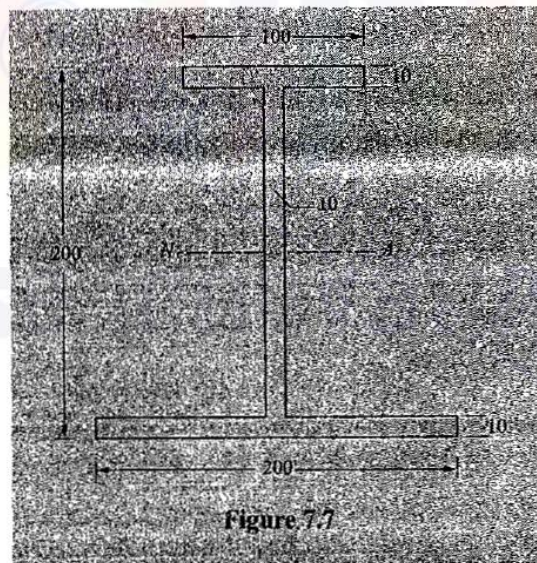
$$< 33.33 \text{ mm}$$

which is all right.

Note Purlins are usually not checked for shear and bearing since loads and shear are small.

BEAMS:

Determine the plastic moment capacity and plastic modulus of section of the unsymmetric section shown in Fig. 7.7.

Figure 7.7

Solution:

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

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

Plastic N-A is at a depth 'h' from top fibre where h is given by

$$100 \times 10 + (h - 10) \times 10 = 2400$$

$$\therefore \quad h = 150 \text{ mm}$$

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

$$M_p = \left(100 \times 10 \times (150 - 5) + 10 \times (150 - 10)\frac{(150 - 10)}{2} \right.$$

$$\left. + 10 \times (50 - 10)\frac{(50 - 10)}{2} + 200 \times 10 \,(50 - 5) \right) f_y$$

$$= 341000 \, f_y \text{ mm}^2 \qquad \textbf{Answer}$$

$$\therefore Z_p = \frac{M_p}{f_y} = 341000 \text{ mm}^3 \qquad \textbf{Answer}$$

Example 7.3

A roof of a hall measuring 8 m × 12 m consists of 100 mm thick R. C. slab supported on steel I-beams spaced 3 m apart as shown in Fig. 7.8. The finishing load may be taken as 1.5 kN/m² and live load as 1.5 kN/m². Design the steel beam.
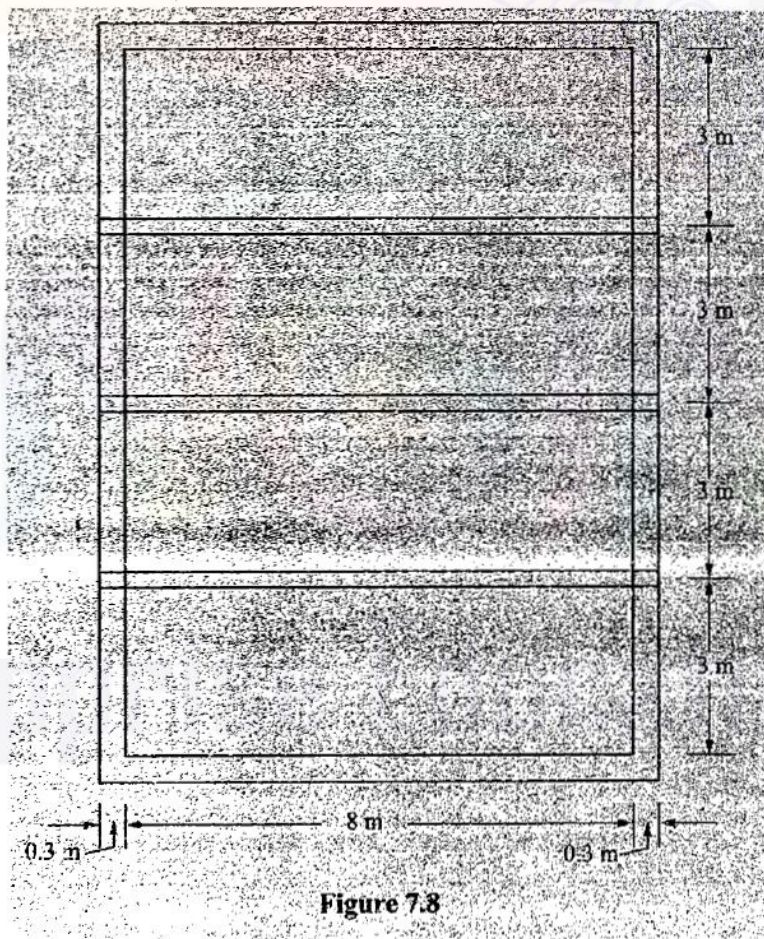
Solution:

Each beam has a clear span of 8 m and takes care of 3 m width of slab. Hence the load per metre length of the beam is as follows:

Weight of R.C. slab $= 0.1 \times 1 \times 3 \times 25 = 7.5 \text{ kN/m}$

Finishing load $= 1.5 \times 3 = 4.5 \text{ kN/m}$

Self weight (assumed) $= 0.8 \text{ kN/m}$

Figure 7.8

$\therefore$ Total dead load = 12.8 kN/m.

Live load = 1 × 3 × 1.5 = 4.5 kN/m.

$\therefore$ Factored dead load = 1.5 × 12.8 = 19.2 kN/m

Factored live load = 1.5 × 4.5 = 6.75 kN/m

$\therefore$ Total factored load = 25.95 kN/m.

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

Assuming width of support = 0.3 m,

Effective span = 8 + 0.3 = 8.3 m.

$\therefore$ Design moment, $M = \dfrac{wL^2}{8}$

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

Design shear force $V = \dfrac{25.95 \times 8.3}{2} = 107.69 \text{ kN}$

$\therefore$ Section modulus required $= \dfrac{M}{f_y} \times \gamma_{mo}$

$$Z_p = \dfrac{223.46 \times 10^6 \times 1.1}{250} = 983224 \text{ mm}^3$$

Try ISMB 400 which has $Z_p = 1176.163 \times 10^3 \, \text{mm}^3$.

The properties of the section are as follows:

Depth of section $h = 400 \, \text{mm}$

Width of flange $b = 140 \, \text{mm}$

Sectional area $A = 7845.58 \, \text{mm}^2$

Thickness of flange $t_f = 16.0 \, \text{mm}$

Thickness of web $t_w = 8.9 \, \text{mm}$

Depth of web $d = h - 2 \, (h_2)$

$$= 400 - 2(32.8) = 333.4 \, \text{mm}$$

Moment of inertia about z-z axis

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

Elastic section modulus $Z_e = 1022.7 \times 10^3 \, \text{mm}^4$

Outstanding leg of comp. flange, $b = \dfrac{140}{2} = 70$

Section Classification:

$$\in = \left(\frac{250}{f_y}\right)^{1/2} = \left(\frac{250}{250}\right)^{1/2} = 1.0$$

$$\frac{b}{t_f} = \frac{70}{16} = 4.38 < 9.4 \in$$

$$\frac{d}{t_w} = \frac{333.4}{8.9} = 37.57 < 84 \in$$

Hence the section is classified as plastic section:

Weight of the section = 0.604 kN/m.

Assumed weight = 0.8 kN/m.

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

Check for shear strength:

Design shear V = 107.69 kN

Design shear strength of the section

$$V_d = \frac{f_y}{\sqrt{3}} \times \frac{1}{1.1} \times \text{shear area}$$

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

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

$$= 467128 \text{ N} = 467.128 \text{ kN} > 107.61 \text{ kN}$$

Hence the section is adequate.

$$0.6 \, V_d = 0.6 \times 467.128 = 280.277 > 107.61 \text{ kN}$$

Hence it is not high shear case.

Check for moment capacity:

$$\frac{d}{t_w} = 38.2 \text{ which is less than } 67 \in, \text{ since } \in = 1.$$

Hence, $M_d = \beta_b \, Z_p \dfrac{f_y}{\gamma_{mo}} \leq 1.2 \dfrac{Ze \, f_y}{\gamma_{mo}}$

$\beta_b = 1.0$ since it is plastic section.

$$\therefore \quad M_d = 1.0 \times 1176.163 \times 10^3 \times \frac{250}{1.1} \leq 1.2 \times 1022.7 \times 10^3 \times \frac{250}{1.1}$$

$$= 267.310 \times 10^6 \leq 278.918 \times 10^6$$

Hence adequate. $\quad \therefore \quad M_d = 267.310 \times 10^6$ N-mm $= 267.310$ kN-m.

Check for deflection:

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

$$= 17.3 \text{ N/mm}$$

Maximum deflection

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

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

$$= 26.127 \text{ mm.}$$

Permissible deflection for a beam in building (Ref. Table 7.2) $= \dfrac{l_e}{300} = \dfrac{8300}{300} = 27.67$ mm
Hence deflection is within the permissible limit.

$\therefore$ Provide ISMB 400.

ECCENTRIC CONNECTION

Bracket type of connections are made wherever two members to be secured together do not intersect (Fig. 14.9). Structurally, this is not a desirable type of connection because a pronounced eccentricity is introduced both in the connection and in one of the members to be jointed. These type of joints are subjected to direct shear and torque/bending moment due to eccentric shear. When subjected to direct shear and shear due to torque these are called *bracket connection-type I* and in case shear is accompanied with tension (due to bending moment) these are called *bracket connection-type II.*

Two general approaches prevail for the analysis of bracket connection-type I, the elastic method and the ultimate strength method. In the elastic method the friction or the slip resistance between the connected parts is neglected. Further, the connected parts are assumed to be perfectly rigid and the connectors to be perfectly elastic. The elastic analysis has been in use since decades despite of being very conservative. The ultimate strength method though provides the most realistic values, but is extremely tedious to apply and even the calculations with handheld calculators are cumbersive. The code, however, does not specify any particular method for the analysis of bracket connection-type I, and has left it to the designer to decide which method to use.

