Chapter 1: Steel Structures and their Analysis and Design [4 hours]

- A steel structure is an assemblage of a group of members (elements) expected to sustain their share of applied forces and to transfer them safely to the ground.
- Depending on the orientation of the member in the structure and its structural use, the member is subjected to forces, either axial, bending, or torsion, or a combination thereof.

(a) Framed building

(b) Industrial building

ansmission line tower

(c) Rail-road bridge

(d) Overhead water tank

- A significant difference between steel and concrete constructions is that the designer has more control over the shape of reinforced cement concrete elements.
- For building a steel structure, the designer is normally compelled to use standard rolled sections.
- Fortunately, the variety of steel sections available is so great that any desired structural effect can be achieved in steel.
- The main advantages of steel structures are their smaller weight-to-strength ratio, speed of erection and dismantling, and its scrap value.
- However, the faster degradation of their strength in the events of fire, requirement of skilled, personnel and the accuracy desired in fabrication are the major drawbacks.

Complex
at Jwagal
A mordern mercantile building with commercial facilities enhancing urban fabric.
ArBCon Pvt. Ltd.
+977-016524457, 6591150
arbconpvtltd@gmail.com
www.arbcon.com.np
facebook.com/arbcon
ArBCon Pvt. Ltd.

- The design of steel structures involves the planning of the structure for specific purposes, proportioning of members to carry loads in the most economical manner, and considerations for erection at site.
- First, the structure should serve the purpose for which it is intended and this is achieved by proper functional planning.
- Secondly, it should have adequate strength to withstand direct and induced forces to which it may be subjected during its lifespan. An inadequate assessment of forces and their effects on the structure may lead to excessive deformation and its failure.
- Therefore, the design of structures includes functional planning, acknowledgement of the various forces, strength of materials and the design methods. In addition, the structure should be economical and easy to erect.

- To facilitate safe and economical design specifications, standards and codes are prepared on the basis of past design practice, experience based on behavior of existing structures, knowledge gained from failures, and from research data. The information thus gathered is critically examined, assessed and approved by a body of experts and published in the form of codes and standards. For this purpose, the Bureau of Indian Standards has published a number of codes, standards and handbooks, some of which will be referred to frequently and are listed below.

1. IS Handbook No. 1 Properties of Structural Steel Rolled Sections
2. IS: 875–1987 Code of Practice for Design Loads for Building and Structures
3. IS: 800–2007 Code of Practice for use of Structural Steel in General Building Construction

Structural Steel and Classification of Steel Sections

- Structural steel has been classified by the Bureau of Indian Standards based on its ultimate or yield strength.
- For example, Fe-410 steel has minimum tensile strength of 410 N/mm2.

Table 1.1 Mechanical properties of some typical structural steels

Type of Steel	*Designation*	*UTS (MPa)*	*Yield Strength (MPa) Thickness (mm)*			*Min. Percentage Elongation (Gauge Length = 5.65 $\sqrt{A_0}$)*	*Charpy V-notch Impact Energy (min)*
			<20	*20–40*	*>40*		
Standard structural steel (IS 2062)	Fe 410A	410	250	240	230	23	—
	Fe 410B	410	250	240	230	23	27
	Fe 410C	410	250	240	230	23	27
			<16	**16-40**	**41-63**		
Micro-alloyed	Fe 440B	440	300	290	280	22	30
Medium-/ high-strength steel (IS 8500)	Fe 540B	540	410	390	380	20	25
	Fe 490B	490	350	330	320	22	25
	Fe 590B /570B	590/570	450	430	420	30	20

Fe stands for the steel followed with the number the characteristic ultimate tensile stress in MegaPascals. The letters A, B and C indicate the grade of steel.

Grade A Steel: used in structures subject to normal conditions.
Grade B Steel: used in structures subject to non-critical applications, where service temperature does not fall bellow 0°C. Usually, specified for structural parts prone to brittle fracture or subject to severe fluctuation of stresses.
Grade C Steel: has guaranteed low temperature (up to – 40°C) and impact properties.

Table 1 (*Concluded*)

Sl No.	Indian Standard	Grade/Classification	Properties				
			Yield Stress MPa, *Min*			Ultimate Tensile Stress MPa, *Min*	Elongation, Percent, *Min*
(1)	(2)	(3)	(4)			(5)	(6)
			d or *t*				
			< 20	20-40	> 40		
viii)	IS 2062	E 165 (Fe 290)	165	165	165	290	23
		E 250 (Fe 410 W) A	250	240	230	410	23
		E 250 (Fe 410 W) B	250	240	230	410	23
		E 250 (Fe 410 W) C	250	240	230	410	23
		E 300 (Fe 440)	300	290	280	440	22
		E 350 (Fe 490)	350	330	320	490	22
		E 410 (Fe 540)	410	390	380	540	20
		E 450 (Fe 570) D	450	430	420	570	20
		E 450 (Fe 590) E	450	430	420	590	20
			d or *t*				

Some of the other mechanical properties of structural steel are as follows:

1. Modulus of elasticity (E) = 2×10^5 N/mm2
2. Shear modulus (G) = 0.769×10^5 N/mm2
3. Poisson's ratio (μ)
   (i) elastic range = 0.3
   (ii) plastic range = 0.5
4. Coefficient of thermal expansion = 12×10^{-6}/ ºC
5. Unit mass (r) = 7.85×10^3 kg/m3

Products of structural steel

- In the design process of steel elements one of the objectives is the selection of the appropriate cross sections for the individual members of the structure being designed.
- This selection will entail choosing a standard cross-sectional shape that is widely available rather than requiring the fabrication of a shape with unique dimensions and properties.
- Depending upon the manufacturing process, these sections are classified as :
  - Hot rolled and
  - Cold rolled section

Hot rolled steel

- Hot rolled steel is heated to over 1700 degrees Fahrenheit. At this temperature it is to shape and form the steel.
- The steel shrinks when cooled meaning there is less control over the final size and shape. So it is cheaper.
- Used when precise shape and tolerances are required.
- Scaly surface finish, rounded edges, non oily surface

Hot Rolled Sections

- In this manufacturing process, which takes place in a rolling mill, molten steel is taken from the furnace and poured into a continuous casting system where the steel solidifies but is never allowed to cool completely. The hot steel passes through a series of rollers that squeeze the material into the desired cross-sectional shapes.
- Rolling the steel while it is still hot allows it to be deformed without any loss of ductility
- While cooling, the variation in the cooling rates for the different thicknesses of various elements of the rolled section, residual stresses, are introduced which may be removed by subsequent strengthening processes.
- During the rolling process, the member increases in length and is cut to standard lengths, which are subsequently cut (in fabricating shop) to the length required for particular structure.

- The types of steel shapes rolled are described as follows:

1. Rolled steel I-Sections
2. Rolled steel channel sections
3. Rolled steel angle sections
4. Rolled steel T-Sections
5. Rolled steel tube sections
6. Rolled steel bars
7. Rolled steel flats
8. Rolled steel plates
9. Rolled steel sheets
10. Rolled steel strips

(a) Rolled beams and columns (b) Rolled channels (c) Angles (unequal)

SECTION 2 BEAM SECTIONS

Table 2.1 Nominal Dimensions, Mass and Sectional Properties of Indian Standard Medium Flange Beams

Designation	Mass M	Sectional Area, a	Dimensions							Sectional Properties					
			D	B	t	T	Flange Slope, Max α, deg	R_1	R_2	I_x	I_y	r_x	r_y	Z_x	Z_y
	kg/m	cm^2	mm	mm	mm	mm		mm	mm	cm^4	cm^4	cm	cm	cm^3	cm^3
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
MB 100	8.9	11.4	100	50	4.7	7.0	98.0	9.0	4.5	183	12.9	4.00	1.05	36.6	5.16
MB 125	13.3	17.0	125	70	5.0	8.0	98.0	9.0	4.5	445	38.5	5.16	1.51	71.2	11.0
MB 150	15.0	19.1	150	75	5.0	8.0	98.0	9.0	4.5	718	46.8	6.13	1.57	95.7	12.5
MB 175	19.6	25.0	175	85	5.8	9.0	98.0	10.0	5.0	1 260	76.7	7.13	1.76	144	18.0
MB 200	24.2	30.8	200	100	5.7	10.0	98.0	11.0	5.5	2 120	137	8.29	2.11	212	27.4
MB 225	31.1	39.7	225	110	6.5	11.8	98.0	12.0	6.0	3 440	218	9.31	2.34	306	39.7
MB 250	37.3	47.5	250	125	6.9	12.5	98.0	13.0	6.5	5 130	335	10.4	2.65	410	53.5
MB 300	46.0	58.6	300	140	7.7	13.1	98.0	14.0	7.0	8 990	486	12.4	2.86	599	69.5

SECTION 4 CHANNEL SECTIONS

Table 4.1 Dimensions, Mass and Sectional Properties of Sloping Flange Channels

Designation	Mass M	Sectional Area, a	Dimensions							Sectional Properties						
			D	B	t	T	Flange Slope, α	R_1	R_2	C_y	I_x	I_y	r_x	r_y	Z_x	Z_y
	kg/m	cm²	mm	mm	mm	mm	deg	mm	mm	mm	cm⁴	cm⁴	cm	cm	cm³	cm³
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
Medium Weight Channels																
MC 75	7.14	9.10	75	40	4.8	7.5	96	8.5	2.4	1.32	78.5	12.9	2.94	1.19	20.9	4.81
MC 100	9.56	12.2	100	50	5.0	7.7	96	9.0	2.4	1.54	192	26.7	3.97	1.48	33.5	7.71
MC 125	13.1	16.7	125	65	5.3	8.2	96	9.5	2.4	1.95	425	61.1	5.05	1.91	68.1	13.4
MC 125*	13.7	17.5	125	66	6.0	8.1	96	9.5	2.4	1.92	435	64.4	4.98	1.92	69.6	13.8
MC 150	16.8	21.3	150	75	5.7	9.0	96	10.0	2.4	2.20	788	103	6.08	2.20	105	19.5
MC 150*	17.7	22.6	150	76	6.5	9.0	96	10.0	2.4	2.17	813	110	6.00	2.20	108	20.2
MC 175	19.6	24.9	175	75	6.0	10.2	96	10.5	3.2	2.19	1 240	122	7.04	2.21	141	23.0
MC 175*	22.7	27.6	175	76	7.5	10.2	96	10.5	3.2	2.14	1 310	136	6.89	2.22	150	24.5

*The heavier sections in each size intended for use in wagon industry are to be obtained from same set of rolls as the corresponding lightest section in that size group, by raising the rolls.

SECTION 5 EQUAL LEG ANGLES

Table 5.1 Nominal Dimensions, Mass and Sectional Properties of Indian Standard Equal Leg Angles

Designation	Mass M	Sectional area, a	Dimensions				Sectional Properties											
			A×B	t	R_1	R_2	C_x	C_y	I_x	I_y	I_u (Max)	I_v (Min)	r_x	r_y	r_u (Max)	r_v (Min)	Z_x	Z_y
	Kg/m	Cm^2	mm×mm	mm	mm	mm	cm	cm	cm^4	cm^4	cm^4	cm^4	cm	cm	cm	cm	cm^3	cm^3
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
∠ 20 20× 3	0.9	1.12	20×20	3.0	4.0	Should be resonably square	0.59	0.59	0.4	0.4	0.6	0.2	0.58	0.58	0.73	0.37	0.3	0.3
× 4	1.1	1.45		4.0			0.63	0.63	0.5	0.5	0.8	0.2	0.58	0.58	0.72	0.37	0.4	0.4
∠ 25 25× 3	1.1	1.41	25×25	3.0	4.5		0.71	0.71	0.8	0.8	1.2	0.3	0.73	0.73	0.93	0.47	0.4	0.4
× 4	1.4	1.84		4.0			0.75	0.75	1.0	1.0	1.6	0.4	0.73	0.73	0.91	0.47	0.6	0.6
× 5	1.8	2.25		5.0			0.79	0.79	1.2	1.2	1.8	0.5	0.72	0.72	0.91	0.47	0.7	0.7
∠ 30 30× 3	1.4	1.73	30×30	3.0	5.0		0.83	0.83	1.4	1.4	2.2	0.6	0.89	0.89	1.13	0.57	0.6	0.6
× 4	1.8	2.26		4.0			0.87	0.87	1.8	1.8	2.8	0.7	0.89	0.89	1.12	0.57	0.8	0.8
× 5	2.2	2.77		5.0			0.92	0.92	2.1	2.1	3.4	0.9	0.88	0.88	1.11	0.57	1.0	1.0
∠ 35 35× 3	1.6	2.03	35×35	3.0	5.0		0.95	0.95	2.3	2.3	3.6	0.9	1.05	1.05	1.33	0.67	0.9	0.9
× 4	2.1	2.66		4.0			1.00	1.00	2.9	2.9	4.7	1.2	1.05	1.05	1.32	0.67	1.2	1.2
× 5	2.6	3.27		5.0			1.04	1.04	3.5	3.5	5.6	1.5	1.04	1.04	1.31	0.67	1.4	1.4
× 6	3.0	3.86		6.0			1.08	1.08	4.1	4.1	6.5	1.7	1.03	1.03	1.29	0.67	1.7	1.7
∠ 40 40× 3	1.8	2.34	40×40	3.0	5.5		1.08	1.08	3.4	3.4	5.5	1.4	1.21	1.21	1.54	0.77	1.2	1.2
× 4	2.4	3.07		4.0			1.12	1.12	4.5	4.5	7.1	1.8	1.21	1.21	1.53	0.77	1.6	1.6
× 5	3.0	3.78		5.0			1.16	1.16	5.4	5.4	8.6	2.2	1.20	1.20	1.51	0.77	1.9	1.9
× 6	3.5	4.47		6.0			1.20	1.20	6.3	6.3	10.0	2.6	1.19	1.19	1.50	0.77	2.3	2.3
∠ 45 45× 3	2.1	2.64	45×45	3.0	5.5		1.20	1.20	5.0	5.0	8.0	2.0	1.38	1.38	1.74	0.87	1.5	1.5
× 4	2.7	3.47		4.0			1.25	1.25	6.5	6.5	10.4	2.6	1.37	1.37	1.73	0.87	2.0	2.0
× 5	3.4	4.28		5.0			1.29	1.29	7.9	7.9	12.6	3.2	1.36	1.36	1.72	0.87	2.5	2.5
× 6	4.0	5.07		6.0			1.33	1.33	9.2	9.2	14.6	3.8	1.35	1.35	1.70	0.87	2.9	2.9

(d) Tees

Square hollow sections (SHS)

Rectangular hollow sections (RHS)

(e) Hollow sections (tubes)

Circular hollow sections (CHS)

(f) Bars

(g) Flats

(h) Plates

Cold rolled steel

- Hot rolled steel is processed further
- Cold rolled steel is heated, cooled at room temperature, followed by annealing and tempering
- These additional processes allows for a wider range of finishes
- Greasy or oily finish
- Smooth surface
- Very sharp edges
- Mostly used for machine parts

Hot Rolled
Cold Rolled

Stress-strain characteristics of mild steel

OAB— represents a straight line, (obeys Hooke's law.)

A —limit of proportionality

B —elastic limit

C'/C— represents the upper/lower yield point

CD — plastic yielding

DE — strain hardening

E — ultimate stress

EF — necking

F — breaking stress

A— represents the limit of proportionality—the stress beyond which linear variation ceases.

B— represents the elastic limit—the maximum stress up to which a specimen regains its original length on removal of the applied load. This stress, in general, is not measured and B is assumed to coincide with A. Actually, it is difficult to determine the actual stress–strain relation in the elastic range near the yield point; proportional and elastic limit are therefore considered to be coincident.

- C’/C— represents the upper/lower yield point—the magnitude of stress corresponding to the upper yield point C’ depends on the cross sectional shape of the specimen and the type of equipment used to perform the test. The upper yield point is observed if load is applied rapidly whereas the lower yield point is observed if the rate of loading is slow. In many of the structural steel hot-rolled sections, the upper yield point is not obtained due to residual stresses from the hot rolling process. Hence, it has no practical significance. Point C represents the yield point at which there is a definite increase in strain without any further increase in stress. The stress corresponding to the lower yield point C is thus yield stress with a typical magnitude of 250 N/mm2 for mild steel.

- For the region OC, the material is elastic, and the slope E, is the Young's modulus. On average E is **2 × 10^5 N/mm2.** The strain at the yield stress is about **0.00125**.
- CD— represents plastic yielding—it is the strain which occurs after the yield point, with no increase in stress.
- The point D of plastic yield range is somewhat variable but a typical strain is 0.015. The strain in the range CD lies between ten to fifteen times the strains at the yield point.
- DE—represents strain hardening—it is a range where additional stress produces additional strain. Strain increases fast with stress till ultimate load is reached. The slope of the stress–strain curve in the strain hardening range, is typically 5 to 15% of the Young's modulus for structural steels.
- E—represents the ultimate stress—the stress corresponding to the ultimate load. The initial slope of this region is about 4 per cent of Young's modulus. At a strain of at least 0.2, the stress reaches its maximum value, the ultimate strength of steel. It is also known as minimum ultimate tensile strength.

METHODS OF DESIGN

- ***Simple design***: In this method it is assumed that end connections between the members will not develop restraint moments adversely affecting the members and the structures as a whole be assumed to be pin jointed for the purpose of design.
- ***Semi – rigid design***: This method permits a reduction in the maximum bending moment in beams suitably connected to their supports, so as to provide a degree of direction fixity, and in case of triangulated frames, it permits in account being taken of the rigidity of the connections and the moment of interaction of the members.
- ***Fully – rigid design:*** The end connections are considered with sufficient rigidity to hold the original angle between such members and the members they connect virtually unchanged.

Analysis and design

Structural design is the methodical investigation of the stability, strength and rigidity of structures. The basic objective in structural analysis and design is to produce a structure capable of resisting all applied loads without failure during its intended life.

Analysis

Structural analysis deals with a calculation of load coming on the members and analyze them i.e. finding axial force, shear force, bending moment and/or deflection in the member.

Design

Structural design decides the dimension (shape and size) of the structural members on the basis of calculated load from structural analysis.

DESIGN PROCESS

- ✓ Functional planning and structural planning.
- ✓ Preliminary design.
- ✓ Idealization and analysis of the structure.
- ✓ Design and detailing of the structure.
- ✓ Preparation of structural drawing.
- ✓ Construction
  - ➢ Mobilization of personnel
  - ➢ Procurement of materials and equipments
  - ➢ Redesign may be required.

WORKING STRESS DESIGN METHOD

WORKING STRESS DESIGN METHOD

- Traditional method of designing steel structures,
- Based on elastic theory
- Attainment of the initial yielding forms the design criteria for the members in this approach

The working stress in the member should
be less than the permissible stress.

Basic assumptions in working stress method

- Material is homogeneous and isotropic
- Material behaves on a linear elastic manner
- Factor of safety is applied only for material.
- Adequate safety can be ensured by suitably restricting the stresses in the material induced by the expected working loads (service loads) on the structure.

Table 1.5 Permissible stresses in steel structural members

S.No.	*Types of Stress*	*Notation*	*Permissible Stress (MPA)*	*Factor of Safety*
1.	Axial tensile stress	σ_{at}	$0.6f_y$	1.67
2.	Maximum axial compressive stress	σ_{ac}	$0.6f_y$	1.67
3.	Bending tensile stress	σ_{bt}	$0.66f_y$	1.515
4.	Maximum bending compressive stress	σ_{bc}	$0.66f_y$	1.515
5.	Average shear stress	τ_{va}	$0.4f_y$	2.5
6.	Maximum shear stress	τ_{vm}	$0.45f_y$	2.22
7.	Bearing stress	σ_p	$0.75f_y$	1.33
8.	Stress in slab base	σ_{bs}	185	–

- The permissible stresses are some fraction of the yield stress of the material and may be defined as the ratio of the yield stress to the factor of safety.

$$permisible\ stress = \frac{yield\ stress}{factor\ of\ safety}$$

The concept of introducing a factor of safety is to make the structure safe. It accounts for the following:

- The analysis methods are based on assumptions and do not give the exact stresses.
- Structural members may temporarily be overloaded under certain circumstances.
- Underestimation of the future live loads.
- The stresses due to fabrication and erection are not considered in the design of ordinary structures.
- The secondary stresses may be appreciable.
- Stress concentrations.
- Unpredictable natural calamities.

Disadvantages of WSM

1. Stress–strain behavior of the material is considered to be linear and the

   Structure is assumed to behave linearly in an elastic manner.

   (Structures designed based on these assumptions possess considerable reserve of strength beyond elastic limit until they reach their ultimate strength, and lead to uneconomic design. This reserve strength is derived from ductility (the ability of steel member to deform inelastically without major loss of strength) of steel and redundancy. One of the major drawbacks of designing steel structure by WSD approach is that the reserve strength beyond elastic limit is neither quantified nor utilized.)

Disadvantages of WSM

2. Steel is assumed to be stressed well below its elastic limit under the working loads which result in over design of the steel members.
3. Design parameters such as loads, material properties, strength, etc., are assumed to have unique values, though they are variable. The design parameters are predicted on experience or on field data. The design of structural elements therefore depends upon how closely the prediction is made. This uncertainty is non-deterministic.

Disadvantages of WSM

4. Many of the sources of loading vary with time. However, since all the sources of loads were considered to act simultaneously with their maximum value, while maintaining the same factor of safety, would lead to unconservative design.

Disadvantages of WSM

5. Redistribution of moment during yielding is not known. It does not consider the redistribution of forces and moments in statically indeterminate structures. Factor of safety x Working load is the failure load which is not true. Actually it is much more because a material can resist the load after the yield appears at a fibre. In indeterminate structures just formation of a plastic hinge is not the failure criteria, since it can resist load till some more hinges are formed resulting into collapse mechanism. Thus, the redistribution of moments give rise to the additional load carrying capacity.

SERVICE LOAD & PERMISSIBLE STRESSES

Service load:

- The service load (working load) refers to the expected load to which the structure is subjected to its normal use/ service.
- It does not include any possible overloading or any safety factors.

Permissible stress

a) **Direct tensile stress**

The permissible stress in axial tension on the net effective area of the section shall not exceed $\sigma_{at} = 0.66\, f_y$

b) **Direct compressive stress**

The direct stress in compression on the gross sectional area of axially loaded compression members shall not exceed 0.6 fy nor the permissible stress calculated using the following formula

$$\sigma_{ac} = 0.6\, \frac{f_{cc} \cdot f_y}{[(f_{cc})^n + (f_b)^n]^{\frac{1}{n}}}$$

LIMIT STATE DESIGN METHOD

- The limit state design (LSD) method was developed to take account of all conditions that can make the structure unfit for use, considering actual behaviour of materials and structures
- The structure shall be designed to withstand safely all loads likely to act on it throughout its life.
- In limit state design, basically statistical methods have been used for determination of loads and material properties with a small probability of structure reaching the limit states of strength and serviceability.
- The objective is to achieve a structure that will remain fit for use during its lifetime with acceptable target reliability. In other words, the probability of a limit state being reached during a lifetime should be very low.
- ***Limit states are the states beyond which the structure no longer satisfies performance requirements specified.***

Two categories of limit states:

- Strength
- Serviceability

LIMIT STATE OF STRENGTH

- Associated with failures (or imminent failure), under the action of probable and most unfavorable combination of factored loads, which may endanger the safety of life and property.
- The failure of ductile steel structure may either be due to geometric nonlinearity associated with buckling or large deflection, or due to material nonlinearity due to yielding, or due to both.

Limit state of strength, also called ultimate limit state, includes

- Loss of equilibrium of the structure as a whole or any of its parts or components.
- Loss of stability of the structure (including the effect of sway, where appropriate, and overturning) or any of its parts including supports and foundations.
- Failure by excessive deformation/formation of mechanism, general yielding, rupture of the structure or any of its parts or components, or buckling.
- Fracture due to fatigue which occurs due to repeated fluctuation of loading. Fatigue cracking can initiate and propagate in the stress concentration areas of the structure.
- Brittle fracture which occurs below a certain temperature when the ultimate tensile strength of steel diminishes sharply.

IS 800:2007 Clause 5.4 (page 29)

LIMIT STATE OF SERVICIABILITY

- Limit state beyond which service criteria—
  - specified deflection,
  - vibration (for example, wind-induced oscillations),
  - repairable damage due to fatigue, corrosion durability, and fire resistance—are no longer met.
- This limit state is related to the criteria governing normal use of structure.

DESIGN CRITERIA FOR LSM

$$\text{Design Action } (Q_d) \leq \text{Design Strength } (S_d)$$

The left-hand side of refers to the load effects on the structure, while the right-hand side refers to the strength or capacity of the structure.

The design action Q_d is expressed as

$$Q_d = \sum_k \gamma_f Q_{ck}$$

and, the design strength S_d is obtained as

$$S_d = S_u / \gamma_m$$

where,

Q_{ck} = characteristic load

S_u = ultimate strength

γ_f = partial safety factor for loads,

γ_m = partial safety factor for materials

Partial safety factors for loads, γ_f

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

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

Abbreviations: DL = Dead Load, LL = Imposed Load (Live Loads), WL = Wind Load, SL = Snow Load, CL = Crane Load (Vertical/horizontal), AL = Accidental Load, ER = Erection Load, EL = Earthquake Load.

IS 800:2007
page 29

Table 4 Partial Safety Factors for Loads, γ_f, for Limit States

(*Clauses* 3.5.1 *and* 5.3.3)

Combination	Limit State of Strength					Limit State of Serviceability			
	DL	LL[1]		WL/EL	AL	DL	LL[1]		WL/EL
		Leading	Accompanying				Leading	Accompanying	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
DL+LL+CL	1.5	1.5	1.05	—	—	1.0	1.0	1.0	—
DL+LL+CL+ WL/EL	1.2	1.2	1.05	0.6	—	1.0	0.8	0.8	0.8
	1.2	1.2	0.53	1.2					
DL+WL/EL	1.5 (0.9)[2]	—	—	1.5	—	1.0	—	—	1.0
DL+ER	1.2 (0.9)[2]	1.2	—	—	—	—	—	—	—
DL+LL+AL	1.0	0.35	0.35	—	1.0	—	—	—	—

[1] When action of different live loads is simultaneously considered, the leading live load shall be considered to be the one causing the higher load effects in the member/section.

[2] This value is to be considered when the dead load contributes to stability against overturning is critical or the dead load causes reduction in stress due to other loads.

Abbreviations:

DL = Dead load, LL = Imposed load (Live loads), WL = Wind load, CL = Crane load (Vertical/Horizontal), AL = Accidental load, ER = Erection load, EL = Earthquake load.

NOTE — The effects of actions (loads) in terms of stresses or stress resultants may be obtained from an appropriate method of analysis as in **4**.

Partial safety factors for materials, γ_m

S.No.	Definition	Partial Safety Factor	
1.	Resistance, governed by yielding γ_{m0}	1.10	
2.	Resistance of member to buckling γ_{m0}	1.10	
3.	Resistance, governed by ultimate stress γ_{m1}	1.25	
4.	Resistance of connection:	Shop Fabrications	Field Fabrications
	(i) Bolts—Friction Type, γ_{mf}	1.25	1.25
	(ii) Bolts—Bearing Type, γ_{mb}	1.25	1.25
	(iii) Rivets, γ_{mr}	1.25	1.25
	(iv) Welds, γ_{mw}	1.25	1.50

IS 800:2007 page 30

Table 5 Partial Safety Factor for Materials, γ_m

(*Clause* 5.4.1)

Sl No.	Definition	Partial Safety Factor	
i)	Resistance, governed by yielding, γ_{m0}	1.10	
ii)	Resistance of member to buckling, γ_{m0}	1.10	
iii)	Resistance, governed by ultimate stress, γ_{m1}	1.25	
iv)	Resistance of connection:	*Shop Fabrications*	*Field Fabrications*
	a) Bolts-Friction Type, γ_{mf}	1.25	1.25
	b) Bolts-Bearing Type, γ_{mb}	1.25	1.25
	c) Rivets, γ_{mr}	1.25	1.25
	d) Welds, γ_{mw}	1.25	1.50

LSM more popular than WSM:

- LSM uses multiple safety factor format, which attempts to provide adequate safety at ultimate loads as well as adequate serviceability at service loads.
- LSM involves separate consideration of different kinds of failure, types of materials and types of loads.
- WSM considers design based on working loads and bound by elastic theory but LSM considers design loads and design strengths
- LSM gives more economical section than WSM.

- For each material and load, a partial safety factor is assigned individually depending on the material properties and load properties.
- The material strength can be utilized to its maximum value during its lifespan
- Loads can be assessed with reasonable probability of occurrence.
- This method is used because it ensures the utilization if material strength with lowest construction cost input.

Example:

Fe 410 grade steel

fy = yield stress = 250 n/mm2
fu = ultimate stress = 410 N/mm2

WSM

Permissible stress = 0.6 fy = 0.6 x 250 = 150 N/mm2
Working stress < Permissible stress
Working load = 50 kN c/s area= 333 mm2

$$\frac{Applied\ force}{c/s\ area} = 150\ MPa$$

LSM

Design action(loads) < Design strength

Design load= factored load= 50x1.5 = 75 kN

Ultimate stress = 410 N/mm2

Partial factor of safety= 1.25

c/s area=228 mm2

$$\frac{design\ load}{c/s\ area} = \frac{stress}{partial\ factor\ of\ safety}$$

CHAPTER 4: CONNECTIONS

WELDING

WELDED CONNECTION

- Two structural members are jointed by means of welds
- Welded designs offer the opportunity to achieve a more efficient use of materials.
- Welding is the only process that produces a one piece construction.
- The speed of fabrication and erection helps compress production schedules.
- No deductions are there for holes; thus the gross section is effective in carrying loads.

- A properly welded joint is stronger than the jointed material.
- Fused joints create a rigid structure in contrast to the non-rigid structures made using other types of joints. The compactness and greater rigidity of welded joints permits design assumptions to be realised more accurately.
- Welding saves weight and consequently cuts costs. Connecting steel plates are reduced or eliminated since they often are not required.

ADVANTAGES

- **No holes** on the main member are needed
- Saves **weight** of structure
- Less noise pollution compared to riveting.
- Welding offers **airtight and watertight jointing** of plates and hence is employed in the construction of water/oil storage tanks, ships
- More **rigid** (due to the direct connection of members by welding)
- **Aesthetic** in appearance and appear less cluttered
- Offers more freedom to the designer in choosing sections
- **More versatile**, you can weld (nearly) any two surfaces together. Where as not everything can be bolted.

DISADVANTAGES

- Electricity is required
- Failure can be brittle
- Internal stresses are developed due to heating.
- requires highly skilled human resources
- inspection is difficult and expensive

TYPES OF WELD CONNECTION

(a) Groove welds

(b) Fillet welds

(c) Slot welds

(d) Plug welds

- It has been found that fillet welds are used extensively (about 80%) followed by groove welds (15%). Slot and plug welds are used rarely (less then 5%) in structural engineering applications. Fillet welds are suitable for lap and T-joints and groove welds are suitable for butt, corner, and edge joints.

(a) Groove welds

- Groove welds are used to connect structural members that are aligned in the same plane and often used in butt joints.

Fig. 11.4 Types of groove welds

Single V butt weld

Double V butt weld

- Square groove weld is used to connect plates up to 8-mm thickness.
- Partial penetration groove welds should not be used especially in fatigue situations.
- The choice between single or double penetration depends on access on both sides, the thickness of the plate, the type of welding equipment, the position of the weld, and the means by which the distortion is controlled.
- Since groove welds will transmit the full load of the members they join, they should have the same strength **as the members they join. Hence, only full penetration** groove welds are often used.
- groove welds are better in highly stressed structures where smooth flow of stress is necessary.
- If the butt joint has the same characteristics as the parent metal, is finished smooth with it on both sides (by proper grinding or the like), and has complete penetration with no unweld zones, it approaches the condition of no joint at all and, for most common types of parent metal, may have impact and fatigue resistance equal or superior to that of the base metal.

Terms used to describe the parts of a groove weld

Design specification of butt/groove weld

- Generally for section or thickness ≤ 8 mm, provide square or single U and V
- For greater than 8 mm, provide Double U and V

<u>Size of weld</u>

- Throat thickness → thickness of thinner plate for full penetration of double U and V

<u>For calculating strength,</u>

- Effective throat thickness = $5/8^{th}$ of thickness of thinner plate
- Effective area = throat thickness x length of weld
- Length of weld ≥ 4 times thickness of throat
- Strength of weld ≥ Strength of plates
- Effective length = Actual length – 2s

  ($l_e = l - 2s$)

IS 800 PAGE 79

10.5.7 *Design Stresses in Welds*

10.5.7.1 *Shop welds*

10.5.7.1.1 *Fillet welds*

Design strength of a fillet weld, f_{wd} shall be based on its throat area and shall be given by:

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

where

$f_{wn} = f_u / \sqrt{3}$,

f_u = smaller of the ultimate stress of the weld or of the parent metal, and

γ_{mw} = partial safety factor (*see* Table 5).

10.5.7.1.2 *Butt welds*

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

should generally not exceed 3/4 of the thickness of the section at the toe (*see* Fig. 17B).

10.5.8.3 Where the size specified for a fillet weld is such that the parent metal will not project beyond the weld, no melting of the outer cover or covers shall be allowed to occur to such an extent as to reduce the throat thickness (*see* Fig. 18).

10.5.8.4 When fillet welds are applied to the edges of a plate or section in members subject to dynamic loading, the fillet weld shall be of full size with its leg length equal to the thickness of the plate or section, with the limitations specified in **10.5.8.3**.

10.5.8.5 End fillet weld, normal to the direction of force shall be of unequal size with a throat thickness not less than 0.5*t*, where *t* is the thickness of the part, as shown in Fig. 19. The difference in thickness of the welds shall be negotiated at a uniform slope.

10.5.9 *Stresses Due to Individual Forces*

When subjected to either compressive or tensile or

Design strength in tension and compression

Strength of weld $T_{dw} = \frac{f_y}{\gamma_{mw}} \cdot L_w \cdot t_e$

$$
\begin{aligned}
L_w &= length\ of\ weld \\
t_e &= throat\ thickness \\
\gamma_{mw} &= Partial\ factor\ of\ safety \\
&= 1.5\ for\ site\ welding \\
&= 1.25\ for\ shop\ welding \\
f_y &= smaller\ of\ yield\ stress\ of\ weld\ and\ parent\ metal
\end{aligned}
$$

Design strength in shear

Design Strength $V_{dw} = \frac{f_{yw}}{\gamma_{mw}} \cdot L_w \cdot t_e$

$$L_w = length\ of\ weld$$
$$t_e = throat\ thickness$$
$$\gamma_{mw} = Partial\ factor\ of\ safety$$
$$= 1.5\ for\ site\ welding$$
$$= 1.25\ for\ shop\ welding$$

$$f_{yw} = smaller\ of\ shear\ stress\ of\ weld\ \left(\frac{f_{yw}}{\sqrt{3}}\right) and\ parent\ metal\ \left(\frac{f_y}{\sqrt{3}}\right)$$

Shear stress = $\frac{f_y}{\sqrt{3}}$

Problem:

Two plates of 18 mm and 15 mm thickness are joined by groove weld. The joint is subjected to working load of 300 kN. Calculate the length of the weld considering single groove and double groove weld. Use Fe 410 grade steel.

solution:

Working load = 300 kN

Design load (factored load) = $P_d = 1.5 \times 300 = 450\ kN$

Yield stress for Fe 410 $f_y = 250\ MPa$ **[table 1 of IS 800]**

Assume shop yielding, then $\gamma_{mw} =$ …… **[table 5 of IS 800]**

1. **Single butt**

Throat thickness = 5/8 th of thickness of thinner plate =

$\therefore t_e = \frac{5}{8} \times 15 = \ \ldots\ldots mm$

For the calculation of length of weld, $T_{dw} = \frac{f_y}{\gamma_{mw}} . L_w . t_e$

$$450 \times 10^3 = \frac{250}{1.25} \times L_w \times 9.375$$

$$L_w = \ \ldots\ldots$$

2. **Double butt**

Throat thickness $t_e = thickness\ of\ thinner\ member$

$$T_{dw} = \frac{f_y}{\gamma_{mw}} . L_w . t_e$$

(b) Fillet welds

- Fillet welds are most widely used due to their economy, ease of fabrication, and adoptability at site.
- Approximately triangular in cross section
- Since they do not require any edge preparation, they are cheaper than groove welds.
- Fillet welds are assumed to fail in shear.

- Fillet welds are normally easier to make, require less material preparation, and are easier to fit than groove welds.
- On the other hand, for a given amount of weld material, they are not as strong and they cause greater concentration of stress.
- In lightly stressed structures where stiffness rather than strength controls design and fatigue or brittle fracture is not a problem, fillet welds are entirely adequate and generally more economical.
- Reliable fillet welds may be, and frequently are, designed for severe stress and service conditions, but, as a rule, groove welds are better in highly stressed structures where smooth flow of stress is necessary.

- Idealization of fillet weld : Right angled triangle with equal sides ‘s’; s is size of fillet weld

Maximum size of fillet weld, smax

- Along the edge of the plate, less than 6-mm thick, the maximum size is equal to the thickness of the plate.
- Where the fillet weld is applied to the square edge of a plate of thickness greater than 6 mm, size of the weld should be at least 1.5 mm less than the edge thickness. (This limit is specified such that the total strength may be developed without overstressing the adjacent metal.)

Max value of size 's' = thickness – 1.5 mm (for square edge)

[IS 800 , Clause 10.5.8.1]

- Where the fillet weld is applied to the rounded toe of the rolled section, the size of the weld should not exceed **3/4 of the thickness of the section at the toe [Cl 10.5.8.2]**

Minimum size of fillet weld, Smin

IS 800 : 2007

10.5 Welds and Welding

10.5.1 *General*

Requirements of welds and welding shall conform to IS 816 and IS 9595, as appropriate.

10.5.1.1 *End returns*

Fillet welds terminating at the ends or sides of parts should be returned continuously around the corners for a distance of not less than twice the size of the weld, unless it is impractical to do so. This is particularly important on the tension end of parts carrying bending loads.

10.5.1.2 *Lap joint*

In the case of lap joints, the minimum lap should not be less than four times the thickness of the thinner part joined or 40 mm, whichever is more. Single end fillet should be used only when lapped parts are restrained from openings. When end of an element is connected only by parallel longitudinal fillet welds, the length of the weld along either edge should not be less than the transverse spacing between longitudinal welds.

10.5.1.3 A single fillet weld should not be subjected to moment about the longitudinal axis of the weld.

shall not be less than 3 mm and shall generally not exceed 0.7*t*, or 1.0*t* under special circumstances, where *t* is the thickness of the thinner plate of elements being welded.

Table 21 Minimum Size of First Run or of a Single Run Fillet Weld

(*Clause* 10.5.2.3)

Sl No.	Thickness of Thicker Part mm		Minimum Size mm
	Over	Up to and Including	
(1)	(2)	(3)	(4)
i)	–	10	3
ii)	10	20	5
iii)	20	32	6
iv)	32	50	8 of first run 10 for minimum size of weld

NOTES

1 When the minimum size of the fillet weld given in the table is greater than the thickness of the thinner part, the minimum size of the weld should be equal to the thickness of the thinner part. The thicker part shall be adequately preheated to prevent cracking of the weld.

2 Where the thicker part is more than 50 mm thick, special precautions like pre-heating should be taken.

Effective throat thickness

- The shortest distance from the root of the fillet weld to the face of the diagrammatic weld (line joining the toes)
- Effective throat thickness ≥ 3 mm

  and

  < 0.7 x thickness of thinner plate

Effective throat thickness = $K \times$ size of weld = KS

Fig. Fillet weld dimensions

where S is the size of weld in mm and K is a constant. The value of K depends upon the angle between the fusion faces

10.5.2 *Size of Weld*

10.5.2.1 The size of normal fillets shall be taken as the minimum weld leg size. For deep penetration welds, where the depth of penetration beyond the root run is a minimum of 2.4 mm, the size of the fillet should be taken as the minimum leg size plus 2.4 mm.

10.5.2.2 For fillet welds made by semi-automatic or automatic processes, where the depth of penetration is considerably in excess of 2.4 mm, the size shall be taken considering actual depth of penetration subject to agreement between the purchaser and the contractor.

10.5.2.3 The size of fillet welds shall not be less than 3 mm. The minimum size of the first run or of a single run fillet weld shall be as given in Table 21, to avoid the risk of cracking in the absence of preheating.

10.5.2.4 The size of butt weld shall be specified by the effective throat thickness.

10.5.3 *Effective Throat Thickness*

10.5.3.1 The effective throat thickness of a fillet weld

10.5.3.2 For the purpose of stress calculation in fillet welds joining faces inclined to each other, the effective throat thickness shall be taken as K times the fillet size, where K is a constant, depending upon the angle between fusion faces, as given in Table 22.

10.5.3.3 The effective throat thickness of a complete penetration butt weld shall be taken as the thickness of the thinner part joined, and that of an incomplete penetration butt weld shall be taken as the minimum thickness of the weld metal common to the parts joined, excluding reinforcements.

10.5.4 *Effective Length or Area of Weld*

10.5.4.1 The effective length of fillet weld shall be taken as only that length which is of the specified size and required throat thickness. In practice the actual length of weld is made of the effective length shown in drawing plus two times the weld size, but not less than four times the size of the weld.

10.5.4.2 The effective length of butt weld shall be taken as the length of the continuous full size weld, but not less than four times the size of the weld.

