
UNIT 8 DETAILING OF STEEL STRUCTURE

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8.1 INTRODUCTION

Detailing of steel structures is an utmost important task before the construction work can be executed. Detailed drawing of every member of the steel-structure needs to be drawn to indicate

- 1) Its alignment and exact position in the structure.
- 2) Elevation plan and Cross-section at typical locations.
- 3) End connection details.

From the structural drawings, the fabrication drawings are required to be prepared so that fabrication details are correctly known for all members. Location of bolt holes and their diameter or length and size of the weld are to be clearly marked on the drawing. The fabricator then makes every member as per the details given upto every mm of dimension, and assembles required parts and erect the structure as imagined by the "Structural Designers"

Objectives

After studying this unit you should be able to

- know how neat sketches and structural drawings are drawn, and

- draw useful drawings of roof-trusses, beams, columns and some steel girders including details of their end connections.

8.2 LOADS

8.2.1 Dead Loads

Dead load (DL) consists of self weight of the materials used for construction. The density of the various materials is indicated in I.S. 875 (Part 1). The rounded off values of weights of a few materials is indicated below in Table 8.1 as per the above referred code.

Table 8.1

Sl.No	Material	Nominal Size or Thickness (mm)	Approximate Weight in Units	
			kN	kg
1)	Asbestos Cement Sheets	6	0.12 to 0.13	12 to 13
	a) Corrugated with pitch (p = 146 mm) b) Plain	5	0.09	9
2)	Bricks	—	16 to 19	1600 to 1900
	Common brick	—	kN/m ³	kg/m ³
	Pressed bricks	—	17 to 19	1700 to 1900
	Refractory bricks	—	18 to 20	1800 to 2000
3)	Cast Iron Manhole cover	500 mm diameter	1.16 kN per unit	118 kg per unit
	a) H.D	500	0.57	58
	b) M.D	—	0.78	80
	Rectangular (M.D) Manhole frame	500	0.83	85
	a) Circular (H.D)	500	0.57	58
	b) Circular (M.D) c) Rectangular (M.D)	—	0.63	64
4)	Cement	—	14.1 kN/m ³	1440 kg/m ³
5)	a) Reinforced Concrete	—	25 kN/m ³	2500 kg/m ³
	b) Plain Concrete	—	24 kN/m ³	2400 kg/m ³
6)	Cement Plaster	—	20.4 kN/m ³	2100 kg/m ³
7)	Glass Sheet	5 mm	0.123 kN/m ³	12.5 kg/m ³
8)	Brick Masonry	—	19 kN/m ³	2000 kg/m ³
	Common Brick Engineering Bricks	—	23.5 kN/m ³	2400 kg/m ³
9)	Stone Masonry	—	25 kN/m ³	2500 to 2600 kg/m ³

The dead load of the floor/roof remains constant at all time.

8.2.2 Imposed Load (IL)/Imposed Live Load

The imposed load (IL) on the floor depends on the functional use of the floor as well as the

type of occupancy. This load is also referred to as Live-load in practice and it varies from 2 kN/m^2 to 5 kN/m^2 in residential to office buildings. The store room will have heavier live/imposed load depending upon the height of the storey and for very heavy store rooms it can be as high as 20 kN/m^2 . The toilet blocks have generally a low imposed load of 2 kN/m^2 . The staircase units, where crowding is possible, the values of imposed load for residential and office buildings can be in the range of (3 to 5 kN/m^2) respectively. In the case of factory floors, the weight of machinery or vessels and their impact effects need to be obtained from the manufacturers. The DL and Imposed Load together from roof-truss with A.C. sheets is generally about 100 to 120 kg/m^2 when false ceiling is added to truss-soffit additional load of 50 to 70 kg/m^2 is to be considered. Imposed load need not be constant at all time. It can vary from its minimum value which can be nil to maximum value, which needs to be properly estimated as given above or on the basis of previous experience.

8.2.3 Wind Load/Earthquake Load

The lateral (horizontal) load is developed on the structure due to wind blowing against the external walls. The wind pressure is a function of probable maximum wind velocity at a given location in a country. It also depends on the height of structure above the mean sea level. The wind pressure at higher altitudes are higher. The effect of wind on roof of steel structure is also to create either suction or pressure depending on the angle of inclination or the slope of roof and direction of wind in reference to the building orientation. In case A.C. sheets are provided on steel roof trusses, the suction load developed due to high wind velocity can be more than the dead load of roof sheets and sheets are likely to fly-off unless properly anchored to trusses which in turn are anchored to columns and held suitably.

“Magnitude of Earthquake Loads” are function of magnitude of mass at every floor and the stiffness of the structure. The intensity of earthquake depends on location or zone on the Indian map where the building is to be constructed. The details of computation of earthquake loads is given in I.S. 1893 of 1985.

In general for steel structures, we will not consider the effect of earthquake loads since 1) Dead load or mass of steel structure is much less as compared to that of RCC structure. Many times industrial sheds have A.C. sheet roof, 2) Generally steel factories are low rise structures for which earthquake forces are not critical.

In general we will consider only the wind load on steel structures in this unit and simplify the work of analysis and then study the transfer of loads finally to the foundations.

Crane loads with impact factor will also be considered when overhead moving crane girder is considered as a functional requirement of the factory.

8.2.4 Computation of Wind Load

Wind loads are generally calculated based on the provisions given in I.S.875 (Part 3) - 1987. The magnitude of the wind pressure (P_z) at any level (Z) above ground is given by

$$P_z = 0.6 V_z^2$$

where,

P_z = design wind pressure in N/m^2 at height (Z) above mean ground level, and

V_z = design wind velocity in m/s at height (Z).

The velocity of the wind at 10 m height from the mean ground level is referred to as basic wind speed and is given in the wind map of India in Figure 1 of the code. The maximum wind speed as indicated in the map is 55 m/sec and the corresponding maximum wind pressure is estimated to be about 1.8 kN/m^2 which occurs in region near about Ladakh in the Himalayan region. The basic wind speed near the eastern coastal region is 50 m/sec , which corresponds to about 1.5 kN/m^2 of wind pressure at level 10 m above the ground. The basic wind speed near the western coastal land from Ratnagiri to Surat which also includes Mumbai (Bombay) port, is about 44 m/sec which corresponds to a maximum wind pressure of 1.2 kN/m^2 . In the region near Delhi the wind pressure is around 1.3 kN/m^2 .

8.3 STRUCTURAL STEEL SYSTEM OF A TYPICAL FACTORY BUILDING

The factory building having overhead travelling crane facility, has generally large column-free

area whereby long span trusses are required to be provided to support the roof with A.C. sheets on the top. When crane capacity is high, the height of column is also substantially large. A built up column of two I sections (Two legged column) is generally provided so that the outer leg (L_0) is extended above the gantry level to support the roof trusses as shown in Figure 8.1 (Section Y1 Y1) while the inner leg (L_1) directly supports the gantry girder. The rails are laid along gantry-girder over which the wheels of crane-girder can move. Figure 8.1 shows the layout plan for a typical factory which has dimensions of 42 m \times 21 m in plan. Reference directions X and Y are marked in the plan. The grid-lines marked (1) to (8) along Y direction show centre to centre of steel columns along X direction. Grid Lines (A) . (B) . (C) . (D) and E parallel to X direction are centre lines of boundary columns which are spaced at 5.25 m c/c along Y direction. The portion of plan between grid lines (3) to (5) show plan details at gantry level.

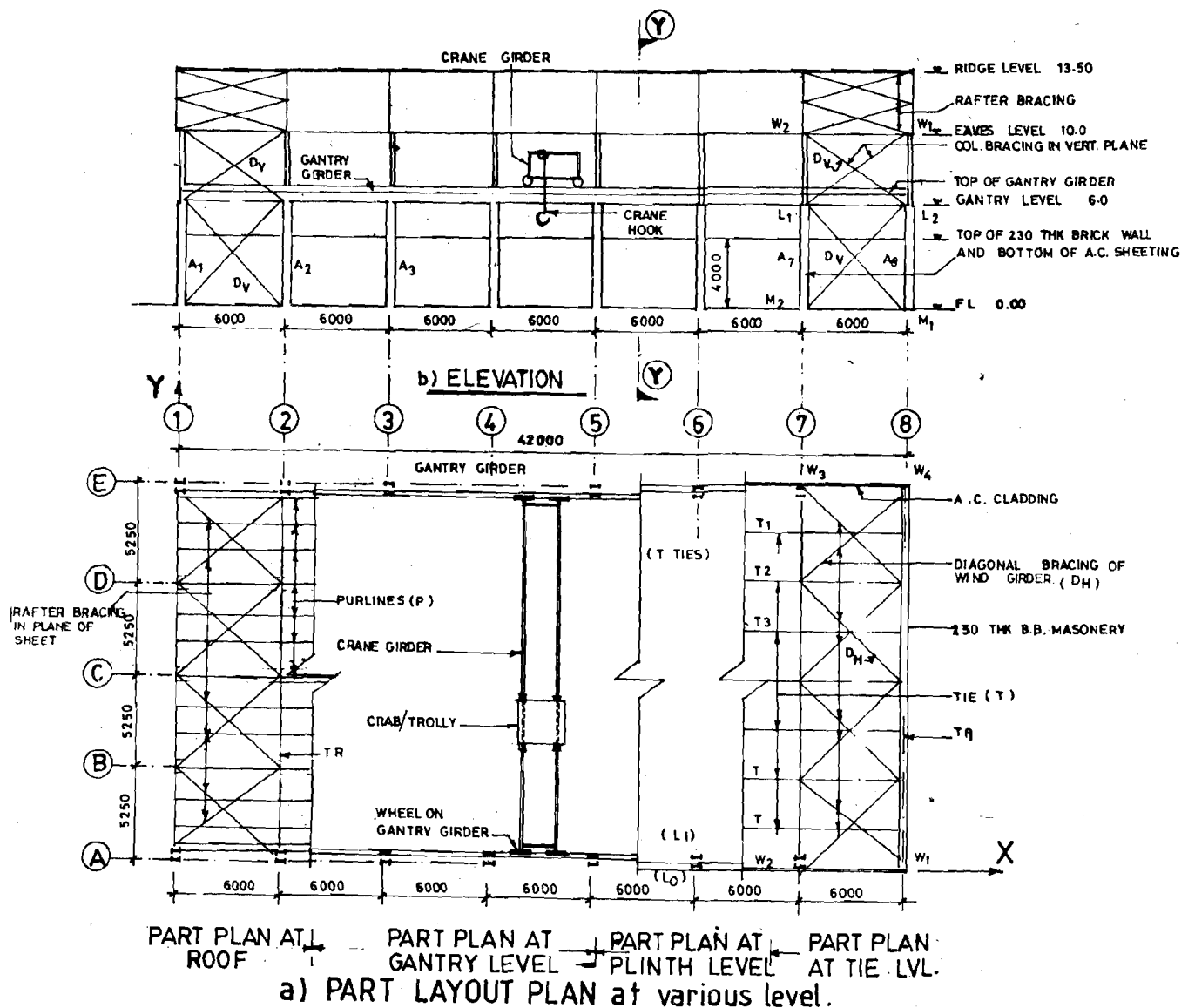


Figure 8.1 (a, b) : Part Layout Plan and Elevation at Various Levels

expansion of the factory when additional Land-space is available beyond these grid line. The extension of column-free area is easily possible if trusses along grid lines (1) and (8) are also provided and supported on columns (A₁, E₁) and (A₈, E₈) respectively. When such provision is made, the steel columns B₈, C₈ on grid lines (B) and (C), which then become internal columns, can be removed, if desired.

8.3.2 Wind Bracing

- a) When wind blows along Y direction (perpendicular to the ridge) it causes horizontal forces on columns A₂, A₃ etc. and also on columns E₂, E₃ etc. which are on the leeward side. The wind pressure acting on the external A.C. cladding of the building is transferred to the column causing high bending moment at the base of the column which mainly act as cantilevers fixed in the pedestals at Ground-level. This bending moment together with B.M. developed due to crane movement is resisted by these two legged structural-steel columns A₂, A₃ etc. in which tension is developed in the roof leg (i.e. outer leg L_o) and compression is developed in the crane leg (inner leg L_i) as shown in Figure 8.1 (e). The foundation of these columns are also to be designed for the axial load and the B.M. When the wind blows in opposite direction, compression is developed in the roof-leg of these columns A₂ and A₃.
- b) When the wind blows along X direction (parallel to the ridge), the following two provisions are generally made in the structural system at eave's level (E) at each end of the building so that columns are relieved of B.M. in this direction.

a) Horizontal Wind Girder and Vertical Wind Bracing

- i) Referring the layout plan and elevation of Figure 8.1, when wind blows on a gable-end columns A₈, B₈, C₈, D₈ and E₈ from Right to Left direction, the horizontal force on the cladding upto ridge level is partly transferred to horizontal wind-girder (W₁, W₂, W₃, W₄) formed with diagonal members (DH) and Ties (T) at tie-level or Eave's-level as shown in Figure 8.1 and partly to the column footings. Horizontal wind girder is assumed to be simply supported in the horizontal plane at top ends of two columns A₇, A₈ at one end and the two columns E₇, E₈ at the other end with a span of 21 metres. It undergoes horizontal bending with its depth (W₁ W₂ = 6 m) as shown in the Figure 8.1 (a).
- ii) Diagonal bracing (W₁ L₁), (W₂ L₂), (L₁ M₁) and (L₂ M₂) are provided in the vertical planes between columns A₇ and A₈. These are called as column bracings (D_v) and are also provided between columns E₇ and E₈. Thus the horizontal load on wind-girder at points W₁ W₂ etc. is transferred downward at points L₁ and L₂ in the first step by virtue of upper diagonals (W₁ L₁) and (W₂ L₂) and then to M₁ and M₂ near the foundation of the columns A₇ and A₈ in the second step through the lower diagonal members. Thus the column carries mainly an axial load and is relieved from effects of B.M. in it due to wind blowing parallel to ridge. Similar wind-girder and vertical bracings in both diagonal-directions are needed at the other gable end between grid lines (1) and (2) when length of workshop is more than 30 metres.

b) Concept of Duplicate Tie-system

It is important to note that generally the length of these diagonal members (L₁ M₁), (L₂ M₂) is quite large. Further for a given direction of wind (say from right to left) acting normally on face of gable on grid line (8), diagonal members W₁ L₁ and L₂ M₂ carry axial compression force. The slenderness ratio of these members being generally large, its resistance to compression force is quite low. Hence in the design practice it will be assumed that these members will not be considered to be effective in resisting any Lateral Load. Since other diagonal members (L₁ M₁) and (W₂ L₂) are in tension these can be easily designed of required area of cross-section without having any stringent limit for its slenderness ratio.

When the wind direction is reversed, the axial forces in the above referred diagonal bracings will reverse in sign. Again the member (L₁ M₁) carrying tension is designed for the full tension assuming that the other diagonal member L₂ M₂ carrying a very small compression force is almost a dummy member.

This system of wind resistance by the Tie-action of two diagonal members, only one of which is effective for a given direction of wind is referred to as "Duplicate-Tie System".

c) Stability of Trusses

The plane roof truss is fully designed for vertical loads in the plane of the truss while it is likely to be unstable under wind loads perpendicular to its plane. To achieve overall integrated action of the roofing system, diagonal bracing (Rafter-Bracing) in both directions are generally provided in the plane of roof sheets (called as Rafter-Plane also) in panels of the boundary bays along all the edges. In case the number of bays are quite more, such stiffening arrangement of diagonal bracing is repeated after about every 8 to 10 bays. During erection of trusses, contractors need to take special measures for its stability.