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. 14.13 (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.

Elastic Analysis The eccentric load P can be made concentric along with a torque M, as shown in Fig. 14.13 (b). The forces in bolts due to direct shear and torque are F_1 and F_2, respectively as shown in Fig. 14.13 (b). The total force in each bolt is the resultant of the direct shear and shear due to torque. This resultant force in each bolt should be less than the strength of the bolt. From the assumption
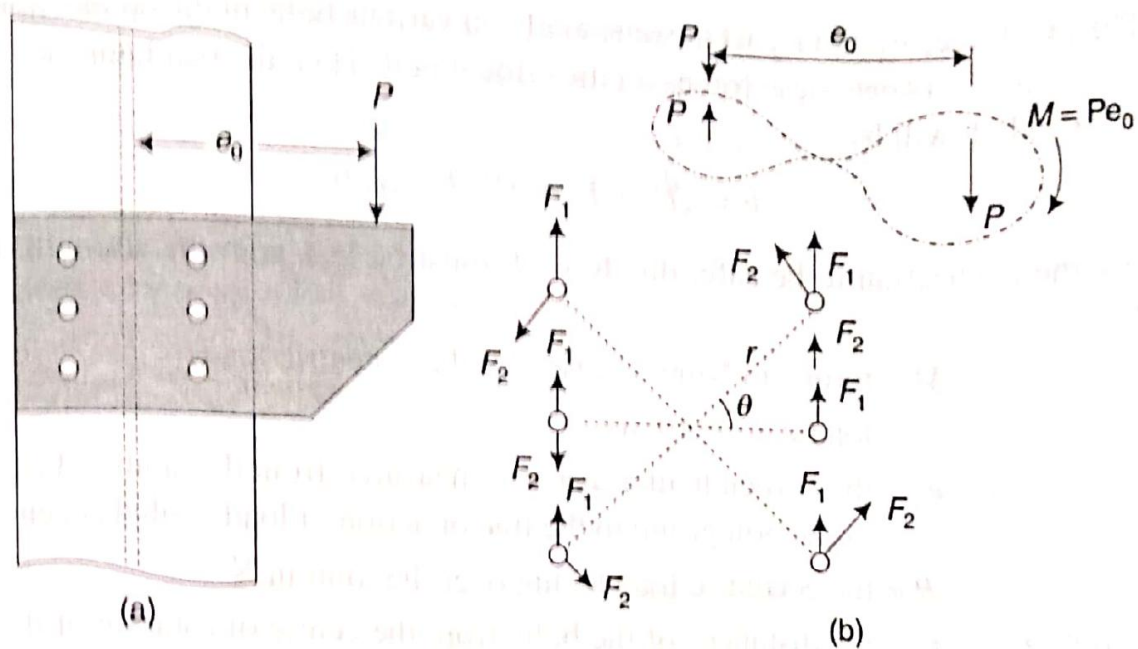
Fig. 14.13 Forces on bolts in bracket connection-type 1

made in the concentric bolted joints, 'the load over the joint is shared equally by all the bolts', force in any bolt due to direct load is,

$$F_1 = \frac{P}{n} \qquad (5)$$

Force in any bolt due to the torque is proportional to its distance from the centre of rotation of the connection.

$$F_2 \propto r \quad \text{or} \quad F_2 = Kr$$

or

$$K = \frac{F_2}{r}$$

Therefore, the torque about the centre of rotation of the bolt group

$$= F_2 r = Krr = Kr^2$$

Total resisting torque $= \Sigma \, Kr^2$

$$= K \Sigma \, r^2$$

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

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

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

or

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

or

$$F_2 = \frac{Pe_0 \, r}{\Sigma r^2} \qquad (6)$$

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

$$F_2 = \frac{Pe_0 \, r_n}{\Sigma r^2} \qquad (7)$$

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

$$F = \sqrt{F_1^2 + F_1^2 + 2F_1 \, F_2 \cos \theta} \qquad (8)$$

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

n = number of bolts in the bolt group

F_1 = force in a bolt due to direct shear P

F_2 = force in a bolt due to torque M

K = constant.

Design Procedure In the elastic method of analysis it is assumed that there is linear relation between loads and deformations for the bolts. It is also assumed that their yield stress is not exceeded when ultimate load on the connection is reached. However, the experiments do not support these assumptions and thus the method is not accurate but very conservative however easy in application.

1. To design bracket connection-type I, the number and diameter of bolts for the connection are assumed (see also note below) and these are placed in two or more vertical rows at a suitable pitch and edge distance.
2. The resultant force on the critical bolt is worked out by Eq. (8).
3. The strength of the bolt is computed. It should be more than the resultant force on the critical bolt.

Note

For convenience the number of bolts required (in one vertical row) may be worked out from Eq. (9).

$$n = \sqrt{6M/n' \, pV_{sd}}$$

(9)

Design of eccentric connection, framed, stiffened and seat connection.

1. How are the building connections classified based on their moment-rotation characteristics?

2. Determine the safe load P that can be carried by the joint shown in Figure. The bolts used are 20 mm diameter of grade 4.6. The thickness of the Flange of I-section is 9.1 mm and that of bracket plate 10 mm. [page No. 717 S.K. Duggal 2nd edition] [5 Marks]
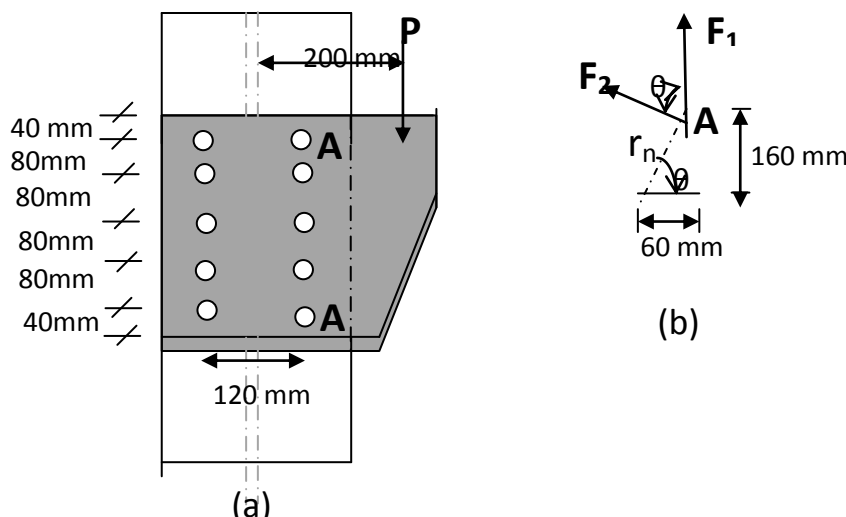
(a)

(b)

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: $\gamma_{mb} = 1.25$

A_{nb} = stress area of 20 mm diameter bolt = 0.78 x $\prod$x 20^2/4 = 245 mm^2

Given: diameter of bolt, d = 20 mm; pitch, p = 80 mm; edge distance, e = 40 mm (2 x20 mm), d = 20 + 2 = 22 mm.

Strength of bolt in single shear, $V_{dsb} = A_{nb}\dfrac{f_{ub}}{1.732 \; x \; 1.25}$

$$= 245 \text{ x } \frac{400}{1.732 \; X \; 1.25} \times 10^{-3} = 45.26 \; kN$$

Strength of bolt in bearing, $V_{dpb} = 2.5 \; k_b \; dt \; \dfrac{f_u}{\gamma_{mb}}$

K_b is least of $\dfrac{e}{3d_0} = \dfrac{40}{3 \; x \; 22} = 0.606$; $\dfrac{p}{3d_0} - 0.25 = \dfrac{80}{3 \; x \; 22} - 0.25 = 0.96$

$\dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.975$; and 1.0

Hence $K_b = 0.606$

$V_{dpb} = 2.5 \; k_b \; dt \; \dfrac{f_u}{\gamma_{mb}} = 2.5 \text{ x } 0.606 \text{ x } 20 \text{x } 9.1 \text{ x } \dfrac{410}{1.25} \times 10^{-3} = 90.44 \; kN$

Hence strength of bolt is $V_{sd} = 45.26$ kN

Let, P_1 be the factored load.

Service load, $P = \dfrac{P_1}{loadfactor} = \dfrac{P_1}{1.50}$

The bolt which is stressed maximum is A

Total number of bolts in the joint , n = 10

The force direct force, $F_1 = \dfrac{P_1}{n} = \dfrac{P_1}{10}$

The force in the bolt due to torque, $F_2 = \dfrac{Pe_0r_n}{\sum r^2}$

$r_n = \sqrt{(80+80)^2 + \left(\dfrac{120}{2}\right)^2} = 170.88$ mm

$\sum r^2 = 4 \times [(160^2 + 60^2) + (80^2 + 60^2)] + 2 \times 60^2 = 164{,}000$ mm^2

$F_2 = (P_1 \times 200 \times 170.88)/(164{,}000) = 0.20839\, P_1$

$Cos\theta = \dfrac{60}{\sqrt{60^2 + 160^2}} = 0.3511$

*The resultant force on the bolt should be less than or equal to the strength of bolt.*

$45.26 \le \sqrt{\left(\dfrac{P_1}{10}\right)^2 + (0.20839 P_1)^2 + 2 \times \dfrac{P_1}{10} \times 0.20839 P_1 \times 0.3511}$

Implies $P_1 = 173.49$ kN

The service load, $P = \dfrac{P_1}{loadfactor} = \dfrac{173.49}{1.5} = 115.65$ kN

L.Date : 24/march/2015.