Table 22 Values of *K* for Different Angles Between Fusion Faces

(*Clause* 10.5.3.2)

Angle Between Fusion Faces	60°–90°	91°–100°	101°–106°	107°–113°	114°–120°
Constant, *K*	0.70	0.65	0.60	0.55	0.50

Minimum Effective Length of Fillet Weld

- Minimum length = four times the size of the weld
- Effective length is taken equal to its overall length minus twice the size of weld.
- ***End returns are made*** equal to twice the size of the weld to relieve the high stress concentration at the ends. End returns must be provided for welded joints that are subject to eccentricity, stress reversals, or impact loads.

Fig. 6.12 End returns

- Effective area of weld = Aw= Le x te
- **Design strength as per code [Cl 10.5.7.1.1]**

10.5.7 *Design Stresses in Welds*

10.5.7.1 *Shop welds*

10.5.7.1.1 *Fillet welds*

Design strength of a fillet weld, f_{wd} shall be based on its throat area and shall be given by:

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

where

$f_{wn} = f_u / \sqrt{3}$,

f_u = smaller of the ultimate stress of the weld or of the parent metal, and

γ_{mw} = partial safety factor (*see* Table 5).

In lap connections,

Min length of weld ≥ 4 x thickness of thinner plate

OR

40 mm **[IS 800 Clause 10.5.1.2]**

$$\left.\begin{array}{l} l_1 > b \\ l_2 > b \end{array}\right\} \text{ in lap connections.}$$

Surface not
in tension
Surface in
tension
Convex
Concave
Mitre

Problem:

- A tie member 60mm x 8 mm is connected to a gusset plate of 12 mm thick to transmit load of 200 kN. Design fillet weld and necessary overlap. Use Fe 410 grade.

Solution

Working load Pw = 200 kN

Design load P_{wd} = 200 × 1.5 = 300 kN

$f_u = 410\ N/mm^2$

$f_y = 250\ N/mm^2$

SIZE:

Smax = t – 1.5 = 8 -1.5 = 6.5 mm

Smin = 5 mm [table 21]

Take size = s = 6 mm

EFFECTIVE THROAT THICKNESS:

te = K . s

for fusion angle 90°, K = 0.7 [table 22]

∴ te = 0.7 × 6 = 4.2 mm

Design strength of fillet weld;

$$P_{wd} = \frac{f_{wn}}{\gamma_{mw}} . A_w = \frac{f_u}{\sqrt{3}} . \frac{A_w}{\gamma_{mw}}$$

$$300 \times 10^3 = \frac{410 \times L_e \times t_e}{\sqrt{3} \times 1.25}$$

$$L_e = 377.189\ mm$$

Provide length of weld = Le + 2 s = 389.19 mm = 390 mm > 4 s (= 24mm) ok.

Minimum size of overalap = max of 4t or 40 mm [clause 10.5.1.2]

= 4 x 8 or 40 mm

=32 or 40 mm

=40 mm

End returns = 2 s = 12 mm

Case : 1

$$\text{Overlap} = \frac{390-60}{2}$$

$$= 165\,mm > 40\,mm$$

(OK)

Case : 2

$$= \frac{390}{2} = 195\,mm$$

$$> 40\,mm$$

(OK)

Intermittent Fillet Welds

- Intermittent fillet welds are provided to transfer calculated stress across a joint when the strength required is less than that developed by a continuous fillet weld of smallest practical size
- The fillet weld length required is computed as a continuous fillet weld and a chain of intermittent fillet welds of total length equal to the computed length, with spacing as per IS specifications is provided.

To what extent intermittent welds should be used?

- Intermittent fillet welds are not economical unless the weld is of minimum size. A smaller size fillet weld of a longer length is usually more economical for the same strength. This is because the strength of a fillet weld increases directly with size but the weight of the weld metal increases with the square of the size.
- If the structure is exposed, the use of continuous welds may be preferable as they are conducive to greater ease of maintenance and longer life of structure.
- If severe dynamic loads, repetitive and alternating act on the structure, intermittent welds must not be used.

- Minimum effective length ≥ 4 s or 40 mm
- Spacing ≤ 12 t (for compression joint)

  ≤ 16 t (for tension joint)

  < 200 mm
- (The clear spacing between the effective lengths of intermittent welds should not exceed 12 and 16 times the thickness of thinner plate jointed for compression and the tension joint respectively, and should never be more than 200mm.)

Plug and Slot weld

- Plug or slot welds are used in addition to fillet welds when sufficient welding length is not available along the edges of the members.
- A slot is cut in one of the overlapping members and welding metal is filled in the slot
- If slot is completely filled with weld metal, it is called plug weld.
- If only periphery of slot is filled with weld metal, it is called slot weld
- IS code stipulates that they should not be designed to carry any stresses.

Plug and Slot weld

- Plug or slot welds are used in addition to fillet welds when sufficient welding length is not available along the edges of the members.
- A slot is cut in one of the overlapping members and welding metal is filled in the slot
- If slot is completely filled with weld metal, it is called plug weld.
- If only periphery of slot is filled with weld metal, it is called slot weld
- IS code stipulates that they should not be designed to carry any stresses.

Slotted connection

An ISLC 300 @ 324.7N/m is to carry factored tensile load of 900 kN. The channel section is welded at site to a gusset plate 12 mm thick. Design a fillet weld if the overlap is limited to 350 mm.

Use Fe 410 grade steel

Size of weld:

Max size = t – 1.5 = 6.7 - 1.5 = 5.2 mm (square edge)

Min size = 5 mm

Adopt size s = 5 mm

Effective throat thickness:

te = k.s = 0.7 x 5 = 3.5 mm

Design strength of weld :

$$P_{wd} = \frac{f_{wn}}{\gamma_{mw}} . A_w = \frac{f_u}{\sqrt{3}} . \frac{A_w}{\gamma_{mw}}$$

le = ...

Length of weld provided = le + 2 s = ... > 4s

Overlap = (1640 - 300) / 2 = 670 mm > 350 mm

So provide slot,

Provide 2 slots

Width of slot > 3 t or 25 mm

- 3 t = 3 x 6.7 =

So width of slot = 25 mm

Truss member

(a)

(b)

Fillet weld for truss member

- Truss members are composed of single-angle or double-angle sections.
- The calculated weld length is placed as longitudinal fillet welds either on the two sides parallel to the axis of the load, or on three sides, i.e., transverse welds along with longitudinal welds.
- A longitudinal fillet weld length should never be placed on one side only as there will be possibility of rotation.
- The centre of gravity of the weld should coincide with the centroid of the section used as a truss member.
- If the member is symmetrical, the welds will be placed symmetrically but if the member is unsymmetrical (angle, channel) as is usually the case of a truss member, the lengths of longitudinal fillet welds are kept different on the two sides.

Let

L_1, L_2 = length of longitudinal fillet welds on two sides

P_1, P_2 = factored design loads along lengths L_1 and L_2, respectively

P = factored load acting on the centroid of the section

Taking moment about the line passing through weld length L_1,

$P_2 \times h - P \times h_2 = 0$

$P_2 = (P \times h_2) / h$

Similarly,

$P_1 = (P \times h_1) / h$

Once the factored design loads P_1 and P_2 are known, the fillet weld lengths can be designed.

$P = P_1 + P_2$

- The effectiveness of a fillet weld can be increased by connecting the two side welds by an end weld as shown in Fig.(b).
- when the weld length to be provided cannot be accommodated over the two parallel sides because of limited overlap, it may be the only choice.
- This arrangement also reduces the size of gusset plate and results in economy.

Let L_1, L_2 = length of longitudinal fillet welds

h = length of end fillet weld

P = factored load acting on the centroid of the section

P_3 = factored design force in the end fillet weld

$$= ht_t \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

Total length of weld required = $l_1 + l_2 + h$

Taking moment about the line passing through l_1,

$$P_2 h + P_3 \frac{h}{2} - Ph_2 = 0$$

Taking moment about the line passing through l_2,

$$P_1 h + P_3 \frac{h}{2} - Ph_1 = 0$$

(b)

Problem 1

- In a truss, ISA 100 x 100 x 8 mm is subjected to factored tension of 200 kN. It is to be connected to a gusset using fillet weld at toe and back. Find the length of weld required so that the centre of gravity of weld lies in the plane of CG of angle.

- Factored load = 200 kN
- Use Fe 410 grade steel ; fu= 410 N/mm2
- Design strength of weld fwd

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}} = \frac{f_u}{\sqrt{3} \cdot \gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.25} = 189.4\ MPa$$

Size of weld (s) $\leq \frac{3}{4} \times 8 = 6\,mm$

S_{max} (at toe) $= \frac{3}{4} \times 8 = 6\,mm$

S_{max} (at top) $= t - 1.5 = 8 - 1.5 = 6.5\,mm$

$S_{min} = 3\,mm$

$\therefore$ Provide $\boxed{s = 6\,mm}$

Now,

~~Taking ma~~ Throat thickness $(t_e) = k_1 s$

$= 0.7 \times s$

$\boxed{t_e = 4.2\,mm}$

Taking moment about welding L1,

$$P_2 \times 100 = P \times 27.6$$

$$P_2 = \frac{200 \times 27.6}{100} = 55.2 \text{ kN}$$

$$P_1 = P - P_2 = 200 - 55.2$$

$$= 144.8 \text{ kN}$$

Now,

$$P_1 = f_{wd} \times t_e \times l_1$$

$$l_1 = \frac{144.8 \times 10^3}{189.4 \times 4.2} = \underline{\underline{182 \text{ mm}}}$$

And, $$l_2 = \frac{P_2}{f_{wd} \times t_e} = \frac{144.8 \times 10^3}{189.4 \times 4.2} = \underline{\underline{69.4 \text{ mm}}}$$

So, it lies in the central axis, there would be no rotation.

Problem 2

- In a truss member consisting of ISA 80 x 50 x 8 mm is connected to transmit the working load of 550 kN. This member is connected to gusset plate of 12 mm thickness at the back and toe. Design the weld.

Size of Weld

S_{max} (at toe) $= \frac{3}{4} t \rightarrow$ thickness of angle section.

$= 3/4 \times 8 = 6$ mm.

$S_{min} = 5$ mm.

Take $\boxed{S = 5mm}$

S_{max} at top $= t - 1.5$

$= 8 - 1.5$

$= 6.5mm$

$t_e = k.s.$

for fusion angle $90°$, $k = 0.7$

$t_e = 0.7 \times 5 = \underline{3.5 mm}$.

Take $L_3 = 80$ mm

Then,

$$P_3 = \frac{f_u}{\sqrt{3}\, \gamma_{mw}} . l_3 . t_e$$

or, $P_3 = \dfrac{410}{\sqrt{2} \times 1.25} \times 80 \times 3.5$ (shop welding)

$= 53023.85\ N$

$\therefore \boxed{P_3 = 53.024\ kN}$.

$\Sigma M_{L_2} = 0$

or, $P_3 \times 40 + P_1 \times 80 = P \times 27.3$

$P_1 = \dfrac{825 \times 27.3 - 53.024 \times 40}{80}$

$\boxed{P_1 = 255.019\ kN}$

$P_2 = P - P_1 - P_3 = 825 - 255.019 - 53.024$

$\boxed{P_2 = 516.957\ kN}$

$$P_1 = \frac{f_u}{\sqrt{3} \cdot \gamma_{mw}} \times l_1 \times t_e$$

$$l_1 = \frac{P_1 \times \sqrt{3} \cdot \gamma_{mw}}{f_u \times t_e} = 384.76 \text{ mm}$$

$$l_2 = 779.96 \text{ mm}$$

Provide $L_1 = 384.76 + 2s = 395$ mm

$L_2 = 779.96 + 2s = 790$ mm.

$L_3 = 80$ mm $> 4s$

$= 4 \times 5$

$= 20$ mm (OK).

Problem 2

- In a truss member consisting of ISA 80 x 50 x 8 mm is connected to transmit the working load of 550 kN. This member is connected to gusset plate of 12 mm thickness at the back and toe. Design the weld.

Size of Weld

S_{max} (at toe) $= \frac{3}{4} t \rightarrow$ thickness of angle section.

$= 3/4 \times 8 = 6$ mm.

$S_{min} = 5$ mm.

Take $\boxed{S = 5mm}$

S_{max} at top $= t - 1.5$

$= 8 - 1.5$

$= 6.5mm$

$t_e = k.s.$

for fusion angle $90°$, $k = 0.7$

$t_e = 0.7 \times 5 = \underline{3.5mm}$.

Take $L_3 = 80$ mm

Then,

$$P_3 = \frac{f_u}{\sqrt{3}\, \gamma_{mw}} . l_3 . t_e$$

or, $P_3 = \frac{410}{\sqrt{2} \times 1.25} \times 80 \times 3.5$ (shop welding)

$= 53023.85\ N$

$\therefore \boxed{P_3 = 53.024\ kN}$.

$\Sigma M_{L2} = 0$

or, $P_3 \times 40 + P_1 \times 80 = P \times 27.3$

$P_1 = \frac{825 \times 27.3 - 53.024 \times 40}{80}$

$\boxed{P_1 = 255.019\ kN}$

$P_2 = P - P_1 - P_3 = 825 - 255.019 - 53.024$

$\boxed{P_2 = 516.957\ kN}$

$$P_1 = \frac{f_u}{\sqrt{3} \cdot \gamma_{mw}} \times l_1 \times t_e$$

$$l_1 = \frac{P_1 \times \sqrt{3} \cdot \gamma_{mw}}{f_u \times t_e} = 384.76 \text{ mm}$$

$$l_2 = 779.96 \text{ mm}$$

Provide $L_1 = 384.76 + 2s = 395$ mm

$L_2 = 779.96 + 2s = 790$ mm.

$L_3 = 80$ mm $> 4s$

$= 4 \times 5$

$= 20$ mm (ok).

(a)

(b)

(c)

Determine the sizes of end fillets to connect two plates with cross sections of 125×8 mm and 125×12 mm which are subjected to a tension of 100 kN at working load. The ultimate strength of the plates, $f_u = 410$ MPa.

Determine the sizes of end fillets to connect two plates with cross sections of 125×8 mm and 125×12 mm which are subjected to a tension of 100 kN at working load. The ultimate strength of the plates, $f_u = 410$ MPa.

Design load $= 1.5 \times 100 = 150$ kN

Design shear strength of the weld, $f_{wd} = 189.4$ MPa

Let the sizes of the welds A and B are in the ratio of the thicknesses of the plates.

i.e., $s_A/s_B = 8/12 \Rightarrow s_B = 1.5\, s_A$

Since the thickness of the thicker plate is 12 mm, minimum size of weld = 5 mm

For the calculation of the effective length of the weld, let $s_A = 5$ mm and

$s_B = 1.5 \times 5 = 7.5$ mm

Let F_{wdA} and F_{wdB} be the design shear forces carried by the welds A and B, respectively.

F_{wdA} = (Shearing area at the throat) × (Design shear strength of the weld)

$= (\text{Effective length of the weld} \times 0.7s_A) \times f_{wd}$

$= (125 - 2 \times 5) \times 0.7s_A \times 189.4$

$= 15246.7s_A$

Similarly, $F_{wdB} = (125 - 2 \times 7.5) \times 0.7s_B \times 189.4 = 14{,}583.8s_B$

The tension is resisted by both the welds A and B.

$\therefore \quad F_{wdA} + F_{wdB} = \text{Design load}$

$15{,}246.7s_A + 14{,}583.8s_B = 150 \times 10^3$

$15{,}246.7s_A + 14{,}583.8 \times 1.5s_A = 150 \times 10^3$

or $s_A = 4\,\text{mm}$

and $s_B = 1.5 \times 4.0 = 6\,\text{mm}$

IS 800:2007

10.5.8.5 End fillet weld, normal to the direction of force shall be of unequal size with a throat thickness not less than $0.5t$, where t is the thickness of the part, as

$0.7s_A > 0.5 \times 8$ from which $s_A = 5.7\,\text{mm}$ and $0.7s_B > 0.5 \times 12$ from which $s_B = 8.6\,\text{mm}$.

$\therefore$ The welds A and B may be of sizes 6 mm and 9 mm, respectively.

A tie member of a roof truss consists of 2 ISA 100 × 75 × 8 mm. The angles are connected to either side of a 10 mm gusset plates and the member is subjected to a working pull of 300 kN. Design the welded connection. Assume connections are made in the workshop.

Solution:

A tie member of a roof truss consists of 2 ISA 100 × 75 × 8 mm. The angles are connected to either side of a 10 mm gusset plates and the member is subjected to a working pull of 300 kN. Design the welded connection. Assume connections are made in the workshop.

Solution:

Step 1: To obtain the thickness of weld:

Working load = 300 kN

Factored load = 300 × 1.5 = 450 kN

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

$$s = \frac{3}{4} \times 8 = 6 \text{ mm}$$

At top the thickness should not exceed

$$s = t - 1.5 = 8 - 1.5 = 6.5 \text{ mm}$$

Hence, provide $s = 6$ mm weld.

Step 2: To obtain the total length of the weld required:

Each angle carries a factored pull of 450/2 = 225 kN

Let L_w be the total length of the weld required.

Assuming normal weld, $t = 0.7 \times 6$ mm

Design strength of the weld = $L_w t f_u/\sqrt{3} \times 1/1.25 = L_w \times 0.7 \times 6 \times 410/\sqrt{3} \times 1/1.25$

Equating it to the factored load, $L_w \times 0.7 \times 6 \times 410/\sqrt{3} \times 1/1.25 = 225 \times 10^3$; $L_w = 283$ mm

Step 3: To obtain the length of top and lower weld:

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

Let L_1 be the length of top weld and L_2 be the length of lower weld.

To make centre of gravity of weld to coincide with that of angle,

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

$$L_1 = (69/31) \times L_2$$

Fig. 3.25 Details of weld at the end connection

Required

$$L_1 + L_2 = 283$$

$$L_2\,((69/31) + 1) = 283$$

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

$\therefore$

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

Provide 6 mm weld of $L_1 = 195$ mm and $L_2 = 87$ mm as shown in the Fig. 3.25

ECCENTRIC WELDING CONNECTION

- Welding connection subjected to combination of shear, bending and twisting are considered as eccentric welding connection
- This type of connection is generally used in beam to column end connection.

Column flange
Cleat angle
Beam
Seat angle

Column flange
Cleat angle
Beam
Packing
Stiffener
Seat angle

Column flange
Framing angle
Beam

Bracket connection

- Bracket means a steel plate or a set of steel plates connected to columns to support beams.
- The bracket may be connected to a column using butt or fillet weld
- Bracket plates are used as a platform for another member to sit/rest on it.
- Welds are subjected to shear force and bending moment or twisting moment

Bracket connection using Butt weld

- Butt weld is subjected to vertical shear force V = P and a bending moment M = P . e

- Consider a bracket connection to flange of a column by a full penetration butt weld
- Eccentric load P causes shear force P and moment at the weld section

- Shear stress in the weld due to factored load

$$q = \frac{V}{l_w . t_w}$$

- Bending stress in the weld due to factored load

$$f_b = \frac{M}{z} = \frac{P.e}{\frac{t_w . l_w^{\,2}}{6}} = \frac{6\,P.e}{t_w . l_w^{\,2}}$$

- As per code [Clause 10.5.10.1 page 80]
- Equivalent stress (*fe*) $f_e = \sqrt{f_b^{\,2} + 3q^{\,2}} < f_d$
- Where, $f_d = \frac{f_y}{\gamma_{mo}}$ for E250

$$= \frac{f_u}{\sqrt{3}\ .\ \gamma_{mw}} \quad for\ other\ grade$$

$\gamma_{mo} = 1.1$ (from Table 5 Page 30)

Approx. length of weld $= \sqrt{\dfrac{6P.e}{t_w\left(f_y/\gamma_{mo}\right)}}$

↓

Considering only bending stress

(Actual eff. length of weld will be more since weld is subjected to shear stress too).

(Q) Design a connection for the bracket using Butt weld carrying factored vertical load of 200 kN acting at 300mm from the face of column. Take thickness of bracket as 12mm, grade of steel is E250 (Fe 410).

Given:-

$P = 200\,kN$.

$e = 300\,mm$.

$t_{bracket} = 12\,mm$ (Generally thickness of bracket >10mm)

E250 ; $f_u = 410\,MPa$

$f_y = 250\,MPa$.

Take thickness of weld $t_w = 12\,mm$.

Approximate length of weld.

$$l_w = \sqrt{\frac{6\,Pe}{t_w\left(f_y/\gamma_{mo}\right)}}$$

$$= \sqrt{\frac{6 \times 200 \times 10^3 \times 300}{12 \times 250/1.1}}$$

$$= 363.318\,mm$$

Take $l_w = 400\,mm$

shear stress

$$q = \frac{V}{l_w \times t_w} = \frac{200 \times 10^3}{400 \times 12} = 41.67 \text{ MPa}$$

Bending stress

$$f_b = \frac{6Pe}{t_w \cdot l_w^2} = \frac{6 \times 200 \times 10^3 \times 300}{12 \times (400)^2}$$

$$= 187.5 \text{ N/mm}^2.$$

Now,

Equivalent stress $f_e = \sqrt{f_b^2 + 3q^2}$

$= 200.91 \text{ MPa} < f_d$ (ok)

$$f_d = \frac{f_y}{\gamma_{mo}} = \frac{250}{1.1} = 227.27 \text{ MPa}.$$

Provide 400 mm deep & 12 mm thick Bracket (full penetration Butt weld).

Bracket Connection Using Fillet Weld.

There are two ways of connecting a bracket to a column using a fillet weld. Bracket connected perpendicular to the flange of column or in the plane of flange of a column i.e. parallel to flange of column.

(i) Bracket perpendicular to flange of Column.

Bracket is connected to flange of a column with fillet welds on either side of bracket.

The fillet welds are subjected to factored shear force $V = P$ and factored B.M. ($M = P.e$), due to which shear stresses in horizontal & vertical directions are developed in the fillet welds at their throats.

Horizontal

Shear stress due to bending moment $q_b = \frac{M}{Z}$

$$q_b = \frac{P.e}{2 \times \frac{t_w . l_w^2}{6}}$$

$$\boxed{q_b = \frac{3 P.e}{t_w . l_w^2}}$$

$$\Rightarrow q_b = \frac{M}{I} \times y$$

$$= \frac{P.e}{2 \times \frac{t_w . l_w^3}{12}} . \frac{l_w}{2}$$

$$= \frac{3 Pe}{t_w . l_w^2}$$

Vertical shear stress at weld due to shear force

$$q_s = \frac{V}{2 . t_w . l_w} = \frac{P}{2 t_w . l_w}$$

$$f_e = \sqrt{f_b^2 + 3q^2} < f_d$$

where,

$$f_{wd} = \frac{f_u}{\sqrt{3}\ \gamma_{mw}}$$

Now, Approx. length may be obtained by considering the shear stress in weld due to bending only.

$$q_b = f_{wd} = \frac{3Pe}{t_w \cdot l_w^2}$$

$$l_w = \sqrt{\frac{3Pe}{t_w \cdot f_{wd}}}$$

Bracket connection using Butt weld

- Butt weld is subjected to vertical shear force V = P and a bending moment M = P . e

- Consider a bracket connection to flange of a column by a full penetration butt weld
- Eccentric load P causes shear force P and moment at the weld section

- Shear stress in the weld due to factored load

$$q = \frac{V}{l_w . t_w}$$

- Bending stress in the weld due to factored load

$$f_b = \frac{M}{z} = \frac{P.e}{\frac{t_w . l_w^2}{6}} = \frac{6\,P.e}{t_w . l_w^2}$$

- As per code [Clause 10.5.10.1 page 80]
- Equivalent stress (*fe*)

$$f_e = \sqrt{f_b^2 + 3q^2} < f_d$$

- Where, $f_d = \frac{f_y}{\gamma_{mo}}$ for E250

$$= \frac{f_u}{\sqrt{3} \cdot \gamma_{mw}} \quad for\ other\ grade$$

$\gamma_{mo} = 1.1$ (from Table 5 Page 30)

Approx. length of weld $= \sqrt{\dfrac{6P.e}{t_w\left(f_y/\gamma_{mo}\right)}}$

↓

Considering only bending stress

(Actual eff. length of weld will be more since weld is subjected to shear stress too).

(Q) Design a connection for the bracket using Butt weld carrying factored vertical load of 200 kN acting at 300mm from the face of column. Take thickness of bracket as 12mm, grade of steel is E250 (Fe 410).

Given:-

$P = 200\,kN$.

$e = 300\,mm$.

$t_{bracket} = 12\,mm$ (Generally thickness of bracket >10mm)

E250 ; $f_u = 410\,MPa$

$f_y = 250\,MPa$.

Take thickness of weld $t_w = 12\,mm$.

Approximate Length of weld.

$$l_w = \sqrt{\frac{6\,Pe}{t_w\left(f_y/\gamma_{mo}\right)}}$$

$$= \sqrt{\frac{6 \times 200 \times 10^3 \times 300}{12 \times 250/1.1}}$$

$$= 363.318\,mm$$

Take $l_w = 400\,mm$

shear stress

$$q = \frac{V}{l_w \times t_w} = \frac{200 \times 10^3}{400 \times 12} = 41.67 \text{ MPa}$$

Bending stress

$$f_b = \frac{6Pe}{t_w \cdot l_w^2} = \frac{6 \times 200 \times 10^3 \times 300}{12 \times (400)^2}$$

$$= 187.5 \text{ N/mm}^2.$$

Now,

Equivalent stress $f_e = \sqrt{f_b^2 + 3q^2}$

$= 200.91 \text{ MPa} < f_d$ (ok)

$$f_d = \frac{f_y}{\gamma_{mo}} = \frac{250}{1.1} = 227.27 \text{ MPa}.$$

Provide 400mm deep & 12mm thick Bracket (full penetration Butt weld).

Bracket Connection Using Fillet Weld.

There are two ways of connecting a bracket to a column using a fillet weld. Bracket connected perpendicular to the flange of column or in the plane of flange of a column i.e. parallel to flange of column.

(i) Bracket perpendicular to flange of Column.

Bracket is connected to flange of a column with fillet welds on either side of bracket.

The fillet welds are subjected to factored shear force $V = P$ and factored B.M. ($M = P.e$), due to which shear stresses in horizontal & vertical directions are developed in the fillet welds at their throats.

Horizontal

Shear stress due to bending moment $q_b = \frac{M}{Z}$

$$q_b = \frac{P.e}{2 \times \frac{t_w . l_w^2}{6}}$$

$$\boxed{q_b = \frac{3 P.e}{t_w . l_w^2}}$$

$$\Rightarrow q_b = \frac{M}{I} \times y$$

$$= \frac{P.e}{2 \times \frac{t_w . l_w^3}{12}} . \frac{l_w}{2}$$

$$= \frac{3 Pe}{t_w . l_w^2}$$

Vertical shear stress at weld due to shear force

$$q_s = \frac{V}{2 . t_w . l_w} = \frac{P}{2 t_w . l_w}$$

$t_w = k.s$.

Now,

$$q_e = \sqrt{q_b^2 + q_s^2} \leq f_{wd}$$

where,

$$f_{wd} = \frac{f_u}{\sqrt{3}\ \gamma_{mw}}$$

Now, Approx. length may be obtained by considering the shear stress in weld due to bending only.

$$q_b = f_{wd} = \frac{3Pe}{t_w . l_w^2}$$

$$l_w = \sqrt{\frac{3Pe}{t_w \cdot f_{wd}}}$$

(Q) Design a connection for bracket using fillet weld when bracket plate is connected to column flange perpendicularly. Take P = 200 kN (factored), e = 300 mm, thickness of bracket = 12 mm. and Fe 410 grade of steel.

Soln. :-

$S_{max} = t - 1.5 = 12 - 1.5 = 10.5\,mm$.

$S_{min} = 5\,mm$ (for $10\,mm \le t \le 20\,mm$).

Take $\boxed{S = 6\,mm}$.

For fusion angle 90°, k = 0.7 (Table 22) Page 78

$t_w = 0.7 \times 6 = 4.2\,mm$

Now,

Effective length of fillet weld may be approximately obtained by considering the shear stress in weld ~~durn~~ due to bending only.

$$l_w = \sqrt{\frac{3\,P_e}{t_w \cdot f_{wd}}}$$

where,

$$f_{wd} = \frac{f_{wn}}{\gamma_{mw}} = \frac{f_u}{\sqrt{3} \cdot \gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.25}$$

$$= 189.37 \text{ MPa}$$

$$\therefore l_w = \sqrt{\frac{3 \times 200 \times 10^3 \times 300}{4.2 \times 189.37}} = 475.72 \text{ mm}.$$

~~Take~~ ~~tw~~ = A 500 mm thick bracket may be tried.

Effective length of fillet weld, $l_w = 500 - 2 \times 5 = 488$

$$q_b = \frac{3Pe}{t_w \cdot l_w^2} = \frac{3 \times 200 \times 10^3 \times 300}{4.2 \times 488^2} = \cancel{186.61 \text{ MPa}}$$

179.96 MPa

$$q_s = \frac{V}{2 \cdot t_w \cdot l_w} = \frac{200 \times 10^3}{2 \times 4.2 \times 488} = 48.8 \text{ MPa}$$

Cl. 10.5.10.1.1 Page 80.

$$\boxed{f_e = \sqrt{q_b^2 + q_s^2}}$$

$= 186.46 < 189.37$ MPa (OK)

$$f_{wd} = \frac{f_u}{\sqrt{3} \cdot \gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.25} = 189.37$$

Hence, 500mm depth bracket is connected by 6mm size fillet weld on either side.

FILLET WELD (ECCENTRIC)

BRACKET IN PLANE OF FLANGE

2. Bracket in plane of flange

- The fillet weld is subjected to a shear force $V = P$ and a twisting moment $M = P \cdot e$
- Due to shear force P, shear stress (qs) is produced at the throat of fillet weld.
- Another shear stress (qt) is developed due to twisting moment M
- Direction of qt at any point on length of weld is perpendicular to the line joining that point with centroid C of the weld.

W
e = x̄ + a.
P =
y
x̄
a
x
G
x.
y
Bracket plate.

Plan

Section

$T = P.e.$

$$\tau = \frac{T}{I_p} \times R$$

Shear stress due to direct shear.

$$q_s = \frac{V}{L_w \cdot t_w} = \frac{P}{L_w \cdot t_w} \quad (\uparrow)$$

Shear stress due to torsion ($M_t = P.e$).

$$q_t = \frac{M_t}{I_p} \times r = \frac{P.e.r}{I_p}$$

where, I_p = Polar moment of inertia of the eff. weld section.

$$q_{tx} = q_t \cos\theta$$
$$q_{ty} = q_t \cdot \sin\theta$$

$$\cos\theta = y/r$$
$$\sin\theta = x/r$$

$$q_{tx} = q_t \cdot \frac{y}{r}$$
$$= \frac{P.er}{I_p} \cdot \frac{y}{r} = \frac{P.e.y}{I_p} \quad (\rightarrow)$$

$$q_{ty} = q_t \cdot \sin\theta$$
$$= \frac{P.e.r}{I_p} \cdot \frac{x}{r} = \frac{P.e.x}{I_p} \quad (\uparrow)$$

Net result,

$$q_x = q_{tx} = \frac{P e y}{I_p}$$

$$q_y = q_{ty} + q_s$$

$$= \frac{P \cdot e \cdot x}{I_p} + \frac{P}{l_w \cdot t_w}$$

$$q_e = \sqrt{q_x^2 + q_y^2} \leq \frac{f_y}{\sqrt{3} \cdot \gamma_{mw}}$$

(Q) A bracket plate is weld to the flange of column section. Factored load = 110 kN. Assume Fe 410 Grade & shop weld.

Soln :-

$P = 110$ kN

$\gamma_{mw} = 1.25$ (shop weld)

$L_w = 200 \times 2 + 260 = 660$ mm

Let t_w be thickness of weld.

Now, Centroid of Weld :-

$$x_c = \frac{\Sigma l_i x_i}{\Sigma l_i}$$

$$= \frac{200 \times 100 + 260 \times 0 + 200 \times 100}{660}$$

$$= 60.61 \text{ mm.}$$

$$e = (200 - 60.61) + 80$$

or, $\boxed{e = 219.39 \text{ mm}}$

$$\begin{cases} x = 139.39 \text{ mm} \\ y = 260/2 = 130 \text{ mm} \end{cases}$$

Now,

$$I_p = I_x + I_y$$

$$I_p = [I_{CG} + Ah^2]_x + [I_{CG} + Ah^2]_y$$

$$I_x = \left[\frac{200 \times tw^3}{12} + 200tw \times 130^2\right] \times 2 + \left[\frac{260^3 \times tw}{12}\right]$$

$$= 8.2247 \times 10^6\ tw$$

$$I_y = \left(\frac{260 \times tw^3}{12} + 60.61^2 \times 260tw\right) + 2 \times \left[\frac{200^3 \times tw}{12} + 200\,tw \times (100 - 60.61)^2\right]$$

$$= 2.8091 \times 10^6\ tw$$

$$I_p = 11.1338 \times 10^6\ tw$$

$$q_x = \frac{P \cdot e \cdot y}{I_p}$$

$$= \frac{110 \times 10^3 \times 219.39 \times 130}{11.1338 \times 10^6 \, t_w}$$

$$= \frac{281.78}{t_w}$$

$$q_y = \frac{P e x}{I_p} + q_s$$

$$= \frac{110 \times 10^3 \times 219.39 \times 139.39}{11.1338 \times 10^6 \, t_w} + \frac{110 \times 10^3}{t_w \cdot 660}$$

$$= \frac{468.8}{t_w}$$

$$q_e = \sqrt{q_x^2 + q_y^2}$$

$$= \frac{546.97}{t_w} \leq f_d$$

$$\frac{546.97}{t_w} \leq \frac{f_y}{\sqrt{3}\,\gamma_{mw}}$$

$$t_w \geq \frac{546.97\sqrt{3} \times 1.25}{410}$$

$$t_w \geq 2.89 \text{ mm}$$

$$t_w = ks$$

$$s = \frac{2.89}{0.7} = 4.12 \text{ mm}$$

$$s \geq 4.12 \text{ mm}$$

Take $s = 5$ mm

(Q) Design the connection using fillet weld subjected to direct shear 50.64 kN & torsional moment 7.09 kNm.

Soln.

Total length of weld provided = 200+90+90
= 380 mm

$$x_c = \frac{\Sigma l_i x_i}{\Sigma l_i}$$

$$= \frac{90 \times 45 + 90 \times 45 + 200 \times 0}{90 + 90 + 200}$$

$$= 21.31 \text{ mm}$$

$$x = 90 + 21.31$$

$$= 68.69$$

$$y = 100$$

Now,

$$I_x = \left[\cancel{\frac{90 \times t_w^3}{12}}^{\,0} + 90 t_w \times 100^2\right] \times 2 + \frac{200^3 \times t_w}{12}$$

$$= 2.467 \times 10^6 \, t_w \ mm^4$$

$$I_y = \left[\frac{90^3 \times t_w}{12} + 90 t_w \times (45 - 21.31)^2\right] \times 2 +$$

$$\left[\cancel{\frac{200 \times t_w^3}{12}}^{\,0} + (200 t_w) \times (21.31)^2\right]$$

$$= 0.3133 \times 10^6 \, t_w \ mm^4$$

Polar moment of inertia of the weld section

$$I_p = I_x + I_y = 2.7803 \times 10^6 \, t_w$$

⇒ Max torsional shear occurs at D or A.

q_t = Max torsional shear at A since A is the farthest point from C.G.

$$q_t = \frac{M.r}{I_p}$$

$$q_{tx} = \frac{M.y}{I_p} = \frac{7.09 \times 10^6 \times 100}{2.7803 \times 10^6 \, t_w} = \frac{255.01}{t_w} \; N/mm^2$$

$$q_{ty} = \frac{M x}{I_p} = \frac{7.09 \times 10^6 \times 68.69}{2.7803 \times 10^6 \, t_w} = \frac{175.16}{t_w} \; N/mm^2$$

$$q_s = \frac{V}{l_w \cdot t_w} = \frac{50.64 \times 10^3}{380 \cdot t_w} = \frac{133.26}{t_w} \ N/mm^2$$

$$q_x = q_{tx} = 255.01 / t_w$$

$$q_y = q_{ty} + q_s = \frac{308.42}{t_w}$$

$$q_e = \sqrt{q_x^2 + q_y^2} = \frac{400.19}{t_w} \leq f_d$$

or, $$\frac{400.19}{t_w} \leq \frac{410}{\sqrt{3} \times 1.25}$$

$$t_w \geq 2.11 \ mm$$

Now,

$$t_w = ks = 0.7s$$

$$s = \frac{t_w}{0.7} = \frac{2.11}{0.7}$$

$$s \geq 3.02 \ mm.$$

$\therefore$ A 4mm size fillet weld will be provided. (This size of 4mm fillet will hold good for up 10mm thick batten).

HOME ASSESSMENT

(Q) Determine the max. load that can be resisted by the bracket shown by fillet weld if size is 6mm if it is shop welding.

Bolted Connection

Introduction

Advantages

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

Disadvantages

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

- There are several types of bolts used to connect structural members. Some of them are listed as follows:
- *Unfinished bolts or black bolts or C grade bolts (IS 1363 : 2002)*
- *Turned bolts*
  - Precision bolts or A grade bolts (IS 1364 : 2002)
  - Semi-precision bolts or B grade bolts (IS 1364 : 2002)
- *Ribbed bolts*
- *High strength bolts (IS 3757 : 1985 and IS 4000 : 1992)*

BLACK BOLTS

- Black bolts are also referred to **as ordinary, unfinished, rough, or common bolts.**
- They are the least expensive bolts.
- However, they may not produce the least expensive connection since the connection may require a large number of such bolts.
- They are primarily used in light structures under static loads such as small trusses, purlins, bracings, and platforms.
- They are also used as temporary fasteners during erection where HSFG bolts or welding are used **as permanent** fasteners.
- They are not recommended for connections subjected to impact, fatigue, or dynamic loads.
- These bolts are made from mild steel rods with a square or hexagonal head and nuts

Hexagonal head black bolt and nut. Figures in brackets are for high-strength bolts and nuts.

- The bolts are available from 5 to 36 mm in diameter and are designated as M5 to M36.
- Most common ones are M16, M20, M24 to M30
- The bolts of property class 3.6 to 12.9 are available, of which most commonly used bolts are of property class 4.6 and 8.8; bolts of property class is more common.
- Here, the number before decimal indicates 1/100th of the nominal ultimate tensile strength and the number after decimal indicates the ratio of yield stress to ultimate stress, expressed as a percentage.
- Thus, the ultimate tensile strength of class 4.6 bolt is 400 N/mm2 and the yield strength is 240 N/mm2 (0.6 times 400).
- For ordinary bolted joints, the force is transferred through the interlocking and bearing of bolts and the joint is called bearing-type joint.

HIGH-STRENGTH BOLTS

- The high-strength bolts are made from bars of medium carbon heat treated steel and from alloy steel.
- Their high strength is achieved through quenching and tempering processes or by alloying steel.
- These bolts may be tightened until they have very high tensile stresses, two or more times that of ordinary bolts, so that the connected parts are clamped tightly together between the bolts and nut heads; this permits loads to be transferred primarily by friction and not by shear.
- This friction is developed by applying a load normal to the joint by tightening these bolts to proof load. That is why these bolts are also known as friction-type bolts.
- The joint using high-strength friction grip bolts is called non-slip connection or slip-critical connection or friction-type connection.

HIGH-STRENGTH BOLTS

(a) Types of high-strength bolts

(b) Slip resistance

Fig. High-strength bolted connection

HIGH-STRENGTH BOLTS

- The high strength bolts are available in sizes from 16 to 36 mm
- Designated as M16, M20, M24 and M30.
- These bolts are identified by the property class identification symbol such as 8S, 8.8S, 10S or 10.9S embossed on the head of the bolt. Here, S stands for high strength.
- Commonly used bolts are of 8.8S and 10.9S property class.

TYPES OF BOLTED JOINTS

(a) Lap joint

(b) Single bolted lap joint

(c) Double bolted lap joint

(d) Eccentricity in lap joint

(e) Single-cover butt joint

(f) Single-cover single bolted butt joint

(g) Single-cover double bolted butt joint

(h) Double-cover single bolted butt joint

(i) Double-cover single bolted butt joint

(j) Double-cover double bolted butt joint

(k) Lap joint, bolt in single shear

(l) Butt joint, bolt in double shear

TYPES OF FAILURE

Shear failure

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

Single shear

Double shear

Bearing failure of bolt

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

Bearing failure of bolt

Bearing on plate and bolt

Bearing failure of plate

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

Bearing failure of plate

Rupture of plate

Tensile failure of plate

(d) Tension failure of plate

Tension failure of bolt

Tensile failure of bolts

Block shear failure

- Bolts may have been placed at a lesser end-distance than required causing the plates to shear out which, however, can be checked by observing the specifications for end-distance.
- This type of failure occurs with shear on one plane and tension on perpendicular plane leading to fall of hatched portion of the plate.

Block shear failure

Spacing of bolt

10.2.2 *Minimum Spacing*

The distance between centre of fasteners shall not be less than 2.5 times the nominal diameter of the fastener.

10.2.3 *Maximum Spacing*

10.2.3.1 The distance between the centres of any two adjacent fasteners shall not exceed $32t$ or 300 mm, whichever is less, where t is the thickness of the thinner plate.