8.3.3 Horizontal Ties at Eave's Level

The ties or Tie Runners (T) of angle section or channel section are generally provided connecting the corresponding bottom joints of adjacent trusses. These may serve also to provide lateral stability and restraints against buckling of bottom-chord members (of the truss) since it becomes compression member under wind suction load, as discussed earlier.

Figure 8.1 shows Ties T_1, T_2, T_3 etc. in the layout plan.

Ties are thus not always required at all the roof joints. In zone of high wind velocity, it is necessary to consider the suction load on roof and design of bottom chord-member (i.e. Tie-member) may be governed as compression member needing lateral restraints to reduce slenderness ratio about both axes of the section.

Ties can also be designed, if required, for walkway/passages to go from one truss to other truss at tie-level. Walkway will be used only by a few people at a time for generally maintenance purposes with expected live-load of 1.2 kN/m^2 . Check for deflection is essential for larger spans.

8.3.4 Other Structural Members of Factory Building

Details of columns, gantry girder etc. will be discussed later at appropriate places.

8.4 ROOFING SYSTEM

Figure 8.1 shows roofing system for a typical factory shed.

This consists of

- 1) Structural Steel Roof Truss (TR) as shown in Figure 8.1 [its Elevation in part (c) and in part layout plan in Figure 8.1 (a)]
- 2) Purlins (P) shown in Figure 8.1 (parts a and c)
- 3) Ties in part plan in Figure 8.1 (part- a)
- 4) Horizontal Wind-girder ($W_1 W_2 W_3 W_4$) in (Part a) at eaves level.
- 5) Wind Bracing for columns in part (b) in the vertical plane.

The details of all these items are discussed below:

8.4.1 Structural Steel Roof Truss

Roof-truss is generally assumed to be a pin-jointed frame supported on columns at its ends as shown in a line diagram in Figure 8.2 (a) as well as Figure 8.2 (c). Sometimes a small overhang is also provided beyond the column to get a better cover/shed against sun or rain for windows and ventilators at higher level, as shown in Figure 8.2 (b). The internal members A.C. sheets on right part of centre line of the truss are symmetrically laid though not shown. The trusses generally have the bottom members in a horizontal line called as bottom chord member. Under the normal dead load of sheets, purlins trusses etc. and the imposed load (which is generally less than 0.75 kN/m^2) acting over the sheets the bottom chord member is in tension and is therefore, also termed as Tie-Member (T). However such a nomenclature as "Tie" is not rational since for many cases of trusses with low value of slope, when suction is created due to wind-load on the upper surface of sheet, the upward wind load on the truss can be more than the D.L. on the truss. In such loading cases the bottom chord member is in compression and does not act as tie. It is also found sometimes that design of these member as compression member under combination of wind-load and dead load is more critical than its design as tension member under D.L. and imposed liveload. This aspect is repeatedly indicated since it forms an important design considerations.

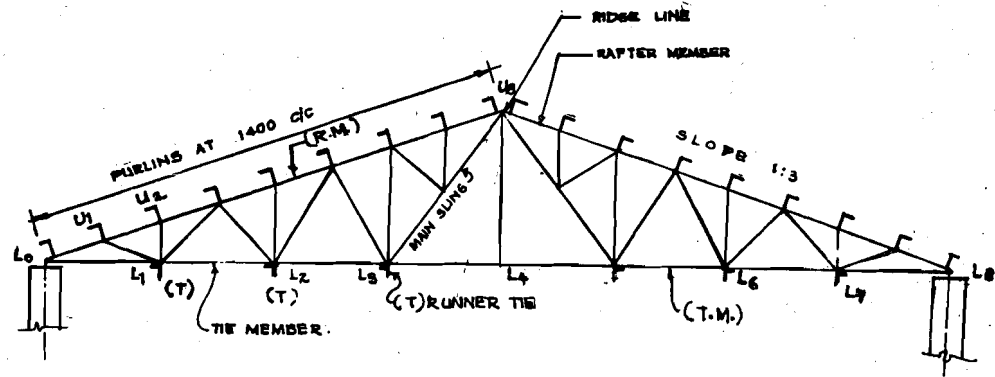


Figure 8.2 (a) : Typical Roof-truss with Ends on RC Columns

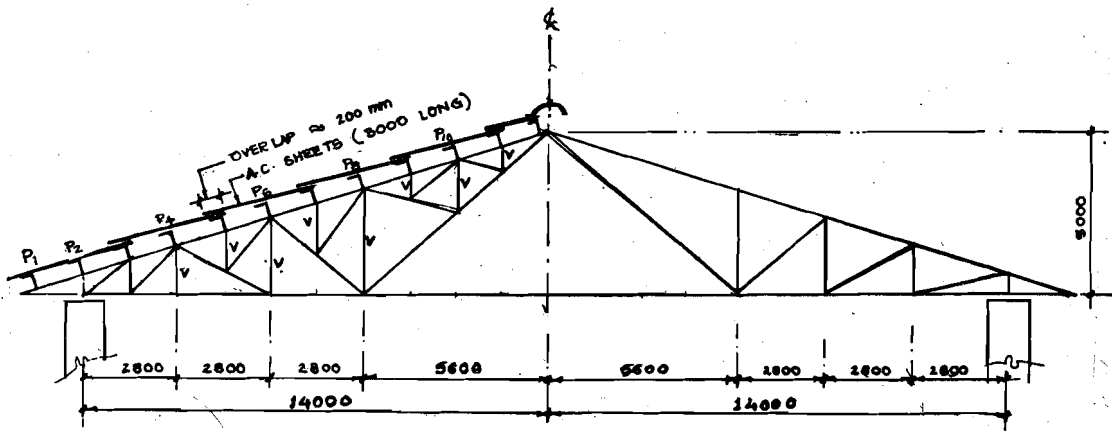


Figure 8.2 (b) : Alternative Internal Pattern of Truss Member with Shorter Vertical Member at Support

The top chord members (i.e. Rafter members) of the truss are generally provided with a uniform slope generally in the range of 1/5 to 1/2.5 i.e. about 11° to 26°. At places where rainfall is quite high it is better to provide a minimum slope of 1:3 from the point of view of avoiding percolation of rain-water and water-leakages. This slope is also desirable where wind velocities are larger than 44 m/sec. A map of India showing wind velocities in different regions is given in I.S. 875, Part III. In a few cases specially for long-span truss, the truss has a end-vertical member and a flatter-slope as shown in Figure 8.2(c).

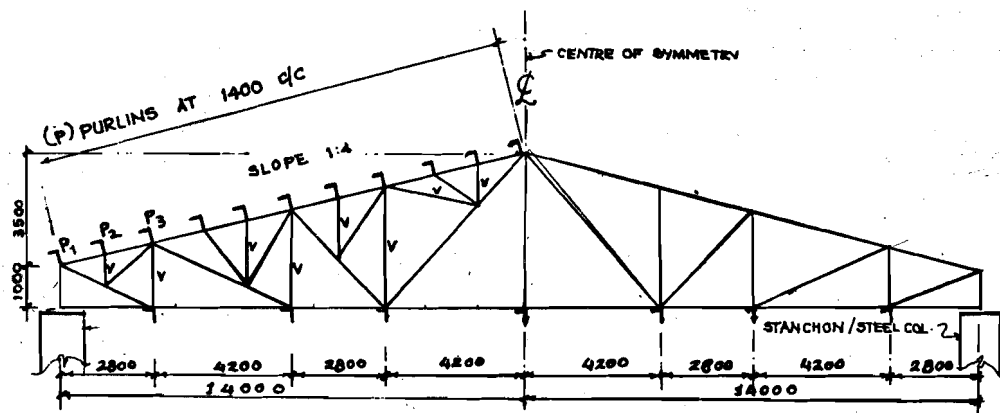


Figure 8.2 (c) : Alternative Internal Pattern of Truss Members with Shorter Vertical End Member

All internal members and purlins on the right-side of centre line of truss are not shown in the Figure. These are assumed to be provided symmetrically. In the industries where ovens/furnaces create flue gases, the outlet for the same can be created by providing monitor-system in trusses as shown in Figure 8.2(d). The top chord member of the above referred types of trusses generally carry large axial compression force under effects of D.L. + L.L. and hence designed as compression member under effects of combination of D.L. + L.L. In case of a truss which acts as a cantilever and supported at one end fully, the top-chord member will be in tension.

The internal members of the truss, which are either vertical or inclined are provided so that effective length of top chord compression member is reduced. Since the A.C. corrugated sheets are to be supported over a span of about 1.4 m, from the deflection criteria, it is preferable to provide the top chord joints at spacing of about 1.4 m so that purlins can be supported at these joints. Since the purlin loads are at the joints of the truss, the truss-members will carry only axial forces.

If the top chord joints are spaced at say 2.8 m c/c, and purlins are spaced at 1.4 m c/c, the top chord members will carry substantial bending moment apart from axial compression force. The design of top chord members will then become heavier, and the weight of such a truss is likely to be higher even though the weights of some of the internal members will be reduced.

Figure 8.2 shows the elevation of 50% (i.e. left half) of the truss on one side of centre line of symmetry central vertical member U8 T5 is also centre line of symmetry.

In order to draw the fabrication drawing of a truss, initially it is required to draw the axis-line (or centroidal axes) of all the members of the truss showing the location of point of intersection of axial forces. Such a point at the joint is called as work-point/node centre of the truss.

Knowing the distance of centroid from the top face/bottom face of the truss member (which is generally an angle-section), the top chord member and bottom chord members can be drawn on the drawing. The angle cleat about 200 mm long is generally welded to top face of top-chord member as shown typically in Figure 8.2 (a) and (d).

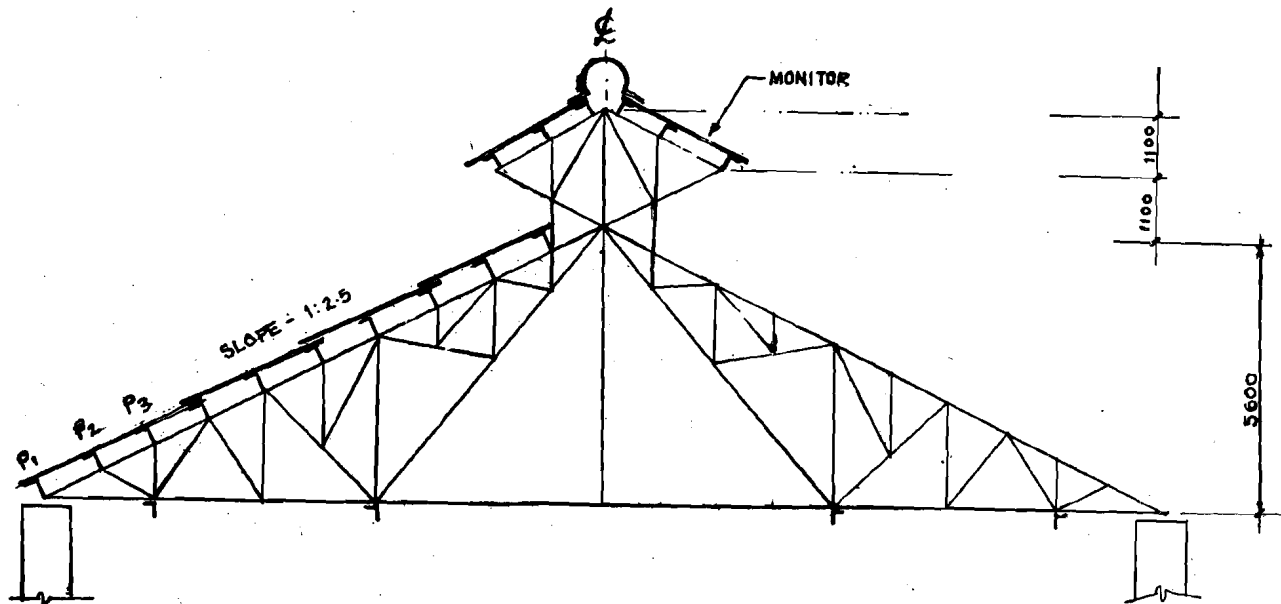


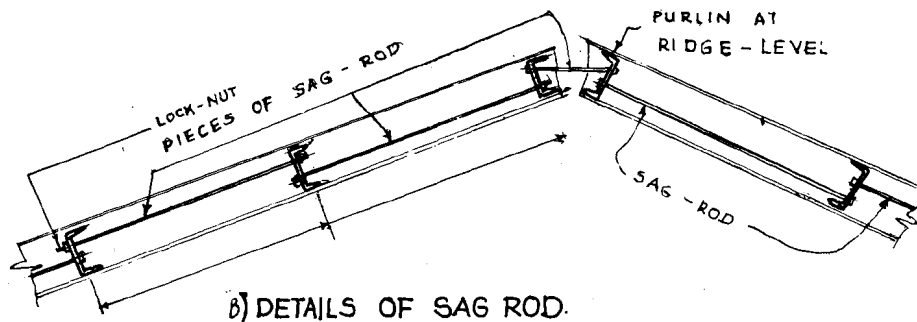
Figure 8.2(d) : Roof with Monitor for Industries Installing Oven/Burner

The angle/channel purlins are then supported on these cleat angles with a minimum of two bolts at each of its ends so that a small moment of resistance is developed at the interior support, of the purlin as a continuity moment.