3. Design a bracket connection to transfer an end reaction of 225 kN due to factored loads as in Figure below. The end reaction from the girder acts at an eccentricity of 300 mm from the face of the column flange. Design bolted joint connecting the Tee-flange with the column flange. Steel is of grade Fe 410 and bolts of grade 4.6. [5 Marks] [Page No. 721, S.K. Duggal 2nd edition]
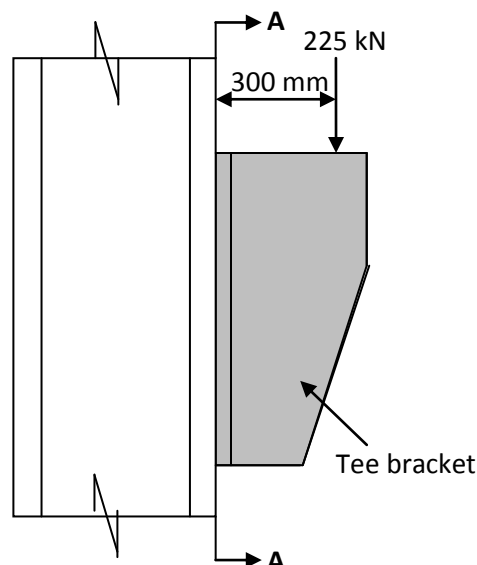
Solution: For Fe 410 grade of steel: f_u = 410 Mpa

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

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

The bolts along section AA are subjected to

(i) Shear due to the load, P = 225kN passing through the c.g. of the joint
(ii) Tension due to bending moment, M = 225 x 300 = 67,500 kNmm
     Let us provide 24 mm diameter bolts for making the connection.

For 24 mm diameter bolts
Stress area, A_{nb} = 353 mm^2
Minimum pitch, p = 2.5 x 24 = 60 mm ≈ 65 mm
Edge distance = 1.5 x (24+2) = 39 mm ≈ 40 mm

Strength of the bolt in single shear,
Vdsb = Vsd = $A_{nb}\dfrac{f_{ub}}{1.732\ x\gamma_{mb}}$ = 353 x $\dfrac{400}{1.732\ x\ 1.25}x10^{-3}$

= 65.22 kN

Strength of bolts in tension T_{db} = T_{nb}/γ_{mb}

T_{nb} = 0.9 f_{ub} A_{nb} = 0.9 x 400 x 353 x10^{-3} = 127.08 kN

$\Big|$ not greater fyb $\dfrac{\gamma_{mb}}{\gamma_{m0}}A_{sb}$ = 250 x $\dfrac{1.25}{1.10}x$ 452 $x10^{-3}$= 128.40 kN

Hence, T_{db} = T_{nb}/γ_{mb} = 127.08 / 1.25 = 101.66 kN

The bolts will be provided in two vertical rows, one on each side of the web of the Tee section, connecting the flanges 0f the two sections.

Number of bolts required in one row , n = $\sqrt{\dfrac{6M}{pn\ \mathcal{V}_{sd}}}=\sqrt{\dfrac{6\times67{,}500}{2\times65\times65.22}}$ =6.91≈ 7

Hence provide 7 bolts in each row at a pitch of 65 mm and edge distance of 40 mm.

Total depth of the bracket plate = 6 x 65 + 2 x 40 = 470 mm

h = 470 – 40 = 430 mm

The neutral axis is assumed to lie at h/7 from the bottom of the bracket, i.e., at 430/7 = 61.42 mm

$\sum y_i$ = 2 x [(65 + 40 – 61.42) + (130 + 40 + 61.42) + (195 + 40 - 61.42) + (260 + 40 -61.42) + 325 + 40 -61.42) +(390 +40 -61.42)]

= 2472.96 mm

$\sum y_i^2$ = 2 x [43.58^2 + 108.58^2 + 173.58^2 + 238.58^2 + 303.58^2 +368.58^2]= 657,502.6 mm^2

M' = $\dfrac{M}{1+\dfrac{2h}{21}\dfrac{\sum y_i}{\sum y_i^2}}$ = $\dfrac{67.5 \times 10^3}{1+\dfrac{2\times430}{21}\dfrac{2472.96}{657502.6}}$ = 58.49 x 10^3 kNm

Tensile force in the critical bolt,

T$_b$ = $\dfrac{M'y_n}{\sum y_i^2}$ = $\dfrac{58.49 \times 10^3}{657502.6}$ × 368.58 = 32.79 kN (y$_n$ = 368.58mm)

Shear force in the critical bolt,

V$_{sb}$ = $\dfrac{P}{number of bolts}$ = $\dfrac{225}{2\times7}$ = 16.07 kN

Check

$\left(\dfrac{Vsb}{Vdsb}\right)^2 + \left(\dfrac{T_b}{T_{db}}\right)^2 \leq 1.0$

$\left(\dfrac{16.07}{65.22}\right)^2 + \left(\dfrac{32.79}{101.66}\right)^2$ =0.1647 ≤ 1.0

Which is as it should be.

4. An ISLB 300 @ 369.8 N/m transmits an end reaction of 385 kN, under factored loads, to the web of ISMB 450 @710.2 N/m. Design a bolted framed connection. Steel is of grade Fe410 and bolts are of grade 4.6. [5 Marks] [Page No. 723 S.k. Duggal 2ndb edition]
5. Design a stiffened seat connection for an ISMB 350@ 514 N/m transmitting an end reaction of 320 kN (due to factored loads) to a column section ISHB 300 @ 576.8 N/m. The steel is of grade Fe 410 and bolts of grade 4.6. [5 Mark s] [Page No. 727, S.k. Duggal 2ndb edition]

<div align="center">

UNIT-V

PLATE GIRDERS

</div>

Design a welded plate girder 24 m in span and laterally restrained throughout. It has to support a uniform load of 100 kN/m throughout the span exclusive of self-weight. Design the girder without intermediate transverse stiffners. The steel for the flange and web plates is of grade Fe 410. Yield stress of steel may be assumed to be 250 MPa irrespective of the thickness of plates used. Design the cross section, the end load bearing stiffener and connections.

Solution:

For Fe 410 grade of steel: f_u = 410 MPa, $f_y = f_{yp} = f_{yw}$ = 250 Mpa
μ = 0.3

$E = 2 \times 10^5$ MPa

Partial safety factors, γ_{mw} = 1.50 (for site welding)

$\qquad\qquad\qquad\qquad\qquad\qquad\qquad$ = 1.25 (For shop welding)

$\varepsilon = \varepsilon_w = \varepsilon_f = \sqrt{\dfrac{250}{f_y}} = \sqrt{\dfrac{250}{250}} = 1.0$

Design Forces
Total superimposed load = 100 kN/m
Factored superimposed load = 1.5 x 100 = 150 kN/m
Let, self-weight of plate girder $= \dfrac{WL}{400} = \dfrac{(100 \times 24) \times 24}{400}$

$\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad$ =144 kN

Self-weight of plate-girder per meter length $= \dfrac{144}{24} = 6$ kN/m

Factored self weight = 1.5 x 6 = 9 kN/m
Total uniform factored load = 150 + 9 = 159 kN/m
Maximum bending moment $= \dfrac{159 \times 24^2}{8} = 11,448$ kNm
Maximum shear force $= \dfrac{159 \times 24}{2} = 1908$ kN

Design of web

Optimum depth of plate girder, $d = \left(\dfrac{M_z k}{f_y}\right)^{0.33}$

When intermediate transverse stiffeners are not to be provided;

$d/t_w \leq 200\varepsilon$ i.e., 200 (from serviceability criteria)

$\qquad\qquad\quad \leq 345\,\varepsilon_f^2$ i.e., 345 (from flange buckling criteria)

and

Let us assume $k = d/t_w = 180$

Optimum depth of plate girder, $d = \left(\dfrac{M_z k}{f_y}\right)^{0.33}$

$$= \left(\dfrac{11448 \times 10^6 \times 180}{250}\right)^{0.33}$$

$= 1871.9$ mm ≈ 1800 mm

Optimum web thickness, $t_w = \left(\dfrac{M_z}{f_y k^2}\right)^{0.33} = \left(\dfrac{11448 \times 10^6}{250 \times 180^2}\right) = 10.95$ mm ≈ 12 mm

(Thickness provided is more since intermediate transverse stiffeners are not to be provided)

Let us try web plate 1800 x 12 mm in size.

Design of Flanges

Let us assume that bending moment will be resisted by the flanges and shear by the web.

Required area of Flange, $A_f = \dfrac{M_z \gamma_{m0}}{f_y d} = \dfrac{11448 \times 10^6 \times 1.10}{250 \times 1800} = 27984$ mm^2

Assuming width of flange equal to 0.3 times depth of girder,

$b_f = 0.3 \times 1800 = 540$ mm ≈ 560 mm

Thickness of flange, $t_f = \dfrac{27984}{560} = 49.97 \approx 50$ mm

Classification of flanges

For the flanges to be classifiable as plastic $b/t_f \leq 8.4\varepsilon$ (ε is yield stress ratio)

The outstand of flange, $b = \dfrac{b_f - t_w}{2} = \dfrac{560 - 12}{2} = 274$ mm

$$\frac{b}{t_f} = \frac{274}{50} = 5.48$$

$$< 8.4 \quad (8.4\varepsilon = 8.4 \text{ x } 1 = 8.4)$$

Hence, the flanges are plastic. ($\beta_b = 1.0$)

Check for bending strength

The trial section of the plate girder is shown in Figure 1. The plastic section modulus of the section,

$Z_{pz} = 2 \text{ b}_f \text{ t}_f \frac{(D-t_f)}{2} = 2 \text{ x } 560 \text{ x } 50 \text{ x } \frac{1900-50}{2} = 51.80 \text{ x } 10^6 \text{ mm}^3$

Moment Capacity,

$M_d = \beta_b Z_{pz} \frac{f_y}{\gamma_{m0}} = 1.0 \text{ x } 51.80 \text{ x } 10^6 \text{ x } \frac{250}{1.10} \times 10^{-6} = 11772.7 \text{ kNm}$

$$> 11448 \text{kNm}$$

which is safe.