10.2.3.2 The distance between the centres of two adjacent fasteners (pitch) in a line lying in the direction of stress, shall not exceed $16t$ or 200 mm, whichever is less, in tension members and $12t$ or 200 mm, whichever is less, in compression members; where t is the thickness of the thinner plate. In the case of compression members wherein forces are transferred through butting faces, this distance shall not exceed 4.5 times the diameter of the fasteners for a distance equal to 1.5 times the width of the member from the butting faces.

Pitch

- It is distance between the centre of two consecutive bolt measured along the row of bolt in the direction of load.

Pitch $\not>$ 16t or 200 mm. (in Tension member)
12 t or 200 mm (in Compression member)

Table 19 Clearances for Fastener Holes

(*Clause* 10.2.1)

Sl No.	Nominal Size of Fastener, *d* mm	Size of the Hole = Nominal Diameter of the Fastener + Clearances mm			
		Standard Clearance in Diameter and Width of Slot	Over Size Clearance in Diameter	Clearance in the Length of the Slot	
				Short Slot	Long Slot
(1)	(2)	(3)	(4)	(5)	(6)
i)	12 – 14	1.0	3.0	4.0	2.5 *d*
ii)	16 – 22	2.0	4.0	6.0	2.5 *d*
iii)	24	2.0	6.0	8.0	2.5 *d*
iv)	Larger than 24	3.0	8.0	[illegible]	[illegible]

Nominal Diameter of Bolt (mm) (d)	*Diameter of Hole (mm) (d_o or d_h)*
12	13
14	15
16	18
18	20
20	22
22	24
24	26
27	30
30	33
Over 33 mm	Bolt diameter +3 mm

DESIGN SPECIFICATION OF BOLTED CONNECTION

Shearing strength of bolt

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

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

(a) Threads included in the shear plane

(b) Threads excluded from the shear plane

IS 800 Clause **10.3.3** *Shear Capacity of Bolt*

The design strength of the bolt, V_{dsb} as governed shear strength is given by:

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

where

V_{nsb} = nominal shear capacity of a bolt, calculated as follows:

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

where

f_u = ultimate tensile strength of a bolt;

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

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

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

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

Bearing strength of bolt

- The tests on the bolted joints reveal that the nominal bearing strength of bolts is independent of grades of bolts and the metal in contact with them.
- Neither the bolts nor the metal in contact with the bolts fail in bearing.
- Tests have also shown that the magnitude of the bearing stresses do affect the efficiency of the connected parts
- It is evident that the bearing strength of a bolted connection is a function of strength of the connected parts and the arrangement of the bolts rather than the grades of bolts.

For the safety of the joint in bearing, the bearing strength of the bolt should be less than the design bearing strength of the bolt ($V_{pb} \leq V_{dpb}$). The design strength of bolt in bearing is given by

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

where γ_{mb} = the partial safety factor for material of bolt
= 1.25

$$V_{npb} = 2.5 k_b d t f_u$$

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

d_0 = diameter of the hole

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

f_{ub} = ultimate tensile stress of the bolt

f_u = ultimate tensile stress of the plate in MPa

d = nominal diameter of the bolt in mm, and

t = aggregate thickness of the connected plates experiencing bearing

10.3.4 *Bearing Capacity of the Bolt*

The design bearing strength of a bolt on any plate, V_{dpb} as governed by bearing is given by:

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

where

V_{npb} = nominal bearing strength of a bolt

$= 2.5\, k_b\, d\, t\, f_u$

where

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

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

d_0 = diameter of the hole;

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

d = nominal diameter of the bolt; and

t = summation of the thicknesses of the connected plates experiencing bearing stress in the same direction, or if the bolts

Tensile strength of plate

6.3.1 *Plates*

The design strength in tension of a plate, T_{dn}, as governed by rupture of net cross-sectional area, A_n, at the holes is given by

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

where

γ_{m1} = partial safety factor for failure at ultimate stress (*see* Table 5),

f_u = ultimate stress of the material, and

A_n = net effective area of the member given by,

$A_n = (b - n d_o) \cdot t$

Dia. of hole d_o = Nominal dia. of bolt + Clearance.

$\gamma_{m1} = 1.25$.

STRENGTH AND EFFICIENCY OF THE JOINT

Strength of a bolted joint is the minimum strength based on strength of bolts, in shear and bearing, in the joint and the strength of the main member at the net section. Efficiency of a bolted joint (η) also called the *percentage strength* of the joint is ratio of the strength of the joint to the strength of the main member expressed as a percentage. The effectiveness of a particular bolted joint is measured by its efficiency.

$$\eta = \frac{\text{Strength of bolted joint/pitch length}}{\text{Strength of solid plate/pitch length}} \times 100$$

BOLTED CONNECTION

LECTURE 2

(Q) Design a lap joint between two plate shown in the figure, so as to transmit the factor load of 70kN using M16 bolt of grade 4.6 & the grade of plate material Fe 410.

$d = 16$ mm

$f_{ub} = 400$ MPa

$f_{yb} = 240$ MPa

$f_u = 410$ MPa

$f_y = 250$ MPa

Clearance = 2mm (Pg 73 Table 19)

dia. of hole $d_0 = 16 + 2 = 18$mm

Design Shear Capacity of Bolt (10.3.3)

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} (n_n A_{nb} + n_s A_{sb})$$

Assume shear plane lies on thread,

$n_n = 1$

$n_s = 0$

$A_{sb} = \frac{\pi}{4} \times 16^2 = 200.96 \text{ mm}^2$

$A_{nb} = 0.78 A_{sb} = 156.75 \text{ mm}^2$

$$V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} (1 \times 156.75)$$

$$= \underline{28.96 \text{ KN}}$$

Design Bearing Capacity of Bolt

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5\, k_b \cdot d \cdot t \cdot f_u}{\gamma_{mb}}$$

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

Page 73 cl 10.2.2. Pitch (p) $\nless 2.5d$

$\nless 2.5 \times 16$

$\nless 40mm$

$P_{min} = 40mm$

$P_{max} = 16t$ or $200mm$

$= 16 \times 12$

$= 192\ mm$ or $200mm$

$P_{max} = 192\ mm$

$e_{max} = 12 t \varepsilon$

$= 12 \times 12 \times \left(\frac{250}{f_y}\right)^{1/2}$

$= 12 \times 12 \times \left(\frac{250}{250}\right)^{1/2}$

$= 144 mm$

$e_{min} = 1.7 d_o = 1.7 \times 18$

$= 30.6 mm$

Take $e = 40 mm$

$p = 50 mm$

$k_b = \frac{40}{3 \times 18}, \frac{50}{3 \times 18} = 0.25, \frac{400}{410}, 1.0$

$= 0.75, 0.6759, 0.9756, 1.0$

$k_b = 0.6759$

$V_{dpb} = \frac{2.5 \times 0.6759 \times 16 \times 12 \times 410}{1.25}$

$= \underline{106.414 kN}$

Bolt strength = Min. of shearing & Bearing
= 28.96 kN

Bolt value $= \frac{70}{28.96} = 2.417 \approx 3$ bolts.

case(I)

Total design load carried by bolt
$= 3 \times 28.96$
$= 86.88$ kN

$A_n = [200 - 18 \times 3] \times 12 = 1752$

$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}$ (Page 32 Cl 6.3.1)

$= \frac{0.9 \times 1752 \times 410}{1.25} = 517.2$ kN > 86.88 kN

(ok)

Case (II)

$$A_n = [200 - 18] \times 12 = 2184 \text{ mm}^2$$

$$T_{dn} = \frac{0.9 A_n f_y}{\gamma_{m1}} = 644.717 \text{ kN} > 86.88 \text{ kN}$$

(safe)

OK

(Q) Two 12 mm thick steel plate are spliced by two 6 mm thick plate with four M16 bolt of property class 4.6. Determine the ultimate load carrying capacity of the connection & check for the cover plate. Use Fe 410 grade of steel. & also check for block shear failure.

Soln :- $d = 16\,mm$ $\qquad d_o = 16 + 2 = 18\,mm$

$f_{ub} = 400\,MPa$,

$f_{yb} = 240\,MPa$

$F_u = 410\,MPa$

$f_y = 250\,MPa$.

Design strength of single bolt,

(1) In Shear :- (Pg 75)

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

$$= \frac{f_u}{\sqrt{3}\,\gamma_{mb}} (n_n A_{nb} + n_s A_{sb})$$

Consider, $n_n = 1$

$n_s = 1$

$$A_{sb} = \frac{\pi}{4} \times d^2 = \frac{\pi}{4}(16)^2 = 201.062\,MPa.$$

$$A_{nb} = 0.78\,A_{sb} = 156.828$$

$$V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} (1 \times 156.828 + 1 \times 201.062)$$

$$= 66.121 \text{ KN}$$

Strength of 4 bolts in shear $= 4 \times 66.121$

$$= 264.484 \text{ KN}$$

(2) In Bearing:

$$e_{min} = 1.7 d_o = 1.7 \times 18 = 30.6 \text{ mm}$$

$$e_{max} = 12t \left(\frac{250}{f_y}\right)^{1/2} = 12 \times 12 \times \left(\frac{250}{250}\right)^{1/2}$$

$$= 144 \text{ mm}$$

$$P_{min} = 2.5 \times d = 2.5 \times 16 = 40 \text{ mm}$$

$$P_{max} = 16t \text{ or } 200 \text{ mm}$$

$$= 16 \times 12 \text{ or } 200$$

$$= 192 \text{ or } 200$$

$$P_{max} = 192 \text{ mm}.$$

Take $e = 40mm$

$P = 80mm$.

$$K_b = \frac{e}{3d_o}, \frac{P}{3d_o} - 0.25, \frac{f_{ub}}{f_u}, 1.$$

$$= 0.7407, 1.2315, 0.9756, 1$$

$$K_b = 0.7407$$

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5 K_b \, d \, t \, f_u}{\gamma_{mb}}$$

$$= \frac{2.5 \times 0.7407 \times 16 \times 12 \times 410}{1.25}$$

$$= 116.616 \text{ kN}.$$

Strength of 4 bolts = 466.463 kN.

Now, Bolt value = Min of V_{dsb} & V_{dpb}

$= 264.484$ kN

(3) Check for Plate / Cover Plate :-

Tension failure of main plate :-

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} = \frac{0.9 \times 1968 \times 410}{1.25} = 580.954 kN$$

$$A_n = (200 - 2 \times 18) \times 12 = 1968 \ mm^2$$

Tension failure of Cover plate :-

$$A_n = (200 - 2 \times 18) \times 6 = 984$$

$$T_{dn} = \frac{0.9 \times 984 \times 410}{1.25} = 290.477 \text{ kN}$$

Strength of two cover plates $= 580.954$ kN

> 264.484 kN

(OK)

4) Check for Block shear failure :- Pg 33 [6.4]

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

OR

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

Shear plane : 1 - 2
3 - 4

Tension plane : 2 - 3

$$A_{vg} = (40+80) \times 2 \times 12$$
$$= 2880 \, mm^2$$

$$A_{vn} = (40+80-1.5 \times 18) \times 2 \times 12$$
$$= 2232$$

$$A_{tg} = 80 \times 12 = 960$$
$$A_{th} = (80-18) \times 12 = 744$$

$$T_{db} = 597.53 \, kN$$
$$= 598.59 \, kN$$

} smaller is taken

$$T_{db} = 597.53 \, kN$$

∴ Ultimate Load carrying capacity of Joint
$= 264.484 \, kN$
(Shear failure of Bolt controls).

(Q) Determine the ultimate load carrying capacity in tension of lap joint shown in figure. If the bolt threads are outside the shear plane. Use M16 bolts of total grade C & property class 4.6. The yield & ultimate strength of the flats are 250 MPa & 410 MPa respectively.

Property class of bolt = 4.6.

$f_{ub} = 400\ MPa$

$f_{yb} = 0.6 \times 400 = 240\ MPa$

$d_o = 16 + 2 = 18\ mm$

Design strength of plate due to rupture at net section

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}} = \frac{0.9\left[(120 - 2\times18)\times12\right]\times410}{1.25}$$

$$= \underline{297.56\ kN} \quad \text{— (1)}$$

Design shear strength of Bolt (Pg 75)

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

$$= \frac{f_{ub}\left(A_{nb}\, n_n + n_s A_{sb}\right)}{\sqrt{3}\cdot \gamma_{mb}}$$

$$= \frac{400\left(A_{nb} \times 0 + 1 \times \pi/4 \times 16^2\right)}{\sqrt{3} \times 1.25}$$

$$= 37.147 \text{ kN}$$

Load carrying capacity in shear $= 4 \times 37.147$

$= \underline{148.586 \text{ kN}}$.

Design bearing strength of bolt (Pg 75)

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5\, k_b \cdot d \cdot t \cdot f_u}{1.25}$$

$$k_b = \frac{30}{3 \times 18}, \frac{80}{3 \times 18} - 0.25, \frac{400}{410}, 1.0$$

$$= 0.55, 1.23, 0.9756, 1.0$$

$$k_b = 0.55$$

$$V_{dpb} = \frac{2.5 \times 0.55 \times 16 \times 12 \times 410}{1.25}$$

$$= 86.592 \text{ kN}$$

Design strength of connection in bearing

$= 4 \times 86.592 = \underline{346.368 \text{ kN}}$

$\therefore$ Design strength in connection $= 148.586$ kN — (2).

Design strength due to Block shear

$$T_{db} = \frac{A_{vg} f_y}{\gamma_{m0}\sqrt{3}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}} = 495.19 \text{ kN}$$

$$= \frac{0.9 A_{vn} f_u}{\gamma_{m1}\sqrt{3}} + \frac{A_{tg} f_y}{\gamma_{m0}} = 503.14 \text{ kN}$$

$A_{vg} = (80+30) \times 12 \times 2 = 2640$

$A_{vn} = (80+30-1.5 \times 18) \times 2 \times 12 = 1992$

$A_{tg} = 60 \times 12 = 720$

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

$\therefore T_{db} = 495.19$ kN.

$\Rightarrow$ Ultimate load carrying capacity of Joint $= 148.586$ kN

Shear failure of bolt controls

(Q) Design a double cover plate butt joint to transmit a working load of 300 kN to connect two flats 120 mm wide & 20 mm thick using M16 high strength bolts of property class 10.9 if

(a) Slip is not permitted at working load.

(b) Slip is not " " ultimate load.

The cover plates are 12 mm thick. Assume that one shear plane intersects the threads of the bolt.

a) **Slip is not permitted at working load**

- The joint is designed for slip resistance at the working load 300 kN
- But a slip may occur at ultimate load.
- So the bolts should be checked for shear and bearing at ultimate load.

10.4 Friction Grip Type Bolting

10.4.1 In friction grip type bolting, initial pretension in bolt (usually high strength) develops clamping force at the interfaces of elements being joined. The frictional resistance to slip between the plate surfaces subjected to clamping force opposes slip due to externally applied shear. Friction grip type bolts and nuts shall conform to IS 3757. Their installation procedures shall conform to IS 4000.

10.4.2 Where slip between bolted plates cannot be tolerated at working loads (slip critical connections), the requirements of **10.4.3** shall be satisfied. However, at ultimate loads, the requirements of **10.4.4** shall be satisfied by all connections.

(b) Slip resistance

Fig. High-strength bolted connection

$$V_{sf} \leq V_{dsf}$$

where¢

V_{dsf} = V_{nsf} / γ_{mf}

V_{nsf} = nominal shear capacity of a bolt as governed by slip for friction type connection, calculated as follows:

$$V_{nsf} = \mu_f \, n_e \, K_h \, F_o$$

where

μ_f = coefficient of friction (slip factor) as specified in Table 20 ($\mu_f = 0.55$),

n_e = number of effective interfaces offering frictional resistance to slip,

K_h = 1.0 for fasteners in clearance holes,

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

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

γ_{mf} = 1.10 (if slip resistance is designed at service load),

= 1.25 (if slip resistance is designed at ultimate load),

F_o = minimum bolt tension (proof load) at installation and may be taken as $A_{nb} f_0$,

A_{nb} = net area of the bolt at threads, and

f_o = proof stress ($= 0.70 f_{ub}$).

NOTE — V_{ns} may be evaluated at a service load or ultimate load using appropriate partial safety factors, depending upon whether slip resistance is required at service load or ultimate load.

where, $\mu_f = 0.55$

$n_e = 2$

$K_h = 1.0$

$F_o = A_{nb} \cdot f_o = 0.78 \times \frac{\pi}{4} \times 16^2 \times 0.70 f_{ub}$

$= 109779.8137$

$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{120890}{1.1} = 109900 N$ (for 1 Bolt)

No. of bolt needed $= \frac{300 \times 10^3}{109900} = 2.73 \approx 3$ nos

The joint should be checked for bearing, shear & block shear at ultimate load.

$e_{min} = 1.7 d_o = 30.6$

$e_{max} = 12t\left(\frac{250}{f_y}\right)^{1/2} = 144$

$P_{min} = 2.5d = 40$

$P_{max} = 16t = 192\,mm$

Take $e = 40\,mm$ & $P = 60\,mm$

⇒ Shear capacity of bolt $V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}$

$$= \frac{f_u(n_n A_{nb} + n_s A_{sb})}{\sqrt{3}\,\gamma_{mb}}$$

$$= \frac{1000(1\times 201 + 1\times 157)}{\sqrt{3}\times 1.25}$$

$$= 165.35 \text{ kN}$$

⇒ Bearing capacity of Bolt

$$V_{dpb} = \frac{2.5\, k_b \cdot d \cdot t \cdot f_u}{\gamma_{mb}}$$

$$k_b = \frac{e}{3d_0},\ \frac{p}{3d_0} - 0.25,\ \frac{f_{ub}}{f_u},\ 1$$

$$= \frac{40}{3\times 18},\ \frac{60}{3\times 18} - 0.25,\ \frac{1000}{410},\ 1$$

$$= 0.7407,\ 0.8611,\ 2.44,\ 1$$

$\therefore K_b = 0.7407$

$$V_{dpb} = \frac{2.5\times 0.7407\times 16\times 20\times 410}{1.25}$$

$$= 194.36 \text{ kN}$$

Design strength of a bolt = 165.35 kN.

Design strength of 3 bolts in ultimate load = ~~388.1~~ 496.05 kN

> 300 × 1.5

> 450 kN

(ok)

⇒ Check for block shear.

Avg = (40+60+60) × 20 = 3200 mm²

(along 1-2-3-4)

$A_{vn} = (160 - 2.5 \times 18) \times 20 = 2300\ mm^2$
(along 1-2-3-4)

$A_{tg} = 60 \times 20 = 1200\ mm^2$ (along 4-5)

$A_{tn} = \left(60 - \frac{18}{2}\right) \times 20 = 1020\ mm^2$
(along 4-5)

$$\frac{A_{vg} f_y}{\gamma_{m0} \sqrt{3}} + \frac{0.9\, A_{tn} f_u}{\gamma_{m1}} = \frac{3200 \times 250}{1.1 \times \sqrt{3}} + \frac{0.9 \times 1020 \times 410}{1.25}$$
$$= 720995.1\ N$$

$$\frac{0.9\, A_{vn} f_u}{\gamma_{m1} \sqrt{3}} + \frac{A_{tg} f_y}{\gamma_{m0}} = \frac{0.9 \times 2300 \times 410}{1.25 \times \sqrt{3}} + \frac{1200 \times 250}{1.1}$$
$$= 664725\ N$$

$T_{db} = 664725\ N = 664.725\ kN > 450\ kN$
(OK)

Hence, 3 bolts may be provided.

(b) Slip is not permitted at ultimate load

(i.e. slip resistance at ultimate load)

$V_{nsf} = 120890$ N

Design frictional force produced by a bolt

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{120890}{1.25} = 96712 \text{ N}$$

$$\text{No. of bolts needed} = \frac{1.5 \times 300 \times 10^3}{96712} = 4.65$$

Five bolts may be provided.

Eccentric Bolted connection

Type II: Perpendicular to flange

Fig. 4.63 (*b*)

(a) When Bracket is connected at right angle to the flange of column.

P
e
Bolt
A
Bolt
B
h

X
X
y2
y1
h/7
Bolt
A
Bolt
B

⇒ Bracket is connected to the column with the help of angle section and it is assumed that depth of neutral axis ~~ex~~ (NA) = $\frac{h}{7}$ from the bottom of the bracket.

⇒ X – X is centroidal axis → Neutral axis.

(1) Factored moment M = P.e is resisted by the tensions developed in the bolts above the N.A.

(2) For equilibrium, the moments of tensions in the bolts above the NA should be equal to P.e.

$$P.e = 2T_1y_1 + 2T_2y_2 + \dots + 2T_ny_n \quad (1)$$

$T_1, T_2 \ldots T_n$ = Tensions in each of the bolts

$y_1, y_2 \ldots y_n$ = distances from the NA to the respective bolts.

Assumptions : The tension in a bolt is proportional to the distance ($y_1, y_2 \ldots y_n$) from the neutral axis.

$$\frac{T_1}{y_1} = \frac{T_2}{y_2} = \ldots = \frac{T_n}{y_n}$$

Then,

$$T_1 = \frac{y_1}{y_n} T_n$$

$$T_2 = \frac{y_2}{y_n} T_n$$

$$T_n = \frac{y_n}{y_n} T_n \quad \text{——— (2)}$$

$$M = Ta_1 + Cb_1$$

Substituting in eq^n ① yields.

$$\frac{2y_1^2}{y_n}T_n + \frac{2y_2^2}{y_n}T_n + \cdots + \frac{2y_n^2}{y_n}T_n = P.e.$$

or, $$\frac{T_n}{y_n}\left[2y_1^2 + 2y_2^2 + \cdots + 2y_n^2\right] = P.e.$$

or, $$\frac{2T_n}{y_n}\Sigma y_i^2 = P.e.$$

or, $$\boxed{T_n = \frac{P.e.y_n}{2\Sigma y_i^2}}$$ $\Rightarrow$ Max. tension developed in bolt of topmost row.

Shear force in each bolt due to direct shear

$$V_{sb} = \frac{P}{N} \quad (N = \text{No. of bolts})$$

Bolt subjected to tension & shear should satisfy the interaction eq^n

$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1$$

Approximate no. of bolt in a single vertical line

$$n = \sqrt{\frac{6M}{m\,p\,T_{db}}}$$

where

m = no. of vertical line

p = pitch of bolt

$M = P.e$

(Q) Design a connection for a bracket using M20 bolts of product grade & property class 4.6 to carry factored vertical load of 200 kN acting at 300 mm from the face of the column. 12 mm thick bracket is connected by 2 ∠100 100 X10. The grade of steel is E250.

Given:-

Dia. of bolt (d) = 20 mm

f_{ub} = 400 MPa

f_{yb} = 240 MPa

P = 200 kN

e = 300 mm

M = P · e = 60 kNm

DESIGN OF BOLT 'A' :-

Bolts 'A' are subjected to SINGLE SHEAR & BENDING MOMENT.

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

Assume, bolt threads are outside shear plane.

$$V_{nsb} = \frac{400}{\sqrt{3}} \left(0 \times A_{nb} + 1 \times 314 \right) = 72.52 \text{ KN}$$

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{72.52}{1.25} = 58 \text{ kN}$$

Design tension capacity of a bolt

T_{db} = Least of T_{dbg} or T_{dbn}.

T_{dbn} = Design strength of a bolt due to rupture at the net section (i.e. at root of thread)

$$T_{dbn} = \frac{0.9\, A_{nb}\, f_{ub}}{\gamma_{mb}} = \frac{0.9 \times 245 \times 400}{1.25} = 70.56\ kN$$

T_{dbg} = Design strength of a bolt due to yielding of a gross section (i.e. at the shank).

$$T_{dbg} = \frac{A_{sb} f_{yb}}{\gamma_{mo}} = \frac{314 \times 240}{1.1} = 68.5 \text{ kN}$$

$\therefore T_{db} = 68.5$ kN

Let n = approx. no. of bolts required in a vertical line.

$$n = \sqrt{\frac{6M}{m p T_{db}}}$$

$$= \sqrt{\frac{6 \times 60 \times 10^6}{2 \times 60 \times 68.5 \times 10^3}}$$

Take
e = 40mm
p = 60mm

$= 6.62 \approx 7$ nos. in a line.

Total no. of bolts $N = 2 \times 7 = 14$

Depth of bracket, $h = 2 \times 40 + 6 \times 60 = 440\,mm$

Depth of neutral axis $= h/7 = 63\,mm$

$y_1 = 100 - 63 = 37.$

$y_2 = 37 + 60 = 97$

$y_3 = 97 + 60 = 157$

$y_4 = 157 + 60 = 217$

$y_5 = 217 + 60 = 277$

$y_6 = y_n = 337 \text{ mm}.$

Max. tension developed in the bolts of topmost row

$$T_n = \frac{P e y_n}{2 \Sigma y_i^2}$$

$$= \frac{60 \times 10^6 \times 337}{545628}$$

$\therefore T_b = T_n = 37.06 \text{ kN}$

$$V_{sb} = \frac{P}{N} = \frac{200 \times 10^3}{14} = 14.286 \text{ kN}$$

↑ factored SF acting on a bolt.

Now,

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

$$\left(\frac{14.286}{58}\right)^2 + \left(\frac{37.06}{68.5}\right)^2 = 0.353 < 1.0$$

(ok)

Oversafe

(0.8 to 0.9 is suitable).

So, the no. of bolts may be reduced to 12 (6 in each row/line) and the calculation is repeated.

$h = 2 \times 40 + 5 \times 60 = 380\,mm$

$h/7 = 54.286\,mm \approx 54\,mm$

$y_1 = 100 - 54 = 46\,mm$

$y_2 = 46 + 60 = 106\,mm$

$y_3 = 166\,mm$

$y_4 = 226\,mm$

$y_5 = y_n = 286\,mm$

$$T_n = \frac{P \cdot e \cdot y_n}{2 \Sigma y_i^2} = 49.37\,kN$$

$$V_{sb} = \frac{P}{N} = \frac{200}{12} = \underline{\underline{16.67}}\,kN$$

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

or, $$\left(\frac{16.67}{58}\right)^2 + \left(\frac{49.37}{68.5}\right)^2 = 0.6 < 1.0$$

(OK)

⇒ Check for Bearing

$$K_b = \frac{e}{3d_0}, \left(\frac{p}{3d_0} - 0.25\right), \frac{f_{ub}}{f_u}, 1.0$$

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

$$= 0.606, 0.659, 0.97, 1.0$$

$$\therefore k_b = 0.606$$

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

bearing on angle

$$= \frac{2.5 \times 0.606 \times 20 \times 10 \times 410}{1.25} = 99.384 \text{ kN}$$

$> V_{sb}$ (16.67 kN)

(OK)

DESIGN OF BOLTS 'B':

These bolts are subjected to DIRECT SHEAR & TWISTING (TORSION)

Twisting moment = 200 x 240 = 48 kNm

$\left(\begin{matrix}100-40\\=60\end{matrix}\right)$

e = 300 − 60 = 240 mm.

As these bolts are in double shear,

$V_{dsb} = 2 \times 58 = 116$ kN

Since there is only one vertical line m = 1.

$$\tau = \frac{T}{I_p} \times R$$

Direct shear is equally resisted by all bolts so.

$$\boxed{V' = \frac{P}{N}} \quad (\uparrow)$$

Twisting moment $M = P.e$ is resisted by the shear forces V_1'', V_2'', . . . , V_N'' developed in the bolt. If the applied moment is clockwise, then the ~~clockwise~~ direction of these shear forces are such that they produce an anticlockwise moment. So for equilibrium,

$$V_1'' r_1 + V_2'' r_2 + \ldots + V_N'' r_N = M = P.e. \quad (1)$$

It is assumed that the shear forces V_1'', V_2'' ... V_N'' are proportional to their distances r_1, r_2 ... r_N respectively from the CG of bolt.

$$\frac{V_1''}{r_1} = \frac{V_2''}{r_2} = \dots = \frac{V_N''}{r_N}$$

or, $V_1'' = \frac{r_1}{r_N} V_N''$

$V_2'' = \frac{r_2}{r_N} V_N''$

$V_N'' = \frac{r_N}{r_N} V_N''$

V_N'' = Max S.F. in the extreme bolt due to twisting moment Pe.

Substituting in (1) we get.

$$\frac{r_1^2}{r_N} V_N'' + \frac{r_2^2}{r_N} V_N'' + \dots + \frac{r_N^2}{r_N} V_N'' = P.e.$$

or, $\frac{V_N''}{r_N} (\Sigma r_i^2) = Pe.$

or, $V_N'' = \frac{P.e\, r_N}{\Sigma r_i^2}$

$$V''_{Nx} = V''_N \cos\theta = \frac{P.e.r_N}{\Sigma r_i^2} \cdot \frac{y_N}{r_N} = \frac{P.e.y_N}{\Sigma r_i^2}$$

$$V''_{Ny} = V''_N \sin\theta = \frac{P.e.r_N}{\Sigma r_i^2} \cdot \frac{x_N}{r_N} = \frac{P.e.x_N}{\Sigma r_i^2}$$

$$V_{Nx} = V''_{Nx} = \frac{P.e.y_N}{\Sigma r_i^2}$$

$$V_{Ny} = V' + V''_{Ny} = \frac{P}{N} + \frac{P.e.x_N}{\Sigma r_i^2}$$

V_N = Max. Resultant SF in the extreme bolt

$$V_N = \sqrt{V_{Nx}^2 + V_{Ny}^2} \leq V_{db}$$

V_{db} = least of V_{dsb} & V_{dpb}

Approx. no. of bolts $n = \sqrt{\frac{6M}{m\,p\,V_{dsb}}}$

m = No. of vertical lines of bolt.

Since there is only one vertical line $m = 1$.

$$n = \sqrt{\frac{6M}{m p V_{db}}}$$

$$= \sqrt{\frac{6 \times 48 \times 10^6}{1 \times 60 \times 116 \times 10^3}} = 6.43 \approx 7.$$

So, 7 Bolts may be provided.

$r_1 = 60$

$r_2 = 120$

$r_3 = 180$

$$V_N'' = \frac{P \cdot e \cdot r_n}{\Sigma r_i^2}$$

$$\Sigma r_i^2 = 2(r_1^2 + r_2^2 + r_3^2) = 100800\,mm^2$$

$$V_{Nx} = V_{Nx}'' = \frac{P.e.y_n}{\Sigma r_i^{\circ 2}}$$

$$= \frac{48 \times 10^6 \times 180}{100800}$$

$$= 85.71 \text{ kN}$$

$$V_{Ny} = V' + V_{Ny}''$$

$$= \frac{P}{N} + \frac{P.e.x_N}{\Sigma r_i^{\circ 2}} \qquad [V_{Ny}'' = 0]$$

$$= \frac{P}{N} + 0$$

$$= \frac{200}{7}$$

$$= 28.57 \text{ kN}$$

$$V_N = \sqrt{85.71^2 + 28.57^2} = 90.35 \text{ kN} < V_{db}$$

(116 kN)

(OK)

Check for Bearing

$$V_{dpb} = \frac{2.5\, k_b\, d.t\, f_u}{\gamma_{mb}}$$

$$= \frac{2.5 \times 0.606 \times 20 \times 12 \times 410}{1.25}$$

$$= 119.26 \text{ kN} > 90.35 \text{ kN} \ (\text{OK})$$

Eccentric Bolted connection

Type I: In plane of flange

BOLTED BRACKET CONNECTIONS

TYPE 1

(b) Bracket connected in the plane of flange of column.

$$\tau = \frac{T}{I_p} \times R$$

Direct shear is equally resisted by all bolts so.

$$\boxed{V' = \frac{P}{N}} \quad (\uparrow)$$

Twisting moment $M = P.e$ is resisted by the shear forces V_1'', V_2'', . . . , V_N'' developed in the bolt. If the applied moment is clockwise, then the ~~clockwise~~ direction of these shear forces are such that they produce an anticlockwise moment. So for equilibrium,

$$V_1'' r_1 + V_2'' r_2 + \ldots + V_N'' r_N = M = P.e. \quad (1)$$

It is assumed that the shear forces V_1'', V_2'' ... V_N'' are proportional to their distances r_1, r_2 ... r_N respectively from the CG of bolt.

$$\frac{V_1''}{r_1} = \frac{V_2''}{r_2} = \dots = \frac{V_N''}{r_N}$$

or, $V_1'' = \frac{r_1}{r_N} V_N''$

$V_2'' = \frac{r_2}{r_N} V_N''$

$V_N'' = \frac{r_N}{r_N} V_N''$

V_N'' = Max S.F. in the extreme bolt due to twisting moment Pe.

Substituting in (1) we get.

$$\frac{r_1^2}{r_N} V_N'' + \frac{r_2^2}{r_N} V_N'' + \dots + \frac{r_N^2}{r_N} V_N'' = P.e.$$

or, $\frac{V_N''}{r_N} (\Sigma r_i^2) = Pe.$

or, $V_N'' = \frac{P.e\, r_N}{\Sigma r_i^2}$

$$V''_{Nx} = V''_N \cos\theta = \frac{P.e.r_N}{\Sigma r_i^2} \cdot \frac{y_N}{r_N} = \frac{P.e.y_N}{\Sigma r_i^2}$$

$$V''_{Ny} = V''_N \sin\theta = \frac{P.e.r_N}{\Sigma r_i^2} \cdot \frac{x_N}{r_N} = \frac{P.e.x_N}{\Sigma r_i^2}$$

$$V_{Nx} = V''_{Nx} = \frac{P.e.y_N}{\Sigma r_i^2}$$

$$V_{Ny} = V' + V''_{Ny} = \frac{P}{N} + \frac{P.e.x_N}{\Sigma r_i^2}$$

V_N = Max. Resultant SF in the extreme bolt

$$V_N = \sqrt{V_{Nx}^2 + V_{Ny}^2} \leq V_{db}$$

V_{db} = least of V_{dsb} & V_{dpb}

Approx. no. of bolts $n = \sqrt{\frac{6M}{m p V_{dsb}}}$

m = No. of vertical lines of bolt.

(Q) Design a connection for bracket using high strength M20 bolts a property class 8.8 if the bracket is connected in the plane of the flange. The width of column is 200mm. The bracket has to carry a factored vertical load of 200kN. The eccentricity of load is 300mm from the edge of the column flange. Also design the connection if

(a) No slip is permitted at the ultimate load.

(b) A slip is permitted at the ultimate load but not a working load.

Dia of bolts $d = 20$ mm

$d_0 = 20 + 2 = 22$ mm

$f_{ub} = 800$ MPa

$f_{yb} = 640$ MPa

$e = 300 + 100 = 400$ mm

$M = P.e$

$= 200 \times 0.4$

$= 80$ kNm

(a) Slip is not permitted at ultimate load

$$V_{nsf} = \mu_f \cdot n_e \cdot K_h \cdot F_o$$
$$= 0.55 \times 1 \times 1 \times 137200$$
$$= 75.46 \text{ kN}$$

$$F_o = A_{nb} f_o$$
$$= 0.78 \times \frac{\pi}{4} \times 20^2 \times 0.7 \times 800$$
$$= 245 \times 0.7 \times 800$$
$$= 137200$$

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{75.46}{1.25}$$
$$= 60.37 \text{ kN}$$

$$n = \sqrt{\frac{6\,M}{m_p V_{dsf}}} = \sqrt{\frac{6 \times 80 \times 10^6}{2 \times 60 \times 60.37 \times 10^3}} = 8.14 \approx 9$$

Hence 18 bolts may be provided.

$g = 100\,mm$ for HB200 section.

$$\Sigma r_i^2 = 4\left[(50^2+240^2)+(50^2+180^2)+(50^2+120^2)+(50^2+60^2)\right]$$
$$= 472000\,mm^2 + 2\times(50)^2$$

$$V_{Nx} = \frac{P.e.y_N}{\sum r_i'^2} = \frac{80\times10^6 \times 240}{472000} = 40.68\ kN$$

$$V_{Ny} = \frac{P}{N} + \frac{P.e.x_N}{\sum r_i'^2} = \frac{200}{18} + \frac{80\times10^6 \times 50}{472000} = 19.6\ kN$$

$$V_N = \sqrt{V_{Nx}^2 + V_{Ny}^2} = 45.15\ kN < 60.37\ kN$$

(OK)

(b) Slip is permitted at ultimate load but not at working load.

If slip is permitted at ultimate load, bolts are designed for bearing & shear at ultimate load.

⇒ Shear capacity of bolt

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}}\left(\cancel{n_n}^{\,0} A_{nb} + \cancel{n_s}^{\,1} A_{sb}\right)$$

$$= \frac{800}{\sqrt{3}}(314)$$

$$= 145 \text{ kN}$$

$$V_{dsb} = \frac{145}{1.25} = 116 \text{ kN}$$

Take $e = 40$ mm

$p = 60$ mm

⇒ Bearing capacity of bolt.

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

$$= \frac{40}{3 \times 22}, \frac{60}{3 \times 22} - 0.25, \frac{800}{410}, 1.0$$

$$= 0.606, 0.659, 1.95, 1.0$$

$$K_b = 0.606$$

$$V_{dpb} = \frac{2.5 K_b \cdot d \cdot t \cdot f_u}{\gamma_{mb}}$$ (Take thickness of bracket = 12 mm)

$$= \frac{2.5 \times 0.606 \times 20 \times 12 \times 410}{1.25}$$

$$= 119.26 \text{ kN}$$

$$\therefore \boxed{V_{db} = 116 \text{ kN}}$$

No. of bolts in a line $n = \sqrt{\dfrac{6M}{m p V_{db}}}$

$$= \sqrt{\frac{6 \times 80 \times 10^6}{2 \times 60 \times 116 \times 10^3}}$$

$$= 5.9 \simeq 6$$

Twelve bolts may be provided.

$$\Sigma r_i^2 = 4\left[(50^2+30^2)+(50^2+90^2)+(50^2+150^2)\right]$$

$$= 156000$$

$$V_{Nx} = \frac{P.e.y_N}{\Sigma r_i^2}$$

$$= \frac{80 \times 10^6 \times 150}{156000 \times 10^3}$$

$$= 77 \text{ kN}$$

$$V_{Ny} = \frac{P}{N} + \frac{P e x_N}{\Sigma r_i^2}$$

$$= \frac{200}{12} + \frac{80 \times 10^3 \times 50}{156000} = 42.3 \text{ kN}$$

$\therefore V_N = 87.85 \text{ kN} < 116 \text{ kN}$ (OK).

The provided should be checked for no slip at the working load.

$$V_{dsf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{75.46}{1.1} = 68.6 \text{ kN}.$$

$$\text{Working load} = \frac{200}{1.5} = 133 \text{ kN}$$

$$V_{Nx} = \frac{133 \times 400 \times 150}{156000} = 51 \text{ kN}$$

$$V_{Ny} = \frac{133}{12} + \frac{133 \times 40 \times 50}{156000} = 28 \text{ kN}$$

$$V_N = 58.18 \text{ kN} < 68.6 \text{ kN} \quad (\underline{OK})$$

Staggered arrangement of bolts

- Empirical equation by V. H. Cochrane
- Along the inclined/diagonal line there is a combination of direct tensile stress and shear stress and thus a smaller area should be used.
- Deduction = holes c/s area – $\frac{\sum p^2 t}{4\,g}$
- $\frac{p^2 t}{4\,g}$ for each line extending between holes at other than right angles
- The chain of lines should be chosen so as to produce max. deductions
- It is merely an approximation or simplification for the staggered arrangement of bolts

$$A_n = \left[b - nd_h + \sum_i \frac{p_{si}^2}{4g_i} \right] t$$

where

b, t = width and thickness of the plate, respectively,

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

g = gauge length between the bolt holes, as shown in Fig. 5,

p_s = staggered pitch length between line of bolt holes, as shown in Fig. 5,

n = number of bolt holes in the critical section, and

i = subscript for summation of all the inclined legs.

FIG. 5 PLATES WITH BOLTS HOLES IN TENSION

Q) Determine min. net area of c/s. of 300mm x 12 mm shown in fig below. The dia. of the holes are 17.5 mm

Soln:-

Along 1 - 2 - 5 - 6

$$be = b - nd$$
$$= 300 - 2 \times 17.5$$
$$= 265\,mm$$
$$A_{net} = be \times t$$
$$= 265 \times 12 = 3180\ mm^2$$

Along 1 - 2 - 3 - 4 - 7

$$be = b - nd + \sum_i \frac{p_i^2}{4g_i}$$

$$= 300 - 3 \times 17.5 + \frac{40^2}{4 \times 60}$$

$$= 254\,mm$$

$$A_{net} = 254 \times 12 = 3048\,mm^2$$

Along 1-2-3-4-5-6

$$be = b - nd + \frac{\Sigma p_i^2}{4g_i}$$

$$= 300 - 4 \times 17.5 + \frac{40^2}{4 \times 60} + \frac{40^2}{4 \times 60}$$

$$= 243 \text{ mm}$$

$$A_{net} = 243 \times 12$$

$$= 2916 \text{ mm}^2$$

$\therefore A_{min} = 2916 \text{ mm}^2$ (Critical Section).

Chapter 5: Tension member

Tension member

- Member carrying a tensile force
- May be wires, cables, bars and rods and flat plates
- May fail by

Yielding of gross section

Rupture of net section

Block shear failure

Tension failure of plate

Tensile failure of plate

(d) Tension failure of plate

Yielding of gross section

- It means considerable deformation of tension member before fracture.
- Design strength of tension member due to yielding

6.2 Design Strength Due to Yielding of Gross Section

The design strength of members under axial tension, T_{dg}, as governed by yielding of gross section, is given by

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

where

f_y = yield stress of the material,

A_g = gross area of cross-section, and

γ_{m0} = partial safety factor for failure in tension by yielding (*see* Table 5).