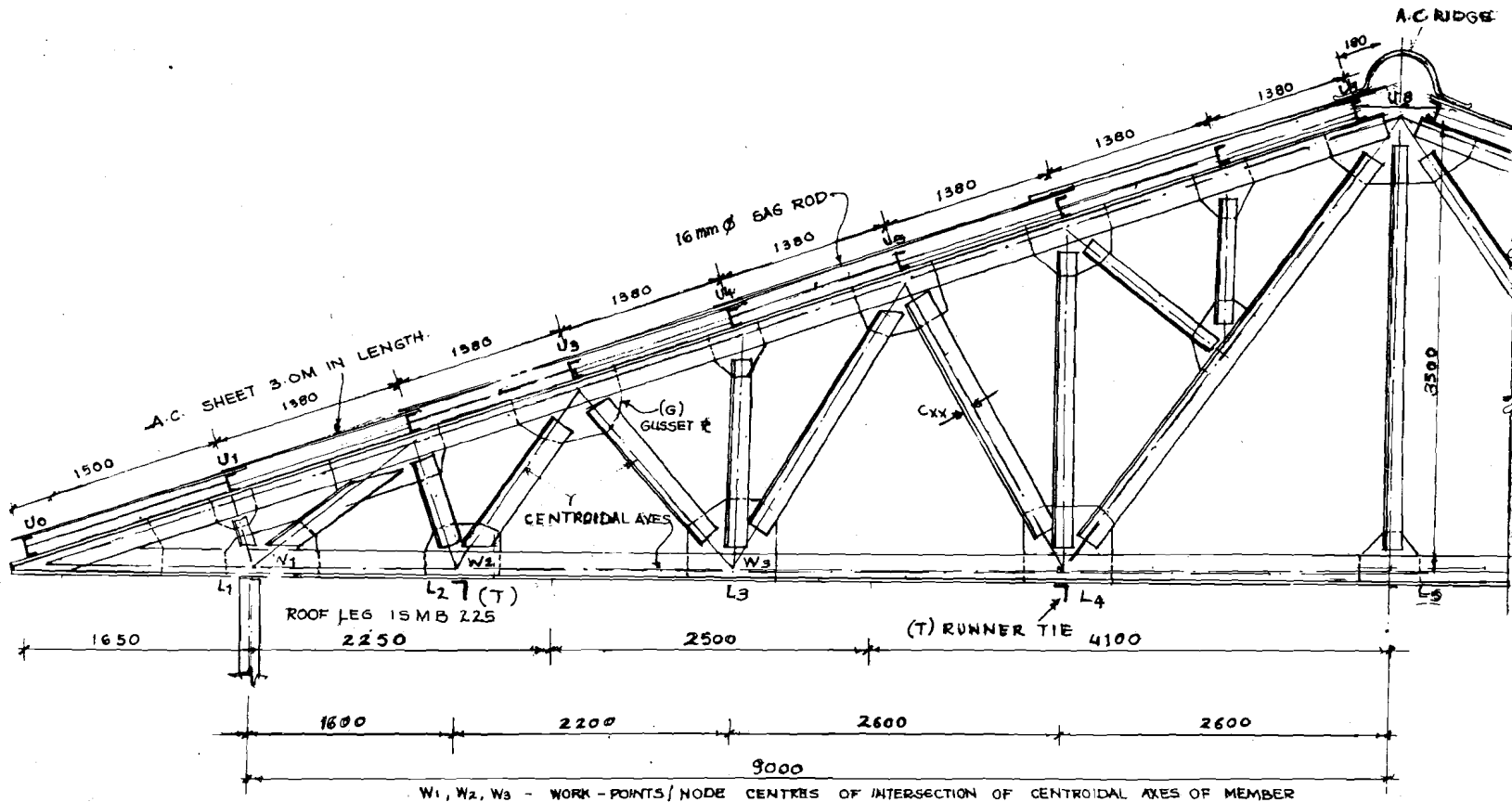
8.4.2 Detailing of Truss Members and Joints

Figure 8.3 shows truss wherein there are joints U_0, U_1, \dots, U_8 along top chord member and joints L_1, L_2 and so on in the bottom/lower chord member of the truss. It is seen that joints U_3 has four members ($U_2 U_3, U_3 U_4, L_2 U_3$ and $L_3 U_3$) meeting through a gusset plate.

Thickness of gusset-plate generally 2mm more than that of members connected to it. Similarly gusset-plates are generally required to make a good joint of various members so that adequate length of each member is lapped over the gusset-plate and welded to it. The length of the lap is generally designed on the basis of full strength of the member using generally 6 mm fillet weld. It is therefore required to keep clear distance between adjacent members on the gusset plate to facilitate welding on all possible edges.



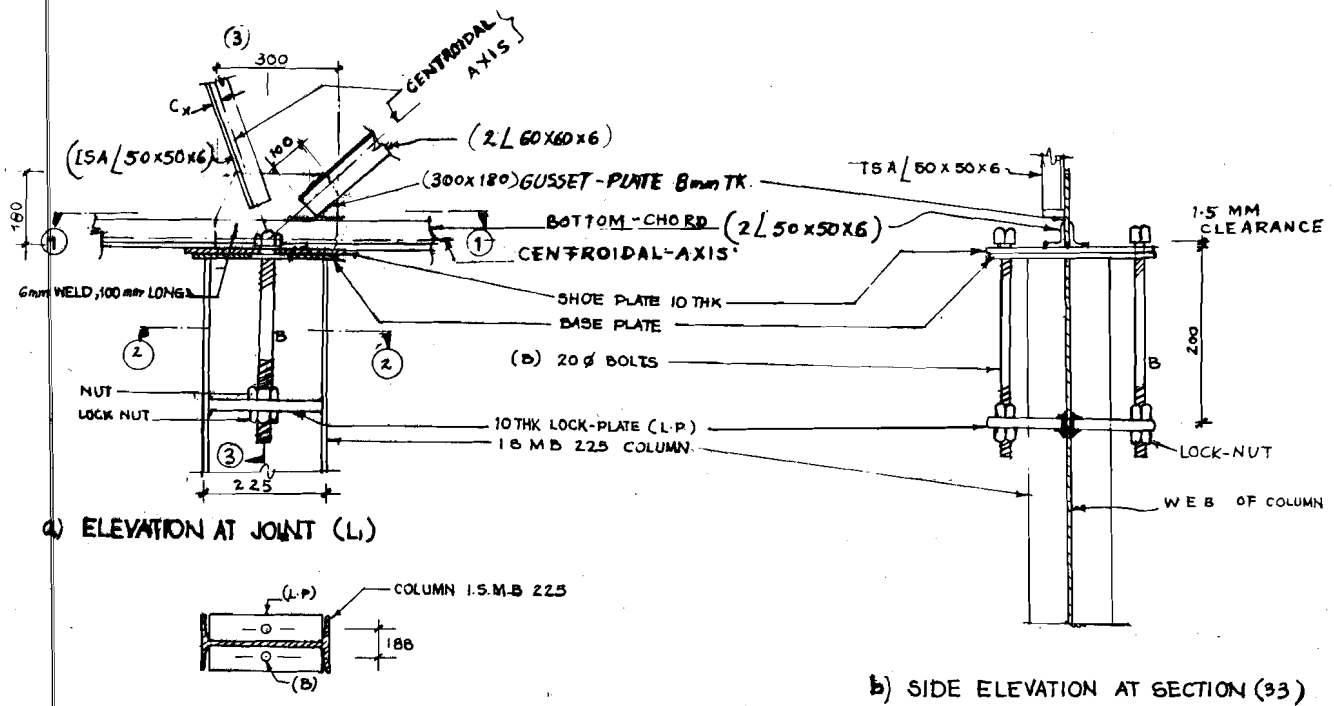
b) DETAILS OF SAG ROD.



a) PART OF ROOF-TRUSS SUPPORTED ON STEEL COLUMNS

Figure 8.3 (a) : Part of Roof-Truss Supported on Steel Column
 Figure 8.3(b) : Details of Sag Rod

Figure 8.4 shows typical details of the truss joints (L₁) supported on the column. It is intended to show a roller support at (L₁). The bottom chord member of the truss is provided with a shoe plate welded to it. The top of the column is provided with a cap-plate (or base-plate) welded on its top as shown in Figure 8.4 (a). Base plate is simply resting on the cap-plate. It is also



d) PLAN AT 2-2

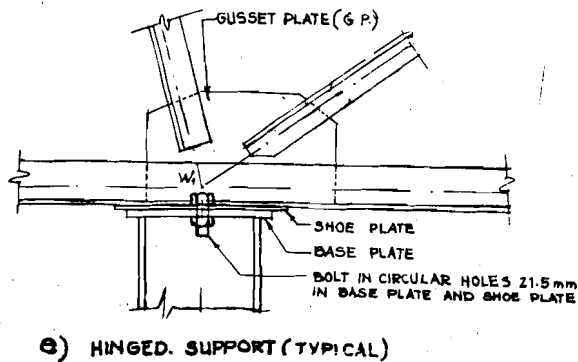
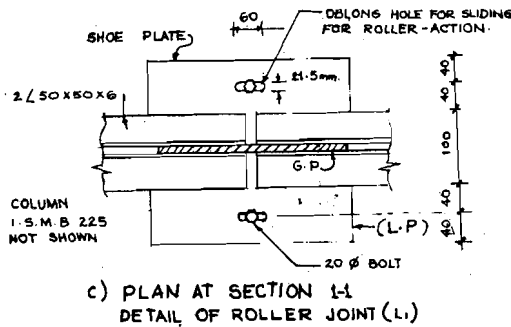


Figure 8.4 (a, b, c, d) : Detailing of Roller Joint (L₁) of Truss on Steel Column

seen that a lock plate is welded to steel column at distance of about 200 to 250 mm below the cap-plate so that anchor bolt provided in shoe plate and base-plate is held fixed by nut and lock-nuts applied on the lock plate as seen in Figure 8.4 (a). The hole provided in the shoe plate is elongated (or oblong hole) with its length about (3) times the bolt diameter. The bolt head of the bolt is kept above the shoe-plate with a clearance of about 1.5 to 2 mm so that shoe-plate can slide over the cap-plate smoothly, to allow for expansion/contraction of truss-length during summer/winter.

Generally, point loads from purlins are transferred on the top-chord joints of the truss and under such condition members of the truss carry only axial-forces either tensile or compressive in nature. The strength of any tension member is given by

$$\bar{T} = \bar{\sigma}_{at} \times A_{eff}, \text{ or } T_1 = \bar{\sigma}_{at} \times (A_{net})$$

Generally, as per I.S 800 the permissible tensile stress ($\bar{\sigma}_{at}$) under axial tensile load is 150 N/mm^2 (i.e $0.6 f_y$). The shape of tension members in the roof truss is generally angle section either single or double angle as shown in Figure 8.5 (a). The members are connected at the joint through a gusset plate. The tension load capacity of the member depends on its net area (A_{net}) which is the effective area of section. The effective area is computed as follows;

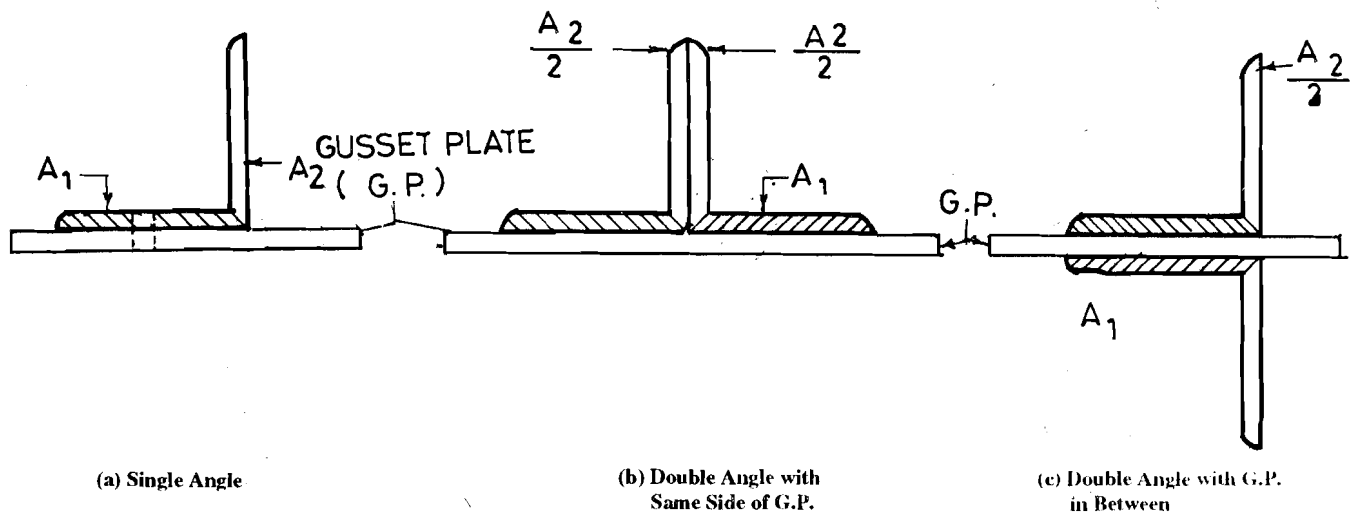


Figure 8.5 : Connection of Tension Member

- a) For a single angle section,

$$A_{net} = A_1 + (K_1) A_2$$

where,

A_1 = Net cross-sectional area of connected leg shown shaded, and

A_2 = The net cross-sectional area of the outstand (i.e. unconnected leg which is perpendicular to the gusset plate). In case there is no hole in the outstand, it is the gross-area of cross-section of the outstand.

$$K_1 = \frac{3A_1}{3A_1 + A_2}$$

N.B : Note that if ed connection is bolted, net Area (A_1) is the shaded position.

- b) In case of a pair of angles back to back (or a single tee) connected by one leg of the angle to the same side of a gusset plate as shown in Figure 8.5 (b) when the two angles are tacked to each other by rivets/weld with spacing less than (1000) mm

$$A_{net} = A_1 + (K_2) A_2$$

where,

$$K_2 = \frac{5A_1}{5A_1 + A_2}$$

- c) In case of double angles placed back to back with gusset plate in between the two as shown in Figure 8.5 (c) provided members are suitably tacked to each other all along their length by Tack bolts/weld at distance less than 1000 mm center to center.

$$A_{net} = A_1 + (K_3) A_2$$

where,

$$K_3 = 1$$

It is to be noted that $K_3 > K_2 > K_1$

The truss-member which carry compression force are also generally angle section and are dealt with separately later in the unit of structural steel column.

8.4.3 Purlins

Refer Figure 8.6 showing layout of purlins, their connection with truss members and Roof-Sheets. The purlins are the structural steel members directly supporting the A.C. sheets or G.I. corrugated sheet of the roof. Normally, the length of sheets available in the market are 1, 1.5, 2, 2.5, 3 m. The corrugated A.C. sheets cannot span more than 1.65 m c/c from the deflection criteria of the structural design consideration. Considering both these aspects maximum span of the sheet is generally adopted as about 1.4 metres so that a sheet of 1.5 metre length covers one span along with a total lap of 100 mm with the adjacent sheets, while a (3) metre long sheet as shown in Figure 8.6 (a) "J bolts" of 6 to 8 mm diameters are used to fix up the A.C. sheets to the purlins. Generally, angle section purlins are used when the spacing of the trusses are upto (4) metres. When the spacing of the trusses is more than or equal to 4.0 m, the channel section is generally preferred. In case the spacing of the trusses exceeds 8.0 m or 50 it is preferable to provide a built up purlin made either of two angles, one at top and one at bottom with a M.S. bar/mild steel angle and lacing members placed diagonally connecting the two angles at a series of equidistant points. Angle of diagonal member with the horizontal varies from 45° to 60° . As an alternative to this a three angle purlin forming a triangular configuration with two angles at the top at a spacing of 1.2 m to 1.4 m and one angle at the bottom connected with diagonal lacing members of m.s bar on all the three faces.

Detailing of Purlin-connection with Truss and Sheets

Figure 8.6 shows the details of A.C. sheetings and purlins as supported on the top chord member of the trusses. Figure 8.6 (a) shows a section of purlin mounted at approximately on the truss joint through an angle cleat welded on the topchord in direction across the truss. The centre of a typical truss joint marked as W_1 is a point intersection of the centroidal axes of the various members of the truss. This point W_1 is sometimes referred to as node centre or the work point. The details of the angle cleat whose length is generally not less than about 160 mm and welded on the top chord members of the truss, is shown in Figure 8.6 (d). Each purlin is generally discontinuous over the trusses and connected to their cleat angle by means of at least two bolts generally 12 mm or 16 mm diameter as shown in Figure 8.6 (d). The advantage of providing two bolts is to provide partial continuity effect on interior truss-joints, so that deflection as well as bonding-moment at mid-span of purlin are reduced. Sometimes a fish plate of 6 mm to 8 mm is also provided to stiffen the joint (i.e. to provide more amount of fixity at the joint). It is to be noted that bolts are not provided at location where cleat angles are welded to purlins. In view of this, the bolts shown in Figure 8.6 (a) are shown dotted and are not indicative of their location. The exact locations of the bolts are shown in Roof-plan layout in Figure 8.6 (b) and (d).