Shear capacity of web

Let us use simple post-critical method.

$$\frac{d}{t_w} = \frac{1800}{12} = 150$$

$< 200 \quad (200\varepsilon = 200 \text{ x } 1 = 200)$

and also < 345

$$(345\varepsilon^2 = 345 \text{ x } 1 = 345)$$

which is all right.

Elastic critical shear stress,

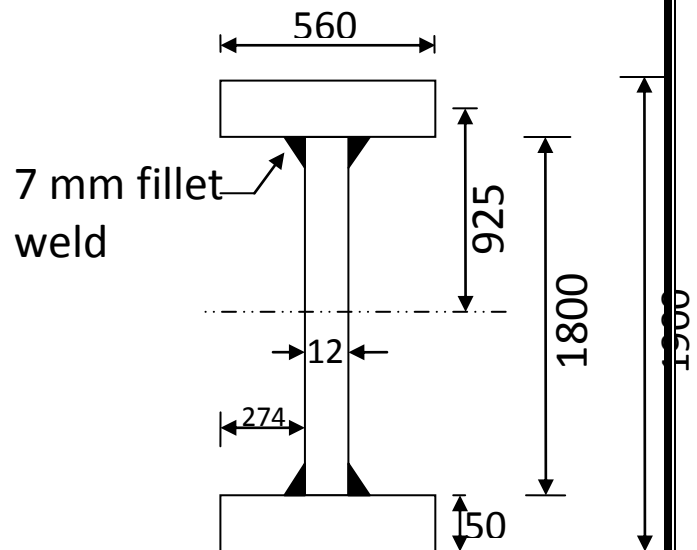$$\tau_{cr,e} = \frac{k_v \pi^2 E}{12(1-\mu^2)\left(\frac{d}{t_w}\right)^2}$$

Figure 1.

Transverse Stiffeners will be provided at supports only. Hence, K_v=5.35

$$\tau_{cr,e} = \frac{5.35 \times \pi^2 \times 2 \times 10^5}{12(1-0.3^2)(150)^2} = 42.98 \text{ N/mm}^2$$

The non-dimensional web slenderness ratio for shear buckling stress,

$$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3} \times \tau_{cr,e}}} = \sqrt{\frac{250}{\sqrt{3} \times 42.98}} = 1.83 \approx 1.80$$

$$> 1.20$$

Shear stress corresponding to buckling (For $\lambda_w > 1.20$),

$$\tau_b = \frac{f_{yw}}{\sqrt{3} \times \lambda_w^2} = \frac{250}{\sqrt{3} \times 1.80^2} = 44.55 \text{ N/mm}^2$$

Shear force corresponding to web buckling,

$V_{cr} = dt_w\tau_b = 1800$ x 12 x 44.55 x $10^{-3} = 962.28$ kN < 1908kN

Which is unsafe.

Let us revise the web thickness from 12 mm to 16 mm.

New values of $\tau_{cr,e}$, λ_w , τ_b, and V_{cr} will be as follows.

$$\frac{d}{t_w} = \frac{1800}{16} = 112.5$$

$$\tau_{cr,e} = \frac{5.35 \times \pi^2 \times 2 \times 10^5}{12(1-0.3^2)(1)^2} = 76.41 \text{ N/mm}^2$$

$$\lambda_w = \sqrt{\frac{f_{yw}}{\sqrt{3} \times \tau_{cr,e}}} = \sqrt{\frac{250}{\sqrt{3} \times 76.41}} = 1.374 = 1.37 > 1.2$$

$$\tau_b = \frac{f_{yw}}{\sqrt{3} \times \lambda_w^2} = \frac{250}{\sqrt{3} \times 1.37^2} = 76.90 \text{ N/mm}^2$$

$V_{cr} = dt_w\tau_b = 1800$ x 12 x 44.55 x $10^{-3} = 2214.7$ kN > 1908kN

Which is safe.

Check for lateral-torsional buckling

Since the compression flange of the girder is laterally restrained throughout, the possibility of lateral-torsional buckling is not there and this check is not required.

Flange to web connection

There will be two weld lengths along the span for each flange to web connection [Figure 1]

$$q_w = \frac{V A_f \bar{y}}{2 \times I_z}$$

$$I_z = \frac{b_f D^3}{12} - \frac{(b_f - t_w)d^3}{12}$$

$$= \frac{560 \times 1900^3}{12} - \frac{(560-16)1800^3}{12}$$

$$= 55702.6 \text{ x } 10^6 \text{ mm}^4$$

$$q_w = \frac{1900 \times 560 \times 50 \times (900 + \frac{50}{2})}{2 \times 55702.6 \ x \ 10^6}$$

Let us provide weld of size, $S = 6$ mm

$KS = 0.7 \text{ x } 6 = 4.2$ mm

Strength of shop weld per unit length,

$$F_{wd} = \frac{4.2 \times 250 \times 10^{-3}}{\sqrt{3} \times 1.25} = 0.485 \text{ kN/mm} > 0.4436 \text{ kN/mm}$$

Which is all right.

End bearing stiffener

Local capacity of the web,

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

B1 = 125 mm

N2 = 50 x 2.5 = 125 mm

F_w = (125 + 125) x 16 x $\frac{250}{1.10} \times 10^{-3}$

= 909.09 kN

< 1908 kN

Hence, stiffener will be required.
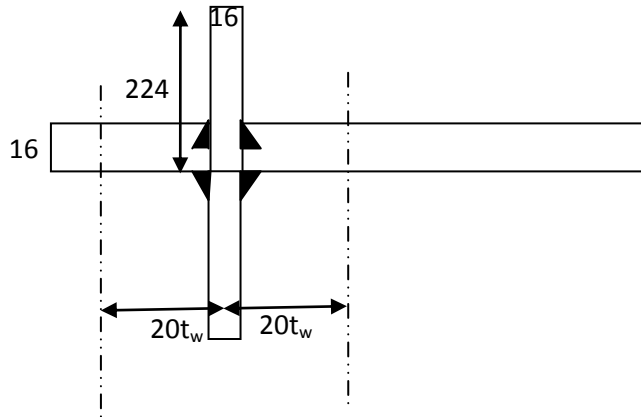
Maximum reaction = 1908 kN

Let us try two flat sections, as stiffener, one on each side of web. Maximum width of flat that can be accommodated = $\frac{560-16}{2}$ = 272 mm

Let us provide 16 mm thick flat section.

Maximum permissible outstand = 2 x t_q ε = 20 x 16 x1 = 224mm

Let us try flat section 224 x 16 mm in size **[Figure 2]**

Check for buckling of the stiffener

Effective area of stiffener = 2 x 224 x 16 + (2 x 20 x 16) x 16 = 17408 mm^2.

Moment of Inertia of the stiffener,

$$I_x = 2 \text{ x } \left[\frac{16 \times 224^3}{12} + 16 \times 224 \times \left(\frac{224}{2} + \frac{16}{2} \right)^2 \right] = 13319.1 \times 10^4 mm^4$$

Radius of Gyration, $r = \sqrt{\frac{I_x}{A}} = \sqrt{\frac{13319.1 \times 10^4}{17408}} = 87.47$ mm

Slenderness ratio, $\lambda = \frac{0.7 \times 1800}{87.47} = 14.41$

For $\lambda = 14.41$, $f_y = 250$ N/mm^2, and buckling curve c, the design compressive stress from Table 8.7,

$f_{cd} = 225.67$ N/mm^2

Buckling resistance, $P_d = A_e f_{cd} = 17408 \times 225.67 \times 10^{-3} = 3928.46$ kN

> *1908 kN*

Which is safe. Hence, stiffener is safe in compression

Check for bearing capacity of the stiffener

Since the stiffener will be coped to accommodate the fillet weld of flange plate to the web, the available effective width of stiffener flat for bearing will be lesser than the actual width. Let the stiffener plate be coped by 15 mm [Figure 3]

Width available for bearing $= 224 - 15 = 209$ mm

Bearing strength of the stiffener,

$F_{psd} = \frac{A_q f_{yp}}{0.8\, \gamma_{m0}} \qquad \geq F_c - F_w$

Area of stiffener in contact with flange,

$A_q = 2 \times 209 \times 16 = 6688\ mm^2$

$F_c - F_w = 1908 - 909.09 = 998.91\ kN$

$F_{psd} = \frac{6688 \times 250 \times 10^{-3}}{0.8 \times 1.10} = 1900$ kN > 998.91 kN

Which is safe.

Check for torsional resistance provided by end bearing stiffener

The ends of the plate girder must have sufficient torsional resistance from transportation and erection view point.

The moment of inertia of the end bearing stiffener at support,

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

$I_y = \frac{2t_f b_f^3}{12} + \frac{dt_w^3}{12} = \frac{2 \times 50 \times 560^3}{12} + \frac{1800 \times 16^3}{12} = 1464.08 \; x \; 10^6 mm^4$

$A = 2 \; x \; 560 \; x \; 50 + 1800x16 = 84,800 \; mm^2$

$r_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{1464.08 \times 10^6}{84,800}} = 131.40 \; mm$

Slenderness ratio, $\lambda = \frac{L_{LT}}{r_y} = \frac{24 \times 10^3}{131.40} = 182.65 > 100$ *(See section 12.17in S.K. Duggal, 2nd edition)*

For $L_{LT} > 100$, $\alpha_s = \frac{30}{\lambda^2} = \frac{30}{182.65^2} = 8.99 \times 10^{-4}$

$I_{s, \; provided} \geq 0.34 \times 8.99 \times 10^{-4} \times (1800 + 2 \times 50)^3 \times 50 = 104.82 \times 10^6 mm^4$

$I_{s, provided} = \frac{16 \times (2 \times 224)^3}{12} = 119.89 \times 10^6 mm^4 > 104.82 \; x \; 10^6 mm^4$

Which is safe.