Rupture of net section

- Occurs when tension member reaches the ultimate stress at net section

For Plates :-

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

For Threaded rod :-

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

For single angles :-

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

6.3 Design Strength Due to Rupture of Critical Section

6.3.1 *Plates*

The design strength in tension of a plate, T_{dn}, as governed by rupture of net cross-sectional area, A_n, at the holes is given by

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

where

γ_{m1} = partial safety factor for failure at ultimate stress (*see* Table 5),

f_u = ultimate stress of the material, and

A_n = net effective area of the member given by,

6.3.2 *Threaded Rods*

The design strength of threaded rods in tension, T_{dn}, as governed by rupture is given by

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

6.3.3 *Single Angles*

The rupture strength of an angle connected through one leg is affected by shear lag. The design strength, T_{dn}, as governed by rupture at net section is given by:

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

where

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

where

w = outstand leg width,

b_s = shear lag width, as shown in Fig. 6, and

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

Stress
distribution
Stress

Stress
distribution
Stress

FIG. 6 ANGLES WITH SINGLE LEG CONNECTIONS

Block shear failure

- Occurs due to insufficient edge and end distance
- Bolts may have been placed at a lesser end-distance than required causing the plates to shear out which, however, can be checked by observing the specifications for end-distance.

This type of failure occurs with shear on one plane and tension on perpendicular plane leading to fall of hatched portion of the plate.

Block shear failure

https://m2ukblog.wordpress.com/2016/05/28/block-shear-failure-in-tension-members/

6.4 Design Strength Due to Block Shear

The strength as governed by block shear at an end connection of plates and angles is calculated as given in **6.4.1**.

6.4.1 *Bolted Connections*

The block shear strength, T_{db} of connection shall be taken as the smaller of,

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

or

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

where

A_{vg}, A_{vn} = minimum gross and net area in shear along bolt line parallel to external force, respectively (1-2 and 3-4 as shown in Fig. 7A and 1-2 as shown in Fig. 7B),

A_{tg}, A_{tn} = minimum gross and net area in tension from the bolt hole to the toe of the angle, end bolt line, perpendicular to the line of force, respectively (2-3 as shown in Fig. 7B), and

f_u, f_y = ultimate and yield stress of the material, respectively.

Design strength due to Block shear (6.4 Pg 33)

Block shear strength of bolted connection is least of

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

OR

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

where,

A_{vg}, A_{vn} = Min. gross & net areas in the shear along the bolt line of action of force respectively (along 1-2-3 & 4-5-6)

Design Steps of Tension member subjected to axial load

1. Find net sectional area $A_n = \dfrac{T}{0.9 f_u / \gamma_{m1}}$

   $(\gamma_{m1} = 1.25)$

2. Find approx. area of tension member

   $A_{approx} = (1.25 \text{ to } 1.4)\ A_g$

   $A_g = T \cdot \dfrac{\gamma_{m0}}{f_y}$ $\quad (\gamma_{m0} = 1.1)$

3. Select appropriate steel section having $A_{g,\,trial} \geq A_{approx}$.

4. Design end connection (Bolted OR Welded).

5. Check tension member fails by yielding, rupture or block shear failure.

   $(T_d \geq T)$

6. Check Slenderness ratio $= \dfrac{l_e}{r}$

   $r = \sqrt{\dfrac{I}{A}}$

(Q1) Determine the effective area of the section.
Take Fe 410.
ISA (100 75 X 10)

Soln

Case (A) BOLTED CONNECTION

$f_u = 410 MPa$.

$d = 18 mm$; $d_o = 18 + 2 = 20 mm$

Area of connected leg $= A_{nc} = \left[L - d_o - \frac{t}{2}\right] \times t$

$$A_{nc} = \left[100 - 20 - \frac{10}{2}\right] \times 10 = 750 \text{ mm}^2$$

Area of outstanding leg $= A_{go} = \left[b - \frac{t}{2}\right] \times t$

$$A_{go} = \left[75 - \frac{10}{2}\right] \times 10 = 700 \text{ mm}^2$$

Total area $= A_n' = A_{nc} + A_{go} = 1450 \text{ mm}^2$.

If single angle section is used as a tension member then net area should be modified by a factor α.

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

where

α = 0.6 for one or two bolts, 0.7 for three bolts and 0.8 for four or more bolts along the length in the end connection or equivalent weld length;

α depends on no. of bolts used.

$\alpha = \underline{0.7}$ for 3 bolts (Pg 33).

$$A_n = \alpha A_n$$
$$= 0.7 \times 1450 = 1015 \, mm^2$$

Case [B] WELDED CONNECTION

$$A_{nc} = \left[100 - \frac{10}{2}\right] \times 10 = 950 \, mm^2$$

$$A_{go} = \left[75 - \frac{10}{2}\right] \times 10 = 700 \, mm^2$$

$$A_n' = A_{nc} + A_{go} = 1650 \, mm^2$$

$$A_n = \alpha A_n' = \underline{0.8} \times 1650 = 1320 \, mm^2$$

(Q3) A single unequal angle ISA 90×60×6mm is connected to a 10mm thick gusset plate at the end with 5 no. of 16mm bolts to transfer tension. Determine the design tensile strength of the angle if gusset is connected to longer side of the angle.

e = 30mm

p = 50mm

Soln :-

Check → Yielding
→ Rupture
→ Block failure.

(a) Design Strength due to yielding.

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

$$A_g = \left(90 - \frac{6}{2}\right) \times 6 + \left(60 - \frac{6}{2}\right) \times 6$$

$$= 864 \, mm^2$$

from code $A_g = 865 \, mm^2$

Assume Fe 410 ; $f_y = 250 MPa$.

$$T_{dg} = \frac{864 \times 250}{1.1} = 196.36 \text{ kN}$$ #

(b) Design Strength due to Rupture :

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

A_{nc} = Area of connected leg

$$= \left(90 - d_o - \frac{t}{2}\right) \times t$$

$$= (90 - 18 - 3) \times 6 = 414 \text{ mm}^2$$

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

$\therefore$ ~~T_{dn}~~ =

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

$$\geq 0.7$$

$w = 60$

$t = 6$

$b_s = w + w_1 - t = 60 + 50 - 6 = 104\ mm$

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

$\therefore \boxed{\beta = 1.16} < 1.44$

> 0.7 (ok).

$\therefore T_{dn} = 212.38\ kN$ #

(c) Design strength due to block shear.

$A_{vg} = (4 \times 50 + 30) \times 6 = 1380 \text{ mm}^2$

$A_{vn} = (4 \times 50 + 30 - 4.5 \times 18) \times 6 = 894 \text{ mm}^2$

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

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

$$T_{db} = \left.\begin{array}{c} 235.98 \text{ kN} \\ \text{OR} \\ 206.91 \text{ kN} \end{array}\right\}.$$

$\therefore \boxed{T_{db} = 206.91 \text{ kN}}$

Design Strength of Angle Section = 196.36 kN

Sairam (3.4)

(Q4) Find the ultimate design strength of angle 100 X 100 X 10 in tension which is connected to gusset 12 mm thick using M20 bolt of property class 4.6 in a single line. Assume the bolt threads are outside the shear plane. The yield & ultimate strengths are 250 MPa & 410 MPa respectively.

Soln :-

Steps

1) Design strength of angle due to yielding
$T_{dg} = 431.82$ kN.

2) Design strength of bolt $\left.\begin{array}{l}\rightarrow \text{Shear} \\ \rightarrow \text{Bearing}\end{array}\right\} = 58.04$ kN

No. of bolts = 8.

3) Design strength of angle in rupture

4) Design strength for Block shear failure.

5) Ultimate Design strength => Min of 1, 3, 4.

Chapter 5: Tension member

Lecture 2

6.2 Design Strength Due to Yielding of Gross Section

The design strength of members under axial tension, T_{dg}, as governed by yielding of gross section, is given by

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

where

f_y = yield stress of the material,

A_g = gross area of cross-section, and

γ_{m0} = partial safety factor for failure in tension by yielding (*see* Table 5).

6.3 Design Strength Due to Rupture of Critical Section

6.3.1 *Plates*

The design strength in tension of a plate, T_{dn}, as governed by rupture of net cross-sectional area, A_n, at the holes is given by

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

where

γ_{m1} = partial safety factor for failure at ultimate stress (*see* Table 5),

f_u = ultimate stress of the material, and

A_n = net effective area of the member given by,

6.3.2 *Threaded Rods*

The design strength of threaded rods in tension, T_{dn}, as governed by rupture is given by

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

6.3.3 *Single Angles*

The rupture strength of an angle connected through one leg is affected by shear lag. The design strength, T_{dn}, as governed by rupture at net section is given by:

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

where

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

where

w = outstand leg width,

b_s = shear lag width, as shown in Fig. 6, and

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

FIG. 6 ANGLES WITH SINGLE LEG CONNECTIONS

Design strength due to Block shear (6.4 Pg 33)

Block shear strength of bolted connection is least of

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

OR

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

where,

A_{vg}, A_{vn} = Min. gross & net areas in the shear along the bolt line of action of force respectively (along 1-2-3 & 4-5-6)

Design Steps of Tension member subjected to axial load

1. Find net sectional area $A_n = \dfrac{T}{0.9 f_u / \gamma_{m1}}$

   $(\gamma_{m1} = 1.25)$

2. Find approx. area of tension member

   $A_{approx} = (1.25 \text{ to } 1.4)\ A_g$

   $A_g = T \cdot \dfrac{\gamma_{m0}}{f_y}$ $\quad (\gamma_{m0} = 1.1)$

3. Select appropriate steel section having $A_{g,\,trial} \geq A_{approx}$.

4. Design end connection (Bolted OR Welded).

5. Check tension member fails by yielding, rupture or block shear failure.

   $(T_d \geq T)$

6. Check Slenderness ratio $= \dfrac{l_e}{r}$

   $r = \sqrt{\dfrac{I}{A}}$

Design tension member carrying a service load of 300 KN using *single angle* and *double angle* with bolted connection.

Design of Single angle section with Bolted connection

Gross - Sectional area $A_g = \frac{T \cdot \gamma_{mo}}{f_y}$

$$= \frac{(1.5 \times 300) \times 10^3 \times 1.1}{250}$$

$$= 1980 \text{ mm}^2$$

Take ISA 150 X 115 X 10 mm.

Area $= 20.1 \text{ cm}^2 = 2010 \text{ mm}^2 > 1980 \text{ mm}^2$.

Design of end Connection

Take M20 Bolts.

$d = 20\,mm$

$d_o = 22\,mm$

Shear capacity of bolt $V_{dsb} = \dfrac{f_u}{\sqrt{3}\,\gamma_{mb}}(n_n A_{nb} + n_s A_{sb})$

Assume threads of bolt lie in shear plane & Class of 4.6

$n_n = 1$

$n_s = 0$

$A_{nb} = \dfrac{\pi d^2}{4} \times 0.78 = 245.04\ mm^2$

$\therefore V_{dsb} = 45.27\ kN$

Bearing strength of Bolts.

$$V_{dpb} = \frac{2.5 K_b \cdot d \cdot t \cdot f_u}{\gamma_{mb}} = \frac{2.5 \times 0.606 \times 20 \times 10 \times 400}{1.25}$$

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

Assume, $e = 40mm$, $p = 60mm$

$K_b = 0.606$

$\therefore V_{dpb} = 96.96$ kN

$\therefore$ Strength of bolt $= 45.27$ kN.

No. of Bolts required $= \frac{1.5 \times 300}{45.27} = 9.94 \approx \underline{\underline{10 \text{ nos}}}$.

Check :-

Strength due to yielding $T_{dg} = A_g \frac{f_y}{\gamma_{m0}}$

$= \frac{2010 \times 250}{1.1}$

$= 456.82\ kN > (1.5 \times 300)$

$> 450\ kN$

(OK)

Strength due to rupture :-

Strength due to Block shear :-

Design of Double angle section

Also see 3.10 Sairam

Gross - sectional area $A_g = 1980\ mm^2$

Area of each angle $= 990\ mm^2$

Take 2 $\angle$ 70 X 70 X 8 mm

Total area $= 2 \times 1060 = 2120\ mm^2 > 1980\ mm^2$

Assuming angles are provided at one side of gusset plate.

Use M20 Bolts of property class 4.6

$d = 20\ mm$

$d_0 = 22\ mm$

$f_{ub} = 400\ MPa$.

→ Shear strength of bolt

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

Assume threads of bolt lies in shear plane

$n_n = 1$ & $n_s = 0$

$$V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} \left(1 \times 0.78 \times \frac{\pi}{4} (20)^2 \right)$$

$$= 45.27 \text{ kN}.$$

→ Bearing capacity of bolt :-

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

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

Min $p = 2.5d = 2.5 \times 20 = 50$ mm

Max $p = 16t = 16 \times 8$ or 200 mm $= 128$ mm

Min $e = 1.7 d_o = 37.4$

Max $e = 12t\sqrt{\frac{250}{f_y}} = 12 \times 8 = 96$ mm

Take $p = 70$ mm, $e = 50$ mm

$K_b = 0.76$

$\therefore V_{dpb} = 2.5 \times 0.76 \times 20 \times 8 \times \frac{410}{1.25} = 99.7$ kN

Strength of bolt $= 45.27$ kN

No. of bolts $= \frac{450}{45.27} = 9.94 \simeq 10$ Nos.

check for yielding

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{2120 \times 250}{1.1} = 481.8\,kN$$

$$> 450\,kN$$

Check for rupture.

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

$$A_{nc} = \left(70 - 22 - \frac{8}{2}\right) \times 8 = 352\ mm^2$$

$$A_{go} = \left(70 - \frac{8}{2}\right) \times 8 = 528\ mm^2$$

$$\beta = 1.4 - 0.076 \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_c} \leq \frac{f_u\, \gamma_{mo}}{f_y\, \gamma_{m1}}$$

$$\geq 0.7$$

$w = 70$ $\quad$ $f_u = 410$

$t = 8$ $\quad$ $b_s = w + w_1 - t = 70 + 35 - 8 = 97$

$f_y = 250$ $\quad$ $L_c = 70 \times 4 = 280$

$$\therefore \beta = 1.26 \leq 1.44$$
$$> 0.7$$

$$\therefore T_{dn} = 255.11\ kN \times 2 = 510.22 > 450\ kN$$

(OK).

Check for Block Shear :

$A_{vg} = (70 \times 4 + 50) \times 8 = 2640$

$A_{vn} = [(70 \times 4 + 50) - 4.5 \times 22] \times 8 = 1848$

$A_{tg} = 35 \times 8 = 280$

$A_{tn} = (35 - \frac{22}{2}) \times 8 = 192$

$T_{db} = 403.1$ kN OR 378.6 kN.

$T_{db} = 378.6 \times 2 = 757.2$ kN > 450 kN

(Q6) A single unequal angle 100 75 X 6 mm is connected to a 10 mm thick gusset plate at the end with a 6 - 16 mm dia bolts to transfer the tension. Determine design tensile strength of the angle assuming that $f_y = 250$ MPa, $f_u = 410$ MPa. If the gusset plate is connected to the 100 mm leg & also design connection for its full capacity.

Area = 1010 mm^2

Soln :-

⇒ Design tensile strength : Min. of (a), (b), (c)

(a) Gross - section yielding :- $T_{dg} = A_g \times f_y / \gamma_{mo} = 229.55\ kN$

(b) Net-section rupture :- $T_{nb} = 250.28\ kN$

$\beta \rightarrow \quad w = 75 ; \; b_s = 75 + 50 - 6 ; \; L_c = 5 \times \text{pitch}$.

(c) Block shear strength :- $T_{db} = 263.50\ kN$

⇒ Design for connection :

(a) Shear strength of bolts.

(b) Bearing " " "

Design strength of bolt = ?

No. of bolts required for full capacity (n) = ?

Chapter 5: Tension member

Lecture 3

Design of Single Angle Section with Welded Connection

Design load = 450 kN

Gross - sectional Area $A_g = \frac{T \cdot \gamma_{mo}}{f_y} = 1980 \, mm^2$

Take ∠ 150 115 x 10 mm

Fe 410, $f_y = 250 \, MPa$

Design strength of weld $P_u = \frac{f_u}{\sqrt{3} \, \gamma_{mw}} \cdot l_w \cdot t_w$

$\gamma_{mw} = 1.25$ (Assume shop weld)

$t_w = k \times s = 0.7 * s$

Size of weld (s) ⇒ $s_{min} = 5 \, mm$

$s_{max} = \frac{3}{4} \times 10 = 7.5 \, mm$

Provide $s = 6 \, mm$

$t_w = 0.7 \times 6 = 4.2 \, mm$

Now, $450 \times 10^3 = \frac{410}{\sqrt{3} \times 1.25} \times l_w \times 4.2$

$\therefore l_w = 565.78 \, mm$

Provide $l_w = 600 \, mm$

$l_w = 300 \, mm$ on both ends.

Check for yielding :-

$$T_{dg} = A_g \frac{f_y}{\gamma_{mo}} = \frac{2010 \times 250}{1.1} = 456.8\ kN$$

$> 450\ kN$ (OK)

Check for Rupture :-

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

$$\beta = 1.4 - 0.076 \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{l_c} \leq \frac{f_u \gamma_{mo}}{f_y \gamma_{m1}}$$

$$\geq 0.7$$

$w = 115\ mm$

$t = 10\ mm$

$b_s = w = 115\ mm$

$\beta = 1.297 < 1.44$

> 0.7 (OK)

$$A_{nc} = \left(150 - \frac{10}{2}\right) \times 10 = 1450\ mm^2$$

$$A_{go} = \left(115 - \frac{10}{2}\right) \times 10 = 1100\ mm^2$$

$\therefore T_{dn} = 752.29 > 450\ kN$

Practice question:

- Design a fillet welded channel section to act as a tension member subjected to tensile load of 300 KN.

Lug angle

- To make the connection at joints, a certain length of the tension member and that of the gusset plate is utilized.
- If the load is heavy and the number of bolts/length of weld required for making the connection are large, the size of the gusset plate required may be uneconomical and may be awkward.
- Lug angle is used along with the tension member to reduce the length of joint and consequently the size of gusset plate.
- In the case of angle members, lug angles and their connection to the gussets or other supporting members should be capable of developing strength not less than 20% in excess of the force in the outstanding leg of the angle.
- The attachment of the lug angle to the angle member should be capable of developing 40% in excess of that force.

Lug angle
Tension
member
Gusset
plate

Gusset
angle
lug angle
Lug angle
Channel
Gusset

- whole area of the member shall be taken as the effective (whole area = gross area – deduction for bolt holes)
- lug angle to gusset plate = 20% more than the force in outstanding leg
- lug angle to main member = 40% more than the force in outstanding leg

Design Steps:

1) Find load shared by connected leg (P_c) & bolts required for connection of connected leg & gusset plate (n_c).

$$P_c = \frac{A_c}{A} \times P$$

$$n_c = \frac{P_c}{\text{Design strength of bolts}}$$

2) Find load shared by outstanding leg (P_o) & bolts required for connection of ~~with~~ lug angle (n_o) with outstanding leg and lug angle with gusset plate (n_g).

$$P_o = \frac{A_o}{A_g} \times P$$

$$P_o = P - P_c$$

$$n_o = \frac{1.4\ P_o}{\text{Design strength of bolt}}$$

$$n_g = \frac{1.2\ P_o}{\text{Design strength of bolt}}$$

3) Find sectional area of lug-angle and select appropriate section for lug-angle.

$$A_g = \frac{T\ \gamma_{mo}}{f_y} = \frac{1.2\ P_o \times 1.1}{250}$$

(Q) Design a tension member to carry a factored tensile force of 450 kN. The length of connection of angle section to the gusset plate is limited to 400 mm. Take M20 bolts of grade 4.6.

Soln:-

Design load = 450 kN

M20, d = 20 mm, do = 22 mm

Length of connection ≤ 400 mm.

Sectional area of angle section: $A_g = \frac{T \times \gamma_{mo}}{f_y}$

$= 1980 \text{ mm}^2$

Take a trial section ISA 150×75×10 mm

$A_g = 2160 \text{ mm}^2 > 1980 \text{ mm}^2$

Design of connection

Design of connection

$$V_{dsb} = \frac{400}{\sqrt{3} \times 1.25}\left(1 \times 0.78 \times \frac{\pi}{4} \times 20^2\right) = 45.3 \text{ kN}$$

Take $e = 40\text{mm}$

$p = 50\text{mm}$

$$V_{dpb} = 2.5 K_b \times d \times t \times \frac{f_{ub}}{\gamma_{mb}} = 2.5 \times 0.507 \times 20 \times 10 \times 410/1.25$$

$$= 83.6 \text{ kN}$$

$$K_b = \frac{40}{3 \times 22}, \frac{50}{3 \times 22} - 0.25, \frac{f_{ub}}{f_u}, 1$$

$$= \underline{0.606}, 0.507, 0.976, 1$$

$\therefore$ Design strength of bolt $= \underline{45.3 \text{ kN}}$

No. of bolt required $= \frac{450}{45.3} = 9.94 \approx \underline{\underline{10}}$

Length of connection $= 9 \times 50 + 2 \times 40 = 530\,mm$
$> 400\,mm$.

So, provide lug angle.

$$P_c = \frac{A_c}{A} \times P$$

$P_c \rightarrow$ Connecting angle section load
$P_o \rightarrow$ lug angle load

$$A_c = \left(150 - \frac{10}{2}\right) \times 10 = 1450\,mm^2$$

$$P_c = \frac{1450}{2160} \times 450 = 302.08\,KN$$

$$n_c = \frac{P_c}{\text{Design strength}} = \frac{302.08}{45.3} = 6.67 \approx 7$$

$$P_o = P - P_c = 450 - 302.08 = 147.92\,KN$$

$$n_o = \frac{1.4\,P_o}{45.3} = \frac{1.4 \times 147.92}{45.3} = 4.57 \approx 5$$

$$n_g = \frac{1.2\,P_o}{45.3} = 3.92 \approx 4$$

$$\text{Area of Lug-angle} = \frac{1.2\,P_o \cdot \gamma_{mo}}{f_y} = \frac{1.2 \times 147.92 \times 1.1}{250}$$

$$= 781\ mm^2$$

Provide $\angle\ 70 \times 70 \times 6\ (800\ mm^2)$ as lug angle.

TENSION SPLICE

- When the available length is less than the required length of tension member, then splices are provided.
- Splice connection for tension member is usually provided in the bolted connection.
- The splice sections as well as the bolts are subjected to the tensile load to be transmitted by the main tension member.

- As per IS specification, the splice connection should be designed for a force of at least 0.3 times the member design capacity in tension or design action(load) whichever is more.
- If the sections to be spliced are not of same thickness steel packing should be provided.
- The design shear capacity of bolt carrying calculated shear through a packing plate in excess of 6 mm should be decreased by the factor β_{pkg}

10.3.3.3 *Packing plates*

The design shear capacity of bolts carrying shear through a packing plate in excess of 6 mm shall be decreased by a factor, β_{pk} given by:

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

where

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

Page 75 clause 10.3.3.3

Chapter 6

COMPRESSION MEMBER

INTRODUCTION

- A compression member is a structural member which is straight and subjected to two equal and opposite compressive forces applied at its ends.
- An ideal compression member is one which is perfectly straight, no imperfections; and the loads are applied uniformly across it, with the centre of gravity of loads coinciding with centre of gravity of the member. Such a compression member will be a truly axially loaded member.
- Compression members are actually more critical than tension members, because if even slightly bent compression member is placed in structure it may have significant bending moment (equal to the column load times the initial lateral deflection) whereas a tension member will try to straighten out.

www.shutterstock.com · 565915783

www.shutterstock.com · 482638447

STEEL STRUCTURE AND PIPE

www.shutterstock.com · 337093031

(a) Channels face-to-face connected by single lacing

(b) Channels back-to-back connected by single lacing

(c) Channels back-to-back connected by battens

(d) Angles connected by single lacing (shown on two faces only)

(a) Multistorey building (b) Industrial building

CLASSIFICATION OF COLUMN

SHORT	➢ usually fail by crushing or yielding. ➢ column shortens in the direction of applied force ➢ shortening of the column continues until the column squashes
INTERMEDIATE	➢ fail by inelastic buckling (yielding and buckling) ➢ Failure occurs after the extreme fibers have reached the yield point and the others remain elastic.
LONG	➢ fail by elastic buckling ➢ At the point of failure, the stress in a long column will not exceed the proportional limit

"---" : Original Shape

"—" : Buckled Shape

Buckled Steel Column

Buckled R.C.C. Column

Effective length

- The distance between the two points of contraflexure in longitudinal distance.
- Can be calculated from the actual length of column by applying a factor that depends upon the end conditions of column provided (Table 11 of IS 800:2007)

Boundary Conditions				*Schematic Representation*	*Effective Length (KL)*
At One End		*At the Other End*			
Translation	*Rotation*	*Translation*	*Rotation*		
Restrained	Restrained	Free	Free		2.0*L*
Free	Restrained	Restrained	Free		2.0*L*
Restrained	Free	Restrained	Free		1.0*L*
Restrained	Restrained	Free	Restrained		1.2*L*
Restrained	Restrained	Restrained	Free		0.8*L*
Restrained	Restrained	Restrained	Restrained		0.65*L*

Slenderness ratio $(\lambda = KL/r)$

- The longer a compression member becomes for the same cross section, the greater becomes its tendency to buckle and the smaller becomes the load it will support.
- The tendency of a member to buckle is usually measured by its slenderness ratio
- The slenderness ratio of a member is the ratio of the effective length to the appropriate radius of gyration

Table 8.4 Maximum slenderness ratio for compression members

S. No.	*Type of Member*	λ
1.	A tension member in which a reversal of direct stress occurs due to loads other than wind or seismic forces.	180
2.	A member carrying compressive loads resulting from dead loads and imposed loads.	180
3.	A member subjected to compressive forces resulting only from combination with wind/earthquake actions, provided the deformation of such members does not adversely effect the stress in any part of the structure.	250
4.	Compression flange of a beam restrained against lateral-torsional buckling.	300
5.	A member normally acting as a tie in a roof truss or a bracing system not considered effective when subjected to possible reversal of stresses resulting from the action of wind or earthquake forces.	350

BUCKLING

- A long column subjected to direct load deflects in lateral direction, is known as **buckling**
- The *maximum limiting load* at which the column tends to have lateral displacement or tends to buckle is called **buckling or crippling load**.
- Buckling takes place about the axis having *minimum radius of gyration* or least moment of inertia.

Slender Column
(Long Column)

Design of axially loaded compression member

1. Assume value of design stress *fcd*:
   - For rolled steel beam sections the slenderness ratio = 70 to 90; *fcd* = 135 N/mm2.
   - For angle struts, slenderness ratio = 100 to 130; *fcd* = 90 N/mm2
   - For compression members carrying large loads; *fcd* = 150-200 N/mm2
2. Calculate the design compressive stress *fcd* using above slenderness ratio and buckling class of required section
3. Calculate the gross sectional area of the member using Ag=Factored load/design compressive stress
4. Take a trial section having area greater than Ag
5. Calculate effective length of column as per given boundary conditions
6. Calculate actual slenderness ratio
7. Compare with permissible ratio table 3
8. Calculate actual design compressive stress by using actual slenderness ratio and buckling class of section
9. Calculate design capacity of section using Pd = A x *fcd* > factored load

1) Determine the load carrying capacity of a single discontinuous angle 80 x 80 x 6 mm which is used as a compression member in a roof truss if it connected to a gusset plate by 2 bolts. The centre to centre distance between the end connection is 2 m.

7.5.1.2 *Loaded through one leg*

The flexural torsional buckling strength of single angle loaded in compression through one of its legs may be evaluated using the equivalent slenderness ratio, λ_e, as given below:

$$\lambda_e = \sqrt{k_1 + k_2 \lambda_{vv}^2 + k_3 \lambda_\varphi^2}$$

where

k_1, k_2, k_3 = constants depending upon the end condition, as given in Table 12,

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 \varepsilon}{250}}} \text{ and } \lambda\varphi = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 \varepsilon}{250}}}$$

where

l = centre-to-centre length of the supporting member,

r_{vv} = radius of gyration about the minor axis,

b_1, b_2 = width of the two legs of the angle,

t = thickness of the leg, and

ε = yield stress ratio $(250/f_y)^{0.5}$.

Table 12 Constants k_1, k_2 and k_3

Sl No.	No. of Bolts at Each End Connection	Gusset/Connecting Member Fixity [1]	k_1	k_2	k_3
(1)	(2)	(3)	(4)	(5)	(6)
i)	≥ 2	Fixed	0.20	0.35	20
		Hinged	0.70	0.60	5
ii)	1	Fixed	0.75	0.35	20
		Hinged	1.25	0.50	60

Correction in IS 800:2007

Clause 7.5.1.2 ***Loaded through one leg***

Page 48

$$\lambda_e = \sqrt{k_1 + k_2 \lambda_{vv}^2 + k_3 \lambda_\phi^2}$$

where k_1, k_2, k_3 = constants depending upon the end condition

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} \quad \text{and} \quad \lambda_\phi = \frac{(b_1 + b_2)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}} \times 2t}$$

Soln :-

For ISA 80 x 80 X 6 mm

Area $A_e = 929\ mm^2$

$r_{min} = r_{vv} = 15.6\ mm$.

Cl 7.5.1.2 . Pg 48

$$\lambda_e = \sqrt{k_1 + k_2 \lambda_{vv}^2 + k_3 \lambda_\psi^2}$$

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon \sqrt{\frac{\pi^2 E}{250}}} = \frac{\left(\frac{2000}{15.6}\right)}{1 \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}} = \underline{1.44}$$

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

$$\lambda_\psi = \frac{(b_1 + b_2)/2t}{\varepsilon \sqrt{\frac{\pi^2 E}{250}}} = \frac{(80+80)/(2 \times 6)}{1 \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}} = 0.15$$

SECTION 5 EQUAL LEG ANGLES

Table 5.1 Nominal Dimensions, Mass and Sectional Properties of Indian Standard Equal Leg Angles

Designation	Mass M	Sectional area, a	Dimensions				Sectional Properties											
			A×B	t	R_1	R_2	C_x	C_y	I_x	I_y	$I_u(Max)$	$I_v(Min)$	r_x	r_y	$r_u(Max)$	$r_v(Min)$	Z_x	Z_y
	Kg/m	Cm^2	mm×mm	mm	mm	mm	cm	cm	cm^4	cm^4	cm^4	cm^4	cm	cm	cm	cm	cm^3	cm^3
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
∠ 80 80 × 6	7.3	9.29	80×80	6.0	8.0		2.18	2.18	56.0	56.0	89.6	22.5	2.46	2.46	3.11	1.56	9.6	9.6
× 8	9.6	12.2		8.0			2.27	2.27	72.5	72.5	116	29.4	2.44	2.44	3.08	1.55	12.6	12.6
×10	11.8	15.0		10.0			2.34	2.34	87.7	87.7	139	36.0	2.41	2.41	3.04	1.55	15.5	15.5
×12	14.0	17.8		12.0			2.42	2.42	102	102	161	42.4	2.39	2.39	3.01	1.54	18.3	18.3
∠ 90 90 × 6	8.2	10.5	90×90	6.0	8.5		2.42	2.42	80.1	80.1	128	32.0	2.77	2.77	3.50	1.75	12.2	12.2
× 8	10.8	13.8		8.0			2.51	2.51	104	104	166	42.0	2.75	2.75	3.47	1.75	16.0	16.0
×10	13.4	17.0		10.0			2.59	2.59	127	127	202	51.6	2.73	2.73	3.44	1.74	19.8	19.8
×12	15.8	20.2		12.0			2.66	2.66	148	148	235	60.9	2.71	2.71	3.41	1.74	23.3	23.3
∠ 100 100 × 6	9.2	11.7	100×100	6.0	8.5		2.67	2.67	111	111	178	44.5	3.09	3.09	3.91	1.95	15.2	15.2
× 8	12.1	15.4		8.0			2.76	2.76	145	145	232	58.4	3.07	3.07	3.88	1.95	20.0	20.0
×10	14.9	19.0		10.0			2.84	2.84	177	177	282	71.8	3.05	3.05	3.85	1.94	24.7	24.7
×12	17.7	22.6		12.0			2.92	2.92	207	207	329	84.7	3.03	3.03	3.82	1.94	29.2	29.2

End connection assumed as fixed (in Roof truss).

$k_1 = 0.20$

$k_2 = 0.35$

$k_3 = 20$

Equivalent Slenderness Ratio

$\lambda_e = \underline{\underline{1.173}}$

Table 12 Constants k_1, k_2 and k_3

Sl No.	No. of Bolts at Each End Connection	Gusset/Connecting Member Fixity [1]	k_1	k_2	k_3
(1)	(2)	(3)	(4)	(5)	(6)
i)	≥ 2	Fixed	0.20	0.35	20
		Hinged	0.70	0.60	5
ii)	1	Fixed	0.75	0.35	20
		Hinged	1.25	0.50	60

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

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}} = \chi f_y / \gamma_{m0} \leq f_y / \gamma_{m0}$$

where

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

λ = non-dimensional effective slenderness ratio

Table 7 Imperfection Factor, α

(*Clauses* 7.1.1 *and* 7.1.2.1)

Buckling Class	a	b	c	d
α	0.21	0.34	0.49	0.76

Table 10 Buckling Class of Cross-Sections

(*Clause* 7.1.2.2)

Cross-Section (1)	Limits (2)	Buckling About Axis (3)	Buckling Class (4)
Rolled I-Sections	$h/b_f > 1.2$: $t_f \le 40$ mm	z-z y-y	a b
	$40 \le$ mm $< t_f \le 100$ mm	z-z y-y	b c
	$h/b_f \le 1.2$: $t_f \le 100$ mm	z-z y-y	b c
	$t_f > 100$ mm	z-z y-y	d d
Welded I-Section	$t_f \le 40$ mm	z-z y-y	b c
	$t_f > 40$ mm	z-z y-y	c d
Hollow Section	Hot rolled	Any	a
	Cold formed	Any	b
Welded Box Section	Generally (except as below)	Any	b
	Thick welds and $h/t_f < 30$ $h/t_w < 30$	 z-z y-y	 c c
Channel, Angle, T and Solid Sections		Any	c

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

α = Imperfection factor (Table 7, Pg 35)

$\alpha = 0.49$ (Buckling class 'c') $\because$ angle section.

$$\phi = 0.5[1 + 0.49(1.173 - 0.2) + 1.173^2]$$
$$= 1.43$$

Now,

Design compressive stress.

$$f_{cd} = \frac{f_y / \gamma_{mo}}{\phi + [\phi^2 - \lambda^2]^{0.5}}$$
$$= \frac{250/1.1}{1.43 + [1.43^2 - 1.173^2]^{0.5}}$$
$$= 101.104 \text{ MPa}.$$

Design capacity $= A_{eff} \cdot f_{cd}$
$= 929 \times 101.104$
$= 93.93$ KN

2) Design of compression member to carry a compression load (factored) of 150 kN. The centre to centre distance between the end connection is 3 m and used M16 bolts of 4.6 class. Design the connection.

Soln :

Assume $\lambda = 100 = \frac{KL}{r}$

Assume angle section.

Using Buckling class c (Table 9c)

$\lambda = \frac{KL}{r} = 100, \quad f_y = 250$

$f_{cd} = 107 \text{ MPa}$

Area of Section $= A_g = \frac{150 \times 10^3}{107} = 1401 \text{ mm}^2$

Provide single angle section

Take $\angle 90 \quad 90 \times 10 \text{ mm} \quad A_g = 1700 \text{ mm}^2$

$r_{min} = r_{vv} = 1.74\,cm = 17.4\,mm$

Assume $l_{eff} = l = 3m$

$$\lambda_{actual} = \frac{3 \times 10^3}{17.4} = 172.4 < 180 \text{ (ok)}$$

Actual capacity = f_{cd} x Area.

for, f_{cd}

$$f_{cd} = \frac{1}{\phi + [\phi^2 - \lambda^2]^{0.5}} \times \frac{f_y}{\gamma_{mo}}$$

$$\lambda = \lambda_e = \sqrt{k_1 + k_2 \lambda_{vv}^2 + k_3 \lambda_\varphi^2}$$

$$\left.\begin{aligned} k_1 &= 0.7 \\ k_2 &= 0.6 \\ k_3 &= 5 \end{aligned}\right\} \geq 2 \text{ bolts \& Hinged.}$$

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\left(3000/17.4\right)}{1\sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}}$$

$\therefore \lambda_{vv} = 1.94$

$$\lambda_\varphi = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{(90+90)/(2\times 10)}{1\sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}}$$

$\therefore \lambda_\varphi = 0.101$

$\lambda = \lambda_e = 1.735$

$$\phi = 0.5\left[1 + \alpha(\lambda - 0.2) + \lambda^2\right]$$
$$= 0.5\left[1 + 0.49(1.735 - 0.2) + 1.735^2\right]$$
$$= 2.381$$

$\therefore f_{cd} = 56.65$ MPa. (UNSAFE)

$P_d = f_{cd} \times \text{Area} = 56.65 \times 1700 = 96.311 < 150$ kN

⇒ We can take two angle sections & connect on same side.

⇒ This gives $P_d = 96.31 \times 2 = 192.62 > 150\,kN$ (safe).

⇒ Not necessary to do another trial taking another angle section.

<u>Next Trial:-</u> Take $\angle 110\ 110 \times 10\ mm$ ($2110\ mm^2$)

Chapter 6

COMPRESSION MEMBER
Numericals

1) Determine the load carrying capacity of a single discontinuous angle 80 x 80 x 6 mm which is used as a compression member in a roof truss if it connected to a gusset plate by 2 bolts. The centre to centre distance between the end connection is 2 m.

7.5.1.2 *Loaded through one leg*

The flexural torsional buckling strength of single angle loaded in compression through one of its legs may be evaluated using the equivalent slenderness ratio, λ_e as given below:

$$\lambda_e = \sqrt{k_1 + k_2 \lambda_{vv}^2 + k_3 \lambda_\varphi^2}$$

where

k_1, k_2, k_3 = constants depending upon the end condition, as given in Table 12,

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 \varepsilon}{250}}} \text{ and } \lambda\varphi = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 \varepsilon}{250}}}$$

where

l = centre-to-centre length of the supporting member,

r_{vv} = radius of gyration about the minor axis,

b_1, b_2 = width of the two legs of the angle,

t = thickness of the leg, and

ε = yield stress ratio $(250/f_y)^{0.5}$.

Table 12 Constants k_1, k_2 and k_3

Sl No.	No. of Bolts at Each End Connection	Gusset/Connecting Member Fixity [1]	k_1	k_2	k_3
(1)	(2)	(3)	(4)	(5)	(6)
i)	≥ 2	Fixed	0.20	0.35	20
		Hinged	0.70	0.60	5
ii)	1	Fixed	0.75	0.35	20
		Hinged	1.25	0.50	60

Correction in IS 800:2007 Clause 7.5.1.2 ***Loaded through one leg***

Page 48

$$\lambda_e = \sqrt{k_1 + k_2 \lambda_{vv}^2 + k_3 \lambda_\phi^2}$$

where k_1, k_2, k_3 = constants depending upon the end condition

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} \quad \text{and} \quad \lambda_\phi = \frac{(b_1 + b_2)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}} \times 2t}$$

Soln :-

For ISA 80 x 80 X 6 mm

Area $A_e = 929\ mm^2$

$r_{min} = r_{vv} = 15.6\ mm$.

Cl 7.5.1.2 . Pg 48

$$\lambda_e = \sqrt{k_1 + k_2 \lambda_{vv}^2 + k_3 \lambda_\psi^2}$$

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon \sqrt{\frac{\pi^2 E}{250}}} = \frac{\left(\frac{2000}{15.6}\right)}{1 \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}} = \underline{1.44}$$

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

$$\lambda_\psi = \frac{(b_1 + b_2)/2t}{\varepsilon \sqrt{\frac{\pi^2 E}{250}}} = \frac{(80+80)/(2 \times 6)}{1 \sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}} = 0.15$$

SECTION 5 EQUAL LEG ANGLES

Table 5.1 Nominal Dimensions, Mass and Sectional Properties of Indian Standard Equal Leg Angles

Designation	Mass M	Sectional area, a	Dimensions				Sectional Properties											
			A×B	t	R_1	R_2	C_x	C_y	I_x	I_y	$I_u(Max)$	$I_v(Min)$	r_x	r_y	$r_u(Max)$	$r_v(Min)$	Z_x	Z_y
	Kg/m	Cm^2	mm×mm	mm	mm	mm	cm	cm	cm^4	cm^4	cm^4	cm^4	cm	cm	cm	cm	cm^3	cm^3
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
∠ 80 80 × 6	7.3	9.29	80×80	6.0	8.0		2.18	2.18	56.0	56.0	89.6	22.5	2.46	2.46	3.11	1.56	9.6	9.6
× 8	9.6	12.2		8.0			2.27	2.27	72.5	72.5	116	29.4	2.44	2.44	3.08	1.55	12.6	12.6
×10	11.8	15.0		10.0			2.34	2.34	87.7	87.7	139	36.0	2.41	2.41	3.04	1.55	15.5	15.5
×12	14.0	17.8		12.0			2.42	2.42	102	102	161	42.4	2.39	2.39	3.01	1.54	18.3	18.3
∠ 90 90 × 6	8.2	10.5	90×90	6.0	8.5		2.42	2.42	80.1	80.1	128	32.0	2.77	2.77	3.50	1.75	12.2	12.2
× 8	10.8	13.8		8.0			2.51	2.51	104	104	166	42.0	2.75	2.75	3.47	1.75	16.0	16.0
×10	13.4	17.0		10.0			2.59	2.59	127	127	202	51.6	2.73	2.73	3.44	1.74	19.8	19.8
×12	15.8	20.2		12.0			2.66	2.66	148	148	235	60.9	2.71	2.71	3.41	1.74	23.3	23.3
∠ 100 100 × 6	9.2	11.7	100×100	6.0	8.5		2.67	2.67	111	111	178	44.5	3.09	3.09	3.91	1.95	15.2	15.2
× 8	12.1	15.4		8.0			2.76	2.76	145	145	232	58.4	3.07	3.07	3.88	1.95	20.0	20.0
×10	14.9	19.0		10.0			2.84	2.84	177	177	282	71.8	3.05	3.05	3.85	1.94	24.7	24.7
×12	17.7	22.6		12.0			2.92	2.92	207	207	329	84.7	3.03	3.03	3.82	1.94	29.2	29.2

End connection assumed as fixed (in Roof truss).