In case of channel purlin, the moment of resistance in the plane of the web is quite high as compared to that of single angle purlin, while the M.R. in the plane parallel to flange or to roof-sheets is quite low. In this case, the span of the channel section can be divided into equal subdivisions of suitable value by providing "Sag-Rods" bolted to each purlin as shown in Figure 8.5 (b) so that purlin acts as continuous beam for bending considered in the plane of flange with a span equal to distance of this subdivisions (i.e. distance between sag-rods). This helps to provide an economical section of the purlin. The reaction for the inplane component of the load on lower most purlin is taken by the sag-rod and is transferred on the next purlin at higher level. Thus loads carried by upper sag-rod is a cumulative addition for reaction of all purlins below it. The two purlins on either side of the ridge can be connected by the sag-rod so that loads on sag-rods on the two slopes are balanced. In case of a North-light type of truss-system, the topmost sag-rod could be taken up inclined towards the top end of truss and horizontal load component is transferred to the framed structure.

8.4.4 Side Cladding of Industrial Structure

In Industrial structures, generally steel columns are provided and the external face of the building is covered with A.C. or G.I. corrugated sheets. Figure 8.7 shows the typical details of such side cladding of the building formed by A.C. sheets supported on horizontal side cladding runners (similar to purlins of the trusses). Figure 8.7 (a) shows a typical layout plan of the structural steel columns along the external face of the structure. Angle cleats of suitable size are welded to the external face of the steel columns at a spacing of about 1400 mm centre to centre and side cladding runner which are generally channel section or angle section are supported over the cleats by means of either the bolted connection as shown in section X₁-X₁, or by the welded connection. The corrugated A.C. sheets are supported over the side cladding runners by means of J bolts as shown in section X₂-X₂. The vertical sheets are generally 1 m wide and a suitable lap between the two adjacent sheets is provided as shown in part plan of the Figure 8.7.

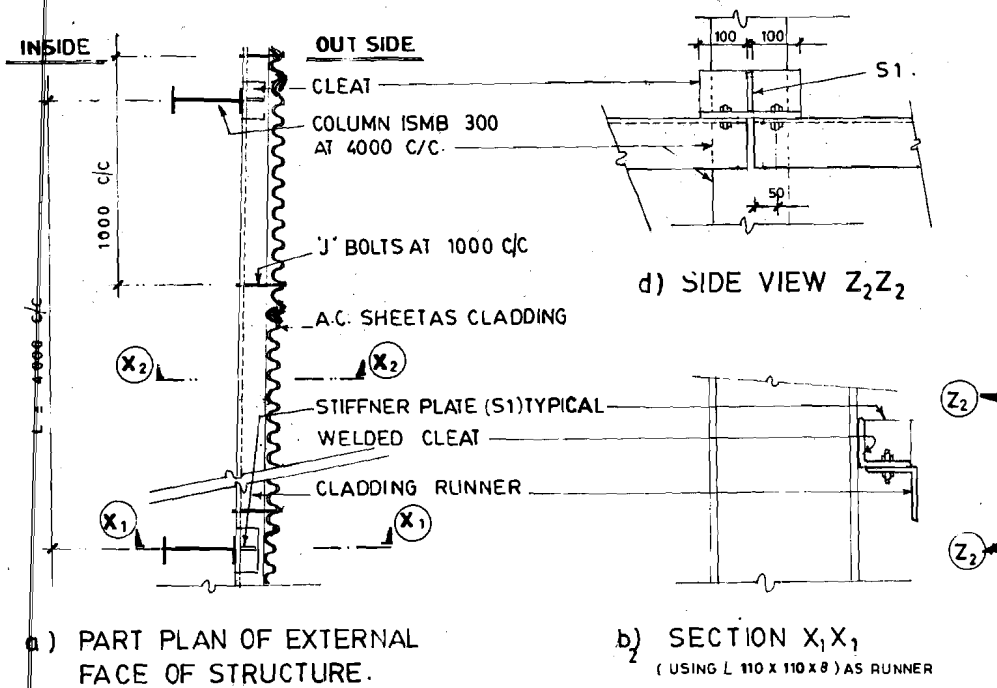


Figure 8.7 (a,b,d) : Details of Side Cladding

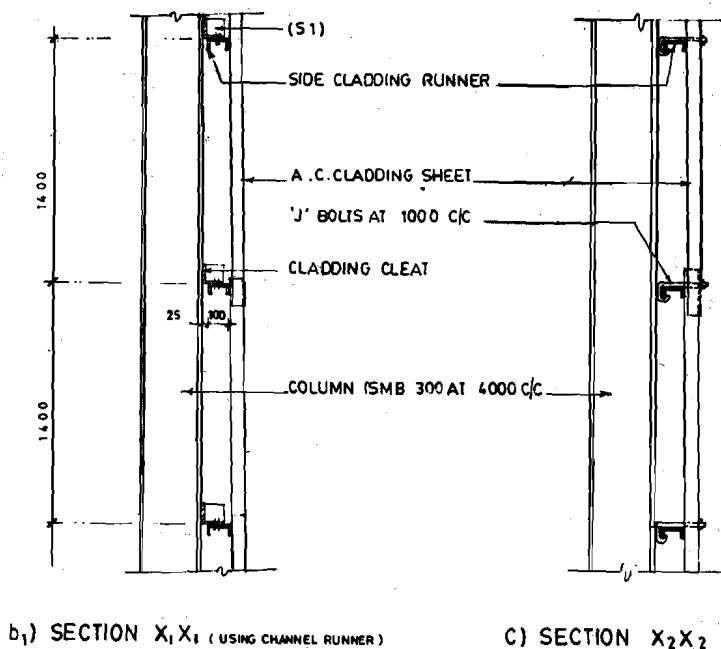


Figure 8.7 (b₁, c) : Details of Side Cladding

For the purpose of design of the side cladding runners, wind load is considered in a horizontal direction based on the velocity of wind at the given place. Wind load is transferred from the A.C. sheeting to the side cladding runners. The span of the A.C. sheets is generally considered to be 1.4 m in vertical direction as shown in section X₁-X₁. The horizontal wind load is then taken up by the side cladding runners in horizontal bending with a horizontal span of (L_r)

which is also equal to the centre to centre distance of the columns as shown in the figure. When the span is less than or equal to about 3.5 m to 4 m angle purlins are provided. For larger spans generally channel section is preferred, mounted with web horizontal as shown in Figure 8.7 (c) and (d).

8.4.5 Ties and Wind Bracings

Ties and bracings are provided in the roofing system to stiffen the same against horizontal forces and uplift forces due to wind. The bottom chord member of a truss carries a tensile force under the condition of dead load and live load acting on it in the downward direction. However, due to the uplift force caused by wind suction effect, when the wind blows parallel to the ridge of the roof, this bottom-chord member carries an axial compression force under which it is likely to buckle in the horizontal plane. Ties T₁, T₂, T₃ as shown in the layout plan in Figure 8.1 help to prevent buckling of the bottom chord member. The top chord member of the adjacent trusses are also connected by means of diagonal bracing members in the plane of the roof sheets. This system of bracing is referred to as Rafter-Bracing as shown in part-plan of Figure 8.1. Many of the aspects of ties and bracings are already covered under the article wind load/earthquake load in section 8.2.

Apart from the aspect of instability of the roof trusses discussed above, the instability of the overall structural framework is also required to be studied. For this purpose many times horizontal wind girders are provided in various floor levels, wherever possible, along a given horizontal direction so that the horizontal force on the floor is transferred to the end-vertical planes of the building. Diagonal wind bracings are provided in these vertical planes along the column lines at each end of the wind girder so that these vertical planes of diagonal bracings are perpendicular to the horizontal plane of the horizontal wind girder. Such a system of bracing is already discussed in detail in section 8.2 and indicated in drawing of Figure 8.1.

8.4.6 Fixtures on A.C. Roof

Figure 8.8 shows details of fixing A.C. half-round gutter at the edge of the sloping roof. The edge purlin is used to fix the metal clamps over it and thus the gutter is supported over a series of such gutter metal clamps.

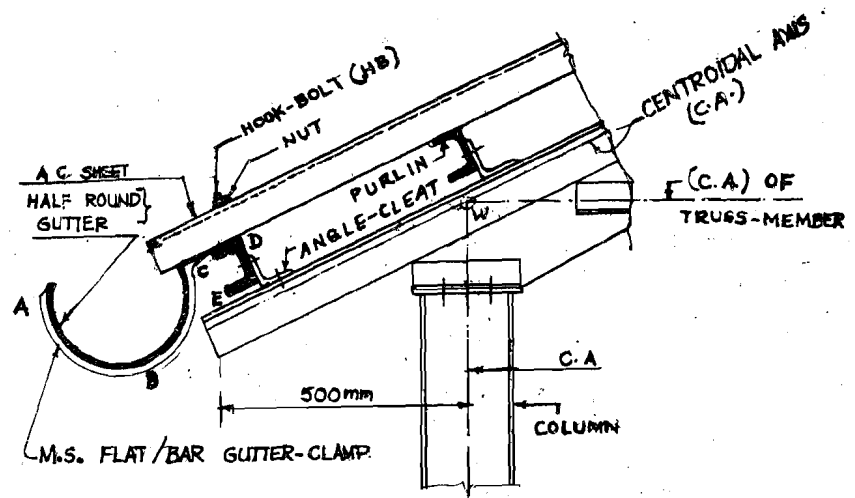


Figure 8.8 : Details of Half Round Gutter

Figure 8.9 shows how north light end of the truss is protected from rains by means of northlight-curves fixed by standard (J) bolts over the purlins. It is also indicated how glazing frame is supported over the runner angles which in turn are supported on vertical end members of north light truss.

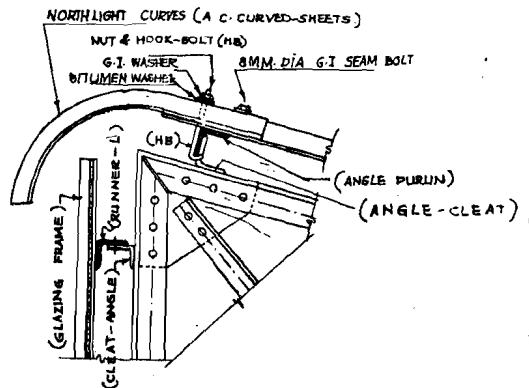


Figure 8.9 : Fixing of North Light Curves

Figure 8.10 shows how A.C. ridge can be constructed at the crown edge of the roof-truss. Adjustable ridges are also available so that (J) bolts can be conveniently located.

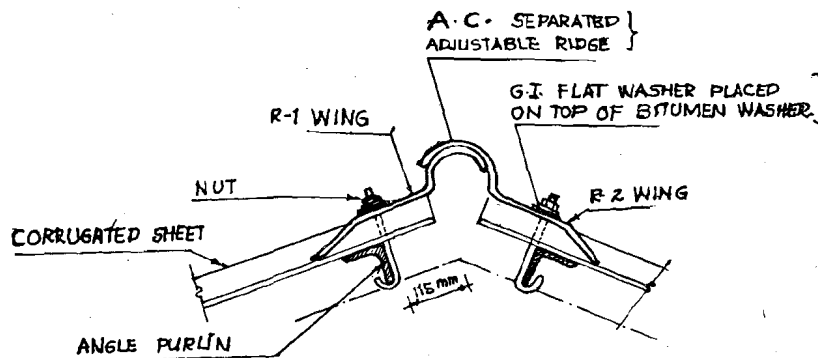


Figure 8.10 : Typical Details of Adjustable Ridger at the Ridge Line of Truss

SAQ 1

- i)
 - a) Distinguish between Dead load and Imposed live load.
 - b) Referring I.S. 875, state the dead load of following material in square metre/cubic metre.
 - 1) Bricks (common clay bricks) 220 mm × 110 mm × 75 mm
 - 2) Brick masonry with cement and sand in proportion 1:3 with overall thickness of 260 mm including plaster of 20 mm thickness on each face.
 - 3) Cement (ordinary portland)
 - 4) Weight of fully filled cement bag
 - 5) Reinforced cement concrete
- ii) Which are the places in India where wind velocity is maximum? What is the magnitude of intensity of wind pressure in these places?
- iii)
 - a) Draw a typical detail of a purlin supported on Truss.
 - b) Draw a typical detail of a purlin supporting A.C. sheets.
- iv)
 - a) Indicate the need of wind-bracings in vertical plane.
 - b) What is meant by Duplicate Tie system?

8.5 FLOORING SYSTEM

Flooring system consists of floor and the supporting beams. There are two types of floors generally used in the industries :

- 1) Chequered Plate Floor,
- 2) Grating Floor.

Sometimes R.C.C. slab is also used as floor. The floor referred above is supported on secondary or main beams. Finally, beams are supported on the columns.

8.5.1 Floors

- a) **Chequered Plates:** Chequered plates are either plain mild steel plates or mild steel plates with top surface provided with slurrations or shallow notches of about 1.5 mm depth having various patterns – diamond shaped pattern being more common. Generally plate sizes of 1m width are used over the steel beams of channel section or (I) sections. The thicknesses of these plates are generally 8 mm to 12 mm depending upon the span which can vary from 600 to 1200 mm.
- b) **Gratings :** Gratings are fabricated steel floors from flats. Span of the gratings can be varied from 1 m to 2 m and the length of the grating can be chosen in the range of 2 m to 3m. Grating consists of two series of flats called as
 - 1) Main member, and
 - 2) Secondary member.

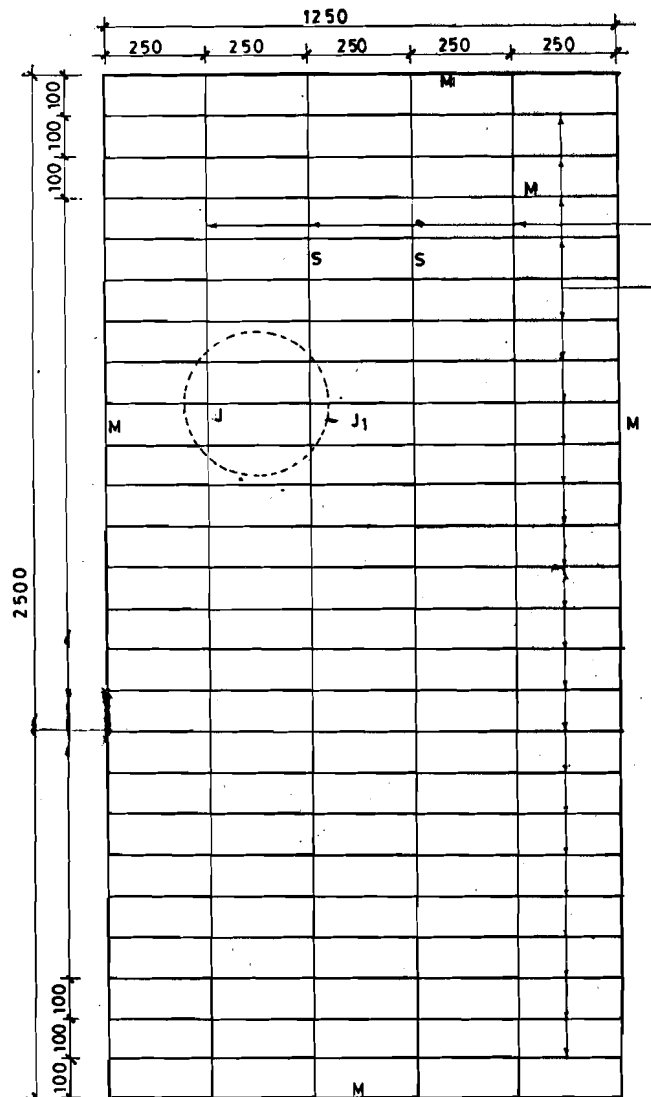


Figure 8.11 (a) : Layout of Member in Grating

Main members, as shown in Figure 8.11 (a, b, c), are generally the flats of thicknesses 4 mm to 6 mm, depth from 25 mm to 50 mm and at a spacing of about 65 mm to 100 mm.