End-stiffener connection

There will two weld lengths along the depth of web on each side of stiffener plates.

$b_s = 224 - 15 = 209 \; mm$

Tension capacity of one flat,

$T_{dn} = \frac{0.9 \; A_n f_u}{\gamma_{m1}} = \frac{0.9 \times (209 \times 16) \times 410}{1.25} \times 10^{-3} = 987.15 \; kN$

Shear per unit length $q_l = \frac{987.15}{2 \times (1800 - 2 \times 15)} = 0.278 \; kN/mm$

Let us provide weld of size, S = 5 mm

KS = 0.7 x 5 = 3.5 mm

Strength of shop weld per unit length, f_{wd1} = $\frac{3.5 \times 250}{1.732 \times 1.25} \times 10^{-3}$ = 0.404 $\frac{kN}{mm}$ > 0.278 kN/mm.

Hence provide 5 mm fillet weld to connect the end bearing stiffener with the web plate (Figure 4).
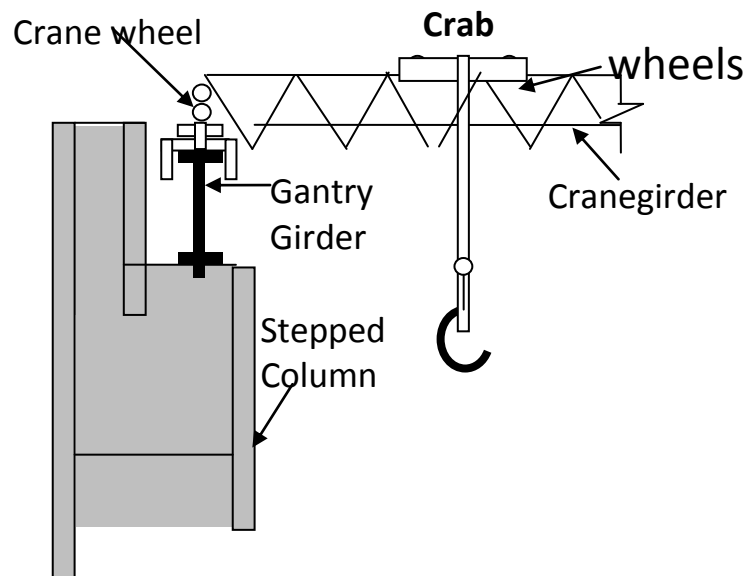
Design of Gantry Girder

Figure: Typical arrangement of gantry girder on a stepped column

Expected question

Design I-section purlin with and without sag bars for a trussed roof from the following data, (expected in June 2015)-1

Span of roof = 10 m
Spacing of purlin along slope or truss = 2.5m
Spacing of truss = 4 m
Slope of roof truss = 1 vertical, 2 horizontal
Wind load on roof surface normal to roof = 1100 N/m^2
Vertical loads from roof sheets = 150 N/m^2.

Solution:

Given data,

Span of roof = 10 m
Spacing of purlin along slope or truss = 2.5m
Spacing of truss = 4 m
Slope of roof truss = 1 vertical, 2 horizontal

Slope $\theta = \frac{1}{2}$

Tan $\theta = \frac{1}{2}$

$\theta = \tan^{-1}\left(\frac{1}{2}\right)$

$\theta = 26.565$ degrees

$Sin\theta = 0.447$

$Cos\theta = 0.894$

Tan $\theta = 0.5$

Wind load on roof surface normal to roof = 1100 N/m^2
Vertical load from roof sheets = 150 N/m^2

Calculating the Dead Load (D.L.)
Load from roof sheeting = 150 x Spacing of purlin
$\qquad\qquad\qquad\qquad\qquad = 150 \times 2.5 = 375$ N/m^2
Assume self weight = 120 N/m^2
Total dead weight $(W_{DL}) = 495$ N/m^2

Calculation of Wind Load
Given, Wind Load on roof surface = 1100 N/m^2
Total wind load = $(W_{w.L.})$ = 1100 x spacing of purlin = 1100 x 2.5 2750 N/m^2

(i) Design of I-Section Purlin Without Sag Bars

It is assumed that the load combination of (Dead Load + Wind Load) creates greater effect on purlin than that of load combination of (Dead Load + Live Load)

Consider the load combination (Dead load + Wind load) for I-section purlin.

Dead load + Wind load

$W_{D.Wx}$ = Load normal to the slope

$W_{w.L.} + W_{D.L.}\,Cos\theta = 2750 + 495\cos(26.565)$

$= 3192.742$N

$W_{D.Wy}$ = Load parallel to the slope

$= W_{D.L.} \sin(26.565) = 495 \sin(26.565)$

$= 221.37$ N

$M_{xx} = \frac{(W_{D.wx})^2 L^2}{10} = \frac{(3192.742) \times 4^2}{10} = 5108.387 \times 10^3$ Nmm

$M_{yy} = \frac{(W_{D.wy})^2 L^2}{10} = \frac{(221.37) \times 4^2}{10} = 354.192 \times 10^3$ Nmm

Assume $\frac{Z_{xx}}{Z_{yy}} = 6$ and $\sigma_{bt} = 0.66$ $f_y = 0.66 \times 250 = 165$ N/mm^2

$E = 2 \times 10^5$ N/mm^2

Finding the Required Sectional Modulus

$Z_{xx_{req}} = \frac{M_{xx} + \frac{Z_{xx}}{Z_{yy}} \times M_{yy}}{\sigma_{bt}} = \frac{5108.387 \times 10^3 + [6 \times 354.192 \times 10^3]}{165} = 43.84 \times 10^3$ mm^3

Select ISMB 100 @ 11.5 kg/m

$Z_{xx} = 51.5 \times 10^3$ mm^3

$Z_{yy} = 10.9 \times 10^3$ mm^3

Check for Permissible Stress

$\sigma_{bt} = \frac{M_{xx}}{Z_{xx}} + \frac{M_{yy}}{Z_{yy}}$

$= \frac{5108.387 \times 10^3}{51.5 \times 10^3} + \frac{354.192 \times 10^3}{10.9 \times 10^3} = 131.687$ N/mm^2 < 165 N/mm^2

Hence safe.

(ii) Design of I-Section with Sag Bar

Dead load + Wind load

$W_{D.Wx}$ = Load normal to the slope

= $W_{w.L.}$ + $W_{D.L.}$ $\cos\theta$ = 2750 + 495 cos(26.565)

= 3192.742N

$W_{D.Wy}$ = Load parallel to slope

= $W_{D.L.}$ Sin(26.565) = 495 sin(26.565)

=221.37 N

Bending Moment

$M_{xx} = \frac{(W_{D.wx})^2 L^2}{10} = \frac{(3192.742)\times 4^2}{10}$ = 5108.387 x 10^3 Nmm

$M_{yy} = \frac{(W_{D.wy})^2 \left(\frac{L}{2}\right)^2}{10} = \frac{(221.37)\times \left(\frac{4}{2}\right)^2}{10}$ = 88.548 x 10^3 N-mm

<u>Finding the Required Sectional Modulus</u>

$Z_{xx_{req}} = \frac{M_{xx} + \frac{Z_{xx}}{Z_{yy}}\times M_{yy}}{\sigma_{bt}} = \frac{5108.387 \times 10^3 + [6\times 88.548 \times 10^3]}{165}$ = 34.180 x 10^3 mm^3

Select ISJB 150 @ 7.1 kg/m from steel Tables

Z_{xx} = 42.9 x 10^3 mm^3

Z_{yy} = 3.7 x 10^3 mm^3

<u>Check for Permissible Stress</u>

$\sigma_{bt} = \frac{M_{xx}}{Z_{xx}} + \frac{M_{yy}}{Z_{yy}}$

$= \frac{5108.387 \times 10^3}{42.9\times 10^3} + \frac{88.548\times 10^3}{3.7\times 10^3}$ = 143.009 N/mm^2< 165 N/mm^2

Hence safe.

<u>Expected Question-2</u>

Example: Compute the loads on a steel roof truss to suit the following data,
Span of the truss = 12 meters
Type of truss = Fan type
Roof cover = Galvanised corrugated G.C. sheeting
Spacing of roof truss = 4.5 meters
Wind pressure = 1.2 kN/m^2

Solution:

Given that,

Span of the truss, l = 12 m,

Spacing of roof truss, S = 4.5 m

Wind pressure = 1.2 kN/m2

Pitch of roof truss, P = ¼ (assumed)

Let, Slope of roof truss be 'θ'

Therefore tanθ = 2p

Tanθ = 2 x (¼)

Tanθ = ½

$\theta = \tan^{-1}(1/2)$

θ = 26.565

Rise of roof truss, R = ¼ x l = ¼ x 12 = 3 m.