$k_1 = 0.20$

$k_2 = 0.35$

$k_3 = 20$

Equivalent Slenderness Ratio

$\lambda_e = 1.173$

Table 12 Constants k_1, k_2 and k_3

Sl No.	No. of Bolts at Each End Connection	Gusset/Connecting Member Fixity [1]	k_1	k_2	k_3
(1)	(2)	(3)	(4)	(5)	(6)
i)	≥ 2	Fixed	0.20	0.35	20
		Hinged	0.70	0.60	5
ii)	1	Fixed	0.75	0.35	20
		Hinged	1.25	0.50	60

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

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}} = \chi f_y / \gamma_{m0} \leq f_y / \gamma_{m0}$$

where

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

λ = non-dimensional effective slenderness ratio

Table 7 Imperfection Factor, α

(*Clauses* 7.1.1 *and* 7.1.2.1)

Buckling Class	a	b	c	d
α	0.21	0.34	0.49	0.76

Table 10 Buckling Class of Cross-Sections

(*Clause* 7.1.2.2)

Cross-Section (1)	Limits (2)	Buckling About Axis (3)	Buckling Class (4)
Rolled I-Sections	$h/b_f > 1.2$: $t_f \le 40$ mm	z-z y-y	a b
	$40 \le$ mm $< t_f \le 100$ mm	z-z y-y	b c
	$h/b_f \le 1.2$: $t_f \le 100$ mm	z-z y-y	b c
	$t_f > 100$ mm	z-z y-y	d d
Welded I-Section	$t_f \le 40$ mm	z-z y-y	b c
	$t_f > 40$ mm	z-z y-y	c d
Hollow Section	Hot rolled	Any	a
	Cold formed	Any	b
Welded Box Section	Generally (except as below)	Any	b
	Thick welds and $h/t_f < 30$ $h/t_w < 30$	 z-z y-y	 c c
Channel, Angle, T and Solid Sections		Any	c

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

α = Imperfection factor (Table 7, Pg 35)

$\alpha = 0.49$ (Buckling class 'c')

$\because$ angle section.

$$\phi = 0.5[1 + 0.49(1.173 - 0.2) + 1.173^2]$$
$$= 1.43$$

Now,

Design compressive stress.

$$f_{cd} = \frac{f_y / \gamma_{mo}}{\phi + [\phi^2 - \lambda^2]^{0.5}}$$
$$= \frac{250/1.1}{1.43 + [1.43^2 - 1.173^2]^{0.5}}$$
$$= 101.104 \text{ MPa}.$$

Design capacity $= A_e \cdot f_{cd}$

$= 929 \times 101.104$

$= 93.93$ KN

2) Design of compression member to carry a compression load (factored) of 150 kN. The centre to centre distance between the end connection is 3 m and used M16 bolts of 4.6 class. Design the connection.

Soln :

Assume $\lambda = 100 = \frac{KL}{r}$

Assume angle section.

Using Buckling class C (Table 9c)

$\lambda = \frac{KL}{r} = 100$, $f_y = 250$

$f_{cd} = 107$ MPa

Area of Section $= A_g = \frac{150 \times 10^3}{107} = 1401 \text{ mm}^2$

Provide single angle section

Take $\angle 90$ 90×10 mm $A_g = 1700 \text{ mm}^2$

Table 10 Buckling Class of Cross-Sections

(*Clause* 7.1.2.2)

Cross-Section (1)	Limits (2)	Buckling About Axis (3)	Buckling Class (4)
Rolled I-Sections	$h/b_f > 1.2$: $t_f \le 40$ mm	z-z y-y	a b
	$40 \le$ mm $< t_f \le 100$ mm	z-z y-y	b c
	$h/b_f \le 1.2$: $t_f \le 100$ mm	z-z y-y	b c
	$t_f > 100$ mm	z-z y-y	d d
Welded I-Section	$t_f \le 40$ mm	z-z y-y	b c
	$t_f > 40$ mm	z-z y-y	c d
Hollow Section	Hot rolled	Any	a
	Cold formed	Any	b
Welded Box Section	Generally (except as below)	Any	b
	Thick welds and $h/t_f < 30$ $h/t_w < 30$	 z-z y-y	 c c
Channel, Angle, T and Solid Sections		Any	c

$r_{min} = r_{vv} = 1.74\,cm = 17.4\,mm$

Assume $l_{eff} = l = 3m$

$$\lambda_{actual} = \frac{3 \times 10^3}{17.4} = 172.4 < 180 \text{ (ok)}$$

Actual capacity = f_{cd} x Area.

for, f_{cd}

$$f_{cd} = \frac{1}{\phi + [\phi^2 - \lambda^2]^{0.5}} \times \frac{f_y}{\gamma_{mo}}$$

$$\lambda = \lambda_e = \sqrt{k_1 + k_2 \lambda_{vv}^2 + k_3 \lambda_\phi^2}$$

$$\left.\begin{aligned} k_1 &= 0.7 \\ k_2 &= 0.6 \\ k_3 &= 5 \end{aligned}\right\} \geq 2 \text{ bolts \& Hinged.}$$

$$\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{\left(3000/17.4\right)}{1\sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}}$$

$\therefore \lambda_{vv} = 1.94$

$$\lambda_\varphi = \frac{(b_1 + b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}} = \frac{(90+90)/(2\times 10)}{1\sqrt{\frac{\pi^2 \times 2 \times 10^5}{250}}}$$

$\therefore \lambda_\varphi = 0.101$

$\lambda = \lambda_e = 1.735$

$$\phi = 0.5\left[1 + \alpha(\lambda - 0.2) + \lambda^2\right]$$
$$= 0.5\left[1 + 0.49(1.735 - 0.2) + 1.735^2\right]$$
$$= 2.381$$

$\therefore f_{cd} = 56.65$ MPa. (UNSAFE)

$P_d = f_{cd} \times \text{Area} = 56.65 \times 1700 = 96.311 < 150$ kN

⇒ We can take two angle sections & connect on same side.

⇒ This gives $P_d = 96.31 \times 2 = 192.62 > 150\,kN$ (safe).

⇒ Not necessary to do another trial taking another angle section.

<u>Next Trial:-</u> Take $\angle 110\ 110 \times 10\ mm$ ($2110\ mm^2$)

Q4) Calculate the strength of strut 3m long & consists of 2 unequal angles 150×115×8 mm with long leg connected to both sides of 10 mm thick gusset plate by two bolts.

V.S. Sairam
Example 4.3
Page 89

Pg 48 Cl 7.5.2.1.

7.5.2 *Double Angle Struts*

7.5.2.1 For double angle discontinuous struts, connected back to back, on opposite sides of the gusset or a section, by not less than two bolts or rivets in line along the angles at each end, or by the equivalent in welding, the load may be regarded as applied axially. The effective length, *KL*, in the plane of end gusset shall be taken as between 0.7 and 0.85 times the distance between intersections, depending on the degree of the restraint provided. The effective length, *KL*, in the plane perpendicular to that of the end gusset, shall be taken as equal to the distance between centres of intersections. The calculated average compressive

$KL_x = (0.7 \text{ to } 0.85)L$

$KL_y = L$

$KL_x = (0.7 \text{ to } 0.85)\,L = 0.7 \times 3000$

(plane of gusset plate) $= 2100\ mm$

$KL_y = 1 \times L = 3000\ mm$

(perpendicular to gusset plate)

From code,

$I_y = 590 \times 10^4\ mm^4$ $\quad C_y = 27.6\ mm$

$r_x = 47.8\ mm$ $\quad A = 2070\ mm^2$

$r_y = 34.3\ mm$

$$\lambda_x = \frac{kl_x}{r_{x\,min}} \quad ; \quad \lambda_y = \frac{kl_y}{r_{y\,min}}$$

$$r_{x\,min} = \sqrt{\frac{I_x}{A_{total}}} = \sqrt{\frac{2\,I_x}{2A}} = \sqrt{\frac{I_x}{A}} = r_x$$

$$= 47.8\,mm$$

$$r_{y\,min} = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{2\,I_y}{2A}}$$

$$= \sqrt{\frac{I_y + A\,(c_y + t/2)^2}{A}}$$

$$= \sqrt{\frac{590 \times 10^4 + 2070\,(27.6 + 10/2)^2}{2070}}$$

$$= 62.55\,mm$$

$$\lambda_x = \frac{2100}{47.8} = 43.93$$

$$\lambda_y = \frac{kl_y}{r_{y\,min}} = \frac{3000}{62.55} = 47.96$$

Use Table 9c

for f_{cdx}, $\frac{KL}{r} = 43.93$, $f_y = 250$.

40	198
50	183

$\therefore f_{cdx} = 192.105$ MPa

$\therefore f_{cdy} = 186.06$ MPa.

Design capacity, $P_{dx} = 2(2070 \times 192.105) = 795.31$ kN

$P_{dy} = 2(2070 \times 186.06) = 770$ kN.

$\therefore$ Capacity of Section $= 770$ kN.

NOTE :- $\lambda \uparrow$ $f_{cd} \downarrow$

Example 6.4: Design a column 3.5 m long in a building, restrained in direction and position both the ends. The column subjected to a factored load of 550 KN. Use steel of grade Fe 410.

Solution:

For the steel of grade Fe 410, $f_y = 250$ MPa

The column ends are restrained in direction and position; K= 0.65

Let us provide on I-section column. From IS code 800-2007, Table 10, the design buckling curve about Z-Z axis is 'a' and that about Y-Y axis is 'b'.

Let us assume the effective slenderness ratio to be 90.

For I-section $r_y < r_z$ so the column will buckle about Y-Y axis and buckling class b is will be used for the design.

From table 9(b) of code: The design compressive stress corresponding the effective slenderness ratio 90 is 134 MPa

Now,

$$A = \frac{550 \times 10^3}{134} = 4104 \text{ mm}^2$$

Let us try ISHB 150 @ 34.6 kg/m with the following properties:

$h = 150$ mm

$b_f = 150$ mm

$t_f = 9$ mm

$t_w = 11.8$ mm

$A = 4410$ mm²

$r_z = 60.9$ mm

$r_y = 33.5$ mm

Boundary Conditions				*Schematic Representation*	*Effective Length (KL)*
At One End		*At the Other End*			
Translation	*Rotation*	*Translation*	*Rotation*		
Restrained	Restrained	Free	Free		2.0*L*
Free	Restrained	Restrained	Free		2.0*L*
Restrained	Free	Restrained	Free		1.0*L*
Restrained	Restrained	Free	Restrained		1.2*L*
Restrained	Restrained	Restrained	Free		0.8*L*
Restrained	Restrained	Restrained	Restrained		0.65*L*

Table 10 Buckling Class of Cross-Sections

(*Clause* 7.1.2.2)

Cross-Section (1)	Limits (2)	Buckling About Axis (3)	Buckling Class (4)
Rolled I-Sections 	$h/b_f > 1.2$: $t_f \leq 40$ mm	z-z y-y	a b
	$40 \leq$ mm $< t_f \leq 100$ mm	z-z y-y	b c
	$h/b_f \leq 1.2$: $t_f \leq 100$ mm	z-z y-y	b c
	$t_f > 100$ mm	z-z y-y	d d
Welded I-Section 	$t_f \leq 40$ mm	z-z y-y	b c

Design a column to support a factored load of 1050 kN . The column section taken as I section and the effective length in the direction of X is 7 m and in direction of Y is 5 m .

Soln :-

Design of I - section .

Assume $\lambda = 100$.

Assuming buckling class in (from Table 10)

Y - dir^n → class b

X - dir^n → class a

Using Table 9(b) $f_{cdy} = 118$ MPa (for $\lambda = 100$ & $f_y = 250$ MPa)

Using Table 9a

$f_{cdx} = 132$ MPa

$$\text{Area} = \frac{1050 \times 10^3}{118} = 8898.30 \text{ mm}^2$$

Take SC 220, Area = 8980 mm²

$r_x = 9.35\text{cm} = 93.5$ mm

$r_y = 49.0$ mm

Buckling class classification (Table 10)

$$\frac{h}{b_f} = \frac{220}{220} = 1 < 1.2$$

$t_f = 16 \text{ mm} < 100 \text{ mm}$

X - X axis → Class b

Y - Y axis → Class c.

Actual slenderness ratio

$$\lambda x = \frac{KLx}{r_x} = \frac{7 \times 10^3}{93.5} = 74.86 < \underline{180} \text{ (ok)}$$

Table 3
Page 20.

$$\lambda y = \frac{KLy}{r_y} = \frac{5000}{49} = 102.04 < 180 \text{ (ok)}$$

Actual compressive stress (fcd)

In x-dir^n (fcd)

$\lambda x = 74.86$, $fy = 250$

$fcdx = 158$ mPa

In Y-dir^n.

$\lambda_y = 102$, $f_y = 250$

$f_{cdy} = 104.52$ MPa.

Capacity :-

X-dir^n = fcd x Area

= 158 x 8980

= 1418.84 kN > 1050 kN (safe)

Y-dir^n = 104.52 x 8980

= 938.59 kN < 1050 (unsafe)

Since, column is unsafe in Y-dir^n take another I-section SC 250 & repeat the process.

Design of Built-up column

- The size and shape of standard rolled steel sections are limited
- When rolled sections do not furnish required sectional area or when a special shape or large radius of gyration is required in two different directions, a built-up section is fabricated.
- For economical design of heavily loaded long columns, the least radius of gyration of column is increased to maximum (ry>rz)
- To achieve this, the rolled sections are kept away from centroidal axis of column and are connected by lattice system.

LACING

- Lacing bars are not load carrying elements. Their function is primarily to hold the main component members (different sections) of the column in their relative position and equalize the stress distribution in them.
- The connection is designed to transfer only the axial forces in the lacing bars to the main components and hence it is not moment resistant
- Lacing is designed for shear force (V) in columns
- The shear force may be due to incidental eccentricity of the applied axial load, moments or lateral forces acting on the columns and erection stresses

(a) Channels face-to-face connected by single lacing

(b) Channels back-to-back connected by single lacing

(c) Channels back-to-back connected by battens

(d) Angles connected by single lacing (shown on two faces only)

(a) Failure of column as a whole (b) Failure of main component (c) Failure of lacing

- The sections used to form the column act as a single section, withstanding buckling with respect to the axis about which the sections were spaced.

(a) Single flat lacing non-intersecting

(b) Single flat lacing inter-secting

(c) Single flat lacing, non-mirror image (not recommended)

(d) Double flat lacing intersecting

Single

Double lacing

forces in lacing.

Design steps of lacing:

1. The design compressive stress is assumed. For steel with fy = 250 MPa a trial value in the range of 125–175 MPa may be used.
2. Approx. cross-sectional area to carry the factored load Pu at the assumed compressive stress is computed.

$$A = \frac{P_u}{\text{assumed compressive stress}}$$

3. Take a trial section of area > approx. area

   For 2 channels: Ag = approx. area/2

   For 4 angles : Ag = approx. area/4
4. The sections are so spaced that the radius of gyration of the section about the axis perpendicular to the plane of lacing is not less than the radius of gyration about the axis in the plane of lacing. This is achieved by making the radius of gyration about the yy-axis equal to or greater than that about the zz-axis i.e. ry ≥ rz.

5. The effective slenderness ratio, (KL/r)e, of the laced column should be taken as 1.05 times the (KL/r), the actual maximum slenderness ratio.
6. Design compressive stress fcd is computed
7. Calculate load carrying capacity Pd
8. Assume angle of inclination of lacing. Angle of inclination of the lacing bar with the longitudinal axis of the component member should be kept between 40º–70º.
9. Calculate spacing of lacing bar
10. Max spacing of lacing bar should be such that the minimum slenderness ratio of the component member (a/ry) is not greater than 50 (to prevent local buckling) or 0.7 times the slenderness ratio of the member as a whole, where a is the length of component member and ry is the radius of gyration about yy-axis of the component member

Force in lacing and connector

11. Calculate length of lacing bar and spacing of lacing bar
12. The lacing for compression members should be proportioned to resist a total transverse shear Vt equal to 2.5% of the axial force in the column. This shear Vt is divided equally in all parallel planes N in which there is shear resisting elements, such as lacing or continuous plates. Hence Vt /N is the transverse force to which the lacing is subjected
13. The compressive force in the lacing bar is computed, which is equal to (Vt /N) cosecθ for single lacing system and (Vt /2N) cosecθ , for a double lacing system. Take N=2
14. Width of lacing bar should not be less than 3 times nominal diameter of end connector

15) Thickness of lacing bar

$$t \not< \frac{l_e}{40} \quad \text{for single lacing}$$

$$t \not< \frac{l_e}{60} \quad \text{for double lacing}$$

16) The min. radius of gyration of lacing bar

$$r_{min} = \sqrt{\frac{I}{A}} = \frac{t}{\sqrt{12}}$$

17) The slenderness ratio of lacing bar should not be greater than 145

$$\frac{KL}{r_{min}} < 145 \qquad K = 0.7$$

---

18) Calculate design comp. stress in lacing bar by using Table 9 C.

19) $P_{lacing} = A_{lacing} \times f_{cd,lacing} > \text{Load on lacing}$

Design a built up column 10 m long. It carry an axial load of 1200 KN and composed of 2 channels placed back to back. Design for lacing (single) system. Design end connection of lacing bar using site weld.

Soln:-

Design load $= 1.5 \times 1200 = 1800$ kN.

$L_{eff} = KL = 10$ m.

Assume, $f_{cd} = 150$ MPa.

$$A_{approx} = \frac{P_u}{f_{cd}} = \frac{1800}{150} = 12000 \text{ mm}^2$$

$$A_{req} = \frac{12000}{2} = 6000 \text{ mm}^2$$

Take ISMC 400 @ 50.1 kg/m.

$A = 6380$ mm^2

$I_x = 15200 \times 10^4$ mm^4

$I_y = 508 \times 10^4$ mm^4

$C_y = 24.2$ mm

$r_x = 154$ mm

$r_y = 28.2$ mm

Actual slenderness ratio,

$$\lambda_{actual} = \frac{L_{eff}}{r_{min}} = \frac{L_{eff}}{r_x} = \frac{10 \times 10^3}{154} = 64.94$$

$$\lambda_e = 1.05\, \lambda_{actual} = 68.18$$

Design compressive stress (f_{cd})

For $\lambda_e = 68.18$, buckling class c, $f_y = 250$

$\therefore f_{cd} = 154.91$ MPa

Design capacity $P_d = 2 \times 6380 \times 154.91 \times 10^{-3}$

$= 1976.65$ kN > 1800 kN (OK)

Design of Spacing (d)

$$r_y \geq r_x$$

or, $\sqrt{\frac{I_y}{A}} \geq \sqrt{\frac{I_x}{A}}$ or, $I_y \geq I_x$

For critical value,

$$I_y = I_x$$

$$2\left[I_y + A\left(c_y + \frac{d}{2}\right)^2\right] = 2I_x$$

or, $508 \times 10^4 + 6380\left(24.2 + \frac{d}{2}\right)^2 = 15200 \times 10^4$

$\therefore d = 255.1$ mm

Provide $d = 260$ mm

Design of lacing

Angle of inclination $\theta = 45°$

Centroidal distance between two channels.

$= d + C_y + C_y$

$= 260 + 24.2 + 24.2$

$= 308.4$ mm

Provide 310 mm.

Length of lacing bar

$$\sin 45° = \frac{310}{\text{length of lacing bar}}$$

$\therefore$ length of lacing bar $= 438.4$ mm

Provide 440mm

Spacing of lacing bar = $2 \times 310 = 620$ mm.

Check for spacing

$$\frac{620}{r_y} = \frac{620}{28.2} = 21.98 < 50$$

$$0.7\lambda_e = 0.7 \times 68.18 = 47.726 > 21.98$$

(Ok)

Width of lacing

Take $b = 50$ mm (50 to 70 mm)

Thickness of lacing bar

$$t \nless \frac{l_e}{40} \nless \frac{440}{40} \nless 11$$

Provide $t = 12$ mm

Forces in lacing bar

Transverse shear $V_t = 2.5\%$ of P_u

$$= \frac{2.5}{100} \times 1800 = 45 \text{ kN}$$

Compressive force in lacing bar

$$V_{c_1} = \frac{V_t}{N} \operatorname{cosec} \theta = \frac{45}{2} \operatorname{cosec} 45^\circ = 31.82 \text{ kN}$$

Slenderness ratio of lacing bar

$$\lambda = \frac{l_e}{r_{min}} = \frac{440 \times 0.7}{3.464} = 88.9 < 145$$

OK

Table 10 Buckling Class of Cross-Sections

(*Clause* 7.1.2.2)

Cross-Section (1)	Limits (2)	Buckling About Axis (3)	Buckling Class (4)
Rolled I-Sections	$h/b_f > 1.2$: $t_f \le 40$ mm	z-z y-y	a b
	$40 \le$ mm $< t_f \le 100$ mm	z-z y-y	b c
	$h/b_f \le 1.2$: $t_f \le 100$ mm	z-z y-y	b c
	$t_f > 100$ mm	z-z y-y	d d
Welded I-Section	$t_f \le 40$ mm	z-z y-y	b c
	$t_f > 40$ mm	z-z y-y	c d
Hollow Section	Hot rolled	Any	a
	Cold formed	Any	b
Welded Box Section	Generally (except as below)	Any	b
	Thick welds and $h/t_f < 30$ $h/t_w < 30$	 z-z y-y	 c c
Channel, Angle, T and Solid Sections		Any	c

$$r_{min} = \frac{t}{\sqrt{12}} = \frac{12}{\sqrt{12}} = 3.464'$$

<u>For fcd</u>,

for $\lambda = 88.9$, $f_y = 250$ MPa, Table 9C.

$f_{cd,\,lacing} = 121$ MPa.

Capacity of lacing bar $= 121 \times (12 \times 50)$

$= \cancel{80.75\ MPa}\ 72.6$ ~~MPa~~ KN

> 31.82 KN

(OK).

<u>Design of tie plate</u> → Refer design of batten.

Design of end connection

Strength of weld

$t_{bracing} = 12mm$

$t_f = 15.3mm$

Size of weld.

$S_{min} = 5mm$

$S_{max} = 12 - 1.5 = 10.5mm$

Provide $S = 8mm$

$t_e = K.S = 0.7 \times 8 = 5.6mm$

$$P_d = t_e \cdot l_w \cdot \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

$$31.82 = 5.6 \times l_w \times \frac{410}{\sqrt{3} \times 1.25}$$

$$l_w = 36.006mm + 2 \times 8$$

~~≈ 40mm~~

$\therefore$ length of both $= 53mm$

COMPRESSION MEMBER

BATTENS

Channels back-to-back connected by battens

- Battens are plates or any other rolled sections used to connect the main components of compression members.
- Batten should be placed opposite to each other on the two parallel faces of the compression member and should be spaced and proportioned uniformly throughout.

DESIGN STEPS

STEPS 1 to 4 : Same as Lacing

5. Effective slenderness ratio = 1.1 x Actual slenderness ratio
6. Calculate fcd; Pd = A x fcd
7. Battens will be designed to a total shear force 2.5 % of total axial load
8. Atleast 3 battens should be provided within the actual length of column from centre to centre of end connection
9. Effective depth of end batten shall not be less than the centroidal distance between main members of built-up column. Effective depth of intermediate batten is taken as $3/4^{th}$ of depth of end batten
10. Thickness of batten shall not be less than $1/50^{th}$ distance between innermost connecting lines of bolts and welds

11. Overlap length of batten shall not be less than 4 x Thickness of batten
12. The maximum spacing of the battens C should be such that the minimum slenderness ratio of the component member C/r_y is not greater than 50 or 0.7 times the slenderness ratio of the member as a whole, about an axis parallel to battens
13. Battens should be able to resist the longitudinal shear and moment arising from the transverse shear V_t

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

(a)

(b)

$$M = \frac{V}{N} \cdot \frac{C}{2}$$

(c)

$$V_b \cdot S = \frac{V}{N} \cdot C$$

$$\Rightarrow V_b = \frac{VC}{NS}$$

(d)

14. Check for shear stress and bending stress of batten plates:

Shear stress = (V_b /A)= V_b /(t x d) < $(f_y/\sqrt{3}\,\gamma_{m0})$

Bending stress:

$$\sigma_{bc,\,cal} = \frac{M}{Z} = \frac{M}{\frac{1}{6}td^2} = \frac{6M}{td^2} < \frac{f_y}{\gamma_{m0}}$$

Design a built-up column of effective length of 6 m to carry an axial load of 1000 KN using two channels and battens. Design connection using weld.

$L_{eff} = KL = 6m.$

Axial load $= 1000\,kN$

Factored load $P_u = 1.5 \times 1000 = 1500\,kN$

Assume permissible stress $f_{cd} = 150\,MPa$.

$$A_{approx} = \frac{1500 \times 10^3}{150} = 10{,}000\,mm^2$$

Area of each channel $A = 5000\,mm^2$

Take ISMC ~~350~~ 400

Area $= 6380\,mm^2$

$I_x = 15200 \times 10^4\,mm^4$

$I_y = 508 \times 10^4\,mm^4$

$r_x = 154\,mm$

$r_y = 28.2\,mm$

$C_y = 24.2\,mm$

$b_f = 100\,mm$

Slenderness ratio (λ_{actual}) $= \frac{KL}{r_{min}} = \frac{6 \times 10^3}{154}$

$= 38.96$

Effective $\lambda_{eff} = 1.1 \times 38.96$

$= 42.86$

Using Table 9C, $f_{cd} = 193.71$ MPa.

Design capacity $= 193.71 \times 2 \times 6380$

$= 2471.74$ kN > 1500 kN (ok).

$\Rightarrow$ for design of spacing $r_y \geq r_x$

$r_{min} = r_x = 154$ mm

Design of spacing

$$r_y \geq r_x$$

$$I_{YY} \geq I_{XX}$$

Critical condition, $I_{YY} = I_{XX}$

$$2\left[I_y + A\left(b_f - C_y + \frac{d}{2}\right)^2\right] = 2 I_x$$

$$\left[508\times10^4 + 6380\left(100 - 24.2 + \frac{d}{2}\right)^2\right] = 15200\times10^4$$

$$\therefore d = 151.9 \text{ mm}$$

Provide 200 mm spacing.

Design of Batten

End Batten :-

Min depth (width) of end batten

= Centroidal distance betn 2 channels

$= 200 + 2(b_f - c_y)$

$= 200 + 2(100 - 24.2)$

$= 351.6$ mm

Provide depth $= 360$ mm

Depth of intermediate batten $= \frac{3}{4} \times 360$

$= 270\,mm$

Thickness of batten $\nless \frac{351.6}{50} = 7.032\,mm$

Provide 8 mm thick batten plate.

Overlap $\nless 4 \times$ Thickness of batten

$\nless 4 \times 8$

$\nless 32\,mm$

Provide overlap 40 mm

Length of batten $= 200 + 40 + 40$

$= 280\,mm$

Provide $280 \times 360 \times 8\,mm$ end batten

and $280 \times 270 \times 8\,mm$ intermediate batten

Spacing of Batten (c)

$$\frac{c}{r_y} < 50 \text{ or } 0.7\lambda e$$

$$\frac{c}{28.2} < 50 \text{ or } 0.7 \times 42.86$$

$$< 50 \text{ or } 30.002.$$

$$c < 28.2 \times 30.002 = 846.06 \text{ mm}$$

Provide $c = 800$ mm.

Design strength of Batten

Batten is designed for bending moment & shear force developed due to transverse shear force.

$$V_t = 2.5\% \text{ of } 1500 = 37.5 \text{ kN}.$$

Shear force in batten $V_b = \dfrac{V_t \cdot C}{NS}$

$$S = 200 + \frac{40}{2} + \frac{40}{2} = 240 \text{ mm}$$

$$\therefore V_b = \frac{37.5 \times 800}{2 \times 240} = 62.5 \text{ kN}$$

Moment $M_b = \dfrac{V_t \cdot C}{2N} = \dfrac{37.5 \times (800 \times 10^{-3})}{2 \times 2}$

$$= 7.5 \text{ kNm}$$

[S – distance betn CG of weld]

Check :-

End batten :-

Shear stress $= \frac{V_b}{A} = \frac{62.5 \times 10^3}{8 \times 360} = 21.7$ MPa

$< \frac{250}{\sqrt{3} \times 1.10}$

< 131.2 MPa (ok)

Bending stress $= \frac{6M}{td^2}$

$= \frac{6 \times 7.5 \times 10^6}{8 \times 360^2}$

$= 43.4$ MPa $< \frac{250}{1.1}$

< 227.27 MPa

(ok)

Intermediate batten:-

Shear stress $= \frac{62.5 \times 10^3}{8 \times 270} = 28.94$ MPa (ok)

Bending stress $= \frac{6 \times 7.5 \times 10^6}{8 \times 270^2} = 77.16$ MPa (ok).

Design of connection

Eccentric welded connection in the plane of column.

fig:- End batten connection

$$x_c = \frac{\Sigma L_i x_i}{\Sigma L_i}$$

$$= \frac{40 \times 20 + 360 \times 0 + 40 \times 20}{40 + 360 + 40}$$

$$= 3.64 \text{ mm}.$$

$I_x =$

$I_y =$

$I_p = I_x + I_y$

$$q_x = \frac{P.e.y}{I_p}$$

$$q_y = \frac{P.e.x}{I_p} + \frac{P}{l_w . t_w}$$

$$q = \sqrt{q_x^2 + q_y^2} < \frac{f_y}{\sqrt{3}\ \gamma_{mw}}$$

$t_w = 0.7\,s.$

here,

$$\boxed{\begin{aligned} P &= V_b \\ P.e &= M_b \end{aligned}}$$

Column bases

Column Base

- A column base consists of horizontal steel plate which is provided at the bottom most end of the column and placed on a concrete pedestal.
- The column base ensures that the bearing pressure between the column end and the column pedestal is within the permissible limit so that no crushing of concrete occurs.
- The base plate is usually butt welded to the column and is connected to the concrete pedestal using anchor bolts.

Column
Cleat angle
Foundation bolts
Base plate
Concrete pedestal

Column Base

- These bases may be used for axially loaded columns and beam-columns.
- Slab base/ column base is designed to resist the load of a column on a greater area so that the bearing pressure on concrete or masonry on which column stand is not exceeded.
- For axially loaded column supported on symmetrical base plate, the intensity of pressure between same plate and concrete base will be assumed uniform.

Column Base

- Steel columns are normally supported over concrete blocks.
- However, when the loads supported by these columns are large and the bearing pressure of concrete from below is insufficient to resist the loads, they may fail.
- Therefore, it is a normal practice to distribute the column loads to steel base plates which are placed over these concrete blocks.
- In addition to transferring safely the column loads, a base plate also maintains alignment of the column in plan, verticality of the column and controls the column and frame deflections.

Column Base

- Column bases are primarily subjected to bearing pressure from below, and bending moment and shear force.
- The main problems in the design of the column bases are to determine the size of the plates and their thicknesses.
- The size is determined by the required bearing area on foundation; and the required thickness is determined so that bending strength of the plate is not exceeded.

Fig. 11.2 Assumed pressure distribution under base plate

Fig. 11.3 Slab base

(a) Column subjected to axial load

(b) Column subjected to axial load and bending moment

$$M = \frac{wa^2}{2} = w \times 1 \times a \times \frac{a}{2}$$

$$M = \frac{wb^2}{2}$$

$$M_{net,x} = \frac{wa^2}{2} - \nu\frac{wb^2}{2}$$

$$= \frac{wa^2}{2} - 0.3\frac{wb^2}{2}$$

$$= \frac{w}{2}(a^2 - 0.3b^2)$$

As the slab tends to bend simultaneously about two principal axes of slab, the stress caused by bending about one axis is influenced by the stress due to bending about other axis. Poison's ratio of 0.3 is used to account for this effect.

From flexure capacity of member, Pg 53
Cl 8.2.1.2

$$\boxed{M_v = 1.2\, f_y \frac{Z_e}{\gamma_{m0}}}$$

For safety, $M_{net} = M_v$

or, $$\frac{\omega}{2}(a^2 - 0.3b^2) = 1.2 f_y \frac{Z_e}{\gamma_{m0}}$$

or, $$\frac{\omega}{2}(a^2 - 0.3b^2) = 1.2 \frac{f_y}{\gamma_{m0}} \cdot \frac{t^2 \times 1}{6}$$

or, $$t = \sqrt{2.5\,\omega(a^2 - 0.3b^2)\frac{\gamma_{m0}}{f_y}} > t_f$$

7.4.3 *Slab Bases*

Columns with slab bases need not be provided with gussets, but sufficient fastenings shall be provided to retain the parts securely in place and to resist all moments and forces, other than direct compression, including those arising during transit, unloading and erection.

7.4.3.1 The minimum thickness, t_s, of rectangular slab bases, supporting columns under axial compression shall be

$$t_s = \sqrt{2.5\,w\,(a^2 - 0.3b^2)\gamma_{m0} / f_y} \quad > t_f$$

where

w = uniform pressure from below on the slab base under the factored load axial compression;

a, b = larger and smaller projection, respectively of the slab base beyond the rectangle circumscribing the column; and

t_f = flange thickness of compression member.

1. Design the base plate for a column section ISHB 300 which carries a factored axial compression of 1500 KN. Grade of steel is E250 and grade of concrete is M20.

Soln :-

(0.45 fck)

Bearing pressure of concrete $= 0.6\, f_{ck}$ (cl. 7.4.1)

$= 0.6 \times 20$

$= 12\, MPa$

Factored load $= 1500\, kN$

$$\text{Area required} = \frac{P_u}{\text{Bearing pressure}}$$

$$= \frac{1500 \times 10^3}{12}$$

$$L \times B = 125000\, mm^2$$

$$\frac{L}{B} = \frac{300}{250} = 1.2$$

$$L = 1.2B$$

$$1.2B \times B = 125000$$

$$B = 322.75\, mm > 250\, mm$$

$$L = 387.3\, mm > 300\, mm$$

Provide $L = 400\,mm$

$B = 350\,mm$

Projection along $L = a = \frac{400-300}{2} = 50\,mm$

" " $B = b = \frac{350-250}{2} = 50\,mm$

$$\omega = \frac{P}{L \times B} = \frac{1500 \times 10^3}{400 \times 350} = 10.71\,N/mm^2$$

$< 12\,MPa$ (OK)

$$t = \sqrt{2.5\, w\left(a^2 - 0.3 b^2\right) \frac{\gamma_{mo}}{f_y}}$$

$$= \sqrt{2.5 \times 10.71 \times \left(50^2 - 0.3 \times 50^2\right) \times \frac{1.1}{250}}$$

$$= 14.36\,mm > \underset{(t_f)}{10.6\,mm} \text{ (ok)}$$

Provide 400 X 350 X 16 mm slab base.

Provide 4 no. of M20 bolt to connect base plate with the concrete.

2. Design a slab base for a beam column SC 250 to transfer a factored axial compression of 750 KN and bending moment of 75 KNm. Take M30 concrete and E250 grade of steel.

$M = 75$ kNm

$P_u = 750$ kN

M30, E250.

$$e = \frac{M}{P} = \frac{75}{750} = 0.1\,m = 100\,mm$$

Length of slab base $\geq 6 \times e$

$\geq 6 \times 100$

≥ 600 mm

Provide $L = 600$ mm

Bearing pressure $= 0.6 \times 30$ $(0.6 f_{ck})$

$= 18$ MPa

$$q_{max} = \frac{P}{L \times B}\left(1 + \frac{6e}{L}\right)$$

$$18 = \frac{750 \times 10^3}{600 \times B}\left(1 + \frac{6 \times 100}{600}\right)$$

$\therefore B = 138.89$ mm < 250 mm

Min. width $= 250 + 2 \times$ projections

$= 250 + 2 \times 50$

$= 350$ mm

(50 – 100 mm)

Min. width = 250 + 2 × projections

= 250 + 2 × 50

= 350 mm

(50 – 100 mm)

$$q = \frac{750 \times 10^3}{600 \times 350}\left(1 \pm \frac{6 \times 100}{600}\right)$$

$$= 0 \text{ to } 7.14 \text{ MPa} < 18 \text{ MPa}.$$

(Ok).

In case of slab base, max. bending moment occurs at the face of column.

M_{max} (at X-X) per mm length

$$= 5.06 \times 175 \times \frac{175}{2} + \frac{1}{2} \times 175 \times (7.14 - 5.06) \times \frac{2}{3} \times 175$$

$$= 98714.58 \text{ Nmm/mm}$$

$$= 98.71 \text{ kN/mm}$$

Now,

$$M = 1.2 Z_e f_y / \gamma_{mo}$$

$$98.71 = 1.2 \times \frac{1 \times t^2}{6} \times \frac{250}{1.1}$$

or, $t = 46.6 \text{ mm} > t_f (17\text{mm})$

(OK)

Column Splices

Splices

- A joint when provided in the length of a member is called a splice.
- Compression will be transmitted by direct bearing, and column sections could be rested one on top of each other and function satisfactorily.
- Column sections can be spliced in the following cases:

1. When the length of the column is more than the length of the column section available, a number of pieces are jointed to furnish the full length of the column.
2. In case of multistorey buildings, the section of the column required for the various storeys may be different, as the load goes on increasing for columns of the lower storeys.

Splices

Fig. 5.35 Splicing

(a) Column flanges having complete bearing (b) Web splice for shear

Splices

- In multistorey buildings, it is very convenient to splice the column just above the floor and it is economical to provide a splice near the inflection point in the column.
- Column is preferably spliced at point of inflection which is usually 150 to 300 mm above floor line.
- In case of column splices with bolting, splice plates are to be used on the flanges as well as on the web. Splice plates are designed to resist an axial compression and bending moment (on flange).
- Splice plates on web are designed for shear force.

structura

Specification for column splicing

- Splice plate and the connection should be designed to carry 50 % of the axial load and tension(if any due to bending moment).
- Column splice plate may be assumed to act as a short column of zero slenderness ratio i.e. these plates will be subjected to yield stress.
- Assume $f_{cd}=f_y/\gamma_{mo}$ and calculate required area of splicing plate.
- The width of splicing plate is taken equal to that of width of flange of column and calculate required thickness. The thickness of splice plate should not be less than 6mm for mild exposure condition.
- For selected diameter, the bolt value is calculated and required **number of bolt** can be calculated by **Design load/ Bolt value**.

Specification for column splicing

- When **shear force** acts in addition to gravity loads, a splice plate will be provided on the **web** (as the web takes the maximum of shear stress).
- If moment and shear force are also acting in addition to axial load, splice plates are provided to the flange as well as web.
- *Splice plates attached to the flange are designed to resist additional axial load equal to M/a where, a is the distance between centre to centre of flange splice plate (lever arm).*
- *Web splice plates are designed to resist max shear force.*
- The splices should, as far as possible, be so placed that the centroidal axis of the splice coincides with the centroidal axis of the members jointed. This is done to avoid any eccentricity.

Specification for column splicing

The load P_{u1} for the design of splice and connection due to axial factored load P_u on the column will be $P_u/4$ (machined ends) and $P_u/2$ (non machined ends).

The load due to bending moment $P_{u2} = \dfrac{M_u}{\text{lever arm}}$

where, lever arm is the c/c distance of the two splice plates and P_{u2} is the design load due to the ultimate moment.

Total load = $Pu_1 + Pu_2$

When bearing plates are provided to join two columns of unequal size, following steps are followed:

- Bearing plate may be assumed as short beam to transmit axial load to the lower column
- Axial load of the column is assumed to be taken by flanges only
- Maximum moment in bearing plate =P/2 * a

When bearing plates are provided to join two columns of unequal size, following steps are followed:

- Maximum moment in bearing plate=P/2 * a
- Thickness of bearing plate t

$$t = \sqrt{\frac{6M}{b \cdot f_{bs}}}$$

$$f_{bs} = f_y / \gamma_{m0}$$

1. A column ISHB 300 @ 576.8N/m is to support a factored load of 900 KN. The column section is to be spliced at a height of 2.5 m. Design the splice plate at connection using M16 bolts of property class 4.6. Use Fe410.

Soln:

Factored load = 900 kN

Load to be designed for column splice

= 50% of 900 kN

= 450 kN

As we have to provide two column splices so load to be designed for each splice plate

= 225 kN.

Permissible stress $f_{cd} = \frac{f_y}{\gamma_{m0}} = \frac{250}{1.1} = 227.27$ MPa

Area of each splice plate $= \frac{225 \times 10^3}{227.27}$

$= 990 \text{ mm}^2$.