Secondary members are generally lighter sections of thickness 3 mm to 4 mm, depth varying from 10 mm to 15 mm and spaced at 200 mm to 300 mm centre to centre. The details of grating along with a typical joint (J) of the main member of size 35 mm \times 4 mm with the secondary member 12 mm \times 4 mm are shown separately in plan and section in Figure 8.11 (b) & (c) respectively.

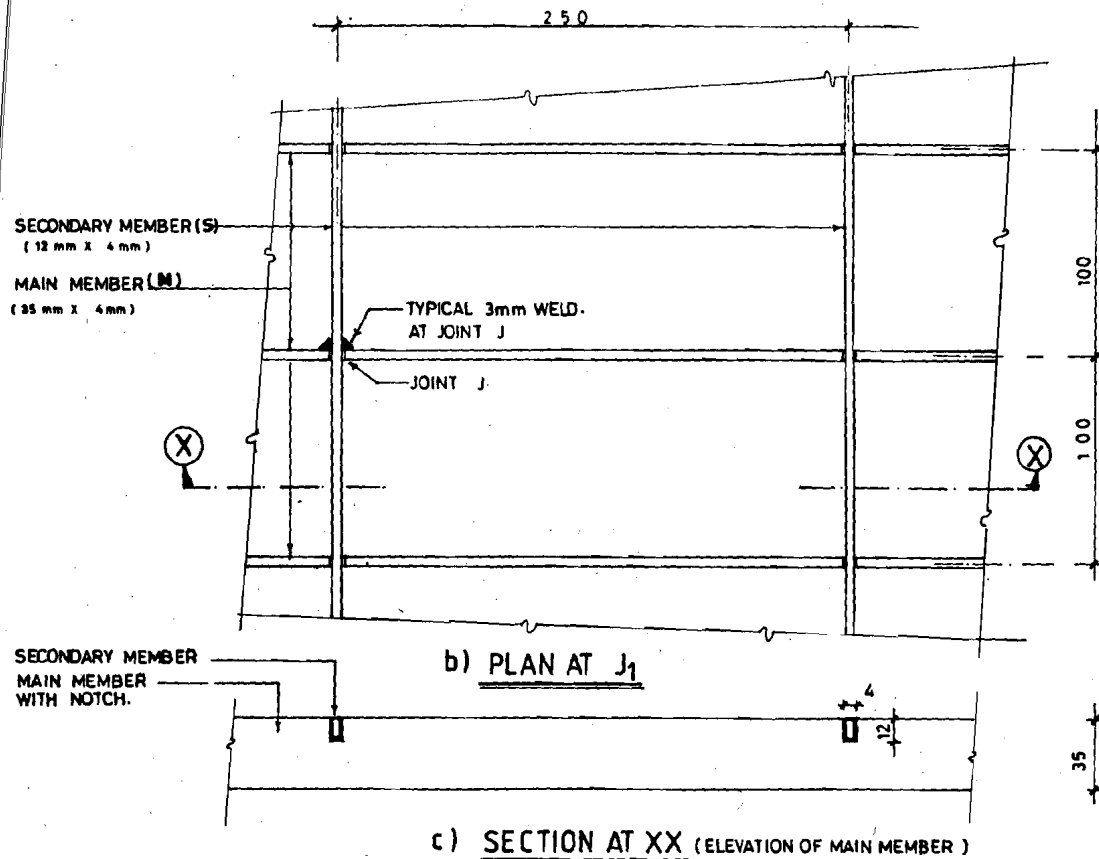


Figure 8.11 (b, c) : Grating

In this case rectangular notches of size 12 mm \times 4 mm are provided at a spacing of 250 mm centre to centre on the top portion of the main member so that the secondary members can be placed fully inside the notch and fillet weld is provided on atleast two sides of the join J_1 as shown in the plan. Secondary members are useful to provide lateral restraints to main member which carries load in vertical bending.

- c) R.C.C floor slab can also be provided over the steel beams. In such cases the spacing of the steel beams can be increased to around 2.5 m. Figure 8.12 (a) shows the layout plan of a typical industrial building having steel columns and steel beams. As shown in the figure the columns are spaced at 6 m c/c in 'X' direction and 8 m c/c in 'Y' direction. The columns carrying heavier loads and of larger heights are generally two-legged columns (I)-sections or channel section suitably laced/battened. The steel columns carrying light loads or medium loads can be of single (I) section also. It is observed in the Figure that the columns along grid line (B) are of double channel section while those along grid lines (A) & (C) are only single (I) section. Referring to section Y_1-Y_1 in Figure 8.12 (b) it can be seen that beams of MB 450 are provided at 2 m centre to centre and R.C.C. slab is laid on the top of these beams. In order to provide restraint to the compression flange (i.e. top flange) of the beams shear connectors (SC) of 12 mm diameter bars are provided as shown in the section Y_1-Y_1 in Figure 8.12 (b) and (b₁). The bottom portion of this shear connectors are welded to the top flange. Such shear connectors can be provided at a spacing of 600 mm centre to centre along the length of the beam.

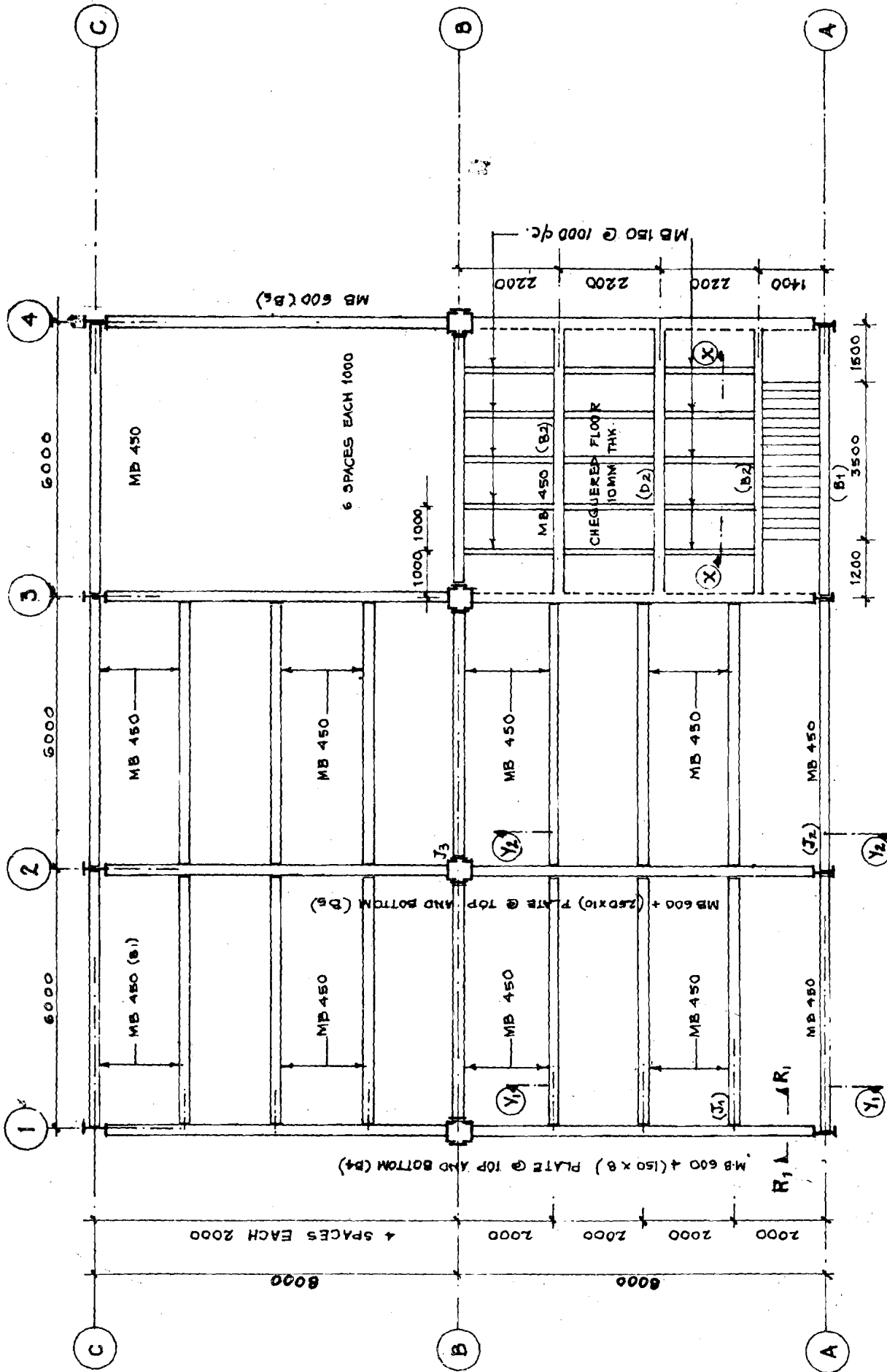


Figure 8.12(a) : Layout Plan of Stenctions (Column, Beam, Chequered Plates) & R.C.C Slab at a Typical Floor of Industrial Building.

The restraint for the compression flange of the (I) section can also be provided by embedding only the top flange in the R.C.C. slab. In the part layout plan between grid lines (3) & (4) chequered floor are shown over the steel beams ISMB 150 spaced at 1 m centre to centre. Section X-X is drawn separately in Figure 8.12 (c) to indicate how the chequered plates are placed on the top of the (I) beams either with welded connections or with bolted connections.

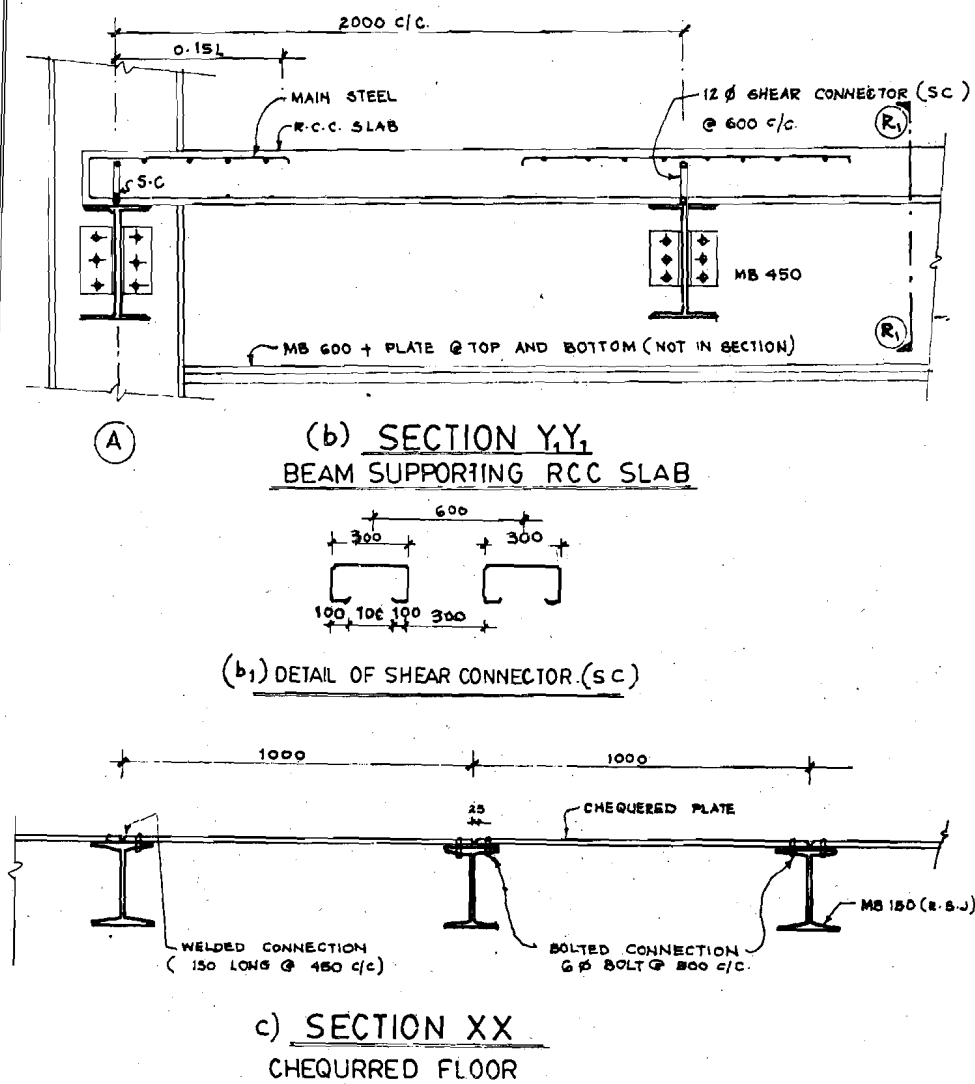


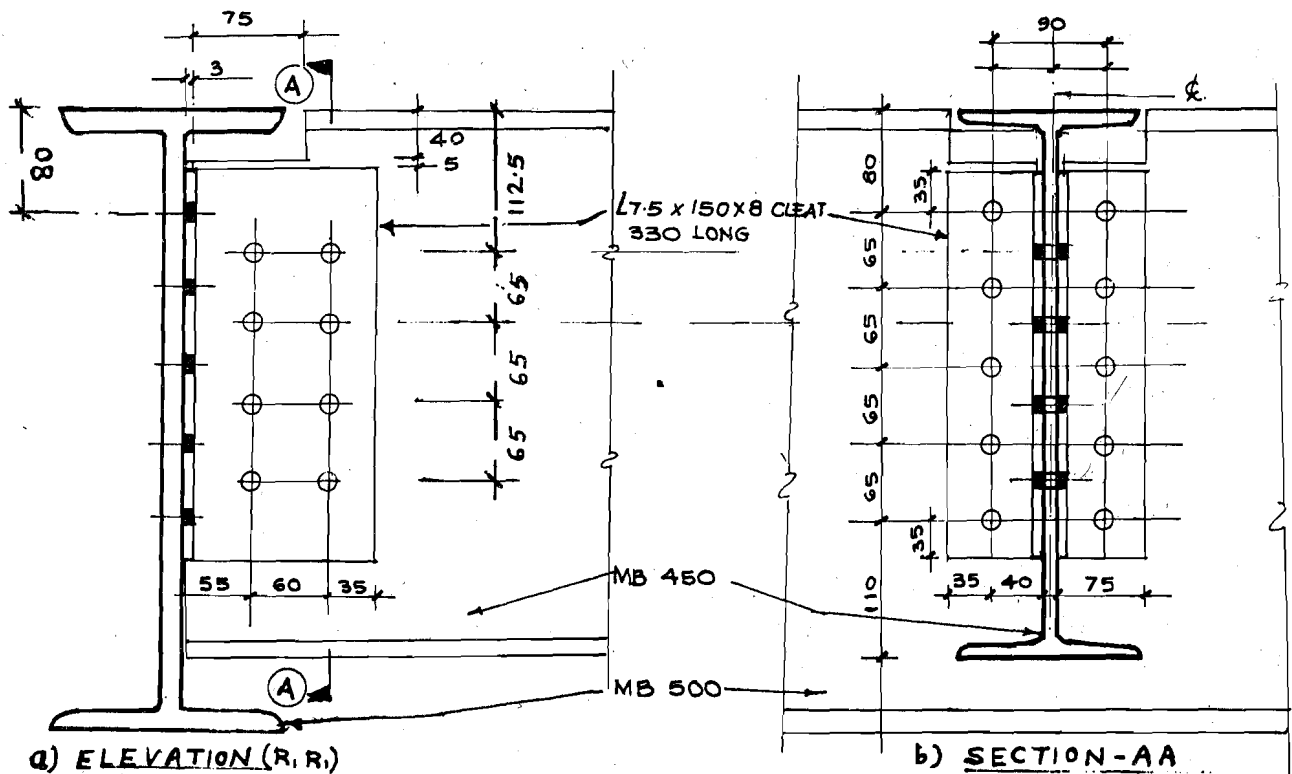
Figure 8.12 (b, b1, c) : Typical Structural Steel Floor System