Length along the sloping roof, $L = \left[\left(\frac{12}{2} \right)^2 + (3^2) \right]^{1/2}$

L = 6.708 m

Length/panel = 6.708/4 = 1.677m

 (i) **Load at Each Panel**
    (a) Dead Load
       Assuming, weight of galvanized corrugated iron sheets,
       $W_{GI} = 0.133 \text{ kN/m}^2$
       Weight of Purlins, Wp = 0.150 kN/m^2
       Weight of bracing, $W_b = 0.015 \text{ kN/m}^2$

Self weight of roof truss,

$$W_s = \frac{1}{100}\left(\frac{l}{3} + 5\right) = \frac{1}{100}\left(\frac{12}{3} + 5\right) = 0.09 \text{kN/m}^2$$

Total dead load, $W_{D.L.} = W_{GI} + W_p + W_b + W_s$

$$= 0.133 + 0.150 + 0.015 + 0.09 = 0.388 \text{ kN/m}^2$$

Length of panel in plan,

$$L_p = 1.677 \text{ cos}\theta$$

$$L_p = 1.677 \text{ cos}(26.565)$$

$$= 1.499 \approx 1.5\text{m}$$

Load acting on each intermediate panel,
$W_1 = 0.388 \text{ x } 4.5 \text{ x } 1.5 = 2619 \text{ kN}$

Load acting at end panel,
$W_2 = W1/2 = 2.619/2 = 1.310 \text{ KN}$

(b) Live Load

1. Give briefly the design steps to be followed in the design of a roof truss [5 Marks] [Page Number. 770, S.K. Duggal, 2nd edition]
2. Design a steel roof truss to suit the following data:
    Span of the truss = 10 m
    Type of truss = Fan-type
    Roof cover = Galvanised corrugated (GC) sheeting
    Materials : Rolled steel angles
    Spacing of roof trusses = 4.5 m
    Wind pressure Pd = 1.0 kN/m^2
Draw the elevation of the roof truss and the details of joints

Example:

The girder showed in figure is fully restrained against lateral buckling throughout its span. The span is 36m and carries two concentraed loads 870 kN acting at 9 m from left support and 9mm fromm right support. Design the plate girder.
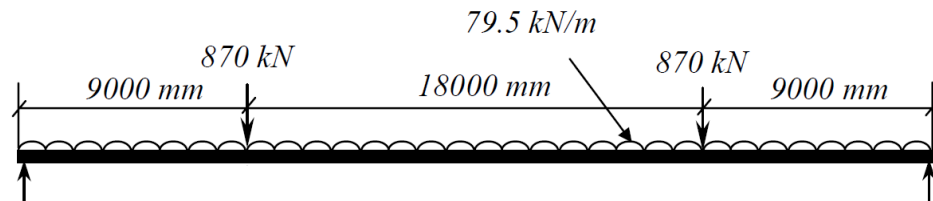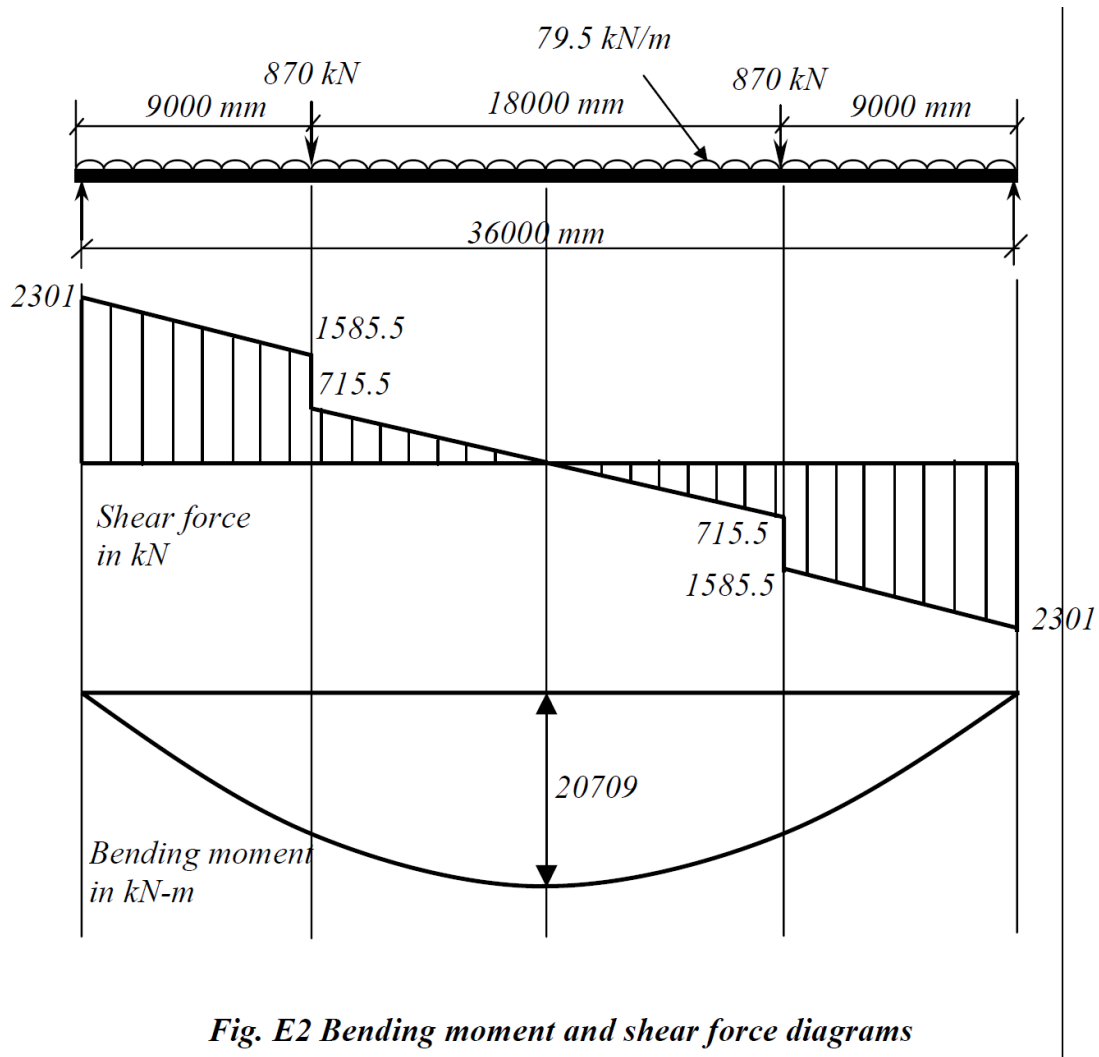

Solution:

Fig. E2 Bending moment and shear force diagrams

Flange:

$p_y = 250/1.15 = 217.4 \ N/mm^2$

Single flange area,

$$A_f = \frac{M_{max}}{d \ p_y} = \frac{20709 * 10^6}{2400 * 217.4} = 39690.7 \quad mm^2$$

By thumb rule, the flange width is assumed as 0.3 times the depth of the section. Try 720 X 60 mm, giving an area = 43200 mm^2.

Web:

Minimum web thickness for plate girder in buildings usually varies between 10 mm to 20 mm. Here, thickness is assumed as 14 mm.

Hence, web size is 2400 X 14 mm

4.0   SECTION CLASSIFICATION

Flange:

$$\varepsilon = \left\{ \frac{250}{f_y} \right\}^{\frac{1}{2}} = \left\{ \frac{250}{250} \right\}^{\frac{1}{2}} = 1.0$$

$$b = \frac{B - t}{2} = \frac{720 - 14}{2} = 353$$

$$\frac{b}{T} = \frac{353}{60} = 5.9 < 7.9 \varepsilon$$

Hence, Flange is PLASTIC SECTION.

Web:

$$\frac{d}{t} = \frac{2400}{14} = 171.4 > 66.2\,\varepsilon$$

Hence, the web is checked for shear buckling.

5.0 CHECKS

Check for serviceability:

$$\frac{d}{250} = \frac{2400}{250} = 9.6\ mm < t$$

Since, $t > \dfrac{d}{250}$

Web is adequate for serviceability.

Check for flange buckling in to web:

Assuming stiffener spacing, a > 1.5 d

$$t \ge \frac{d}{294}\left(\frac{P_{yf}}{250}\right)^{1/2} = \frac{2400}{294} \times \left(\frac{217.4}{250}\right)^{1/2} = 7.6\ mm$$

Since, t (= 14 mm) > 7.6 mm, the web is adequate to avoid flange buckling into the web.

Check for moment carrying capacity of the flanges:

The moment is assumed to be resisted by flanges alone and the web resists shear only.

Distance between centroid of flanges, $h_s = d + T = 2400 + 60 = 2460\ mm$

$$A_f = B * T = 720 * 60 = 43200\ mm^2$$

$$M_c = p_{yf} * A_f * h_s = 217.4 * 43200 * 2460 * 10^{-6} = 23103.5 \ kN\text{-}m$$
$$> 20709 \ kN\text{-}m$$

Hence, the section in adequate for carrying moment and web is designed for shear.

6.0   WEB DESIGN

The stiffeners are spaced as shown in Fig. E5. The spacing of stiffeners is taken as 3000 mm. The spacing can be increased towards the centre of the span for economy.
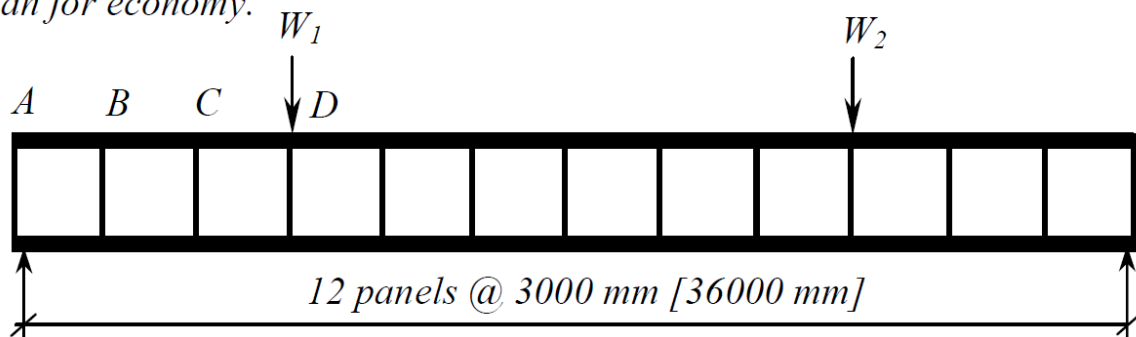

Fig.E3 Trial stiffener arrangement

Panel AB is the most critical panel (Maximum shear zone), so design checks for the web are made for panel AB only.