For ISHB 300 @ 576.8 N/m, bf = 250 mm

Width of splice plate = Width of flange (bf)
= 250 mm

Thickness of plate $= \frac{990}{250} = 3.96\,mm < 6\,mm$

Provide 250 mm × 6 mm splice plate.

Design of connection.

M16 Bolts of property class 4.6.

Shear strength $= \frac{400}{\sqrt{3} \times 1.25} \times 1 \times 0.78 \times \frac{\pi}{4} \times 16^2$

$= 28.97$ kN

Bearing strength $= 2.5\, K_b \times d \times t \times \frac{f_u}{\gamma_{mb}}$

$=$

$\therefore$ Bolt value $= 28.97$ kN

No. of bolts $= \frac{225}{28.97} = 7.76 \approx 8$ Nos.

$P_{min} = 2.5d = 2.5 \times 16 = 40\,mm$

$P_{max} = 32t$ or $300\,mm$

Provide pitch $= 60\,mm$ &

edge distance $= 50\,mm$

$\therefore$ Length of splice plate $= 6 \times P + 4 \times e$

$= 6 \times 60 + 4 \times 50$

$= 560\,mm$

Provide two $560\,mm \times 250 \times 6\,mm$ splice plates.

2. An ISHB 250 column carrying a factored axial load of 900 KN, bending moment 30 KNm and shear force of 100 KN. The column end is machined. Design the splice connection using M16 bolt of 4.6 grade.

Soln

$P_u = 900\ kN$

$M = 30\ kNm$

$S.F. = 100\ kN$

Assume thickness of splice = flange

$= 250 + \frac{6}{2} + \frac{6}{2}$

$= 256\ mm$

Splicing on flange.

$$P_{u1} = \frac{P_u}{4} = \frac{900}{4} = 225\ kN$$

$$P_{u2} = \frac{M_u}{\text{lever arm}} = \frac{30 \times 1000}{(250+6)} = 117.19\ kN$$

Total design load for splice $= 225 + 117.19$

$= 342.19\ kN$

$$\text{Area} = P/f_{rd}$$

$$\text{Area} = \frac{347.19 \times 10^3}{250/1.1}$$

$$= 1505.636\,mm^2$$

Width of splice = Width of flange

$= 250\,mm$

$$\text{Thickness of splice} = \frac{1505.636}{250}$$

$$= 6.022\,mm.$$

Adopt $t = 6\,mm$

Design of connection.

i) Strength of bolt in single shear

$V_{dsb} = 28.975\ kN$

ii) Strength of bolt in bearing

$e = 40$

$p = 60$

$V_{dpb} = 24.56\ kN$

Bolt value $= 28.975\ kN$.

No. of bolts $= \dfrac{342.19}{\text{Bolt value}}$

$= 11.81$

≈ 12 bolts. (multiple of 2)

$L = 4 \times e + 10 \times p$

$= 4 \times 40 + 10 \times 60 = 760\ mm$

Web splice

Width of web between the flanges

$= 250 - 2 \times t_f$

$= 250 - 2 \times 9.7$

$= 230.6\ mm$

Adopt width of web $= 200\ mm$

Splice plates are provided on either sides of web of column to resist the shear

Shear on each splice plate $= 50\ kN$

Design shear stress $= \dfrac{f_y}{\sqrt{3} \cdot \gamma_{m0}} = \dfrac{250}{\sqrt{3} \times 1.1} = 131.2\ N/mm^2$

Equating max. shear stress to design shear stress

$$\frac{3}{2} \times \text{Average shear stress} = 131.2$$

$$\frac{3}{2} \times \frac{50 \times 10^3}{200 \times t} = 131.2$$

$$\therefore \quad t = 2.85 \text{ mm}$$

Provide 6mm thick splice plate.

Connection design

Strength of bolt in Double shear

$V_{dsb} =$

Strength of bolt in bearing

$e = 40mm$

$p = 60mm$

$V_{dpb} =$

$\therefore$ Bolt value =

No. of bolts = 2

Gussetted base

- For columns subjected to heavy axial loads with or without a bending moment, the slab base becomes uneconomical since the required thickness of slab base is very high.
- In such case, use a gusset base .
- In gusseted bases, vertical plates known as gussets are provided which are connected to base plate usually by butt welds.
- These gusset plates stiffen base plate thereby reducing bending of base plates.

Gussetted base

- Gusset materials used in base increase the bearing area consequently reducing the thickness of base plate.
- Also it supports the base plate against bending and hence results in thinner base plate.
- This kind of base plate may be considered to be rigid
- The gusset plate should not be less than 16 mm thick
- The gusset angle is chosen so as to accommodate 2 rows of bolts in vertical leg and one row in horizontal leg. So an unequal angle section is provided. Thickness of gusset angle should be almost same as that of gusset plate.
- Length of gusset angle and gusset plate is kept equal to the length of base plate, parallel to the flange of column
- Holding down bolts 2 or 4 in numbers and of 20 mm diameter is provided
- The critical section is assumed to lie at the root of the fillet of angle section as load from flanges of column is transferred to the base plate through gusset material.

Plan

Elevation

Flange
Web
Column
Gusset Plate
Cleat Angle
(Gusset Angle)
Base Plate
Holding Down Bolt
(Rag Bolt)
Cement Concrete Base

1. A column ISHB 350 @661.2 N/m carries an axial compressive factored load of 1700 KN. Design a suitable bolted gusset base. The base rests on M15 grade concrete pedestal. Use 24 mm diameter bolts of grade 4.6 for making the connections.

Given:

ISHB 350 → $h = 350mm$

$P_u = 1700$ KN — $b_f = 250$

M15 concrete — $t_f = 11.6$

$\phi = 24mm$ of 4.6 grade. — $t_w = 8.3$ mm

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

$= 0.45 \times 15 = 6.75 N/mm^2$

Required area of base plate, $A = \frac{1700 \times 10^3}{6.75}$

$= 251.85 \times 10^3 mm^2$

let us provide 16mm thick gusset plates on two flanges of column section & two gusset angles ISA 150 X 115 X 15 mm.

350
X
X

Min. width required $= 350 + 2 \times 16 + 2 \times 115$

$= 612\,mm \approx 620\,mm$

length of base plate $= \dfrac{251.85 \times 10^3}{620}$

$= 406.2\,mm$

$\approx 410\,mm$

Provide a base plate of $620\,mm \times 410\,mm$.

Bearing pressure of concrete

$$\omega = \frac{P}{A} = \frac{1700 \times 10^3}{620 \times 410}$$

$$= 6.68\,N/mm^2 < 6.75\,N/mm^2$$

(OK).

350
X
X

The critical section of base for gusset base will be at section X - X.

The length of base plate acting as cantilever.

$$C = 115 - 15 + 4 = 104 \text{ mm}$$

$$\text{Max. moment} = \frac{wc^2}{2} = \frac{6.68 \times 104^2}{2}$$

$$= 36125.44 \text{ Nmm}$$

Moment capacity is

$$M_d = 1.2\, Z_e \cdot f_y/\gamma_{m0}$$

$$36125.44 = 1.2 \times \left(\frac{1 \times t^2}{6}\right) \times \frac{250}{1.1}$$

$$t = 28.19\ mm$$

Thickness of base plate $= 28.19 - 15 = 13.19\ mm$

$= 16\ mm > 11.9\ mm$ (t_f)

(Ok)

∴ Provide a base plate of 620 x 410 x 16 mm in size

Bolted connection :-

Strength of bolt in single shear:

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} A_{nb}$$

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

$$= 65.19 \text{ kN}$$

Strength of bolt in Bearing:

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

$\therefore$ Strength of Bolt $= 65.19$ kN

Assuming column end and gusset material to have complete bearing, 50% of load will be assumed to pass directly and 50% of load will pass through the connections.

No. of bolts required to connect column flanges with gusset plates :-

Load on two gusset plates $= \frac{1700}{2} =$

Load on each gusset plate $= 425$ kN

No. of bolts $= \frac{425}{65.19} = 6.52$ Nos. $\simeq 8$ Nos.

Provide 8-24mm ϕ bolts on each flange in two rows. The no. of bolts required to connect the gusset angle with gusset plate will be the same.

Height of gusset plate $= 150 + 2 \times 40 + 60$
$= 290$ mm.

$\therefore$ Provide $410 \times 290 \times 16$ mm in size gusset plate.

CHAPTER: SEVEN
FLEXURE MEMBER

featurePics

INTRODUCTION

- A structural member subjected to transverse loads (loads perpendicular to its longitudinal axis) is called a **beam**.
- When provided in buildings to support roofs, they are called **joists**.
- A large beam supporting a number of joists is called a **girder**.
- The usual method of selecting a beam section is based upon section modulus.
- The criterion of economy is weight rather than the modulus of section.
- It is, therefore, desirable to choose a light beam furnishing the required modulus of section.

INTRODUCTION

- Usually, the most efficient and economical shapes are I-shapes, either rolled or built-up.
- An I-section with cover plates is provided when a large modulus of section (which cannot be furnished by a single rolled I-section) is required.
- Generally ISLB or ISMB sections are provided in such cases.

INTRODUCTION

- When heavy loads are encountered, as may be in the case of a bridge, the I-section is built-up with plates for welded constructions or with plates and angles for riveted/bolted constructions.

CLASSIFICATION OF CROSS SECTIONS

- Depending upon the yield and plastic moments, and rotational capacities:
  - Plastic
  - Compact
  - Semi-compact
  - Slender

CLASSIFICATION OF CROSS SECTIONS

Plastic: $M_d = Z_p f_y$

Compact: $M_d = Z_p f_y$

Semi-compact: $M_d = Z_e f_y$

Slender: $M_d = Z_{ee} f_y$

Z_p = plastic section modulus

Z_e = elastic section modulus

Z_{ee} = effective elastic section modulus

CLASSIFICATION OF CROSS SECTIONS

M is moment of resistance

My is Yield moment ; Mp is full plastic moment.

CLASSIFICATION OF CROSS SECTIONS

Plastic (class 1) section	develops plastic hinges Allow sufficient rotation at or above plastic moment Mp so that redistribution of bending moments can take place in the structure until complete failure mechanism is formed.
Compact (class 2) section	develop their full plastic moment Mp but the local buckling prevents the required rotation at this moment to take place.
Semi-compact (class 3)section	extreme fibre compressive stress can reach yield stress fy but cannot develop Mp due to buckling
Slender (class 4) section	plate elements buckle locally before extreme fibre compressive stress reaches yield stress.

CLASSIFICATION OF CROSS SECTIONS

- The category is based on the least width-to-thickness ratio of the flanges and of the web.
- If the flanges are compact and the web is plastic, the shape is classified as compact.
- Only plastic sections can be used in indeterminate frames.
- The compact sections are suitable for simply supported beams which fail after reaching plastic moment at one section.
- Semi-compact sections can be used for elastic design, where the section fails after reaching the first yield moment My, at the extreme fibers of the beam.
- Slender sections are used for plate girders and cold-formed sections only.

Laterally supported beam

- In laterally supported beam, no local buckling of plate element occur.
- If a beam is progressively loaded, it will fail by flexure, shear and bearing.
- The beam deflects and curvature varies along the length of beam.

Laterally supported beam

IS 800:2007 Page 53
Clause 8.2.1.2

$$M_d = \beta_b Z_p f_y / \gamma_{m0}$$

To avoid irreversible deformation under serviceability loads, M_d shall be less than $1.2\ Z_e f_y / \gamma_{m0}$ incase of simply supported and $1.5\ Z_e f_y / \gamma_{m0}$ in cantilever beams;

where

β_b = 1.0 for plastic and compact sections;

β_b = Z_e / Z_p for semi-compact sections;

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

f_y = yield stress of the material; and

γ_{m0} = partial safety factor (*see* **5.4.1**).

Laterally supported beam

- For beams with plastic, compact or semi-compact flanges and slender web ($d/tw > 67\ \varepsilon$), the web is susceptible to shear buckling before yielding.
- When ($d/tw < 67\ \varepsilon$) there is no shear buckling in web. The web fails by yielding.
- **Design shear strength** $V_d = V_n/\gamma_{m0}$
  - V_n = Nominal shear resistance equals plastic shear strength V_p of a beam.
  - Plastic shear resistance V_p = shear area x $f_y/\sqrt{3}$
  - Pure shear stress = $f_y/\sqrt{3}$ will cause material to yield

0 mm 100 200

Laterally supported beam

- $V < 0.6\ V_d$ the web will be fully effective and the entire cross section of the beam will be effective in resisting the moment (denoted by M_d).
- Since the web area will be ineffective when the shear exceeds this limit, i.e. $V > 0.6\ V_d$, only the flanges will resist the moment.
  - Because of this for high shear case the moment capacity of the beam, denoted by M_{dv}, is reduced.
- For example, sections at supports of a continuous beam will be subjected to relatively high shears as well as high bending moments.
- Accordingly, there will be two cases: $V < 0.6\ V_d$ (**low shear case**), and $V > 0.6V_d$ (**high shear case**).

Case I $V \leq 0.6V_d$

When the factored design shear force V is less than or equal to 0.6 times design shear strength V_d of the section, the shear force does not influence the bending moment capacity. The shear in such cases is characterised as low shear force. The design bending strength is given by

$$M_d = \beta_b Z_p f_y / \gamma_{m0}$$

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

or

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

as appropriate.

where $\beta_b = 1.0$ (for plastic and compact sections)

$= Z_e/Z_p$ (for semi-compact sections)

Z_e, Z_p = elastic and plastic section moduii of the cross section, respectively (Appendix VIII)

f_y = yield stress of the material

$\gamma_{m0} = 1.1$, the partial safety factor

Case II $V > 0.6\ V_d$ When the factored design shear force V is greater than 0.6 times design shear strength V_d, the design bending strength will be less than the moment capacity of the section due to the interaction between the moment and the shear. Shear in such cases is characterised as high shear force. This reduced moment capacity is denoted by M_{dv} and may be determined for different classes of sections and is as follows.

(a) Plastic or Compact Section The moment capacity of a plastic or compact section is described by the following equation when the applied moment is uniform over the unbraced length of the beam and the design shear force is high.

$$M_{dv} = M_d - \beta\,(M_d - M_{fd})$$
$$\leq 1.2\, Z_e f_y / \gamma_{m0}$$

where M_d = plastic design moment of the whole section disregarding high shear force effect but considering web buckling effect

M_{fd} = plastic design strength of the area of the cross section excluding the shear area, considering partial safety factor γ_{m0},

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

V_d = design shear strength as governed by web yielding or web buckling

V = factored applied shear force

Z_e = elastic section modulus of the whole section

As per IS: 800, the shear area in different cases may be taken as follows:

For I-sections and channel sections:

Major axis bending

Hot-rolled	ht_w
Welded	dt_w

Minor axis bending

Hot-rolled or welded	$2bt_f$

Numerical

- Determine design bending strength of ISLB 350 @495 N/m considering the beam to be laterally supported. Assume the design shear force is less than the design shear strength. The unsupported length is 3m. Use E250 grade of steel.

ISLB 350 @ 495 N/m.

$h = 350mm$

$b_f = 165mm$

$t_f = 11.4mm$

$t_w = 7.4mm$

$Z_p = 851.11 \times 10^3 mm^3$

$Z_e = 751.9 \times 10^3 mm^3$

$R_1 = 16$

$r_y = 31.7mm$

Classification of section
Table 2 Page 18

<u>Check for flange</u>

$$\frac{b}{t_f} = \frac{b_f}{2} \times \frac{1}{t_f} = \frac{165}{2 \times 11.4}$$

$$= 7.24 < 9.4\varepsilon$$

<u>Check for web</u>

$$\frac{d}{t_w} = \frac{h - 2(t_f + R_1)}{t_w}$$

$$= \frac{350 - 2(11.4 + 16)}{7.4}$$

$$= 39.89 < 84\varepsilon$$

Section is PLASTIC.

$\beta_b = 1.0$

Laterally supported beam

IS 800:2007 Page 53
Clause 8.2.1.2

$$M_d = \beta_b Z_p f_y / \gamma_{m0}$$

To avoid irreversible deformation under serviceability loads, M_d shall be less than $1.2\ Z_e f_y / \gamma_{m0}$ incase of simply supported and $1.5\ Z_e f_y / \gamma_{m0}$ in cantilever beams;

where

β_b = 1.0 for plastic and compact sections;

β_b = Z_e / Z_p for semi-compact sections;

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

f_y = yield stress of the material; and

γ_{m0} = partial safety factor (*see* **5.4.1**).

Consider shear force $V < 0.6 V_d$

$$M_d = \beta_b \cdot Z_p \cdot \frac{f_y}{\gamma_{mo}} \leq 1.2 Z_e \cdot \frac{f_y}{\gamma_{mo}}$$

$$= 1 \times 851.11 \times 10^3 \times \frac{250}{1.1} \leq 1.2 \times 751.9 \times 10^3 \times \frac{250}{1.1}$$

$$= 193.43 \text{ kNm} < 205.06 \text{ kN-m.}$$

(ok)

$\therefore$ Design bending moment $= 193.43$ kN-m.

Numerical

- A simply supported steel joist of 4 m effective span is laterally supported throughout. It carries a total UDL of 10 KN/m inclusive of self weight. Design appropriate section.

Total UDL = 10 kN/m

Factored load = 1.5 × 10 = 15 kN/m

$$\text{Max B.M} = \frac{W l_e^2}{8}$$

$$= \frac{15 \times 4^2}{8}$$

$$\therefore M = 30 \text{ kNm}$$

$$\text{Max. S.F} = \frac{W l_e}{2} = \frac{15 \times 4}{2} = 30 \text{ kN}.$$

Required plastic modulus,

$$Z_{p,req} = \frac{M.\gamma_{mo}}{f_y}$$

$$= \frac{30 \times 10^6 \times 1.1}{250} = 132000 \text{ mm}^3$$

$$= 132 \text{ cm}^3$$

Take a trial section ISLB200 @ 19.8 kg/m

$Z_{pz} = 184.34 \text{ cm}^3$ (plastic)

$Z_e = 170 \text{ cm}^3$ (elastic).

Depth of section (h) = 200 mm

Width of flange bf = 100 mm

Thickness of web tw = 5.4 mm

Thickness of flange tf = 7.3 mm

Radius of root R_1 = 9.5 mm.

$$b = \frac{b_f}{2} = \frac{100}{2} = 50 \text{ mm}$$

$$d = h - 2(t_f + R_1)$$
$$= 200 - 2(7.3 + 9.5)$$
$$= 166.4 \text{ mm}$$

$$I_z = 1700 \text{ cm}^4 = 1700 \times 10^4 \text{ mm}^4.$$

<u>Classification of section</u> Pg 18, Table 2

$$\frac{b}{t_f} = \frac{50}{7.3} = 6.85 < 9.4\varepsilon \quad \text{plastic}$$

$$\frac{d}{t_w} = \frac{166.4}{5.4} = 30.81 < 84\varepsilon \quad \text{plastic}$$

Section is plastic.

No web buckling under shear before yielding since $\frac{d}{t_w} = 30.81 < 67\varepsilon$.

For welded :-

$A_v = d \times t_w$

Check for Shear Capacity

Design shear strength of section

$$V_d = \frac{V_n}{\gamma_{m0}} = \frac{A_v \cdot f_y}{\sqrt{3}\ \gamma_{m0}}$$

$$= \frac{(d \times t_w) \times f_y}{\sqrt{3} \times \gamma_{m0}}$$

$$= \frac{(200 \times 5.4) \times 250}{\sqrt{3} \times 1.1}$$

$$= 141.713 \text{ kN} > 30 \text{ kN}$$

(OK)

$$0.6\,V_d = 0.6 \times 141.713$$
$$= 85.03 \text{ kN} > 30 \text{ kN}.$$

$$V < 0.6\,V_d \Rightarrow \text{Low Shear Case}.$$

<u>Check for Design Bending Strength (M_d)</u>

$$M_d = \beta_b \cdot Z_p \cdot f_y / \gamma_{m0}$$
$$= \left(\frac{1 \times 184.34 \times 250 \times 10^3}{1.1} \right) \times 10^{-6}$$
$$= 41.895 \text{ kNm} > 30 \text{ kNm}$$

$M_d > M$ Safe in Bending.

<u>OK</u>.

For simply supported beam,

$$M_d \leq 1.2 Z_{pz} \cdot f_y / \gamma_{m0}$$

$$= 1.2 \times 170 \times 10^3 \times 250 / 1.1$$

$$41.895 < 46.36 \text{ kNm} \quad (\text{ok})$$

Check for Deflection. (under working load)

Permissible deflection $\delta = \frac{l}{300} = \frac{4 \times 10^3}{300}$

$= 13.33 \text{ mm}$.

Max. deflection $\delta_{cala} = \frac{5}{384} \frac{wl^4}{EI}$.

$= \frac{5 \times 10 \times (4 \times 10^3)^4}{384 \times 2 \times 10^5 \times 1700 \times 10^4}$

$= 9.8 \text{ mm} < 13.33 \text{ mm}$

(OK)

Table 6 Deflection Limits

Type of Building	Deflection	Design Load	Member	Supporting	Maximum Deflection
(1)	(2)	(3)	(4)	(5)	(6)
Industrial Buildings	Vertical	Live load/ Wind load	Purlins and Girts	Elastic cladding	Span/150
				Brittle cladding	Span/180
		Live load	Simple span	Elastic cladding	Span/240
				Brittle cladding	Span/300
		Live load	Cantilever span	Elastic cladding	Span/120
				Brittle cladding	Span/150
		Live load/ Wind load	Rafter supporting	Profiled Metal Sheeting	Span/180
				Plastered Sheeting	Span/240
		Crane load (Manual operation)	Gantry	Crane	Span/500
		Crane load (Electric operation up to 50 t)	Gantry	Crane	Span/750
		Crane load (Electric operation over 50 t)	Gantry	Crane	Span/1 000
		No cranes	Column	Elastic cladding	Height/150
				Masonry/Brittle cladding	Height/240
				Crane (absolute)	Span/400

Web crippling

- Loads and reactions concentrated along a short length of flange of beam are resisted by compressive stresses in the web.
- The webs of rolled steel sections are, therefore, subjected to a large amount of stresses just below the concentrated loads and above the reactions from the support.
- Stress concentration occurs at the junction of the web and the flange.
- The web near the portion of the stress concentration tends to fold over the flange.
- This type of local buckling phenomenon is called crippling or crimpling of the web.
- The bearing strength is given by $F_w = A_e \dfrac{f_{yw}}{\gamma_{m0}}$

where f_{yw} = the design yield strength of the web.

A_e = effective area of the web = $b_1 t_w$

Web failure by local crippling

The angle of dispersion of the load is assumed to be 1:2.5

Bearing length, $b_1 = b + 2n_1$ (under concentrated loads)

$b_1 = b + n_1$ (under reactions at support)

where P = concentrated load or reaction from support in N.

In some cases the required bearing length, b_1 may be very small and negligible. But a bearing of minimum 100 mm length should be provided. The bearing strength thus calculated should be greater than concentrated load.

- Aside from spreading the load over a longer portion of the flange, there are two remedies to the problem.
- The first is to provide stiffeners which bear against the flanges at the load points and are connected to the web so as to transfer the force to it gradually, not abruptly.
- The other is to make the web thicker.
- Since web crippling is a problem at only a few sections of a beam, it is economical to provide bearing stiffeners at these sections than to increase the thickness of the entire web.

Check for web bearing (web crippling)

Assume stiff bearing length = 100 mm (at support).
(b = 100 mm).

$$n_1 = 2.5(t_f + R_1)$$
$$= 2.5(7.3 + 9.5)$$
$$= 42\,mm$$

$$b_1 = 100 + 42$$
$$= 142\,mm.$$

Bearing strength $F_w = A_e \times \frac{f_y}{\gamma_{m0}}$

$$= (b_1 \times t_w) \times \frac{250}{1.1}$$

$$= 142 \times 5.4 \times \frac{250}{1.1}$$

$$= 174.27\,kN > 30\,kN$$

(OK)

Rxn at support.

Web buckling

- The web in a rolled steel section behaves like a column when placed under concentrated loads.
- The web is quite thin and is, therefore, subjected to buckling.
- Web buckling (vertical buckling) occurs when the intensity of vertical compressive stress near the center of section becomes greater than the critical buckling stress for the web acting as column.
- Generally, vertical buckling of webs is not a problem with rolled beam sections.
- This possibility exists mainly in the rather thin webs of deep plate girders where this ratio may exceed the permissible limit.

Web buckling

The maximum diagonal compression occurs at the neutral axis and will be inclined at 45° to it. The web buckling strength at support will be

$$F_{wb} = B_1 t_w f_{cd}$$

and that below concentrated load P will be

$$F_{wb} = B t_w f_{cd}$$

The maximum diagonal compression occurs at the neutral axis and will be inclined at 45º to it. The web buckling strength at support will be

$$F_{wb} = B_1 t_w f_{cd}$$

and that below concentrated load P will be

$$F_{wb} = B t_w f_{cd}$$

where

- *fcd* is the allowable compressive stress corresponding to the assumed web strut according to buckling curve C, and
- *B* is the length of stiff portion of the bearing plus the additional length given by dispersion at 45° to the level of neutral axis.
- *tw* is thickness of web

Fig. Diagonal compression

- Consider an element of plate girder at the neutral axis.
- The element shown is subjected to vertical shear τ on its vertical faces.
- Complementary shear stresses of the same magnitude are developed on the horizontal faces.
- The stresses produce compression along one diagonal and tension along the other (equal to the shear τ).
- This compression is called diagonal compression and may cause buckling of the web.

Consider a column of unit thickness and of length equal to $d_1\sqrt{2}$ (as inclined at 45° with NA). The effective length will be $(d_1\sqrt{2})/2$, if the ends are assumed to be fixed, whereas for pin connected ends, the effective length will be $d_1\sqrt{2}$.

For a simplified solution, ends are assumed to be fixed.

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

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

$$\text{Slenderness ratio.} \qquad \lambda = \frac{d_1/\sqrt{2}}{t_w\sqrt{12}} = \frac{d_1\sqrt{6}}{t_w} = 2.45\frac{d_1}{t_w}$$

Here, $d_1 = h - 2\,(t_f + R_1)$

Check for web buckling (at support or under concentrated load).

$$\frac{d}{t_w} = 30.81 < 67\,\epsilon$$

Check for web buckling not required.

$$B = b + n = b + \frac{h}{2} \quad \text{(at the support)}$$

$$B = b + 2n = b + h \quad \text{(under concentrated load)}$$

$$\lambda = \frac{KL}{r_{min}} = 2.45 \frac{d_1}{t_w}$$

$$= 2.45 \times \frac{166.4}{5.4}$$

$$= 75.5$$

Buckling class 'C'

From Table 9c ; $f_{cd} = 144$ MPa

$\therefore$ Capacity of section $= F_{wb} = B_1 t_w \cdot f_{cd}$

$= 200 \times 5.4 \times 144$

$= 155.52$ KN > 30 KN (Safe)

here,

$$B_1 = b + n = b + \frac{h}{2} = 100 + \frac{200}{2} = 200$$

$t_w = 5.4$

(8) Design laterally supported beam which is subjected to factored B.M of 150 kNm, factored shear force of 210 kN. Use Fe410. Effective span is 6m.

Soln :-

$M_u = 150$ kNm

$V_u = 210$ kN.

Plastic section modulus required,

$$Z_{pz,req} = \frac{M \cdot \gamma_{mo}}{f_y}$$

$$= \frac{150 \times 10^6 \times 1.1}{250}$$

$$= 660 \times 10^3 \text{ mm}^3$$

Take ISLB 350 @ 49.5 kg/m

$h = 350$ mm
$b_f = 165$ mm
$t_w = 7.4$ mm
$t_f = 11.4$ mm
$R_1 = 16$ mm
$I_z = 13200 \times 10^4 \text{ mm}^4$
$Z_{ez} = 752 \text{ cm}^3 = 752 \times 10^3 \text{ mm}^3$
$Z_{pz} = 851.11 \times 10^3 \text{ mm}^3$

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

<u>Section Classification</u>

$$b = \frac{b_f}{2} = \frac{165}{2} = 82.5$$

$$\frac{b}{t_f} = \frac{82.5}{11.4} = 7.24 < 9.4\varepsilon \quad \text{Plastic}$$

$$\frac{d}{t_w} = \frac{295.2}{7.4} = 39.89 < 84\varepsilon \quad \text{Plastic}$$

Check for shear capacity :

$$V_d = \frac{V_n}{\gamma_{mo}} = \frac{A_v \cdot f_y}{\sqrt{3}\ \gamma_{mo}} = \frac{h \times t_w \times f_y}{\sqrt{3} \cdot \gamma_{mo}}$$

$$= \frac{350 \times 7.4 \times 250}{\sqrt{3} \times 1.1}$$

$$= 339.85 \text{ kN} > 210 \text{ kN} \ \underline{(OK)}$$

Check for high or low shear.

$$0.6 V_d = 203.91 \text{ kN} < 210 \text{ kN}$$

$\therefore$ $V > 0.6 V_d$, <u>High Shear</u>

Since $V > 0.6 V_d$, the design bending strength is less than the moment capacity of section due to the interaction between the moment & the shear. The moment capacity of the section will be reduced & is denoted by M_{dv}. The web area is ineffective in resisting the moment & only the flanges will resist the moment.

$$Z_p = A_c \cdot y_c + A_t \cdot y_t$$

for Plastic Section, Pg 70.

$$M_{dv} = M_d - \beta (M_d - M_{fd}) \leq 1.2 Z_e f_y / \gamma_{m0}$$

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

$$= \left(\frac{2 \times 210}{339.85} - 1\right)^2$$

$$= 0.056$$

$$\leq 1.2 \times 752 \times \frac{10^3 \times 250}{1.1}$$

$$\leq 205.09 \text{ kNm}$$

$$M_d = \beta_b \cdot Z_p \cdot f_y / \gamma_{m0}$$

(M_d is plastic moment of whole section disregarding high shear but considering web buckling effect)

$$= 1 \times 851.11 \times 10^3 \times \frac{250}{1.1}$$

$$= 193.43 \text{ kNm}$$

$$M_{fd} = Z_{fp} \cdot f_y / \gamma_{m0}$$

(here $Z_{fp} \Rightarrow$ plastic section modulus of flanges)

$$Z_{fp} = Z_{pz} - A_w \cdot y_w$$

$$= Z_{pz} - \left(\frac{A_w}{2} \times y_w\right) - \left(\frac{A_w}{2} \times y_w\right)$$

$$= Z_{pz} - A_w \cdot y_w$$

$$\boxed{y_w = h/4}$$

$$y_w = \tfrac{1}{2} \times \frac{350}{2} = 350/4 = h/4$$

$$Z_{fp} = 851.11 \times 10^3 - (350 \times 7.4) \times \frac{350}{4}$$

$$= 624485 \text{ mm}^3$$

$\therefore M_{fd} = 141.93 \text{ kNm}$

$$\therefore M_{dv} = M_d - \beta_b (M_d - M_{fd})$$
$$= 193.43 - 0.056(193.43 - 141.93)$$
$$= 190.546 \text{ kNm} > 150 \text{ kN}$$
$$M_{dv} > M \quad \underline{(OK)}$$

Also, $M_{dv} = 190.546 < 205.09 \text{ kNm}$

$\underline{\underline{(OK)}}$.

Check for web buckling (at support or under concentrated load).

$$\frac{d}{t_w} = 39.89 < 67\,\epsilon .$$

Check for web buckling not required.

$$B = b + n = b + \frac{h}{2} \quad \text{(at the support)}$$

$$B = b + 2n = b + h \quad \text{(under concentrated load)}$$

$$\boxed{F_{wb} = B \cdot t_w \cdot f_{cd}}$$

Assume a stiff bearing length $b = 100mm$

Now,

$$B' \cdot t_w = \left(100 + \frac{350}{2}\right) \times 7.4$$

$$= 2035\ mm^2 \text{ (at support)}$$

<u>For. fcd</u>,

$$KL = 0.7d = 0.7 \times 295.2 = 206.64mm$$

$$r = \frac{t_w}{\sqrt{12}} = 2.136\ mm$$

$$\lambda = \frac{KL}{r} = \frac{206.64}{2.136} = 96.74$$

$\therefore$ Buckling class C

$\therefore$ fcd = _____ MPa

$\therefore$ F_{wb} = _____ > 210 kN (Ok)

Check for web crippling (web bearing)

Assuming dispersion at 1V : 2.5H

$$n_1 = 2.5(t_f + R_1) = 2.5(11.4 + 16) = 68.5\,mm$$

$$b_1 = 100 + 68.5 = 168.5\,mm.$$

Web bearing strength at support,

$$F_w = A_e \cdot \frac{f_y}{\gamma_{m0}}$$

$$= b_1 \cdot t_w \cdot \frac{250}{1.1}$$

$$= \frac{168.5 \times 7.4 \times 250}{1.1} = 283.39\,kN > 210\,kN$$

(OK)

Hence, the section ISLB 350 is safe. #

Note :-

Check for deflection :

Consider pt. load on mid span of simply supported beam

$$M = \frac{PL}{4} \Rightarrow P = \frac{4M}{L} =$$

$$\text{Working load} = \frac{P}{1.5} =$$

$$\Delta max = \frac{PL^3}{48EI}.$$

Numerical

- A hall measuring 15 m x 6 m consists of beams space at 3 m c/c. RCC slab of 110 mm is cast over the beam. The imposed load is 4 KN/m2. The beam is supported on 250 mm wall. Design intermediate beam and check for shear, deflection and lateral stability.

Soln :- (i)

Since RCC is casted on beam, it is laterally supported.

Here,

Clear span of beam = 6 m

Load per metre length of beam is :-

Wt. of RCC slab = $\gamma \times l \times h$

$= 25 \times 3 \times 0.11$

$= 8.25$ kN/m

Imposed load $= 4 \times 3 = 12$ kN/m

Self weight $= 0.8$ kN/m (Assumed).

3m
3m
3m
3m
3m
6 m

$\therefore$ Total working load $= 8.25 + 12 + 0.8$
$= 21.05$ kN/m

$\therefore$ Total factored load $= 1.5 \times 21.05$
$= 31.58$ kN/m.

Eff. length (l_{eff}) $= 6 + \frac{0.25}{2} + \frac{0.25}{2}$
$= 6.25$ m

$\therefore$ Design moment $M = \frac{wl^2}{8}$

Design Shear force $V = \frac{wl}{2}$

FLEXURE MEMBER

BUILT UP BEAM

INTRODUCTION

Built-up or compound beams are provided when:

- large bending moments are generated for long spans and heavy loads .
- Available rolled-beam sections do not have sufficient strength to resist the external bending moment; they may be reinforced along the entire length or part of it.
- Depth of the beam may be restricted due to the head room requirements

In the above situations, the top and bottom flanges of rolled I-sections are reinforced along the length of beam by providing plates (i.e. cover plates)

Fig. Built-up beams

Design a steel beam section for supporting roof of a big hall for the following data and apply the usual checks. Assume steel of grade Fe 410.

Clear Span: 6.5 m
End bearings: 150 mm
c/c spacing of beams: 3 m
Imposed load on the beam: 10 kN/m^2
Dead load (inclusive of self weight): 4 kN/m^2
Restriction on beam depth: 375 mm
The compression flange of the beam is laterally supported throughout.

Factored DL $= 1.5 \times 4 = 6\ kN/m^2$

Factored L.L $= 1.5 \times 10 = 15\ kN/m^2$

c/c spacing $= 3\ m$.

Hence,

Total UDL $= (6 \times 3 + 15 \times 3)$

$\therefore w = 63\ kN/m$

Eff. span (l_e) = Clear span + $2 \times \frac{\text{Bearing length}}{2}$

$= 6.5 + 0.15$

$= 6.65\ m$.

Max B.M $= \frac{w l_e^2}{8}$

$= \frac{63 \times 6.65^2}{8}$

$= 348.25\ kNm$

$$\text{Max. S.F.} = \frac{wl_e}{2} = \frac{63 \times 6.65}{2} = 209.475\ kN$$

For a beam laterally supported throughout,

$$Z_{pz,req} = \frac{M_{u}}{f_y}$$

$$= 1532300\ mm^3$$

Since the depth of beam is restricted to 375 mm

Try ISWB 350 @ 558.19 N/m.

$h = 350\ mm$

$b_f = 200\ mm$

$t_w = 8\ mm$

$t_f = 11.4\ mm$

$R_1 = 12\ mm$

$d = h - 2(t_f + R)$

$= 303.2\ mm$

$I_z = 15500 \times 10^4\ mm^4$

$I_y = 1180 \times 10^4\ mm^4$

$r_z = 146\ mm$

$r_y = 40.3\ mm$

$Z_{ez} = 887 \times 10^3\ mm^3$

$Z_{pz} = 995.49 \times 10^3\ mm^3$

Section modulus required by plates

$$Z_a = Z_{pz,req} - Z_{pz} \text{ of I-section}$$
$$= 1532300 - 995.49 \times 10^3$$
$$= 536810 \text{ mm}^3$$

$$Z_a = A_a \cdot h = A_a \cdot h/2 + A_a \cdot h/2$$
$$A_a = \frac{Z_a}{h} = \frac{536810}{350} = 1533.743 \text{ mm}^2$$

(<u>Assumption</u>: distance betn c.g. of flange cover plates equal to depth of I-section)

Width of cover plate = Width of flange
= 200 mm

Thickness of flange cover plate $t_{fp} = \frac{A_a}{B}$

$= \frac{1533.743}{200}$

$= 7.67\,mm$

$\nless 6\,mm$

Provide 8mm thick cover plate on each side of flange.

Overall beam depth $= 350 + 2 \times 8$

$= 366\,mm < 375\,mm$ (ok)

$Z_{p,\,provided} = 995.49 \times 10^3 + [(200 * 8) * (350 + 8)]$

$= 1568290\,mm^3$

$> 1532300\,mm^3$

(ok)

Section classification

$$\frac{b}{t_f} = \frac{200/2}{11.4} = 8.77 < 9.4\,\varepsilon$$

$$\frac{d}{t_w} = \frac{303.2}{8} = 37.9 < 84\,\varepsilon$$

$\therefore$ Section is plastic.

Since $\frac{d}{t_w} < 67\,\varepsilon$, check for web buckling is not required.

Check for shear capacity

$$V_d = \frac{A_v \cdot f_y}{\sqrt{3}\,\gamma_{m0}}$$

$$= \frac{h \times t_w \times f_y}{\sqrt{3} \times \gamma_{m0}} = \frac{350 \times 8 \times 250}{\sqrt{3} \times 1.1}$$

$$= 367.405 \text{ kN} > 209.475 \text{ kN} \ \underline{(\text{OK})}$$

Now, $0.6 V_d = 220.44 \text{ kN} > 209.475 \text{ kN}$

Since, $V < 0.6 V_d$, Low shear case.

$$M_d = \beta_b \cdot Z_{pz} \cdot \frac{f_y}{\gamma_{m0}} \leq 1.2\, Z_e \cdot \frac{f_y}{\gamma_{m0}}$$

or, $M_d = 1 \times 1568290 \times \frac{250}{1.1}$

$\therefore M_d = 356.43\ kNm > 348.25\ kNm$ (OK)

Now, $Z_{ez} = \frac{I_z}{y}$

$$I_z = 15500 \times 10^4 + 2\left[\frac{200 \times 8^3}{12} + (200 \times 8)\left(\frac{350+8}{2}\right)^2\right]$$

$$= 257.5483 \times 10^6\ mm^4$$

$$\therefore Z_{ez} = \frac{I_z}{y} = \frac{257.5483 \times 10^6}{\left(\frac{350}{2} + 8\right)} = 1408553.37\ mm^3$$

$$\therefore 1.2\, Z_e \frac{f_y}{\gamma_{m0}} = 384.15\ kNm.$$

$M_d = 356.43\ kNm < 384.15\ kNm$ (OK).

Aa
tp
h/2
350
375
tp
cover
plate
Aa

Check for deflection

Service load $= \frac{63}{1.5} = 42\ kN/m$.

Permissible deflection $\delta = \frac{l}{300} = \frac{6.5 \times 10^3}{300} = 21.67mm$

[Table 6 Pg 31 : For simply supported beam, Brittle cladding]

Max. deflection $= \frac{5}{384} \frac{wl^4}{EI}$

$= \frac{5}{384} \times \frac{42 \times (6500)^4}{2 \times 10^5 \times 257.5483 \times 10^6}$

$= 18.95\ mm < 21.67\ mm$

(OK).

Check for web bearing at support.

Take $b = 100mm$

$$n_1 = 2.5(t_f + R_1)$$
$$= 2.5(11.4 + 12)$$
$$= 58.5 \text{ mm}$$

$$F_w = (b + n_1) \times t_w \times \frac{f_y}{\gamma_{m0}}$$
$$= (100 + 58.5) \times 8 \times \frac{250}{1.1}$$
$$= 288.18 \text{ kN} > 209.47 \text{ kN}.$$

(OK)

Design of connection

Bolts / welds joining the plates & flanges are to be designed for the horizontal shear at that level.

Shear stress at level of plate & flanges

$$= \frac{V}{b_f \cdot I_z} (A\bar{y})$$

$$= \frac{209.475 \times 10^3}{200 \times 257.5483 \times 10^6} \times (200 \times 8 \times 175)$$

$$= 1.14 \ N/mm^2$$

Strength in single shear $= \frac{f_{ub}}{\sqrt{3}} \times 0.78 \times \frac{\pi}{4} d^2 \times \frac{1}{1.25}$

$$= 28974.36 \ N$$

Shear strength of two bolts = Shear force per pitch length.

or, $2 \times 28974.36 = 1.14 \times (200 \times P)$

$\therefore P = 254.16$ mm

Max Pitch in tension member = 16t or 200mm = 128 mm
Max. Pitch in comp. member = 12t or 200 mm = 96mm
Hence, provide 125mm pitch at tension side &
90 mm pitch at compression side.