8.5.2 Steel Beams : Beam to Beam Connection

As seen in Figure 8.12 (b), steel beams ISMB 450 are provided at a spacing of 2 m centre to centre. These beams carry the loads coming from the R.C.C. slab and any other machine load or wall loads placed over the floor. Span of these beams is 6 m and these beams transfer loads to the main beams at their ends shown typically as joint J₁. The main beam has a span of 8 m between grid lines (A) to (B) or between grid lines (B) to (C). The external main beams along grid line (1) will also carry either the wall load or the load from side cladding. Thus the section required for main beams is quite heavier. For span of 8 m, ISMB 600 with additional plate of 150 mm x 8 mm welded to its top and bottom is provided as indicated in plan of the Figure 8.12. The actual sizes of beams are to be designed on the basis of maximum bending moment and shear-force occurring at critical section as also on the permissible values of deflection. The sizes indicated in the above referred figures are assumed for the sake of completing the detailing works. The typical detail of beam to beam connection is shown in Figure 8.13 (a, b) which is very much similar to view R₁ R₁. It is observed that the top faces of the secondary beam ISMB 450 and the (main - beam) at the joint J₁ are kept at same level. Cleat angles are generally provided to connect the webs of the secondary beams and the main beam by means of either the bolted connection or the welded connection Figure 8.13(a) and (b) shows the two view of the details of typical bolted connection. It is required to note the following points.

- 1) Spacing of bolts to be not less than $(3 \times \text{dia of bolt})$.
- 2) Minimum distance of the top-most bolt below the top flange.
- 3) Minimum horizontal distance of first bolt line from web of beam.
- 4) Refer details such as distances g and g_1 of I Section given in Handbook No. (I-S).

It is also seen that some part of the top flange of the secondary beam is required to be cut so that web of the secondary beam is extended upto the web of the main beam. The above referred detail is typically called as a beam to beam connection.



(a) Elevation (R₁ R₁)

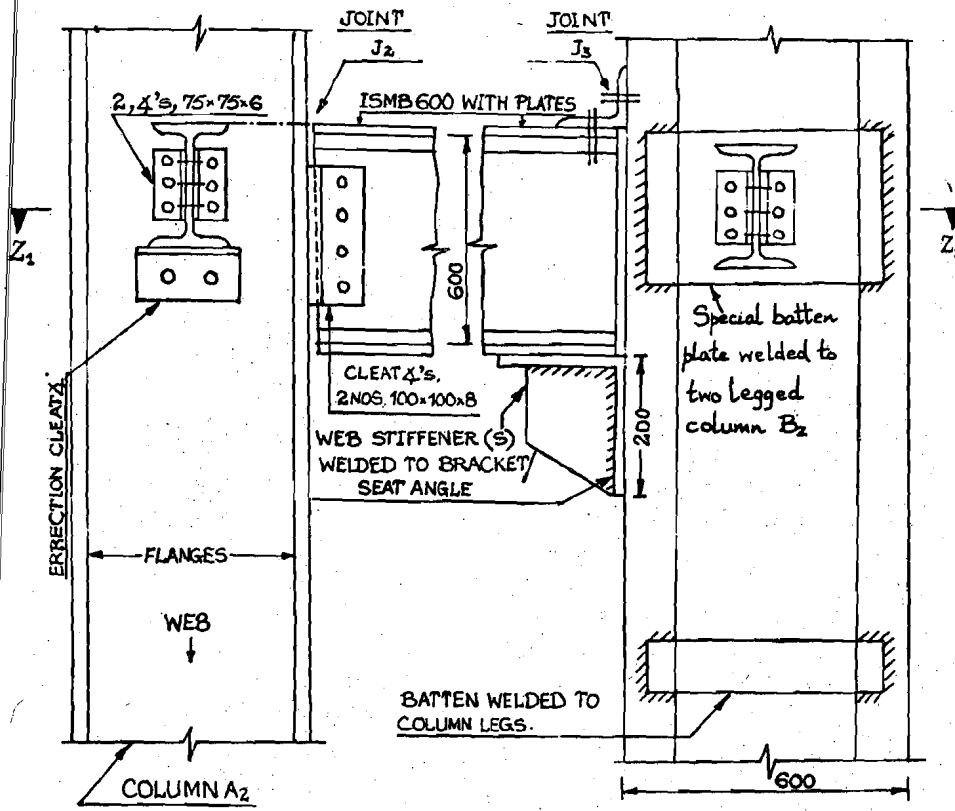
(b) Section AA

Figure 8.13 : Beam to Beam Connection with 20mm ϕ Bolts in 21.5 mm Holes

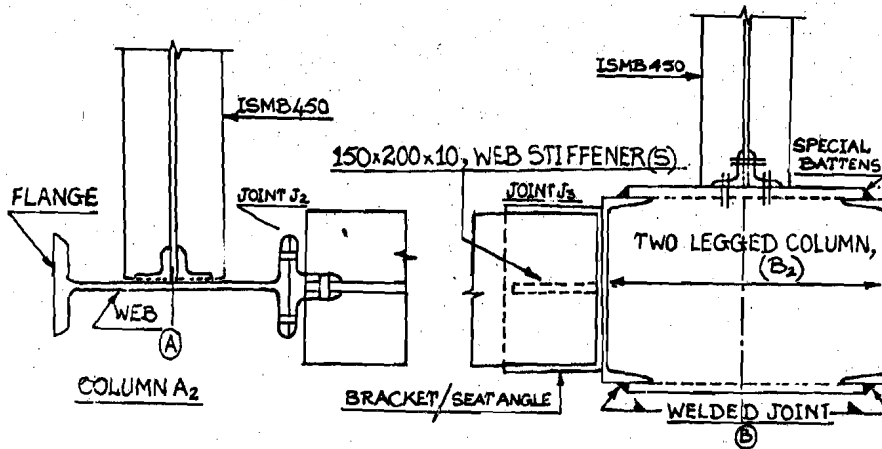
8.5.3 Beam to Column Connection

Figure 8.14 shows a section Y₂-Y₂ and its part plan of the flooring system shown in Figure 8.12. At the left end J₂ of the beam, the main beam is connected to the flange of the column of (I) section while at the right end of the beam, it is connected to the web of the channel section column. In general cleat angles are provided on either side of the web of the beam by means of bolts (or fillet weld) as shown in the figure. The location of the bolts are typically indicated both in elevation and in the plan details in Figure 8.14 considering the section Y₂ Y₂. The end beams ISMB 450 along grid line (A) are connected to the web of the (I) section column while the interior beam ISMB 450 along grid line (B) is connected to the double channel section of the column through a special batten as shown in elevation and sectional plan on (Z₁-Z₁).

Reference can be made to structural steel sections given in ISI Handbook for structural Engineers. A typical detail of the beam to beam connection requires the minimum distances of the bolts in the clear angles from the top face of the flange of the beam. This distance is marked as g_1 in Table 8.1 of dimensions and properties of Rolled steel beams. The distance marked g between the centre lines of the bolt-holes in the flange of the (I) beam is also given in the Table 8.1. The minimum centre to centre distance of the bolts along a given line can be taken as three times the diameter of the bolt. For the purpose of ease of construction and tightening of the bolted joint the location of the bolts in the web of the main beam can be staggered with respect to the locations of bolts in the secondary beam.



(a) ELEVATION ALONG Y_2-Y_2 .



(b) Part Plan between (1) and (2) on Z_1Z_1

Figure 8.14 : Typical Details of Beam - Column Connection at Joint J_2 and J_3

8.5.4 Design Philosophy

Since this chapter is not for study of the design of steel-members it is intended to provide only the guidelines of design.

Following steps should be considered for the purpose of design of the beam member.

- 1) Compute all the vertical loads and their locations on the beam.
- 2) Compute the magnitudes of the horizontal loads, if any and their locations.
- 3) Check up whether the beam is subjected to torsional moment.
- 4) Determine the value of maximum bending moment, maximum shear and maximum torsional moment if any.
- 5) Determine the maximum bending stress at critical locations under combined action of vertical bending moment and horizontal bending moment.
- 6) Determine the maximum bending compressive and tensile stresses and check whether these are less than the permissible bending compression for compressive

stresses and permissible bending tensile stress for tensile stresses. The value of the maximum permissible bending stress in tension can be taken as $\sigma_{bt} = 0.66 f_y$ as recommended in IS 800-1984.

- 7) Compute the maximum shear stress and check whether it is less than the permissible shear stress. Value of the maximum permissible shear stress can be taken as that recommended in I.S. Code.

$$\tau_{vm} = 0.45 f_y$$

where,

$$f_y = \text{Yield stress of the steel.}$$

The average shear stress calculated on the cross-section of the web of the beam shall generally not exceed the value $\tau_{va} = 0.4 f_y$.

- 8) Determine the maximum deflection and check whether it is less than the permissible value of $\frac{L}{320}$.

8.6 STRUCTURAL STEEL COLUMNS

The structural steel columns generally carry axial compression load. Sometimes these are required to be designed for combined effect of axial-load and Bending Moment. Permissible axial compressive stress in steel column depends on its slenderness ratio and is lower than the permissible axial tensile stress.

The slender the column, the smaller is the permissible axial compressive stress. If r_{min} is the least radius of gyrations of the cross-section of the column and (L_{eff}) is the effective length of the column,

$$\lambda = \text{slenderness Rati} = \frac{L_{eff}}{r_{min}}$$

The values of ratio of L_{eff} to actual length of column member are given in Table 5.2 of I.S. 800 -1984 based on end conditions of the column.

The values of permissible axial compressive stress ($\bar{\sigma}_{sc}$) are given in Table 8.1 for various values of its Slenderness - Ratio, where, yield stress of the steel is assumed to be 250 N/mm².

Table 8.1

Slenderness Ratio of Column	10 20 30	40 60 80	100 120 140	160 180 200 250
Permissible Axial Compressive Stress in N/mm ² (σ_{sc})	150 148 145	139 122 101	80 64 51	41 33 28 18

Let the permissible axial compression load that can be carried by a column be (P_c),

$$P_c = \text{Axial load capacity in compression} = \bar{\sigma}_{sc} \times A$$

where A is the cross-sectional area of the column

8.6.1 Computation of Axial Load Capacity

Case (a): If an I section MB 200 is used as column, $A = 32.33 \text{ cm}^2$

$$R_{xx} = 8.32 \text{ cm}, r_{yy} = 2.15 \text{ cm}, r_{min} = r_{yy}$$

The values of P_c for various effective length of column are given in Table 8.2 (Case b): In case a column is made of two legs (Double I section) as shown in Figure 8.15 (a) it can be provided with lacing or battens as shown in Figure 8.15 (a) and (b) so that the combined section is effective as single column. The moment of inertia of column about YY axis is larger than (I_{xx}) where XX and YY are axes through the centroid of the combined section as shown in Figure 8.15.

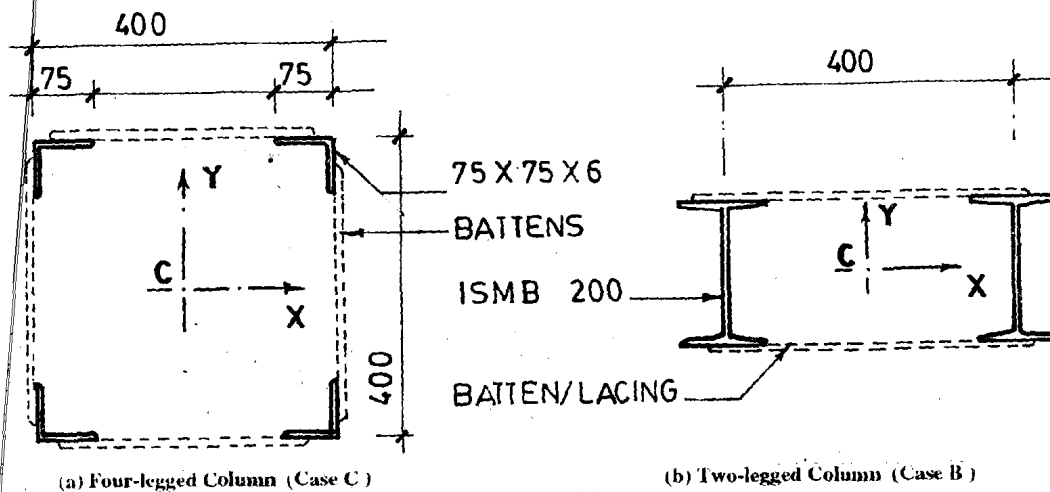


Figure 8.15: Built up Columns

Thus an $r_{xx} = 8.32$ cm is the critical radius of gyration and $\frac{L_x}{r_{xx}}$ is the Slenderness - Ratio. The values of Axial load Capacity (P_c) for this two-legged column is also Tabulated in Table 8.2.

Case (C) : Further when column heights are quite large, the axial load capacity of a built-up four-legged column with each leg of $< 75 \times 75 \times 6$ and external dimensions of $400 \text{ mm} \times 400 \text{ mm}$ as shown in Figure 8.15 (b) is computed and Tabulated for comparison. Since $C_{xx} = 2.06$ $r_{\min} > (20 - C_{xx})$. Assume $r_{\min} \approx 18$ cm (Approximately)

Referring to Table 8.3 and Figure 8.15 (a, b), it is noted that columns under cases a) and c) have almost same area of cross-section. Column of case (a) cannot have effective length more than 4 metres since its slenderness ratio (λ) is then beyond (180). However, the column under case (c) can be constructed even with effective length of 12 metres when its value of $\lambda = 66.7$ only and its load carrying capacity is (398 kN) more than the corresponding value (310 kN) of 2(ISMB 200) of case (b) for which area of cross-section is almost double of that of case (C).