End panel (AB) design:

$d = 2400 \ mm$
$t = 14 \ mm$
$$\frac{a}{d} = \frac{3000}{2400} = 1.25$$

$$\frac{d}{t} = \frac{2400}{14} = 171.4$$

Calculation of critical shear strength, q_{cr}:

Elastic critical stress, q_e (when a/d > 1) = $[1.0 + 0.75/(a/d)^2][1000/(d/t)]^2$

$$= [1 + 0.75/(1.25)^2][1000/(171.4)]^2$$

$$= 50.4 \ N/mm^2$$

Slenderness parameter, λ_w $= [0.6(f_{yw}/\gamma_m)/q_e]^{1/2}$

$$= [0.6(250/1.15)/50.4]^{1/2}$$

$$= 1.61 > 1.25$$

Hence, Critical shear strength ($q_{cr} = q_e$) = 50.4 N/mm^2

$$f_v = \frac{F_{VA}}{dt} = \frac{2301*10^3}{2400*14} = 68.5 \ N/mm^2$$

Since, $f_v > q_{cr}$ *(68.5 > 50.4)*

Panel AB is designed using tension field action.

Calculation of basic shear strength, q_b:

$$\phi_t = \frac{1.5 q_{cr}}{\sqrt{1 + \left(\frac{a}{d}\right)^2}} = \frac{1.5 * 50.4}{\sqrt{1 + (1.25)^2}} = 47.2$$

$y_b = (p_{yw}^2 - 3q_{cr}^2 + \phi_t^2)^{1/2} - \phi_t = (217.4^2 - 3*50.4^2 + 47.2^2)^{1/2} - 47.2 = 157.4$

$$q_b = q_{cr} + \frac{y_b}{2\left[\frac{a}{d} + \sqrt{1 + \left(\frac{a}{d}\right)^2}\right]} = 50.4 + \frac{157.4}{2\left[1.25 + \sqrt{1 + (1.25)^2}\right]} = 78.0 \ N/mm^2$$

Since, $q_b > f_v$ *(78.0 > 68.5)*

Panel AB is safe against shear buckling.

Checks for the web panel:

End panel AB should also be checked as a beam (Spanning between the flanges of the girder) capable of resisting a shear force R_{tf} and a moment M_{tf} due to anchor forces.
(In the following calculations boundary stiffeners are omitted for simplicity)

Check for shear capacity of the end panel:

$$H_q = 0.75 dt \; p_y \left[1 - \frac{q_{cr}}{0.6 \, p_y} \right]^{\frac{1}{2}} \left[\frac{f_v - q_{cr}}{q_b - q_{cr}} \right]$$

$$q_{cr} = 50.4 \, N/mm^2$$

$$H_q = 0.75 * 2400 * 14 * 217.4 \left[1 - \frac{50.4}{0.6 * (250/1.15)} \right]^{\frac{1}{2}} \left[\frac{68.5 - 50.4}{78 - 50.4} \right] = 2814 \; kN.$$

$$R_{tf} = \frac{H_q}{2} = \frac{2814}{2} = 1407 \, kN$$

$$A_v = t.a = 14 * 3000 = 42000 \; mm^2$$

$$P_v = 0.6 \, p_{yw} \, A_v = 0.6 * (250/1.15) * 42000/1000 = 5478 \; kN$$

Since, $R_{tf} < P_v$, the end panel can carry the shear force.

Check for moment capacity of end panel AB:

$$M_{tf} = \frac{H_q d}{10} = \frac{2814 * 2400}{10} * 10^{-3} = 675.4 \ kN - m$$

$$y = \frac{a}{2} = \frac{3000}{2} = 1500$$

$$I = \frac{1}{12} ta^3 = \frac{1}{12} * 14 * 3000^3 = 3150 * 10^7 \ mm^4$$

$$M_q = \frac{I}{y} p_y = \frac{3150 * 10^7}{1500} * (250/1.15) * 10^{-6} = 4565 \ kN - m$$

Since, $M_{tf} < M_q$ *(675.4 < 4565)*

.: The end panel can carry the bending moment.

7.0 DESIGN OF STIFFENERS

Load bearing stiffener at A:

Design should be made for compression force due to bearing and moment.

Design force due to bearing, F_b = 2301 kN

Force(F_m) due to moment M_{tf}, is

$$F_m = \frac{M_{tf}}{a} = \frac{675.4}{3000} * 10^3 = 225 \ kN$$

Total compression = $F_c = F_b + F_m = 2301 + 225 = 2526 \ kN$

Area of stiffener in contact with the flange, A:

Area (A) should be greater than $\dfrac{0.8\,F_c}{p_{ys}}$

$$\frac{0.8\,F_c}{p_{ys}} = \frac{0.8 * 2526}{217.4} * 10^3 = 9295 \ mm^2$$

Try stiffener of 2 flats of size 240 X 25 mm thick

Allow 15 mm to cope for web/flange weld

$A = 225 * 25 * 2 = 11250 \ mm^2 > 9295 \ mm^2$

$\therefore$ Bearing check is ok.

Check for outstand:

Outstand from face of web should not be greater than $20\,t_s\varepsilon$.

$$\varepsilon = \left\{ \frac{250}{f_y} \right\}^{1/2} = \left\{ \frac{250}{250} \right\}^{1/2} = 1.0$$

Outstand b_s = 240 mm < $20\,t_s\varepsilon$ (= 20 * 25 * 1.0 = 500)

b_s = 240 mm < $13.7\,t_s\varepsilon$ (= 13.7 * 25 * 1.0 = 342.5)

Hence, outstand criteria is satisfied.

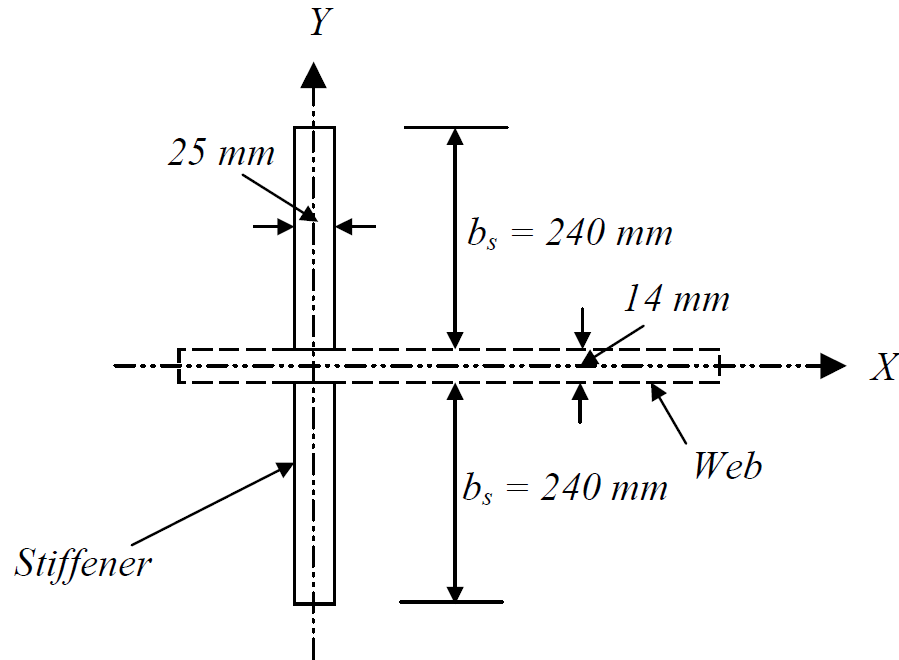**_Check stiffener for buckling:_** *(The effective stiffener section is shown in Fig. E4)*

Fig. E4 End bearing stiffener

The buckling resistance due to web is neglected here for the sake of simplicity.

$$I_x = \frac{25*494^3}{12} - \frac{1}{12}*25*14^3 = 25115*10^4 \ mm^4$$

A_e = Effective area = 240 * 25 * 2 = 12000 mm^2

$$r_x = \left[\frac{I_x}{A_e}\right]^{\frac{1}{2}} = \left[\frac{25115*10^4}{12000}\right]^{\frac{1}{2}} = 144.7 \ mm$$

Flange is restrained against rotation in the plane of stiffener, then

l_e = 0.7 l = 0.7 * 2400 = 1680 mm

$$\lambda = \frac{l_e}{r_x} = \frac{1680}{144.7} = 11.6$$

For $f_y = 250$ N/mm^2 and $\lambda = 11.6$

$\sigma_c = 250$ N/mm^2 from table (3) of chapter on axially compressed columns

Buckling resistance of stiffener is

$P_c = \sigma_c A_e / \gamma_m = (250/1.15) * 12000 * 10^{-3} = 2609$ kN

Since $F_c < P_c$ (2526 < 2609), stiffener provided is safe against buckling.

Check stiffener A as a bearing stiffener:

Local capacity of the web:

Assume, stiff bearing length $b_1 = 0$

*n_2 = 2.5 * 60 * 2 = 300   BS 5950: Part – 1, Clause 4.5.3*

P_{crip} = $(b_1 + n_2)$ t p_{yw}

*   = (0 + 300) * 14 * (250/1.15) * 10^{-3} = 913 kN*

Bearing stiffener is designed for F_A

$F_A = F_c - P_{crip}$ = 2526 – 913 = 1613 kN

Bearing capacity of stiffener alone

$P_A = p_{ys} * A = (250/1.15) * 12000/1000 = 2609$ kN

Since, $F_A < P_A$ (1613 < 2609)

The designed stiffener is OK in bearing.

Stiffener A – Adopt 2 flats 240 mm X 25 mm thick

Design of intermediate stiffener at B:

Stiffener at B is the most critical intermediate stiffener, hence it will be chosen for the design.