Flexure member

LATERALLY UNSUPPORTED BEAM

Lateral – torsional buckling

Fig. Lateral-torsional buckling of I-section beam

Lateral – torsional buckling

- The compression flange of an I-beam acts like a column and will buckle sideways if the beam is not sufficiently stiff or the flange is not restrained laterally.
- The bending of a beam in the plane of its strong axis, and buckling about its weak axis and accompanied by twisting (torsion) is characterized as **lateral-torsional buckling** and the flanges therefore warp.
- The warping of beam-flanges is characterized by out-of-plane deformation due to rotation of the beam cross section about the longitudinal axis of the beam.

Lateral – torsional buckling

Normal stress
distribution
when warping
is restrained
Counterclockwise
rotation
Rotation axis
Clockwise
rotation

Shear centre

- Point in the plane of cross-section through which the resultant load has to pass so that the cross section is free from twisting.
- It is not necessary for laterally supported beam because compression flange is restrained.
- In case of laterally unsupported beams, if load does not pass through the shear centre, it may aggravate(destabilize) or reduce (stabilize) the lateral torsional buckling.

FIGURE 5.11 Effect of the point of application of the load

Numerical

Design a laterally unsupported beam for the following data:

- Eff. Span = 4m
- Max. BM = 550 KNm (factored)
- Max. SF = 200 KN (factored)
- Steel grade = Fe 410
- Conditions of restraints at supports:-
  - Torsional restraints – Fully restrained
  - Warping restraints – Warping not restrained in both flanges

Table 15 Effective Length for Simply Supported Beams, L_{LT}

(*Clause* 8.3.1)

Sl No.	Conditions of Restraint at Supports		Loading Condition	
	Torsional Restraint	Warping Restraint	Normal	Destabilizing
(1)	(2)	(3)	(4)	(5)
i)	Fully restrained	Both flanges fully restrained	0.70 L	0.85 L
ii)	Fully restrained	Compression flange fully restrained	0.75 L	0.90 L
iii)	Fully restrained	Both flanges fully restrained	0.80 L	0.95 L
iv)	Fully restrained	Compression flange partially restrained	0.85 L	1.00 L
v)	Fully restrained	Warping not restrained in both flanges	1.00 L	1.20 L
vi)	Partially restrained by bottom flange support connection	Warping not restrained in both flanges	1.0 $L + 2D$	1.2 $L + 2D$
vii)	Partially restrained by bottom flange bearing support	Warping not restrained in both flanges	1.2 $L + 2D$	1.4 $L + 2D$

Soln:-

From Table 15 Pg 58.

$L_{LT} = KL = 1.0L = 1.0 \times 4 = 4m = 4000mm$
(Normal loading).

Plastic section modulus (Z_{pz}) required,

$$Z_{pz,req} = 1.30 \times \frac{M \gamma_{mo}}{f_y}$$

$$= 1.30 \times \frac{550 \times 10^6 \times 1.1}{250}$$

$$= 3146000 \ mm^3$$

$$= 3140 \ \cancel{\times 10} \ cm^3$$

Z_{pz} may be increased by 25 - 50% for lateral torsional buckling

Try ISMB600 @ 1202.71 N/mm

$h = 600$ $\qquad r_y = 41.2$ mm

$b_f = 210$ $\qquad r_z = 242$ mm

$t_f = 20.3$ mm

$t_w = 12$ mm

$R_1 = 20$ mm

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

$= 519.4$ mm

$I_z = 91813 \times 10^4$ mm^4

$I_y = 2651 \times 10^4$ mm^4

$Z_{pz} = 3510.63 \times 10^3$ mm^3

$Z_{ez} = 3060.4 \times 10^3$ mm^3.

<u>Section classification</u>

$\frac{b}{t_f} = \frac{210/2}{20.3} = 5.172 < 9.4\varepsilon$ plastic flange

$\frac{d}{t_w} = \frac{519.4}{12} = 43.28 < 84\varepsilon$ Plastic web.

$\therefore$ Section is Plastic.

Check for Design Bending strength

$$M_d = \beta_b \cdot Z_{pz} \cdot f_{bd}$$

$$f_{bd} = \chi_{LT} \cdot \frac{f_y}{\gamma_{m0}}$$

$$\chi_{LT} = \frac{1}{\{\phi_{LT} + [\phi_{LT}^2 - \lambda_{LT}^2]^{0.5}\}} \leq 1.0$$

$$\phi_{LT} = 0.5[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2]$$

here, $\alpha_{LT} = 0.21$ for Rolled section..

$$\lambda_{LT} = \sqrt{\beta_b \cdot Z_p f_y / M_{cr}} \leq \sqrt{1.2\, Z_e f_y / M_{cr}}$$

$$= \sqrt{\frac{f_y}{f_{cr,b}}}$$

$$f_{cr,b} = \frac{1.1\pi^2 E}{(L_{LT}/r_y)^2}\left[1 + \frac{1}{20}\left(\frac{L_{LT}/r_y}{h_f/t_f}\right)^2\right]^{0.5}$$

8.2.2 *Laterally Unsupported Beams*

Resistance to lateral torsional buckling need not be checked separately (member may be treated as laterally supported, *see* **8.2.1**) in the following cases:

a) Bending is about the minor axis of the section,

b) Section is hollow (rectangular/ tubular) or solid bars, and

c) In case of major axis bending, λ_{LT} (as defined herein) is less than 0.4.

The design bending strength of laterally unsupported beam as governed by lateral torsional buckling is given by:

$$M_d = \beta_b Z_p f_{bd}$$

where

β_b = 1.0 for plastic and compact sections.

= Z_e/Z_p for semi-compact sections.

Z_p, Z_e = plastic section modulus and elastic section modulus with respect to extreme compression fibre.

f_{bd} = design bending compressive stress, obtained as given below [*see* Tables 13(a) and 13(b)]

$f_{bd} = \chi_{LT} f_y / \gamma_{m0}$

χ_{LT} = bending stress reduction factor to account for lateral torisonal buckling, given by:

$$\chi_{LT} = \frac{1}{\left\{\phi_{LT} + \left[\phi_{LT}^2 - \lambda_{LT}^2\right]^{0.5}\right\}} \leq 1.0$$

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2\right]$$

α_{LT}, the imperfection parameter is given by:

α_{LT} = 0.21 for rolled steel section

α_{LT} = 0.49 for welded steel section

The non-dimensional slenderness ratio, λ_{LT}, is given by

$$\lambda_{LT} = \sqrt{\beta_b Z_p f_y / M_{cr}} \leq \sqrt{1.2 \ Z_e f_y / M_{cr}}$$

$$= \sqrt{f_y / f_{cr,b}}$$

where

M_{cr} = elastic critical moment calculated in accordance with **8.2.2.1**, and

$f_{cr,b}$ = extreme fibre bending compressive stress corresponding to elastic lateral buckling moment (*see* **8.2.2.1** and Table 14).

8.2.2.1 *Elastic lateral torsional buckling moment*

In case of simply supported, prismatic members with symmetric cross-section, the elastic lateral buckling moment, M_{cr}, can be determined from:

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

$f_{cr,b}$ of non-slender rolled steel sections in the above equation may be approximately calculated from the values given in Table 14, which has been prepared using the following equation:

$$f_{cr,b} = \frac{1.1 \ \pi^2 E}{(L_{LT}/r_y)^2}\left[1 + \frac{1}{20}\left(\frac{L_{LT}/r_y}{h_f/t_f}\right)^2\right]^{0.5}$$

The following simplified equation may be used in the case of prismatic members made of standard rolled I-sections and welded doubly symmetric I-sections, for calculating the elastic lateral buckling moment, M_{cr} (*see* Table 14):

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

where

I_t = torsional constant $= \sum b_i t_i^3 / 3$ for open section;

I_w = warping constant;

I_y, r_y = moment of inertia and radius of gyration, respectively about the weaker axis;

L_{LT} = effective length for lateral torsional buckling (*see* **8.3**);

h_f = centre-to-centre distance between flanges; and

t_f = thickness of the flange.

Table 14 Critical Stress, $f_{cr,b}$

(*Clause* 8.2.2.1)

KL/r	h/t_f													
	8	10	12	14	16	18	20	25	30	35	40	50	60	80
10	22 551.2	22 255.1	22 092.6	21 994.1	21 929.8	21 885.7	21 854.0	21 805.4	21 779.0	21 763.1	21 752.7	21 740.5	21 733.8	21 727.2
20	6 220.5	5 947.9	5 794.5	5 700.0	5 637.8	5 594.7	5 563.8	5 515.8	5 489.7	5 473.8	5 463.5	5 451.4	5 444.8	5 438.2
30	3 149.3	2 905.9	2 764.6	2 676.0	2 616.7	2 575.3	2 545.3	2 498.5	2 472.8	2 457.1	2 447.0	2 434.9	2 428.3	2 421.7
40	2 036.1	1 821.2	1 693.0	1 610.8	1 555.1	1 515.8	1 487.0	1 441.7	1 416.5	1 401.1	1 391.0	1 379.0	1 372.5	1 365.9
50	1 492.9	1 303.2	1 187.3	1 111.8	1 059.9	1 022.7	995.3	951.7	927.1	912.0	902.0	890.2	883.7	877.1
60	1 178.0	1 009.5	905.0	835.6	787.4	752.4	726.4	684.6	660.9	646.1	636.4	624.7	618.2	611.7
70	973.9	823.2	728.5	664.8	620.1	587.4	562.9	522.9	500.0	485.5	476.0	464.4	458.0	451.7
80	831.3	695.4	609.2	550.7	509.1	478.4	455.3	417.2	395.1	381.2	371.8	360.5	354.1	347.7
90	725.9	602.6	523.6	469.5	430.9	402.2	380.4	344.2	322.9	309.3	300.2	289.1	282.8	276.5
100	644.7	532.0	459.3	409.3	373.2	346.4	325.8	291.4	270.9	257.7	248.8	237.9	231.8	225.5
110	580.4	476.6	409.3	362.9	329.2	303.9	284.5	251.8	232.1	219.3	210.8	200.1	194.0	187.8
120	527.9	431.9	369.5	326.0	294.5	270.7	252.3	221.2	202.4	190.1	181.6	171.2	165.2	159.1
130	484.3	395.0	336.8	296.1	266.5	244.1	226.7	197.1	179.0	167.1	158.8	148.6	142.8	136.7
140	447.6	364.2	309.5	271.5	243.4	222.3	205.8	177.5	160.2	148.7	140.7	130.8	125.0	119.0
150	416.0	337.8	286.6	250.6	224.2	204.2	188.4	161.5	144.8	133.7	126.0	116.3	110.6	104.7
160	388.7	315.2	266.8	232.8	207.8	188.8	173.9	148.2	132.0	121.3	113.9	104.3	98.8	93.0
170	364.9	295.4	249.6	217.5	193.7	175.6	161.4	136.7	121.3	111.0	103.6	94.4	89.0	83.2
180	343.9	278.0	234.6	204.1	181.5	164.2	150.6	127.1	112.2	102.2	95.2	86.0	80.7	75.0
190	325.2	262.6	221.3	192.3	170.7	154.2	141.2	118.6	104.3	94.6	87.8	79.0	73.7	68.1
200	308.3	248.8	209.6	181.7	161.2	145.4	133.0	111.3	97.5	88.1	81.4	72.8	67.8	62.2
210	293.3	236.5	198.9	172.4	152.7	137.6	125.7	104.8	91.5	82.4	75.9	67.5	62.6	57.1
220	279.5	225.3	189.3	163.9	145.1	130.6	119.1	99.0	86.2	77.4	71.2	62.9	58.1	52.7
230	267.1	215.2	180.7	156.3	138.2	124.3	113.3	93.9	81.5	72.9	66.9	58.9	54.1	48.8
240	255.8	205.8	172.8	149.4	132.0	118.6	108.0	89.3	77.2	69.0	63.1	55.3	50.6	45.4
250	245.3	197.3	165.6	143.0	126.3	113.4	103.2	85.1	73.5	65.5	59.7	52.1	47.5	42.4
260	235.7	189.5	159.0	137.3	121.1	108.7	98.8	81.3	70.1	62.3	56.7	49.3	44.8	39.7
270	226.8	182.3	152.8	131.9	116.3	104.3	94.7	77.9	67.0	59.4	53.9	46.8	42.2	37.3

Table 13(a) Design Bending Compressive Stress Corresponding to Lateral Buckling, f_{bd}, $\alpha_{LT} = 0.21$

(*Clause* 8.2.2)

$f_{cr,b}$	f_y															
	200	210	220	230	240	250	260	280	300	320	340	360	380	400	420	450
10 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.
8 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.
6 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.
4 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.
2 000	181.8	190.9	200	209.1	218.2	227.3	236.4	254.5	272.7	290.9	309.1	327.3	345.5	363.6	381.8	409.
1 000	169.1	179.5	186	196.5	202.9	209.1	219.8	229.1	245.5	261.8	275.1	291.3	300.5	323.6	332.2	355.
900	169.1	179.5	186	194.5	200.7	204.5	215.1	231.6	242.7	258.9	272	291.3	300.5	316.4	328.4	339.
800	167.3	177.5	184	190.3	196.4	206.8	212.7	224	240	258.9	268.9	284.7	293.6	301.8	324.5	335.
700	163.6	171.8	182	188.2	192	202.3	208	226.5	237.3	250.2	259.6	278.2	286.7	294.5	305.5	327.
600	161.8	168	176	181.9	194.2	197.7	203.3	218.9	226.4	244.4	253.5	261.8	276.4	287.3	294	306.
500	161.8	166.1	172	179.8	185.5	188.6	200.9	208.7	218.2	232.7	244.2	248.7	259.1	269.1	274.9	286.
450	158.2	164.2	168	173.5	183.3	186.4	191.5	206.2	215.5	224	231.8	242.2	248.7	258.2	263.5	274.
400	150.9	162.3	166	169.4	174.5	184.1	186.7	196	204.5	215.3	222.5	229.1	238.4	243.6	248.2	257.
350	147.3	152.7	162	165.2	170.2	172.7	179.6	188.4	193.6	200.7	210.2	212.7	221.1	225.5	229.1	233.
300	143.6	147	152	154.7	161.5	163.6	167.8	175.6	182.7	186.2	194.7	196.4	196.9	203.6	206.2	212.
250	134.5	137.5	142	144.3	148.4	152.3	153.6	160.4	163.6	165.8	170	173.5	179.6	178.2	179.5	184.
200	121.8	124.1	126	129.6	130.9	134.1	134.7	137.5	141.8	142.5	145.3	147.3	148.5	149.1	152.7	151.
150	101.8	103.1	104	104.5	106.9	106.8	108.7	109.5	111.8	113.5	114.4	114.5	117.5	116.4	118.4	118.
100	74.5	76.4	76	77.4	76.4	77.3	78	78.9	79.1	78.5	80.4	81.8	79.5	80	80.2	81.
90	67.3	68.7	70	69	69.8	70.5	70.9	71.3	70.9	72.7	74.2	72	72.5	72.7	72.5	73.

$E = 2 \times 105\ N/mm^2$

$L_{LT} = 4000\ mm$

$r_y = 41.2\ mm$

h_f = Centre-to-centre distance betn flanges

$= 600 - 20.3$

$= 579.7\ mm$

$t_f = 20.3\ mm$

$\therefore f_{cr,b} = 289.36\ N/mm^2$

$$\lambda_{LT} = \sqrt{\frac{250}{289.36}} = 0.9295 > 0.4,$$

the effect of lateral torsional buckling has to be considered.

$$M_{cr} = \beta_b \cdot Z_p \cdot f_{cr,b}$$

$$= 1 \times 3510.63 \times 10^3 \times 289.36$$

$$= 1.015836 \times 10^9\ N\,mm$$

$$\sqrt{1.2\, Z_e f_y / M_{cr}} = \sqrt{0.90381} = 0.9507$$

$$\lambda_{LT} = 0.9295 < 0.9507 \quad \underline{(OK)}$$

$$\phi_{LT} = 0.5\left[1 + \alpha_{LT}(\lambda_{LT} - 0.2) + \lambda_{LT}^2\right]$$
$$= 1.0086$$

$$\chi_{LT} = 0.7142 \leq 1.0$$

$$f_{bd} = \chi_{LT} f_y / \gamma_{m0}$$
$$= 162.32 \ N/mm^2$$

$$M_d = \beta_b \cdot Z_p \cdot f_{bd}$$
$$= 1 \times 3510.63 \times 10^3 \times 162.32$$
$$= 569.85 \ kNm > 550 \ kN$$

(OK)

Alternate method:

Simplified formula $M_{cr} = 1190796962 \ Nmm$

$$\lambda_{LT} = 0.859 < 0.8781$$

$$\phi_{LT} = 0.9381 \quad \chi_{LT} = 0.7604 \quad f_{bd} = 172.814$$

Check for shear capacity

$$V = 200\ kN.$$

$$V_d = \frac{f_y \cdot A_v}{\sqrt{3} \cdot \gamma_{m0}}$$

$$= \frac{250 \times (600 \times 12)}{\sqrt{3} \times 1.1}$$

$$= 944.75\ kN > 200\ kN \ (ok).$$

Check for web crippling at support.

$$F_w = (b + n_1) \cdot \times t_w \times \frac{f_y}{\gamma_{m0}}$$

$$= \{100 + 2.5(t_f + R_1)\} \times 12 \times \frac{250}{1.1}$$

$$= 547.5\ kN > 200\ kN \ (ok).$$

Check for web buckling.

$$\frac{d}{t_w} = 43.28 < 67\,\varepsilon.$$

Check for web buckling under shear is not required.

$$F_w = (b + n_1) \times t_w \times f_{cd} > 200\,kN$$

here, $b = 300\,mm$ (Assume)

$$n_1 = \frac{h}{2} = \frac{600}{2} = 300\,mm$$

$$t_w = 12\,mm$$

for f_{cd} :-

$$\boxed{\lambda = \frac{kL}{r} = 3.57 \times \frac{h}{t_w}}$$

$$= 3.57 \times \frac{600}{12}$$

$$= 178.5$$

$\therefore f_{cd} =$ _____

$\therefore F_w =$ _____ $> 200\,kN$ (ok)

Design of Beam.

L.L. = 2 kN/m 2

Thickness of roof + slab = 200mm
= 0.2 m

Soln :-

Assuming unit wt. of slab & roof as 24 kN/m³.

D.L. per m² = 0.2 X 24 = 4.8 kN/m².

L.L = 2kN/m².

Total load = 6.8 kN/m².

(a) Design of Secondary beam :-

Eff. span for intermediate span = 6m

Assume self wt = $\frac{\text{Total Load}}{300}$.

Total load/m = 6.8 X 3 = 20.4 kN/m.

Self wt = $\frac{20.4 \times 6}{300}$ = 0.4 kN/m.

Total D.Load (UDL) = 20.4 + 0.4 = 20.8 kN/m

(b) Design of main beam.

Eff. span = 9 + 0.3 = 9.3 m (Assuming bearing length = 300 mm).

Two point loads act each of value P on main beam.

$P = 20.8 \times 6 = 124.8$ kN

Assume self wt. $= \frac{\text{Total load}}{300} = \frac{2 \times 124.8}{300} = 0.832$ kN/m ≈ 0.85

$M_{max} = 128.75 \times 4.65 - 124.8 \times 1.5 - 0.85 \times \frac{4.65^2}{2} = 402.3$ kNm

$V = 128.75$ kN.

Then design the main beam as a laterally supported or unsupported beam(according to the question) .

Plate Girders

Introduction

- Plate girders are built-up flexural members.
- Their bending resistance can be increased by increasing the distance between the flanges.
- This in turn also increases the shear resistance as the web area increases.
- Plate girders are primarily provided in bridges but are also very common in buildings where heavy concentrated loads (from upper storey columns, etc.) act on a long span beam, e.g., a dining hall floor beam of a restaurant requiring a clear space all through.

Elements of plate girder

1. Web plate
2. Flange angles with or without flange cover plates for riveted/bolted plate girder and only flange plates for welded plate girders
3. Stiffeners — bearing, transverse, and longitudinal
4. Splices — for web and flange

- The analysis and design for each element of a plate girder is dealt with separately and then an overall design procedure is developed.
- Special problem of shear buckling of web and local buckling of flange is also addressed.

0 mm 100 200

Fig. 12.3 Bolted plate girder

Fig. 12.4 Welded plate girder

- Since the flange provides most of the flexural strength, most of the steel must be concentrated in flanges and as far as possible away from the neutral axis of the girder which consequently results in deep, thin web.
- Such slender webs account for many of the special problems associated with plate girders, including local instability.
- The thin webs have a possibility of buckling before yielding.
- The three principal types of buckling to be considered are: shear or diagonal buckling, bending or longitudinal buckling, and bearing or vertical buckling.

Depth of plate girder

- The depth of plate girders vary from about 1/6 to 1/15 of their spans, with average values of 1/8 to 1/12 of their spans, the former being for short spans.
- The depth of the plate girder for which the area of steel used is minimum (thin and deep webs) will have minimum weight and is called optimum depth.
- A girder giving the least weight is not necessarily economical, because if the depth is more the transportation and erection cost will be more.
- Also, for a minimum weight solution adopting thin webs may not be economical if we consider the fabrication cost of the stiffeners.
- In any case, it is desirable to know the optimum depth for economy so that even if it cannot be adopted, it will serve as a guide to be considered along with other requirements.
- Normally, a depth less than the depth giving the minimum weight is provided.
- If the thickness of the plate girder is presumed, an expression can be developed for the optimum depth of plate girder.

Optimum Depth If the moment M is assumed to be resisted entirely by the flanges, then for an I-section beam, approximately

$$M_z = f_y b_f t_f d \tag{1}$$

where f_y = design strength of the flanges

b_f = width of the flange

t_f = thickness of the flange

d = depth of the web

The gross-sectional area of the beam is given by

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

where t_w is the thickness of web.

Eliminating $b_f t_f$ using Eq. (1),

$$A = \frac{2\, M_z}{d f_y} + d t_w \tag{3}$$

Let, the web slenderness ratio be $k = \frac{d}{t_w}$, Eq. (3) reduces to

$$A = \frac{2\, M_z}{k t_w f_y} + k t_w^2 \tag{4}$$

The optimum value of t_w may be obtained by differentiating Eq. (4) with respect to t_w and equating it to zero.

$$\frac{\partial(A)}{\partial t_w} = \frac{\partial}{\partial t_w}\left(\frac{2\,M_z}{kt_w f_y}\right) + \frac{\delta}{\delta t_w}(kt_w^2)$$

or
$$0 = \frac{-2\,M_z}{kf_y t_w^2} + 2\,kt_w$$

or
$$\frac{2\,M_z}{kf_y t_w^2} = 2\,kt_w$$

or
$$t_w^3 = \frac{M_z}{f_y k^2}$$

$$t_w = \left(\frac{M_z}{f_y k^2}\right)^{0.33} \tag{5}$$

For a trial girder section d/t_w ratio of the web may be considered somewhere between 135 and 240.

Equation (3) may be rewritten as

$$A = \frac{2\,M_z}{df_y} + \frac{d^2}{k} \tag{6}$$

The optimum value of d may be obtained by differentiating Eq. (6) with respect to d and equating it to zero.

$$\frac{\partial(A)}{\partial d} = \frac{\partial}{\partial d}\left(\frac{2\,M_z}{d\,f_y}\right) + \frac{\delta}{\delta d}\left(\frac{d^2}{k}\right)$$

$$0 = \frac{-2\,M_z}{d^2\,f_y} + \frac{2\,d}{k}$$

or

$$\frac{2\,M_z}{d^2\,f_y} = \frac{2\,d}{k}$$

or

$$d^3 = \frac{M_z k}{f_y}$$

$$d = \left(\frac{M_z k}{f_y}\right)^{0.33} \qquad (7)$$

After the total girder depth is estimated, the general proportions of the girder are established for the maximum shear and maximum moment. The shear is assumed to be taken up by the web and the moment by flanges.

Stiffeners

- In steel building construction when the web of a plate girder acting alone (that is without stiffeners) proves inadequate, stiffeners may be provided for specific purpose as follows.
- To improve the buckling strength of a slender web due to shear— intermediate transverse and longitudinal web stiffeners; to prevent local buckling of the web due to concentrated loading—load-carrying stiffener; to prevent local crushing of the web due to concentrated loading—bearing stiffener; to provide torsional restraint to beams and girders at supports—torsional stiffener; to provide local reinforcement to a web under shear and bearing—diagonal stiffener; and to transmit tensile forces applied to a web through a flange—tension stiffener are provided.
- Basically there are only two types of stiffeners, intermediate stiffeners and bearing stiffeners.
- The intermediate web stiffeners are used to prevent the web from buckling due to shear and, the bearing stiffeners to prevent crushing of web under concentrated loads.

Design of Plate Girder

- Design of a plate girder basically consists in determining the most efficient and economical size of the web and flanges.
- Application of concentrated loads on the top flange may produce web yielding, web crippling, and side-sway web buckling.
- This form of buckling occurs when the compression in the web causes the flange to buckle laterally.
- In order to prevent this, the relative movements of the flanges should be restrained by lateral bracing.
- Usually in the construction of steel structures it is preferred to brace the plate girders laterally to achieve economy.
- Generally slender webs yield economical design.
- A girder of high strength-to-weight ratio can be designed by incorporating the post-buckling strength of the web in design method employed.
- A decision for the provision of intermediate stiffeners is made easily in the design process because it will affect the web thickness.
- Provisions of bearing stiffeners, torsional stiffeners, etc. are made if required. Finally, the connections between the various elements are designed, preferably with welds.

Design Procedure

The step-by-step design procedure of a plate girder is as follows.

1. It is ascertained whether the girder to be designed can be made laterally restrained throughout or not. If possible the girder should be laterally restrained as this will yield economical section. For cases where compression flange is not restrained, lateral-torsional buckling may occur and will have to be checked.
2. Self weight of the plate girder is assumed. Imposed loads are estimated and the critical moment and shear force in the girder are computed. The factored forces are determined by multiplying the critical moment and shear with load factors.
3. The optimum thickness and depth of the plate girder are determined by using the Eqs. (5) and (7).

$$t_w = \left(\frac{M_z}{f_y k^2}\right)^{0.33}$$

$$d = \left(\frac{M_z k}{f_y}\right)^{0.33}$$

where k can be assumed suitably

Design Procedure

4. The flange area is computed by Eq. (8).

$$A_f = \frac{M_z \gamma_{m0}}{f_y d}$$

The outstand of the flange plate is worked out from serviceability and buckling criteria. Once the width of flange is fixed, the flange thickness is calculated by dividing this to the flange area.

5. The flange is classified. Plastic flanges are preferred. For the flanges to be classifiable as plastic, the ratio of the outstand and the flange thickness,

$$\frac{b}{t_f} < 8.4\,\varepsilon_f$$

where b = outstand of flange = $\dfrac{b_f - t_w}{2}$

6. The trial section is checked for bending. The moment capacity of the beam is determined and this should be greater than the applied moment.
7. The flanges are curtailed in length to achieve the most economic design.

Design Procedure

8. The girder is checked for the maximum shear. The shear strength is a function of web slenderness ratio d/t_w and aspect ratio c/d, where c is the spacing of the intermediate transverse stiffeners, d is the depth of web and t_w its thickness.
9. Stiffeners are designed as per the requirements.
10. The connections of web to flange, and that for stiffeners are designed.
11. All the web and flange splices are made using full strength butt weld or non-slip bolted connections.

Curtailment of Plates

- One economical application of flange reinforcement is to add plates to portions of beam where high bending moment exists. By reinforcing cantilever or continuous spans in the regions of high bending moment, steel can be used economically.
- The size and length of the plates can be tailored to suit the moment diagram as shown in Fig.
- This moment for example in a simple plate girder under uniformly distributed loads occurs near the centre of the span and reduces towards the supports.
- It is assumed that the bending moments are resisted by the flanges.
- It is therefore possible to use a smaller flange area towards the supports.

(a) Curtailed flange plates

(b) Bending moment diagram

Fig. 9.14 Curtailment of flange plates

Graphical method of curtailment is the most widely used method for any system of loads for a bolted plate girder. First, the bending moment diagram is plotted. The bending strength diagram with all cover plates is then plotted (one by one) on the bending moment diagram. The points of intersection of the bending moment diagram with bending strength diagram give the cut-off points of the cover plates.

Chapter 8
PURLIN

PURLIN

- Purlins are members spanning on the roof frames to support the roof coverings.
- The spacing of truss is the span of the purlin.
- The spacing of purlin varies from 1 to 2.5 m
- Purlin consists of angles, channels or I-sections

1 to 2.5 m spacing
L
Spacing of truss
Roofing cover
X (z)
y
W1 (Wind Load)
W2 (Dead Load)
θ
Truss/Rafter
(-z) X
θ
y

Design steps :-

1) Resolve factored force parallel & perpendicular to the sheeting (Roofing).
2) Determine the moment & shear force about X & Y axis.

$$W_x = W_2 \sin\theta$$
$$W_y = W_1 + W_2 \cos\theta$$

(Purlins are designed as beams subjected to bending moment about z & y-axis).

$$M_x = M_z = \frac{W_y \cdot l^2}{10} = (W_1 + W_2 \cos\theta) \cdot \frac{l^2}{10}$$

$$M_y = \frac{W_x \cdot l^2}{10} = W_2 \sin\theta \cdot \frac{l^2}{10}$$

3) Determine the section modulus required for biaxial bending

$$Z_{pz} = M_z \cdot \frac{\gamma_{mo}}{f_y} + 2.5\left(\frac{d}{b}\right) M_y \cdot \frac{\gamma_{mo}}{f_y}$$

Take $\frac{d}{b} = 1$ to 2.

4) Determine the bending capacity of the section by using. (Page 53 Cl 8.2.1.2)

$$M_{dz} = \beta_b \cdot Z_{pz}\left(\frac{f_y}{\gamma_{mo}}\right) \leq 1.2\, Z_{ez} \frac{f_y}{\gamma_{mo}}$$

$$M_{dy} = \beta_b \cdot Z_{py} \frac{f_y}{\gamma_{mo}} \leq 1.2\, Z_{ey} \frac{f_y}{\gamma_{mo}}$$

5) Check for shear (Page 59 Cl 8.4)

$$V_{dz} = \frac{f_y}{\sqrt{3}\,\gamma_{mo}} \cdot A_{vz} \; ; \; V_{dy} = \frac{f_y}{\sqrt{3}\,\gamma_{mo}} A_{vy}$$

A_{vy} → shear area ⇓ $h t_w$

Shear area :-

X – X – Minor axis
Y – Y – Major axis

Shear area ⇒ Major axis bending ⇒ $h\,t_w$
Minor " " ⇒ $2b\,t_f$

(6) Check $\left(\frac{M_z}{M_{dz}}\right) + \left(\frac{M_y}{M_{dy}}\right) \leq 1.0$

(7) Check for deflection

$$\Delta_{cal} < \Delta_{per}.$$

<u>Note</u>: (Purlin is not checked in shear & bearing since loads & shear are small).

Design an I section purlin for a roof truss from the following data:

- Span of roof = 12 m
- Spacing of purlin along the slope of roof = 2m
- Slope of roof truss = 1V:2 H
- Spacing of truss = 5 m
- Wind load on roof surface in normal direction=1000 N/m2
- Vertical load from roof sheet = 200 N/m2

Soln :-

Slope of roof $\alpha = \tan^{-1}(1/2) = 26.56^\circ$

D.L. on Roof $= 200 N/m^2$

Wt. of Roof sheeting $= 200 \times 2 = 400 N/m$.

Assume self. wt. of purlin $= 150 N/m^2$ (100 to 200)

$\therefore$ Self wt $= 150 \times 2 = 300 N/m$.

Total vertical load $(W_2) = 400 + 300$

$= 700 N/m$.

Total wind load $(W_1) = 1000 \times 2 = 2000 N/m$

$W_x = W_2 \sin\alpha = 700 \sin 26.56^\circ = 313 N/m$

$W_y = W_1 + W_2 \cos\alpha = 2626 N/m$

$L = 5m$

$M_x = M_z = \frac{W_y \cdot l^2}{10} = 6565 Nm$

$M_y = \frac{W_x l^2}{10} = 782.5 Nm$

Factored moment : $M_x = M_z = 984.7.5 Nm$

$M_y = 1.5 \times 782.5 = 1173.75 Nm$

Section modulus required :

$Z_{p,req} = M_z \frac{\gamma_{mo}}{f_y} + 2.5\left(\frac{d}{b}\right)\frac{M_y}{f_y}\gamma_{mo}$

$= 69.15 \ cm^3 \quad \left(\text{Take } \frac{d}{b} = 2\right)$

Table 46 (*Concluded*)

Designation	Weight per Metre	Sectional Area	Depth of Section	Width of Flange	Thickness of Flange	Thickness of Web	Radii of Gyration		Section Modulus	Plastic Modulus	Shape Factor
			(D)	(b_f)	(t_f)	(t_w)	(r_z)	(r_y)	(Z_{ez})	(Z_{pz})	(Z_{pz}/Z_{ez})
	kg/m	cm²	mm	mm	mm	mm	cm	cm	cm³	cm³	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
ISMC 150	16.4	20.88	150	75	9.0	5.4	6.11	2.21	103.9	119.82	1.153 3
ISMB 150	14.9	19.00	150	80	7.6	4.8	6.18	1.66	96.9	110.48	1.140 1
ISLC 150	14.4	18.36	150	75	7.8	4.8	6.16	2.37	93.0	106.17	1.141 6
ISLB 150	14.2	18.08	150	80	6.8	4.8	6.17	1.75	91.8	104.50	1.138 4
ISJC 175	*11.2	14.24	175	60	6.9	3.6	7.11	1.88	82.3	94.22	1.144 9
ISJB 200	*9.9	12.64	200	60	5.0	3.4	7.86	1.17	78.1	90.89	1.163 9
ISMB 125	13.0	16.60	125	75	7.6	4.4	5.20	1.62	71.8	81.85	1.139 9
ISMC 125	12.7	16.19	125	65	8.1	5.0	5.07	1.92	66.6	77.15	1.158 5
ISLB 125	11.9	15.12	125	75	6.5	4.4	5.19	1.69	65.1	73.93	1.135 6
ISJC 150	9.9	12.65	150	55	6.9	3.6	6.9	1.73	62.8	72.04	1.147 2
ISLC 125	10.7	13.67	125	65	6.6	4.4	5.11	2.05	57.1	65.45	1.146 2
ISJB 175	*8.1	10.28	175	50	4.6	3.0	6.83	0.97	54.8	64.22	1.179 9
ISMB 100	8.9	11.4	100	50	7.0	4.2	4.00	1.05	36.6	41.68	1.138 9
ISJB 150	*7.1	9.01	150	50	4.6	3.0	5.98	1.01	42.9	49.57	1.155 6
ISJC 125	7.9	10.07	125	50	6.6	3.0	5.18	1.60	43.2	49.08	1.136 2
ISMC 100	9.2	11.70	100	50	7.5	4.7	4.00	1.49	37.3	43.83	1.175 0
ISLB 100	8.0	10.21	100	50	6.4	4.0	4.06	1.12	33.6	38.89	1.157 3
ISLC 100	7.9	10.02	100	50	6.4	4.0	4.06	1.57	32.9	38.09	1.157 6
ISJC 100	*5.8	7.41	100	45	5.1	3.0	4.09	1.42	24.8	28.38	1.144 2
ISMC 75	6.8	8.67	75	40	7.3	4.4	2.96	1.21	20.8	24.17	1.190 4
ISLB 75	6.1	7.71	75	50	5.0	3.7	3.07	1.14	19.4	22.35	1.152 2
ISLC 75	*5.7	7.26	75	40	6.0	3.7	3.02	1.26	17.6	20.61	1.171 0

NOTE — Sections having 'weight per meter' marked with an asterik (*) may be chosen as the section is lighter having high Z_p as compared to sections below it.

ISMB 150 @ 14.9 kg/m for purlin.

$h = 150 mm$

$b_f = 80 mm$

$t_f = 7.6 mm$

$t_w = 4.8 mm$

$Z_{pz} = 110.48 cm^3$

$Z_{ez} = 96.9 cm^3$

Shape factor $(Z_{pz}/Z_{ez}) = 1.1401$

Now,

Section Classification :- (Table 2 page 18)

$$\frac{b}{t_f} = \frac{(80/2)}{7.6}$$

$$= 5.263 < 9.4\varepsilon$$

$$\frac{d}{t_w} = \frac{(150 - 2 \times 7.6)}{4.8}$$

$$= 28.08 < 84\varepsilon$$

The Section is PLASTIC,

$$\beta_b = 1.0$$

Design capacity,

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

$$= 1 \times 110.48 \times \frac{250}{1.1} \leq 1.2 \times 96.9 \times \frac{250}{1.1}$$

$$= 25.109 \text{ kNm} \leq 26.43 \text{ kNm}$$

(OK)

$$Z_{py} = \frac{t_f\, b_f^2}{2} + \frac{(h - 2t_f)}{4} \cdot t_w^2$$

$$= \frac{7.6 \times 80^2}{2} + \frac{(150 - 2 \times 7.6)}{4} \times 4.8^2$$

$$= 25096.448\ mm^3$$

$$Z_{ey} = \frac{I_y}{y} = \frac{I_y}{b_f/2} = \frac{2\, I_y}{b_f}$$

$$= \frac{2 \times 46.8 \times 10^4\ mm^4}{80}$$

$$= 11700\ mm^3$$

$$\left(t_f \times \frac{b_f}{2} \times \frac{b_f}{4}\right) \times 4 = \frac{t_f \, b_f^2}{2}$$

$$\left[(h - 2t_f) \times \frac{t_w}{2} \times \frac{t_w}{4}\right] \times 2$$

$$= \frac{(h - 2t_f)\, t_w^2}{4}$$

$$M_{dy} = \beta_b \cdot Z_{py} \cdot \frac{f_y}{\gamma_{mo}}$$

$$= 1 \times 25096.448 \times \frac{250}{1.1}$$

$$= 5.704 \text{ kNm}$$

$$\leq 1.2 Z_{ey} \frac{f_y}{\gamma_{mo}}$$

$$\leq \frac{1.2 \times 11700 \times 250}{1.1}$$

$$\nleq 3.19 \text{ kNm}$$

So,

Take $M_{dy} = 3.19$ kNm.

Check

$$\frac{M_z}{M_{az}} + \frac{M_y}{M_{ay}} \leq 1.0$$

$$\frac{9.8475}{25.109} + \frac{1.17375}{3.19}$$

$0.76 < 1$ safe (OK)

Check for deflection.

$$\Delta_{max} = \frac{5}{384} \cdot \frac{wl^4}{EI} = 0.0149\,m = 14.9\,mm$$

$$w = w_1 + w_2 \cos\theta$$
$$= 2626\,N/m$$
$$l = 5\,m$$
$$E = 2 \times 10^5\,N/mm^2 = 2 \times 10^{11}\,N/m^2$$
$$I = I_x = I_z = 718\,cm^4 = 7.18 \times 10^{-6}\,m^4$$

$$\Delta_{perm} = \frac{l}{150} = \frac{5}{150} = \frac{1}{30}\,m = 33.33\,mm.$$

Hence,

$$\Delta_{max} < \Delta_{perm} \text{ (OK)}$$

Hence, use ISMB150 @ 14.9 kg/m for purlin.

Table 6 Deflection Limits

Type of Building	Deflection	Design Load	Member	Supporting	Maximum Deflection
(1)	(2)	(3)	(4)	(5)	(6)
Industrial Buildings	Vertical	Live load/ Wind load	Purlins and Girts	Elastic cladding	Span/150
				Brittle cladding	Span/180
		Live load	Simple span	Elastic cladding	Span/240
				Brittle cladding	Span/300
		Live load	Cantilever span	Elastic cladding	Span/120
				Brittle cladding	Span/150
		Live load/ Wind load	Rafter supporting	Profiled Metal Sheeting	Span/180
				Plastered Sheeting	Span/240
		Crane load (Manual operation)	Gantry	Crane	Span/500
		Crane load (Electric operation up to 50 t)	Gantry	Crane	Span/750
		Crane load (Electric operation over 50 t)	Gantry	Crane	Span/1 000
	Lateral	No cranes	Column	Elastic cladding	Height/150
				Masonry/Brittle cladding	Height/240
		Crane + wind	Gantry (lateral)	Crane (absolute)	Span/400
				Relative displacement between rails supporting crane	10 mm
			Column/frame	Gantry (Elastic cladding; pendent operated)	Height/200

Chapter 8: Roof Truss

ROOF TRUSS

Simple fink (6 - 9m)

simple Fan (12m)

ROOF TRUSS

Compound Fink (30 m span max)

Howe (30 m span max)

Pratt (30 m span max)

ROOF TRUSS

⇒ Pitch of roof truss $= \frac{\text{Rise}}{\text{Span}}$

& should be kept between 1/4 to 1/6 for proper drainage.

⇒ Spacing of the roof truss is kept 1/3 to 1/5 of the span (1/4 for 15m, 1/5 for 15-30m)

⇒ Light member are used in roof truss

⇒ Min. angle section ISA 50x50x6 should be provided.