Table 8.2 : Axial Load Capacity (P_c) of Columns of Different Configurations for Various Values of the Effective Length

Effective Length of Column in Metres	Details of Few Cases of Cross-Section of Column								
	Case (a) Single ISMB 200 $A = 32.33 \text{ cm}^2$ $r_{\min} = r_y = 2.15 \text{ cm}$			Case (b) Double ISMB 200 $A = 2 \times 32.33 = 64.66 \text{ cm}^2$ $r_{\min} = r_x = 8.32 \text{ cm}$			Case (c) $4 < 75 \times 75 \times 6$ $A = 4 \times 8.66 = 34.64 \text{ cm}^2$ $r_{\min} 18 \text{ cm}$		
	(λ)	$\bar{\sigma}_{sc}$ N/mm ²	P_c kN	(λ)	$\bar{\sigma}_{sc}$	P_c	(λ)	$\bar{\sigma}_{sc}$	P_c
2	93	87	281	24	147	950	11	150	520
3	139	51	164	36	142	918	16.7	149	516
4	186*	31	100	48	133	860	22	148	512
6	—	—	—	72	109	704	33	144	499
8	—	—	—	96	84	543	44	137	474
10	—	—	—	120	64	414	56	126	436
12	—	—	—	144	48	310	66.7	115	398

* Value of $\lambda = 186$ is marginally higher than a value of (180) which is generally considered as its upper limit for steel column.

In the case of a compression members of a roof-truss, a double angle section is generally used for spans (L) greater than 12 m. Refer placement of two angles in starred connection, in Unit (5) in the solution of SAQs (solved problems). It is noted that

- The minimum radius of gyration for a single angle is (r_v)
- When two angles are connected back to back $r_x < r_v$ and the minimum

gyration is $r_x = r_x$ value of single angle, where X & Y are axes through centroid of combined section of two angles, while x and y are axes of individual section.

- c) When two angles are connected in starred fashion, the minimum value (r_{min}) is further improved and given by r_{uu} which happens to be the maximum value of ($r = r_{uu}$) of a single angle section.
- d) When a compression member is quite long, the starred shape of cross-section of two angles can be chosen suitably to make an economical design.

8.6.2 Battens and Lacing

When a column is built up of two or more members, it is necessary that these member be suitably connected to each other by plates or angles so that combined section of the column can act as an integrated unit. Consider a case of Two I sections as shown in Figure 8.16 (a).

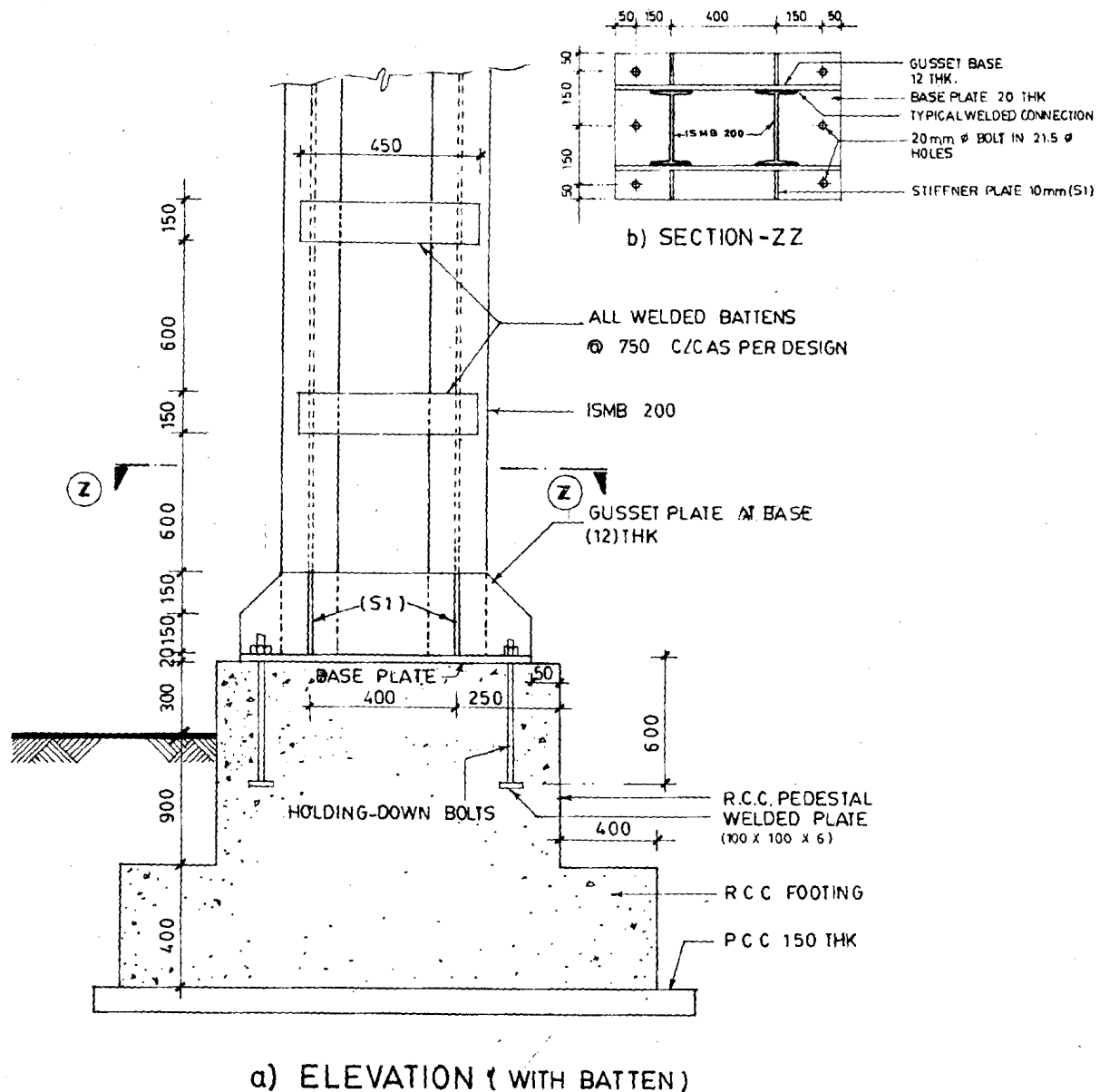


Figure 8.16 (a,b) : Two-legged Braced Column with Base Connection to R.C.C. Footing

Figure 8.16 (a) shows the elevation of such a built-up column with batten-plates of size about 150 mm x 450 mm connecting the two legs (MB 200) by welding. The battens are provided on both sides as indicated by dotted lines in plan of Figure 8.16 (a). The size of battens and their spacing are required to be designed depending on the vertical loads and the horizontal loads to be carried by the column. The details of provision of design is not intended to be covered in this chapter. The design is based on the assumption that battens along with the column forms a rigid-jointed frame in resisting the lateral load. Alternative to batten-connection is the lacing of the column as indicated in Figure 8.16 (c). Lacings are generally either the flats (say of size

(50×8) upto (75 × 8), or Angles (say of size , 35 × 35 × 5) or higher) depending on the codal provisions of the design. The analysis is based on the assumption that along with the column, it forms a pin-jointed frame to resist the horizontal load.

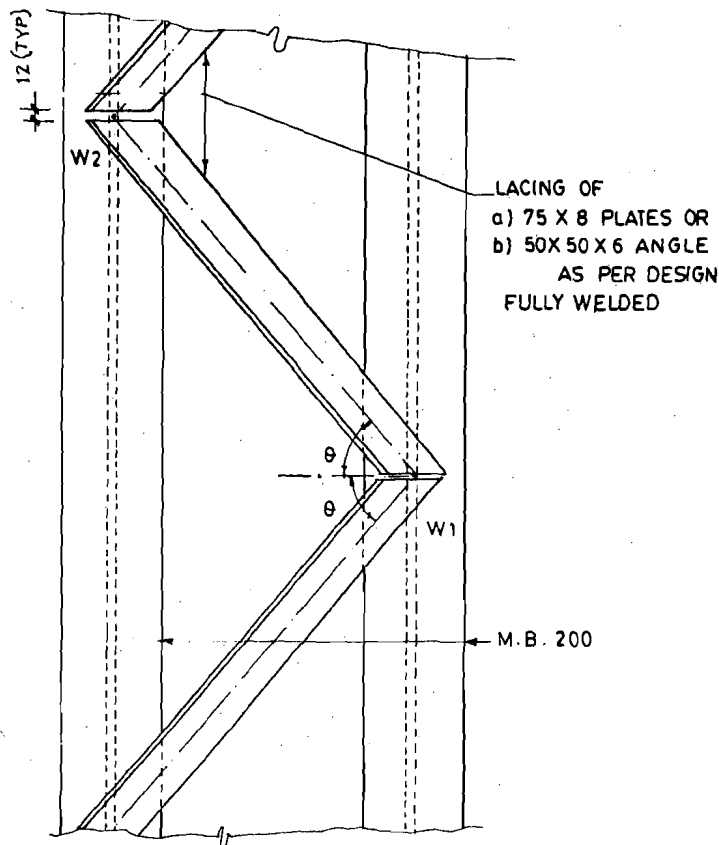


Figure 8.16 (c) : Two-legged Column with Lacing ($30^\circ \leq \theta \leq 60^\circ$)

8.6.3 Column Base

The structural steel column are generally not provided below the ground level. Since it may undergo corrosion when in contact with moist-soil. It needs to be fully encased in concrete if it is taken below ground-level.

Normally a steel column is supported on the top of a R.C.C pedestal (similar to short and sturdy R.C. column) at a level about 300 mm or more above ground level. The steel base plate is generally provided at the bottom face of the column and Anchor bolts/Holding down bolts are provided as shown in Figure 8.16 (a & b). These bolts are well anchored in the concrete of the pedestal. Normally the bolt length embedded in the concrete is about $30 \times$ diameter of the bolt and a small plate is welded at bottom of bolt to further develop additional mechanical bond with the concrete. Bolts help to prevent the overturning or lifting of steel column when designed for wind-load. Sometimes the height of pedestal is quite large and R.C.C. spread footing is provided below it so that it bears on a good soil-strata at appropriate depth. As shown in Figure 8.16, the base plate is stiffened by a) stiffens plates (S1) and b) gusset base plates of suitable thickness. These are welded to the base-plate of the column and also welded/bolted to flanges on both sides of the two-legged-column. The load transfer from steel-column to soil is thus through a continuous medium of R.C.C footing and pedestal. The clear cover of the main-reinforcement bar is also generally ≥ 50 mm so that corrosion effect are reduced.

Figure 8.16 mainly shows the detailing of the base of steel column wherein connection between column to the base-plate and connection details of base-plate to the pedestal are clarified. The details of reinforcement in the R.C.C. pedestal or the footing are not indicated since it has to be studied separately as a part of design of reinforced concrete footings.

8.7 LARGE SPAN STEEL GIRDERS

When span of girders are more than 10 metres, the compound section built up from standard rolled section, which are generally available upto a depth of 600 mm, are uneconomical. For a

range of span from 10 metres to 20 metres solid web plate girder can be designed and fabricated fully with the help of the steel-plates.

When the span is more than about 20 metres or when loads on girder are quite heavy, even for smaller span, it may be economical to fabricate an open-web girder which is basically a lattice girder where bending moments are resisted by top chord member and bottom chord member while shear is resisted by the vertical and diagonal members.

Gantry girders are a special case of girders which carry moving wheel loads causing both vertical as well as horizontal bending moments. Figure 8.1 shows elevation and cross-section (Part d) of the gantry girder. These are required to be designed for both vertical loads with impact as well as horizontal load caused by the Breaking/Accelerating of moving Trolleys.

All the above referred types of girders are explained below. In this unit, design of structural steel member is not included. However, the main emphasis on showing typical details of the girders based on assumed sizes.

8.7.1 Solid Web Plate Girder

Figure 8.17 shows a typical solid plate girder simply supported over a span which is assumed to be 16 metres. Loading diagram is given in Part (a) of the Figure, wherein it is noted that apart from uniformly distributed load (say $W = 40$ kN/m), a concentrated load P (say of 60 kN) is also applied at its midspan. Overall depth (D) of Girder is chosen as 1500 mm at midspan. Generally vertical stiffeners are provided to stiffen the thin web-plate. Web thickness should generally be more than $\left(\frac{D}{180}\right)$.

The vertical stiffener at locations of heavy concentrated loads like load (P) or support reactions are called as bearing stiffeners and have to be designed to carry the concentrated load at that location. The intermediate stiffeners (I.S) are provided at a spacing less than $(1.5 D)$ to prevent the buckling of the web.

The thickness of the flange-plate at top/bottom depends on the maximum Bending Moment (B.M) at midspan. The thickness of flanges can be reduced near the support where the B.M is much lesser. Accordingly the location at which flange thickness can be reduced to 12 mm or 16 mm is to be determined. Figure 8.17 (d) shows a typical butt weld joint where flange thickness is changed, from 16 mm to 30 mm.

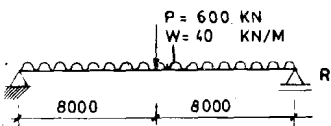


Figure 8.17 (a) : Loading Diagram

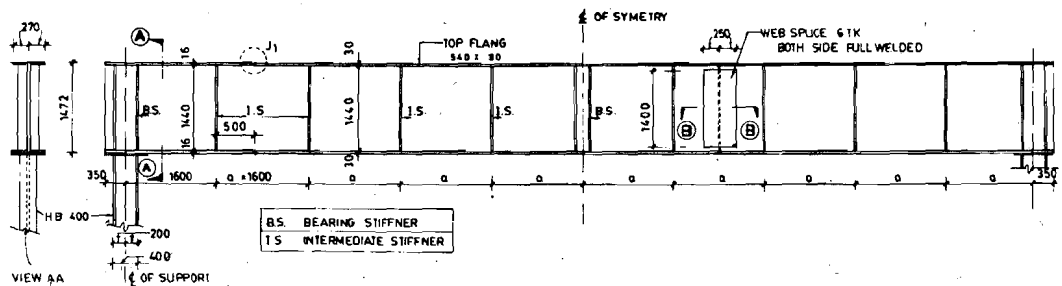


Figure 8.17 (b) : Elevation of Solid Web Plate Girder

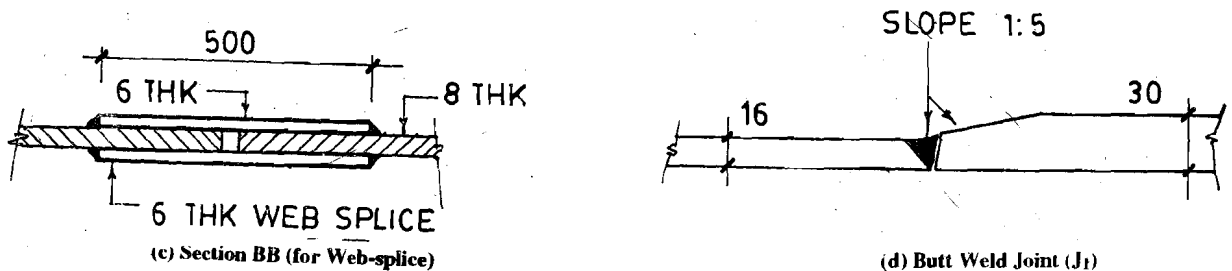


Figure 8.17: Solid-Web Plate Girder (Span -16 m)

Splices

The length of the web plate available in market can be about 10 to 12 metres. Hence it is necessary to make a joint of two web plates at some location (W) as indicated in

Figure 8.17 (b). Plan section BB is shown in Figure 8.17 (c) wherein it is seen that cover plate of about 500 mm wide and 1400 deep can be welded/bolted on each side of web plate. These plates and weld-size are to be designed adequately to resist mainly shear-force and partial B.M. carried by the web at that section.