Minimum Stiffness:

$$I_s \geq 0.75\,dt^3 \quad for \ \ a \geq d\sqrt{2}$$

$$I_s \geq \frac{0.75\,dt^3}{a^3} \quad for \ \ a < d\sqrt{2}$$

$$d\sqrt{2} = \sqrt{2} * 2400 = 3394 \ mm$$

$$\therefore a < d\sqrt{2} \qquad\qquad (3000 < 3394)$$

Conservatively 't' is taken as actual web thickness and minimum 'a' is used.

$$\frac{1.5\,d^3 t^3}{a^2} = \frac{1.5 * 2400^3 * 14^3}{3000^2} = 632 * 10^4 \ mm^4$$

Try intermediate stiffener of 2 flats 90 mm X 12 mm

$$\left(I_s\right)_{\text{Pr}\,ovided} = \frac{12 * 194^3}{12} - \frac{12 * 14^3}{12} = 730 * 10^4 \ mm^4$$

The section provided satisfies the minimum required stiffness.

Check for outstand:

Outstand of the stiffener $\leq 13.7\, t_s \varepsilon$

$13.7\, t_s \varepsilon = 13.7 * 14 * 1.0 = 192\ mm$

Outstand $= 90\ mm$ $(90 < 192)$

Hence, outstand criteria is satisfied.

Buckling check:

Stiffener force, $F_q = V - V_s$

where, V $=$ Total shear force
V_s $= V_{cr}$ of the web.

Elastic critical stress, q_e $= 50.4\ N/mm^2$

$V_{cr} = q_{cr}\, d\, t = 50.4 * 2400 * 14 * 10^{-3} = 1693\ kN$

Shear force at B, $V_B = 2301 - [(2301 - 1585.5)*(3000/9000)] = 2062.5\ kN$

Stiffener force, $F_q = [2062.5 - 1693] = 369.5\ kN$
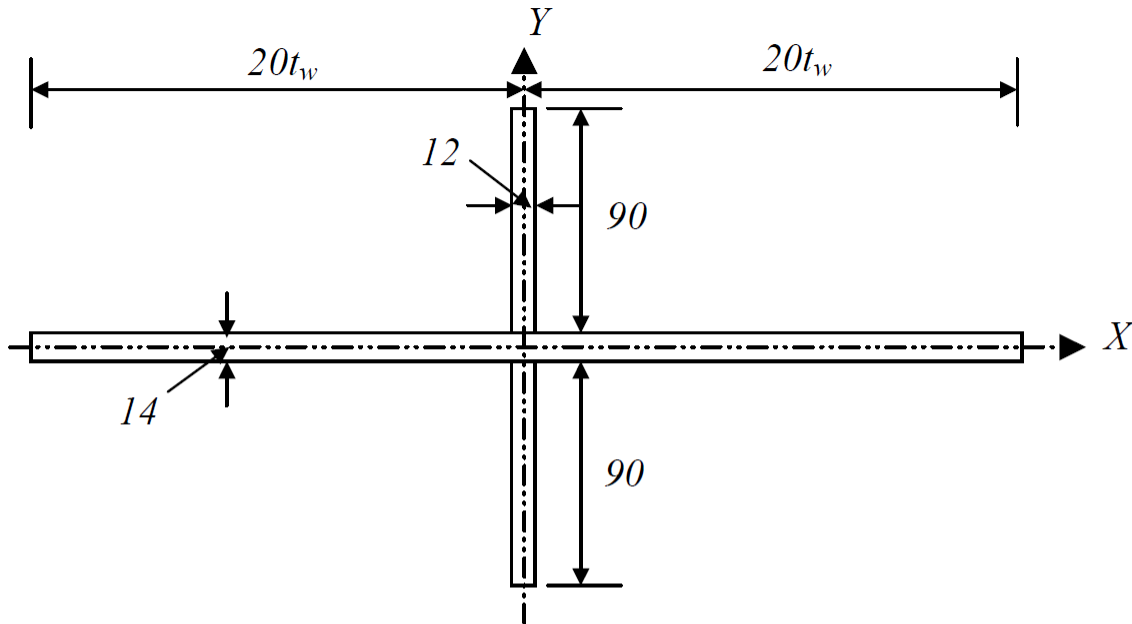
Buckling resistance of intermediate stiffener at B:

Fig.E5 Effective section

$20\ t_w\ =\ 20\ *\ 14\ =\ 280\ mm$

$$I_x\ =\ \frac{1}{12}\ *\ 12\ *194^3\ +\ \frac{560*14^3}{12}\ -\ \frac{12*14^3}{12}\ =\ 743*10^4\ mm^4$$

$A\ =\ 180\ *\ 12\ +\ 560\ *\ 14\ =\ 10000\ mm^2$

$$r_x\ =\ \left[\frac{743*10^4}{10000}\right]^{1/2}\ =\ 27.3$$

$l_e\ =\ 0.7\ *\ 2400\ =\ 1680$

$$\lambda\ =\ \frac{l_e}{r_x}\ =\ \frac{1680}{27.3}\ =\ 61.5$$

$For\ f_y\ =\ 250\ N/mm^2\ and\ \lambda = 61.5$

From table3 of chapter on axially compressed columns,

$\sigma_c\ =\ 182.3\ N/mm^2$

Buckling resistance = (182.3/1.15) * 10000 * 10^{-3}= 1585 kN

F_q < Buckling resistance. (369.5 < 1585)

Hence, intermediate stiffener is adequate

Intermediate stiffener at B - *Adopt 2 flats 90 mm X 12 mm*

Intermediate Stiffener at D (Stiffener subjected to external load):

Try intermediate stiffener 2 flats 90 mm X 12 mm thick

It satisfies the minimum stiffness requirement as in case of stiffener at B.

Buckling check:

$$\frac{F_q - F_x}{P_q} + \frac{F_x}{P_x} + \frac{M_s}{M_{ys}} \leq 1$$

F_q = V- V_s V = 1585.5 kN

V_s = V_{cr} = q_{cr} d t = 50.4* 2400 * 14 * 10^{-3} = 1693 kN
F_q is negative and so we can take $F_q - F_x$ = 0

M_s = 0

F_x = 870 kN

Buckling resistance of load carrying stiffener at D:

(Calculation is similar to stiffener at B)

Buckling resistance, P_x = *(182.3/1.15) * 10000 * 10^{-3} = 1585 kN*

F_x / P_x = *870/1585 = 0.55 < 1.0*

Hence, stiffener at D is OK against buckling

Stiffener at D - *Adopt flats 90 mm X 12 mm thick*

Web check between stiffeners:

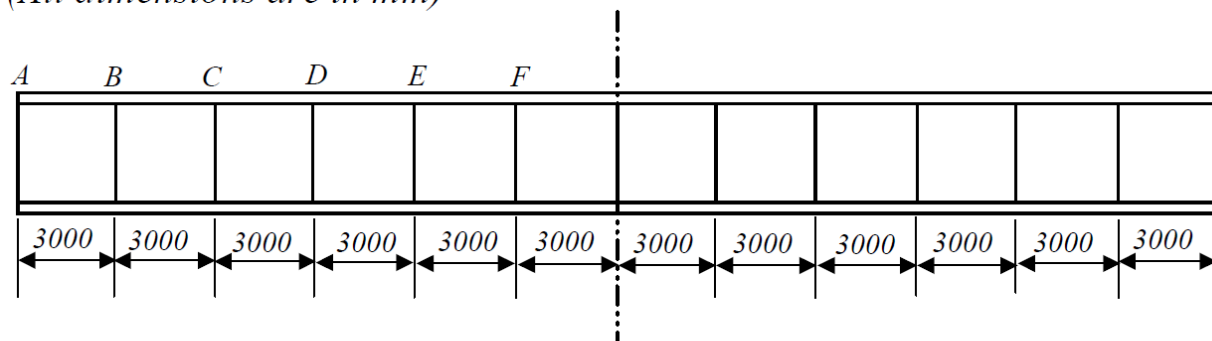$f_{ed} \leq p_{ed}$

f_{ed} = w^1/t = *79.5/14 = 5.7 N/mm²*

when compression flange is restrained against rotation relative to the web

$$p_{ed} = \left[2.75 + \frac{2}{\left(a/d\right)^2} \right] \frac{E}{\left(d/t\right)^2} = \left[2.75 + \frac{2}{\left(\frac{3000}{2400}\right)^2} \right] \frac{200000}{\left(\frac{2400}{14}\right)^2}$$

$$= \frac{3.79*20000}{26406} = 27.4 \ N/mm^2$$

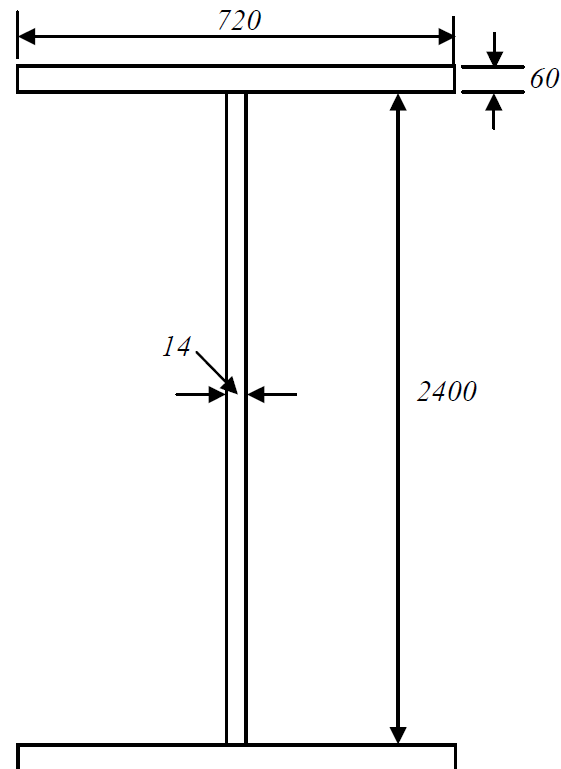*Since,* fB_{ed} < p_{ed} *[5.7 < 27.4], the web is OK for all panels.*

8.0 FINAL GIRDER

(All dimensions are in mm)

(a) Longitudinal section of plate girder

(b) Flange and web proportions