⇒ 6mm thick gusset plate & atleast 2 rivet or bolt should be used.

(for roof truss, members are rigidly bolted (more than 1 bolt) or welded)

LOADS ON ROOF TRUSS

(1) Dead load → Roofing materials, Purlins, Trusses & Bracing systems.

→ Dead load of truss = 10% of loads on truss.

→ Empirical formula $= \left(\frac{\text{Span}}{3} + 5\right) \times 10 \ N/m^2$.

→ A rough estimate of dead wt. of roof truss can be 60% of dead wt. of girder subjected to same design loads but distributed uniformly.

→ Bracing = 12 - 15 N/m^2 of plan area

(2) Live load :- IS 875 (II)
(3) Wind load : - IS 875 (III)
↓
most critical load.

$$V_z = K_1 K_2 K_3 V_b$$

where,

V_z : Design wind speed (m/s) at any height 'z' above ground

K_1 : Probability factor or risk

K_2 : Terrain, height, structure, size factor

K_3 : Topography factor = 1 for wind slope $\theta < 3°$
= 1 to 1.36 for upwind slope $\theta > 3°$

Wind Pressure

$$P_d = 0.6 V_z^2$$

Static Wind method

Wind load F on a building is given by

$F = C_f \times A_e \times P_d$ - for the building as a whole.

$= (C_{pe} - C_{pi}) \times A \times P_d$ - for individual structural elements as roofs, walls, glazing & their fixings.

where,

C_f = Force coeff for building

C_{pe}, C_{pi} = Force coeff. for exterior & interior of bldg.

A_e = Eff. area of structure

A = Eff. area of member.

Ae is taken less than total area due to opening & shielding.

Find wind pressure for design of a sloping roof of span 10 m and pitch ¼. The height of eaves is 5 m above the ground. The building is situated in Madras and permeability is normal.

$$\frac{h}{w} = \frac{5}{10} = 0.5$$

$$\frac{h}{w} \leq \frac{1}{2}$$

$$\text{Pitch} = \frac{1}{4} = \frac{\text{Rise}}{\text{Span}} = \frac{\text{Rise}}{10} = \frac{1}{4}$$

$$\therefore \text{Rise} = 10/4 = 2.5\,m$$

$$\text{Roof angle } \alpha = \tan^{-1}\left(\frac{2.5}{5}\right) = 26.56^\circ$$

Assume Probability factor $K_1 = 1.0$ (General building)
Terrain/Height Structure size factor $K_2 = 0.8$
Topography factor $(K_3) = 1.0$
Basic wind speed at Madras $V_b = 50\,m/s$.

$$\text{Design wind speed } V_z = K_1 K_2 K_3 V_b$$
$$= 1 \times 0.8 \times 1 \times 50$$
$$= 40\,m/s.$$

$$\text{Design wind pressure } P_d = 0.6\,V_z^2$$
$$= 960\,N/m^2$$

TABLE 1 RISK COEFFICIENTS FOR DIFFERENT CLASSES OF STRUCTURES IN DIFFERENT WIND SPEED ZONES

(*Clause* 5.3.1)

Class of Structure	Mean Probable Design Life of Structure in Years	k_1 Factor for Basic Wind Speed (m/s) of					
		33	39	44	47	50	55
All general buildings and structures	50	1·0	1·0	1·0	1·0	1·0	1·0
Temporary sheds, structures such as those used during construction operations (for example, form-work and falsework), structures during construction stages and boundary walls	5	0·82	0·76	0·73	0·71	0·70	0·67
Buildings and structures presenting a low degree of hazard to life and property in the event of failure, such as isolated towers in wooded areas, farm buildings other than residential buildings	25	0·94	0·92	0·91	0·90	0·90	0·89
Important buildings and structures such as hospitals communication buildings / towers, power plant structures	100	1·05	1·06	1·07	1·07	1·08	1·08

TABLE 2 *k*, FACTORS TO OBTAIN DESIGN WIND SPEED VARIATION WITH HEIGHT IN DIFFERENT TERRAINS FOR DIFFERENT CLASSES OF BUILDINGS/STRUCTURES

(*Clause* 5.3.2.2)

Height	Terrain Category 1 Class			Terrain Category 2 Class			Terrain Category 3 Class			Terrain Category 4 Class		
m	*A*	*B*	*C*	*A*	*B*	*C*	*A*	*B*	*C*	*A*	*B*	*C*
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
10	1·05	1·03	0·99	1·00	0·98	0·93	0·91	0·88	0·82	0·80	0·76	0·67
15	1·09	1·07	1·03	1·05	1·02	0·97	0·97	0·94	0·87	0·80	0·76	0·67
20	1·12	1·10	1·06	1·07	1·05	1·00	1·01	0·98	0·91	0·80	0·76	0·67
30	1·15	1·13	1·09	1·12	1·10	1·04	1·06	1·03	0·96	0·97	0·93	0·83
50	1·20	1·18	1·14	1·17	1·15	1·10	1·12	1·09	1·02	1·10	1·05	0·95
100	1·26	1·24	1·20	1·24	1·22	1·17	1·20	1·17	1·10	1·20	1·15	1·05
150	1·30	1·28	1·24	1·28	1·25	1·21	1·24	1·21	1·15	1·24	1·20	1·10
200	1·32	1·30	1·26	1·30	1·28	1·24	1·27	1·24	1·18	1·27	1·22	1·13
250	1·34	1·32	1·28	1·32	1·31	1·26	1·29	1·26	1·20	1·28	1·24	1·16
300	1·35	1·34	1·30	1·34	1 32	1·28	1·31	1·28	1·22	1·30	1·26	1·17
350	1·37	1·35	1·31	1·36	1·34	1·29	1·32	1·30	1·24	1·31	1·27	1·19
400	1·38	1·36	1·32	1·37	1·35	1·30	1·34	1·31	1·25	1·32	1·28	1·20
450	1·39	1·37	1·33	1·38	1·36	1·31	1·35	1·32	1·26	1·33	1·29	1·21
500	1·40	1·38	1·34	1·39	1·37	1·32	1·36	1·33	1·28	1·34	1·30	1·22

NOTE 1 — *See* 5.3.2.2 for definitions of Class A, Class B and Class C structures.

NOTE 2 — Intermediate values may be obtained by linear interpolation, if desired. It is permissible to assume constant wind speed between 2 heights for simplicity.

Design wind pressure $P_d = 0.6 V_z^2$
$= 960 \ N/m^2$

(1) Wind normal to Ridge

External wind pressure coefficient Cpe :-

$\frac{h}{w} \leq \frac{1}{2}$

Roof angle $\alpha = 26.56°$

Wind angle $\theta = 0°$

EF = windward slope

GH = leeward slope

− minus = upward

+ plus = downward

Figure: Sloping roof plan

TABLE 5 EXTERNAL PRESSURE COEFFICIENTS (C_{pe}) FOR PITCHED ROOFS OF RECTANGULAR

(*Clause* 6.2.2.2)

Building Height Ratio	Roof Angle α	Wind Angle θ 0°		Wind Angle θ 90°		Lo	
		EF	*GH*	*EG*	*FH*		
$\frac{h}{w} \leq \frac{1}{2}$	degrees						
	0	−0·8	−0·4	−0·8	−0·4	−2·0	−2
	5	−0·9	−0·4	−0·8	−0·4	−1·4	−1
	10	−1·2	−0·4	−0·8	−0·6	−1·4	−1
	20	−0·4	−0·4	−0·7	−0·6	−1·0	
	30	0	−0·4	−0·7	−0·6	−0·8	
	45	+0·3	−0·5	−0·7	−0·6		
	60	+0·7	−0·6	−0·7	−0·6		
$\frac{1}{2} < \frac{h}{w} \leq \frac{3}{2}$	0	−0·8	−0·6	−1·0	−0·6	−2·0	−2
	5	−0·9	−0·6	−0·9	−0·6	−2·0	−2
	10	−1·1	−0·6	−0·8	−0·6	−2·0	−2
	20	−0·7	−0·5	−0·8	−0·6	−1·5	−1
	30	−0·2	−0·5	−0·8	−0·8	−1·0	
	45	+0·2	−0·5	−0·8	−0·8		
	60	+0·6	−0·5	−0·8	−0·8		

Wall Opening	C_{pi}
Up to 5% of wall area	± 0.2
5-20% of wall area	± 0.5
> 20% of wall area	± 0.7

On windward slope EF = −0.1376 (By interpolation)

On leeward slope GH = −0.4

Internal wind pressure coefficient Cpi :-

Wall opening = 0 to 5% of wall area.

∴ Cpi = ± 0.20 (Normal permeability).

$$F = (C_{pe} - C_{pi}) \times A \times P_d$$

Combined external + internal wind pressures =

Windward slope $= (-0.1376 \pm 0.20) \times 960$

$= \cancel{19867.90}\ 59.9\ N/m^2$ (downward)

$= -324.1\ N/m^2$ (uplift)

Leeward slope $= (-0.4 \pm 0.20) \times 960$

$= -192\ N/m^2$ (uplift)

$= -576\ N/m^2$ (uplift)

(2) Wind <u>parallel</u> to ridge ($\theta = 90°$)

<u>External pressure coeff C_{pe}</u> : on both slopes for 1/4th length of building = -0.7 (EG)

On both slopes for mid 1/2 ~~tot~~ length of bldg = -0.6 (FH)

Internal air pressure coeff. for normal permeability $C_{pi} = \pm 0.2$

Combined external + Internal pressure :-

On both slopes for 1/4th length of bldg

$= (-0.7 \pm 0.2) \times 960 \ N/m^2$

$= -480 \ N/m^2$ (Uplift)

$= -864 \ N/m^2$ (Uplift)

On both slopes for mid half length of bldg

$= (-0.6 \pm 0.2) \times 960$

$= -384 \ N/m^2$ (uplift)

$= -768 \ N/m^2$ (uplift)

The max. wind pressure for design is $59.9 \ N/m^2$ (downward) & $864 \ N/m^2$ (uplift)

Timber structures and design methods

Introduction

- Timber : trees measuring not less than 0.6 m in girth(diameter)
- Timber structures are used quite widely where timber is available cheaply.
- Timber is used for beams, columns, piles, roofing, bridges, trusses, etc.

Characteristics of good timber:

1. Narrow annual rings, closer the rings greater is the strength.
2. Compact medullary rays.
3. Dark colour.
4. Uniform texture.
5. Sweet smell and a shining fresh cut surface.
6. When struck sonorous sound is produced.
7. Free from the defects in timber.
8. Heavy weight.
9. No woolliness at fresh cut surface.

Defects in Timber

- Defects can occur in timber at various stages, principally during the growing period and during the conversion and seasoning process.
- Defects affect the quality, reduce the quantity of useful wood, reduce the strength, spoil the appearance and favor its decay.

Defects due to abnormal growth:

- Checks
- Heart Shakes
- Cup shakes
- Star shakes
- Rindgall
- Knots
- Endsplits
- Upsets
- Foxiness

Defects due to conversion

Defects due to seasoning

Fig. 4.4 Defects in Timber

Advantages

- Easy workable
- Cheap and environment friendly
- Easy to transport and fabricate
- High axial compressive strength

Disadvantages

- Weak in tension and flexure
- Susceptible to fire
- Low strength in compression compared to steel

Properties of Timber

1. **PHYSICAL PROPERTIES**

- Colour and odour
- Specific gravity: depends on type of timber
- Moisture content: 10-20% is preferred
- Grains : wood is strong along the grains so the slope of grains is important. Upto 1 in 10 slope of grain, the compressive strength is hardly affected.
- Shrinkage and swelling
- Bad conductor

Properties of Timber

2. **MECHANICAL PROPERTIES**

- Compression: strength in compression perpendicular to grain increases considerably with drying
- Axial tension: timber is strong in tension parallel to grain.
- Bending: timber is weak in bending compared to steel
- Shear strength : shearing strength increases at base of shakes, checks ; and where there is sudden change of cross-section.

IS 883 : 1994

(Reaffirmed 2005)

भारतीय मानक

निर्माण कार्यों में संरचनात्मक इमारती लकड़ी के डिजाइन — रीति संहिता

(चौथा पुनरीक्षण)

Indian Standard

DESIGN OF STRUCTURAL TIMBER IN BUILDING — CODE OF PRACTICE

(Fourth Revision)

First Reprint JULY 1995

UDC 691·11 : 624·011·1 : 624·04

Classification and Grading of timber

Structural timber can be classified according to

- Grade
- Location
- Durability
- Group
- Treatability
- Availability
- Refractoriness to air seasoning

Grade

Select grade:	Best quality having no defects
Grade I:	Standard grade timber
Grade II:	Common grade timber

Page 9

5.6 Grading of Structural Timber

5.6.1 The cut sizes of structural timber shall be graded, after seasoning, in accordance with IS 1331 : 1975 into the following three grades:

a) Select grade,

b) Grade I, and

c) Grade II.

Location

Inside	Timber remains continuously protected from rain i.e. at dry condition
Outside	Alternate cycle of wetting and drying
Wet	Continuous wet such as pile

Table 1 Safe Permissible Stresses for the Species of Timber

[*Clauses* 5.1, 5.1.2, 5.2, 5.7.1, 5.7.2 (b) 6.2, 6.3, 6.4.1, 6.4.2, 6.4.2.2, 7.5.8.4 (b)]

Species		Locality From Where Tested	Average Unit Mass at 12 Per-cent Moisture Content	Modulus of Elasticity (All Grades and All Locations)	Permissible Stress in N/mm² for Grade I							
					Bending and Tension Along Grain, Extreme Fibre Stress			Shear All Locations		Compression Parallel to Grain		
Botanical Name	Trade Name		kg/m³	N/mm²	Inside Loca-tion	Outside Loca-tion	Wet Loca-tion	Horizon-tal	Along Grain	Inside Loca-tion	Outside Loca-tion	Wet Loca-tion
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
GROUP A												
Acacia catechu	Khair	U. P.	1 009	134·4	20·1	16·8	13·4	1·55	2·21	13·8	12·3	10·1
Acacia chundra	Red kutch	M. P.	1 086	167·9	26·5	22·0	17·6	2·24	3·20	17·9	15·9	13·0
Albizia odoratissima	Black siris	Madras	737	135·4	18·7	15·6	12·5	1·53	2·19	13·3	11·8	9·6
Bruguiera spp.	Bruguiera (Mangrove)	Andmans	897	176·8	21·9	18·3	14·6	1·18	1·69	14·3	12·7	10·4
Grewia tiliifolia	Dhaman	Madras	788	148·2	18·3	15·2	12·2	1·31	1·87	12·0	10·7	8·7
Hopea utilis (*Balanocarpus utilis*)	Karung	Madras	987	169·1	25·1	20·9	16·7	1·51	2·16	16·4	14·6	11·9
Hopea glabra	Hopea	Madras	1 081	147·9	21·3	17·8	14·2	1·53	2·19	14·5	12·9	10·6
Hopea parviflora	Hopea	Madras	923	130·3	18·6	15·5	12·4	1·29	1·84	13·2	11·8	9·6
Manilota polyandra (Syn. *Cynometra polyandra*)	Ping	Assam	903	132·0	19·1	15·9	12·7	1·28	1·84	1·17	10·4	8·5
Mesua ferrea	Mesua	Assam	965	163·0	23·3	19·4	15·5	1·23	1·76	15·5	13·8	11·3
Mimusops littoralis	Bullet-wood	S. Andaman	1 103	173·9	22·7	18·9	15·1	1·47	2·10	14·2	12·7	10·4

Durability

High durability average life of 120 months and over.

Moderate durability average life of less than 120 months but of 60 months or more.

Low durability average life of less than 60 months.

Group

Group A: Modulus of elasticity in bending above 12.5 kN/mm^2

Group B: Modulus of elasticity in bending above 9.8 kN/mm^2 and below 12.5 kN/mm^2

Group C: Modulus of elasticity in bending above 5.6 kN/mm^2 and below 9.8 kN/mm^2

Treatability

This classification is based upon the resistance offered by the heartwood of a species to preservatives under a working pressure of 1.05 N/mm2 as:

(a) Heartwood easily treatable.

(b) Treatable but complete preservation not easily obtained.

(c) Only partially treatable.

(d) Refractory to treatment.

(e) Very refractory to treatment, penetration of preservative being practically nil from the sides and ends.

Availability

According to availability, timber can be of three grades, namely X, Y and Z.

- X—Most common, 1415 m3 or more per year
- Y—Common, 355 m3 to 1415 m3 per year
- Z—Less common, below 355 m3 per year

This is based upon the figures supplied by the forest departments.

Refractoriness to Air-Seasoning

- **Highly refractory (Class A)**: are slow and difficult to season; free from defects.
- **Moderately refractory (Class B)**: may be seasoned; free from surface defects, etc. ;if some protection is given against rapid drying.
- **Non-refractory (Class C)**: can be rapidly seasoned free from defects even in open air and sun. if not rapidly dried they develop blue stain and mould on the surface.

Permissible stresses:

- Given in Table 1 and Table 3 for Grade I
- For other grades, the permissible stresses shall be multiplied by following factor:
- For select grade timber = 1.16
- For grade II timber = 0.84
- [clause 6.2 & 6.3 Page 10]

6.3 For permissible stresses (excepting E) of other grades of timber, values given in Table 1 and Table 3 shall be multiplied by the following factors, provided that the conditions laid down in **6.2** are satisfied:

a) For Select Grade Timber 1·16

b) For Grade II Timber 0·84

6.3.1 When low durability timbers are to be used on outside location, the permissible stresses for all

Factors affecting strength of timber

- Soil condition and weather condition
- Extent and efficiency of seasoning
- Slope of grain and inclination
- Presence of natural defect

Design of compression member

- These consists of single piece of wood
- Generally square in cross-section; round columns are also used
- Round column is designed as a square column having same cross sectional area
- Slenderness ratio= (s/d)= ratio of unsupported length of column and least dimension of column

Types of column

a. Solid Square Column

b. Solid Circular Column

c. Box Column

d. Built up Column

Types of column

Solid column

- Any section (rectangular, circular, square) having solid core throughout the length of the column is called solid column.

Solid column

a. Short column

b. Intermediate column

c. Long column

Based on L/D ratio

where, L = unsupported length of column

D = least lateral dimension of the column section

Classification:

Short column if $\frac{L}{D} < 11$

Intermediate column if $11 < \frac{L}{D} < K_s$

Long column if $K < \frac{L}{D}$

where, $K_s = 0.702\sqrt{\frac{E}{\sigma_{cp}}}$

E = modulus of Elasticity for timber = 10800N/mm^2

σ_{cp} = allowable compressive stress parallel to grains (N/mm^2)

IS 883: 1994

7.6.1.1 For short columns, the permissible compressive stress shall be calculated as follows:

$$f_c = f_{cp}$$

7.6.1.2 For intermediate columns the permissible compressive stress is calculated by using the following formula:

$$f_c = f_{cp}\left[1 - \frac{1}{3}\left(\frac{S}{K_8\, d}\right)^4\right]$$

7.6.1.3 For long columns, the permissible compressive stress shall be calculated by using the following formula:

$$f_c = \frac{0{\cdot}329\, E}{(S/d)^2}$$

Solid column

For the purpose of estimating the L/D ratio, the least lateral dimension *d* shall be taken as follows:

a. Rectangular solid column: d = lesser of the two principle lateral dimensions.

b. Circular solid column: d = side of square having the same area as the given column.

c. Tapered solid column: $d = d_2 + \frac{d_1 - d_2}{3}$

where, d_2 = least lateral dimension of the smaller end

d_1 = least lateral dimension of the larger end

However, d shall not be taken greater than $1.5d_2$

Problem #1

- A column 200 mm x 200 mm is made of sal wood. Determine the safe load for the column if unsupported length is 2m, 3.5 m and 5 m.
- Take fcp = 10.6 N/mm2
- E = 12700 N/mm2

Case 1: L = 2m

- Size of column = 200 x 200 mm
- Unsupported length = 2 m = 2000mm
- Assume select grade, inside location and solid column
- s/d = 2000/200 = 10 < 11
- Column is short column.
- E = 126.7 x 100 N/mm2 (table 1)
- Permissible compressive stress fc = fcp = 12.296 N/mm2
- (Permissible compressive stress for select grade and for inside location ,compression parallel to grain; fcp = 10.6 x 1.16 = 12.296 N/mm2
- Load = fc x 200 x 200 = 12.296 x 200 x 200 = 491840 N

Case 2:L = 3.5 m

- s/d = 3500/200 = 17.5 > 11
- Assume solid column and inside location and grade I
- K8 = 24.27

$$k_8 = 0.702\sqrt{\frac{E}{f_{cp}}}$$

- Column is intermediate column
- Permissible stress
- fc = 9.64 N/mm2

$$f_c = f_{cp}\left[1 - \frac{1}{3}\left(\frac{s}{k_8 . d}\right)^4\right]$$

- Load = fc x 200 x 200 = 9.64 x 200 x200= 386 kN

Case 3: L = 5 m

- s/d = 5000/ 200 = 25 > 11
- k_8 = 24.27 < s/d

$$k_8 = 0.702\sqrt{\frac{E}{f_{cp}}}$$

- Column is long column
- Permissible stress

$$f_c = \frac{0.329\,E}{\left(\frac{s}{d}\right)^2}$$

Problem #2

- A column of 300 mm diameter is made of Deodar wood. Determine the safe axial load if its length is 3 m

- Assume solid column and inside location and grade I
- From table 1, for DEODAR wood

  E = 94.8 x100 N/mm2

  fcp = 7.8 N/mm2 (permissible stress for grade I and for inside location of compression parallel to grain)
- For classification of column, check for s/d ratio:

  For finding d:

  Area of square = Area of circle

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

d = 265.87 mm

s/d = 3000/265.87 = 11.28>11

K8 = 24.47

$$f_c = f_{cp}\left[1 - \frac{1}{3}\left(\frac{s}{k_8 . d}\right)^4\right]$$

- Intermediate column
- Permissible stress fc = 7.68N/mm2
- Load = fc x area = 7.68 x 265.87 x 265.87 = 543 kN

  OR 7.68 x 3.14 x (300x300) /4 = 543 kN

Problem #3

Design solid Deodar column for the following data:

Load on column = 500 KN

Column is located at open area

Effective length = 4 m

Select grade timber

Soln :-

For select grade timber,

Permissible stresses $= fcp' = 6.9 \times 1.16$

$= 8.004 \, N/mm^2$

$S = 4m$

Assume column is short.

$$\frac{S}{d} < 11$$

$$d > \frac{S}{11}$$

$$d > \frac{4000}{11}$$

$$d > 363.64 \, mm$$

Take $d = 400mm$

$P = 500 \, KN$

$$\text{Area} = \frac{500 \times 10^3}{8.004} = 62468.77 \, mm^2$$

$$\text{Other side} = \frac{62468.74}{400} = 156.17 \, mm$$

Take $200 \, mm \times 400 \, mm$ size column.

Take 200 mm x 400 mm size column.

$$s/d = 4000/200 = 20 > 11$$

$$k_s = 0.702\sqrt{\frac{E}{f_{cp}}} = 24.16$$

$$11 < \frac{s}{d} < k_s \quad (\because \text{Intermediate column})$$

$$f_c = f_{cp}\left[1 - \frac{1}{3}\left(\frac{s}{k_s d}\right)^4\right]$$

$$= 8.004\left[1 - \frac{1}{3}\left(\frac{20}{24.16}\right)^4\right] = 6.75\ N/mm^2$$

Permissible compressive stress $= 6.75\ N/mm^2$

Area of column $= \dfrac{500 \times 10^3}{6.75}$

$= 74074.07\ mm^2$

Smaller dimension $= 200\ mm$

Larger " $= \dfrac{74074.07}{200} = 370.37\ mm$

Provide $200\ mm \times 380\ mm$ size solid column.

Load $= \dfrac{200 \times 380 \times 6.75}{1000} = 513\ kN > 500\ kN$

(OK)

Box and built up column:

- Built-up columns are made of two or more rectangular sections (or planks) properly bolted together by metal connector.
- Width of each plank should not be more than 5 times its thickness
- The planks are fastened together at interval not exceeding 6 times thickness of individual plank.

c. Box Column

d. Built up Column

d1
d2
d2
d1
Timber planks

Box and built up column:

These columns are classified as

a. Short column

b. Intermediate column

c. Long column

a. For short columns $\frac{L}{\sqrt{d_1^2 + d_2^2}} < 8$, Safe compressive stress = σ_{cp}

where,

d_1 = the least overall width of box column.

d_2 = the least dimension of core in box column.

b. For intermediate columns $8 \le \frac{L}{\sqrt{d_1^2 + d_2^2}} < K_9$

$$\text{Safe compressive stress} = \sigma_{cp}\left[1 - \frac{1}{3}\left\{\frac{L}{K_9\sqrt{d_1^2 + d_2^2}}\right\}^4\right]$$

$$K_9 = \frac{\pi}{2}\sqrt{\frac{UE}{5\sigma_{cp}}}$$

U = 0.8 for 25mm thick planks

= 0.6 for 50mm thick planks

Problem #4

A built-up column of Salwood consists of four planks of 50 mm by 300 mm with the central core of 250 mm by 250 mm. Calculate safe load capacity of column if its effective length is 4m. Take Salwood used in wet location.

Soln :-

Min. outer dimension of column

$d_1 = 300 + 50$

$= 350$ mm

Min. dimension of core

$d_2 = 250$ mm

$$\frac{S}{\sqrt{d_1^2 + d_2^2}} = \frac{4 \times 10^3}{\sqrt{350^2 + 250^2}}$$

$$= 9.3 > 8$$

$$K_g = \frac{\pi}{2}\sqrt{\frac{U \times E}{5\, q\, f_{cp}}}$$

$E = 126.7 \times 10^2$ N/mm^2

$f_{cp} = 7.7$ N/mm^2

for plank thickness 50mm (Cl 7.6.2.5)

$U = 0.6$

$q = 1.0$

$$K_g = \frac{\pi}{2}\sqrt{\frac{0.6 \times 126.7 \times 10^2}{5 \times 1. \times 7.7}}$$

$$= 22.07$$

$$8 < \frac{s}{\sqrt{d_1^2 + d_2^2}} < K_g$$

∴ Column is Intermediate.

Permissible stress

$$f_c = q \cdot f_{cp}\left[1 - \frac{1}{3}\left(\frac{s}{K_g(\sqrt{d_1^2 + d_2^2})}\right)^4\right]$$

$$= 1 \times 7.7\left[1 - \frac{1}{3}\left(\frac{9.83}{22.07}\right)^4\right]$$

$$= 7.62 \text{ N/mm}^2$$

Safe Load $= \frac{7.62 \times 350 \times 350}{10^3} = 933.45$ KN

Problem #5

- Design a built-up column of Salwood fabricated with 50 mm thick and 250 mm width planks to carry an axial load of 800 KN. Effective length of column is 3.5 m
- Take E = 12700N/mm2
- fcp = 10.6 N/mm2
- U = 0.6
- q = 1

Soln :-

Take 90% of fcp

$f_c = 0.9 \times 10.6 = 9.54\ N/mm^2$

Area of each plank $= 50 \times 250 = 12500\ mm^2$

No. of planks $= \dfrac{800 \times 10^3}{12500 \times 9.54} = 6.7$.

Provide 7 planks.

$d_1 = 250\ mm$

$d_2 = 3 \times 50 = 150\ mm$

$$\frac{S}{\sqrt{d_1^2+d_2^2}} = \frac{3.5 \times 1000}{\sqrt{250^2+150^2}} = 12.005 > 8$$

$$K_g = \frac{\pi}{2}\sqrt{\frac{U E}{5q\, f_{cp}}} = 18.83$$

$$8 < \frac{S}{\sqrt{d_1^2+d_2^2}} < K_g \left(\text{Intermediate Column.}\right)$$

Permissible stress

$$f_c = q \cdot f_{cp}\left[1 - \frac{1}{3}\left(\frac{S}{K_g\sqrt{d_1^2+d_2^2}}\right)^4\right]$$

$$= 10.016\ N/mm^2$$

$$\therefore \text{Safe load} = \frac{10.016 \times 350 \times 250}{10^3}$$

$$= 876.4\ kN > 800\ kN\ \underline{(ok)}.$$

Spaced column:

- These are special form of built-up column of hollow construction consisting of two or more planks with their axis parallel and separated by spacer block and end block.

Lo
Spacer block
Disc dowel
P
P
l
L
l
End block

Problem #6

- A timber column 3 m long has to support a load of 200 KN. Taking column to be of Deodar wood, design column as a spaced column considering inside location.

Soln :-

$S = 3000\,mm$

Assume the end connector to be provided at $S/20$ from each end

$$\frac{S}{20} = \frac{3000}{20}$$

$$= 150\,mm$$

$$\frac{S}{20} \Rightarrow L/\sqrt{2.5}$$

$$\frac{S}{10} \text{ to } \frac{S}{20} \Rightarrow \frac{L}{\sqrt{3}}$$

Distance between centre of end connector

$$l_1 = 3000 - 150 - 150$$

$$= 2700\,mm$$

Effective length $s = l_1/\sqrt{2.5} = 2700/\sqrt{2.5} = 1708\ mm$

Assume thickness of plank $= 60\ mm = d$

Slenderness ratio $= \dfrac{s}{d}$

$= \dfrac{1708}{60} = 28.46 > 11$

$$\boxed{K_{10} = 0.702\sqrt{\frac{2.5E}{f_{cp}}}}$$

From IS 883

$E = 94.8 \times 10^2\ N/mm^2$

$f_{cp} = 7.8\ N/mm^2$

$K_{10} = 0.702\sqrt{\dfrac{2.5 \times 94.8 \times 10^2}{7.8}} = 38.69$

$11 < \dfrac{s}{d} < K_{10}$ (Intermediate Column).

$$f_c = f_{cp}\left[1 - \frac{1}{3}\left(\frac{s}{K_{10}\,d}\right)^4\right]$$

$= 7.04\ N/mm^2$

Area required $= \dfrac{200 \times 10^3}{7.04} = 28409.1\ mm^2$

Width of each plank $= \frac{28409.1}{2 \times 60}$

$= 236.74\ mm \approx 240\ mm$

(Let us provide 2 planks of equal size).

Provide 2 planks of size 60mm × 240mm.

Check:

Safe load $= 7.04 \times 60 \times 240 \times 2 = 202.75\ kN$

$> 200\ kN$.

(OK).

Structural Member subjected to Bending and Axial stress

For Compression

$$\frac{f_{ac}}{f_c} + \frac{f_{ab}}{f_b} \leq 1$$

For tension

$$\frac{f_{at}}{f_t} + \frac{f_{ab}}{f_b} \leq 1$$

$f_t \rightarrow$ perm. stress parallel to grain

$f_{ac} = \frac{P}{A}$, $f_{ab} = \frac{M}{Z} = \frac{M \times y}{I}$

$f_c =$, $f_b =$ from code (Permissible bending stress on extreme fibre)

Problem #7

(Q7) Check the safety of a square column 200mm × 200 mm in c/s. ~~cff~~ The effective length of the column is 3m, the axial load and bending moment in the column are 25 kN & 2 kN m respectively. The material is timber of Salwood and the column is located inside the building.

Soln :- $E = 126.7 \times 10^2 \ N/mm^2$

$f_{cp} = 10.6 \ N/mm^2$

$f_b = 16.9 \ N/mm^2$

Check s/d ; find f_c ; $f_{ac} = \frac{P}{A}$; $f_{ab} = \frac{M}{Z}$

Check $\frac{f_{ac}}{f_c} + \frac{f_{ab}}{f_b} \leq 1.0$.

Assignment

Flexure member

(a) Effective span

l_{eff} = Centre-to-centre distance between two supports

OR

Clear span + $\frac{\text{bearing length}}{2}$ + $\frac{\text{bearing length}}{2}$

whichever is minimum

(b) Bending stress

The max. bending stress due to dead load & superimposed load shall not exceed permissible stress specified by the code.

i.e. $f_{ab} = \frac{M}{Z} \leq f_b$

The allowable stress of beam slightly decreases with the increase of depth so we have to modify the section modulus by a factor known as form factor.

If depth of beam ≥ 300 mm we should take a form factor K_3 for rectangular section, (Pg 11 Cl 7.5.4)

$$M = K_3 \cdot f_{ab} \cdot Z$$

Box Beam & I-section, $M = K_4 \cdot f_{ab} \cdot Z$.

Solid circular, $K_5 = 1.18$; $M = \frac{f_{ab} \cdot Z_{sq}}{K_5}$

Square section, $K_6 = 1.414$; $M = \frac{f_{ab} \cdot Z_{sq}}{K_6}$

7.5.4 *Form Factors for Flexural Members*

The following form factors shall be applied to the bending stress:

a) *Rectangular section* — For rectangular sections, for different depths of beams, the form factor K_3 shall be taken as:

$$K_3 = 0{\cdot}81 \frac{D^2 + 89\,400}{D^2 + 55\,000}$$

NOTE — Form factor (K_3) shall not be applied for beams having depth less than or equal to 300 mm.

b) *Box beams and I-beams* — For box beams and I-beams the form factor K_4 shall be obtained by using the formula:

$$K_4 = 0{\cdot}8 + 0{\cdot}8\,y \left(\frac{D^2 + 89\,400 - 1}{D^2 + 55\,000} \right)$$

where

$$y = p_1^2 \,(6 - 8\,p_1 + 3\,p_1^2)\,(l - q_1) + q_1$$

MISTAKE

CORRECTION

7.5.4 *Form Factors for Flexural Members*

The following form factors shall be applied to the bending stress:

a) *Rectangular section* — For rectangular sections, for different depths of beams, the form factor K_3 shall be taken as:

$$K_3 = 0{\cdot}81 \frac{D^2 + 89\,400}{D^2 + 55\,000}$$

NOTE — Form factor (K_3) shall not be applied for beams having depth less than or equal to 300 mm.

b) *Box beams and I-beams* — For box beams and I-beams the form factor K_4 shall be obtained by using the formula:

$$K_4 = 0{\cdot}8 + 0{\cdot}8\,y \left(\frac{D^2 + 89\,400}{D^2 + 55\,000} - 1 \right)$$

where

$$y = p_1^2 \,(6 - 8\,p_1 + 3\,p_1^2)\,(l - q_1) + q_1$$

(c) Width of Beam

The min. width of beam shall not be less than 50mm or span/50 whichever is greater.

(d) Depth of Beam

Depth of beam shall not be taken 3 times of width of beam without any restrainer.

$$d \not> 3b$$

In practical, $\frac{d}{b} = 2$

(e) Shear (Pg 12 cl 7.5.7)

$$H = \frac{3V}{2bd} < f_{shear}$$

(f) Bearing

Flexural member directly supported on masonry wall or concrete ball, the bearing length provided at each end of the support shall not be less than 75mm. In design, bearing length generally taken as 200 ~ 300 mm.

(g) Deflection

The max. deflection can be calculated for any type of loading then this calculated deflection shall not exceed permissible deflection specified by code.

$\Delta_{calculated} \leq \Delta_{permissible}$.

Problem #1

- A timber beam having a clear span of 6 m carries a UDL of 15 kN/m including the self weight of the beam assuming beam to be made of deodar wood, design the beam.

Assume bearing length at each end be 300mm

Effective length $l_e = 6 + \frac{0.3}{2} + \frac{0.3}{2}$

$= 6.3\,m$

Max. B. M $= \frac{W l_e^2}{8} = \frac{15 \times 6.3^2}{8} = 74.42\ kNm$

Max. permissible bending stress for Deodar

$f_b = 10.2\ N/mm^2$.

Now,

Section modulus required.

$Z_{req} = \frac{M}{f_b} = \frac{74.42 \times 10^6}{10.2} = 7.3 \times 10^6\ mm^3$

Width of Beam $\not< 50mm$ $\not< span/50$ } whichever is greater.

Width of beam = 50mm OR $\frac{6300}{50} = 126mm$

Adopt b = 126mm

Provide 200mm width.

Section modulus = $Z = \frac{bd^2}{6} = 7.3 \times 10^6 mm^3$

or, $200 \times d^2 = 6 \times 7.3 \times 10^6$

$\therefore d = 467.97 mm$

Take 200mm x 470mm size beam.

<u>Check</u> $d \not> 3b$

$470 \not> 600$ (ok)

Check for Shear :- (Horizontal shear)

$$V = \frac{W l_e}{2}\left[1 - \frac{2d}{l_e}\right]$$

$$= \frac{15 \times 6.3}{2}\left[1 - \frac{2 \times 0.47}{6.3}\right]$$

$$= 40.2 \text{ kN}$$

$$H = \frac{3V}{2bd} = \frac{3 \times 40.2 \times 10^3}{2 \times 200 \times 470} = 0.641 \text{ N/mm}^2$$

$$H = 0.641 \text{ N/mm}^2 < 0.7 \text{ N/mm}^2$$

(OK)

Check for deflection :- (Cl 7.5.9 Pg 13)

Max deflection $\Delta_{max} = \frac{5}{384} \frac{Wl^4}{EI}$

$$= \frac{5}{384} \times \frac{15 \times (6.3 \times 10^3)^4}{94.8 \times 10^2 \times 1.73 \times 10^9}$$

$$= 18.76 \text{ mm}$$

$$I = \frac{bd^3}{12} = \frac{200 \times 470^3}{12} = 1.73 \times 10^9$$

$$\Delta_{permissible} = \frac{span}{240} = \frac{6300}{240} = 26.25 \text{ mm}$$

$\Delta_{max} < \Delta_{permissible}$ (OK).

Check for Bearing :-

$$\text{Bearing stress} = \frac{\text{Reaction}}{\text{Bearing area}}$$

$$\text{Reaction} = \frac{W l_e}{2} = \frac{15 \times 6.3}{2} = 47.25 \text{ kN}$$

$$\text{Bearing area} = 200 \times 300 = 60000 \text{ mm}^2$$

$$\text{Bearing stress} = \frac{47.25 \times 10^3 \text{ N}}{60000 \text{ mm}^2}$$

$$= 0.7875 \text{ N/mm}^2 < 2.7 \text{ N/mm}^2$$

Permissible bearing stress = Permissible compressive stress perpendicular to grain

$$= 2.7 \text{ N/mm}^2$$

Safe (ok)

Problem #2

- Design one of the intermediate Deodar joist used in timber roof of a hall from the following data:
- Spacing of joist (c/c) = 50 cm
- Clear span of joist = 2 m
- Bearing at each end = 8 cm
- Dead load = 2 kN/m2
- Live load = 1.5 kN/m2
- Timber of standard grade and inside location

2 m
50 cm
50 cm

Solution :—

For Deodar, $E = 94.8 \times 10^2 \ N/mm^2$

Dead load $= 2 \times 0.5 = 1 \ kN/m$

Live load $= 1.5 \times 0.5 = 0.75 \ kN/m$

Assuming self wt. of beam $= 0.1 \ kN/m$

Total load $= W = 1.85 \ kN/m$

Effective span l_{eff} = Clear span + Bearing length

$\therefore \ l_{eff} = 2 + 0.08 = 2.08 \ m$

$$\text{Max. B.M.} = \frac{W \cdot l_e^2}{8} = \frac{1.85 \times 2.08^2}{8} = 1.0005 \ kNm$$

Max. permissible stress for Deodar $f_b = 10.2 \ N/mm^2$ (Bending)

Section modulus required

$$Z_{req} = \frac{M}{f_b} = 98086.27 \ mm^3$$

Now, $Z = \frac{bd^2}{6}$

or, $bd^2 = 588517.65$

Taking $b/d = 0.5 \Rightarrow b = 0.5d$

$\therefore d = 105.58$ mm & $b = 52.79$ mm.

Min width of beam $b > 50$ mm or $l/50 (= 2.08/50)$

$b > 50$ mm or 41.6 mm

$52.79 > 50$ mm (ok)

Provide $b = 60$ mm & $d = 120$ mm Deodar beam.

Check for shear

Shear $V = \frac{W l_e}{2}\left[1 - \frac{2D}{l_e}\right] = \frac{1.85 \times 2.08}{2}\left[1 - \frac{2 \times 0.14}{2.08}\right]$

$\therefore V = 1.665$ kN

Shear stress $(\tau) = \frac{3V}{2bD} = \frac{3 \times 1.665 \times 10^3}{2 \times 60 \times 120} = 0.347\ N/mm^2$

From code, permissible shear stress $= 0.7\ N/mm^2 > 0.347\ N/mm^2$

(Horizontal) (OK) Safe in Shear.

Check for Deflection :-

$$\Delta_{calculated} = \frac{5 w l_e^4}{384 EI} = \frac{5 \times 1.85 \times (2080)^4}{384 \times 94.8 \times 10^2 \times 8.64 \times 10^6}$$

$$I = \frac{60 \times 120^3}{12} = 8.64 \times 10^6 \, mm^4$$

$$\therefore \Delta = 5.505 \, mm$$

$$\Delta_{permissible} = span/240 = 2080/240 = 8.67 \, mm > 5.505 \, mm$$

(OK).

Check for Bearing :-

$$\text{Bearing stress} = \frac{\text{Reaction}}{\text{Bearing area}}$$

$$\text{Reaction} = \frac{W_{le}}{2} = \frac{1.85 \times 2080}{2} = 1924\,N$$

$$\text{Bearing area provided} = 60 \times 80 = 4800\ mm^2$$

$$\text{Bearing stress} = \frac{1924}{4800}$$

$$= 0.4\ N/mm^2$$

Permissible bearing stress $= 2.7\ N/mm^2 > 0.4\ N/mm^2$
(perpendicular to grain) (ok)