Bearing stiffener (B.S.) over a column of I section is detailed in Figure 8.18 (a, b). The pair of stiffener plate on each side of web of girder are welded to the web and generally these plates are placed in alignment with the flange-plates of column below/above. These plates have their end faces cut to the profile of top and bottom flange angles of the girder and placed with proper bearing over the legs of the angles.

The typical welding details of the bearing stiffener to the web plate are shown in the three views of Figure 8.18 to transfer the concentrated load from reaction provided by the steel column. The typical details of welding between web and flanges are also shown in the part elevation of the figure. In the view (AA), a clamping plate (B) welded to column flanges is shown. The holding-down bolts provided through base-plate of the girder are held with the help of this clamping plate. In case of roller support at right hand side (R) of the girder (Figure 8.17) the bolt hole in the base plate need to be made oblong and the top nut of the bolt is kept above the base plate and the flange plate by a clearance of about 1.5 mm so that the end of the girder along with the base plate can slide while the bolt remains in position since it is locked with lock-nuts on the clamping plate. In these situation it is better to provide a separate cap-plate welded on the top of the column so that base plate can slide over the same smoothly. The circular bolt holes shown in Figure 8.18 (c) are meant of hinged support at end (L) of the girder.

8.7.2 Open Web Girder

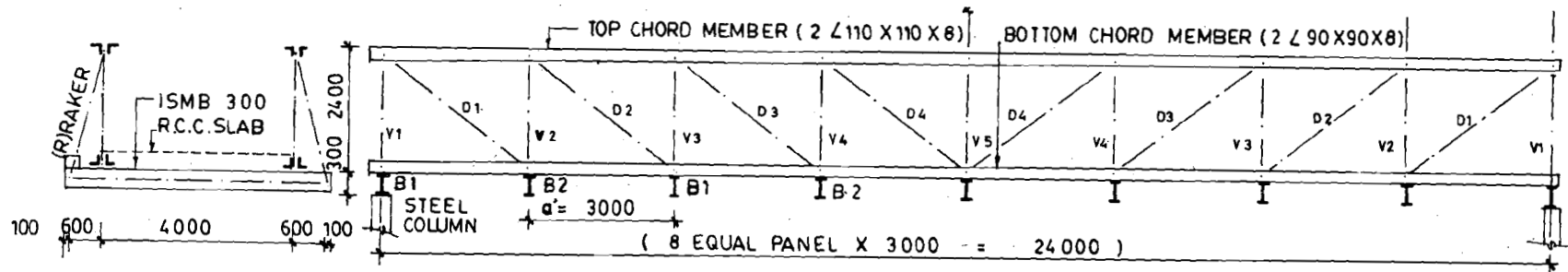
A typical open-web girder used as "Foot-over Bridge" over the railway-lines covering a span of 24 metres and simply supported over steel-column is shown in Figure 8.19 (a,b,c). Part (a) of the Figure 8.19 shows the general elevation of the girder wherein top chord member, bottom chord member are generally of double angle section. For larger span a box section of two channels is also used. The vertical members (V) carry compression load and are generally double angle section. The diagonal members carry tensile force and can be designed as single angle section if so required. Generally the minimum size of the angle used is angle $50 \times 50 \times 6$.

In this girder web plate is not provided and hence the name "Open Web Girder" is appropriate. The cross-beams B₁ and B₂ are supported over the two girders at the joints of horizontal bottom chord and the vertical/diagonal member. The function of the cross-beams which are generally R.S.J (Rolled steel joist of I section is to support the R.C.C. slab over which people can walk. The side elevation in Figure 8.19 (b) shows how connection of girder and R.S.J can be designed. The layout plan of R.S.J. is shown in Figure 8.19 (c). It is a good practice to provide wind bracings (D) designed as ties in diagonal direction in between the R.S.J. as shown in the layout. The horizontal wind load over the height of the girder or over side sheetings if provided will cause horizontal bending of this horizontal open-web girder provided below the R.C.C. slab. This girder is also supported on the same vertical columns. In the layout plan it is noted that alternate cross-beams are extended beyond the centre lines of girder by about 700 mm. The raker arms (R) as shown in Figure 8.19 (b) are provided from these ends to provide lateral restraint to the top-chord member of the girder. It is known that top chord member which carries axial compression forces should be provided with lateral restraints to reduce its effective length against buckling in horizontal plane also. In the vertical plane effective length is controlled by the spacing of vertical members (v) of the girder.

The details of members and their connection at joints are not shown since these are similar to those shown in the case of roof-truss. A detail of raker connection with top chord and bottom R.S.J. is however shown in Figure 8.20.

8.7.3 Gantry Girder

Gantry girders are essentially required in factory building where devices for lifting machinery or its part and transporting the same from any location of the factory floor to any other location as required are required to be provided. Generally the part of transporting the masses of machine-parts is to be done at quite high level over the essential head room so that working of persons in other portion is not affected. Thus electrically operated overhead travelling cranes (O.T.C) are usually provided. The trolley with wheels, carrying the hoisting system and its hook is supported over the rails of the crane-girder and can be moved along the crane-girder. The crane-girder itself is provided with end wheels and can be rolled over the rails of the

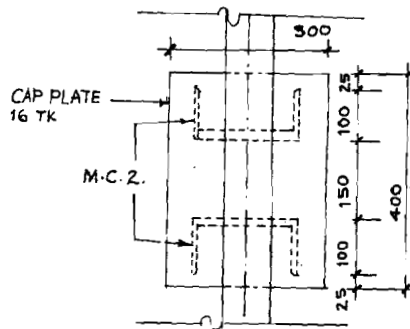


b) SIDE ELEVATION

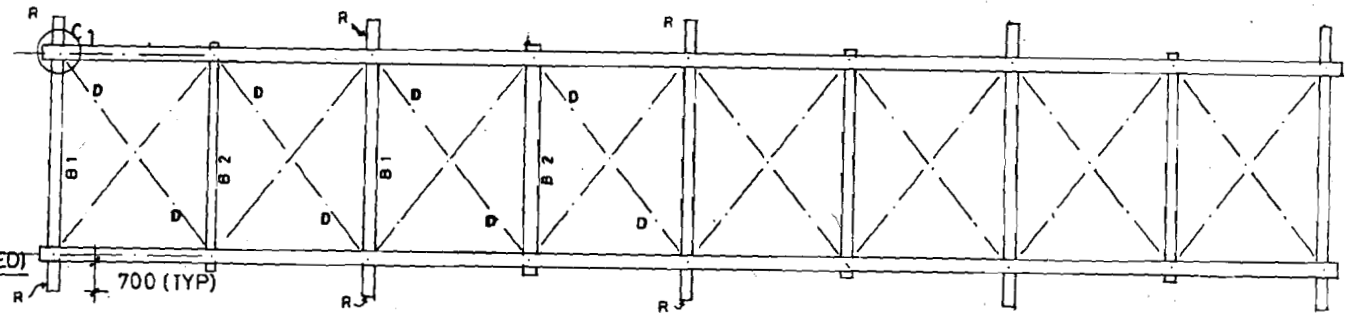
a) ELEVATION

(α) = PANEL LENGTH = 3000 mm

REFER DETAILS OF MEMBER V1D1 IN FIG 8.7.4.



DETAIL OF COLUMN C₁(TWO LEGGED)



(c) Layout Plan of Cross Beams (B1 & B2) Supporting R.C.C. slab and Diagonal wind Bracings
Figure 8.19: Open web Girder for Railway Foot - over Bridge (Typical)

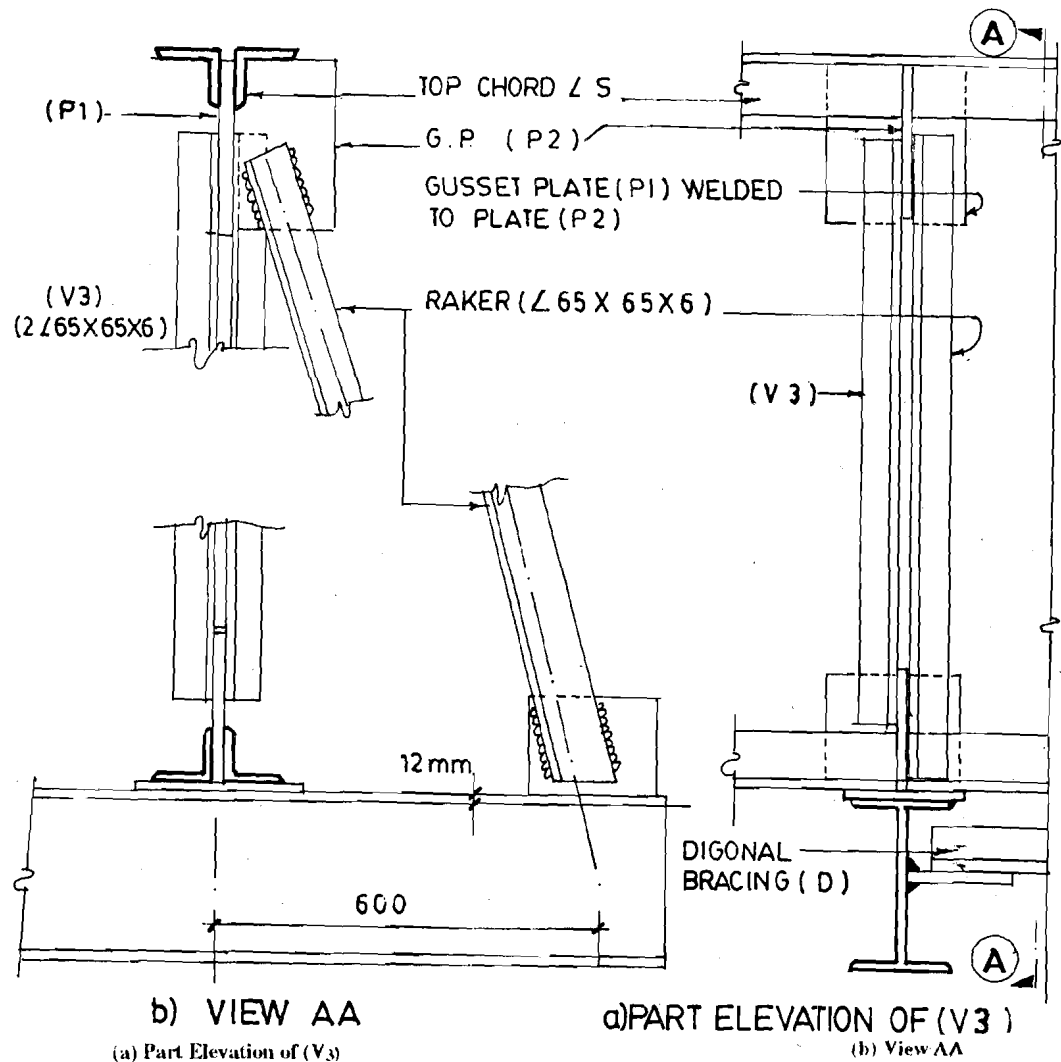


Figure 8.20 : Connection Details of Raker B1 & V3

longitudinal girder (along X direction called as gantry girder supported on steel columns A₁, A₂ etc. as shown is general layout plan and elevation of the factory building in Figure 8.1.

The gantry girder thus carries the vertical loads transferred through the wheels of the moving crane girder. When the trolley moves in cross-direction (Y) and suddenly accelerates or breaks, the lateral forces in horizontal directions are also transferred to the gantry girder apart from the vertical loads. In view of this the gantry girder is to be designed for both vertical and horizontal bending. Hence, generally its cross-section is a compound section with a channel placed with its web horizontal on the top of I section with its web vertical as shown in Figure 8.1 (d).

SAQ 2

- i)
 - a) Draw a neat drawing showing a bolted connection of beam I.S.M.B 350 to another I.S.M.B 500 going in perpendicular direction.
 - b) Draw a similar detail if it is a welded connection, through cleat angles connected web to web.
- ii) Draw a detail of an open web girder at its end supported on steel column of I section
 - a) if it is a hinged connection, and
 - b) if it is a roller connection.
- iii) Draw a welded detail of bearing stiffener of a solid-web girder supported over a steel column of I section.
- iv) Draw a typical detail of a web-splice of solid web plate girder
- v) Draw a built up column of two I sections and show a typical lacing detail.

8.8 SUMMARY

Detailing of steel structure is a part of professional practice and improvement can be done by experience in the various jobs. The details of various drawings indicated are only the introduction. Whenever you move about any structure, you should take interest to inspect carefully how steel members are provided and how the same are jointed to form the structure right from its foundation to the roof. You may draw to scale the details shown below to practice, think improve.

- 1) Trusses, the centre-line of members, cutting length of various internal members, purlins & their connection with top chord member and the A.C. roof-sheets.
- 2) The floor slab such as chequered plates and grating to be shown along with various beams suitably jointed.

The detail of beam to column connection is very important.

- 3) Steel has its weakness when considering axial compression for long-member while it is very strong in Tension. Draw suitable cross-sections for very tall column.
- 4) The long span girders of steel are quite light as against R.C. beams or prestressed concrete beams. Open-web girders are further lighter. It is extremely important that every small detailing has at its background, a design requirement to be fulfilled and thus study of design of steel structure is of utmost importance and should go hand in hand with the detailing aspects.

8.9 KEY WORDS

Flooring	:	Chequered plate, grating, RC slab or timber-plank on which human load or other dead load can be placed.
Rolled Steel Joist	:	Standard rolled member in the form of I, channel, angle section etc.
Compound Beam	:	Beam constructed by joining rolled steel joist and steel plates.
Built-up Beam	:	A beam formed by fabricating it by using steel-plates.
Solid Web Plate Girder	:	web plate Girder is an example of built up girder.
Built-up Column	:	A steel column built up from R.S.J., angles, channels etc. and the plates.
Battens	:	Flats used to make two or more legs of the column to behave as single column and to resist better lateral load.
Lacing	:	Flats or angles used to improve the load capacity of the column.
Purlins	:	A member supported on roof trusses and used for supporting the A.C. sheeting.
Sag Rod	:	A bar or a rod to provide interior support to purlins connected from one purlin to another in a direction parallel to top chord of truss to reduce the span of purlin for load-components in plane of the sheeting. These pair of sag-rod on the two slopes of roof are finally tied to ridge member.

8.10 ANSWERS TO SAQs

The solutions of SAQs are generally in the form of detailing and available in the text of respective Sections.

FURTHER READING

- 1) *The steel designer's manual* - Gray
- 2) The I.S. Handbooks Nos. (1) (2) and (3) for structural Engineers
 - i) Structural Steel Sections
 - ii) Steel Beams
 - iii) Steel Columns and struts.
- 3) I.S. 800-1984 for General Construction in steel
- 4) *Structural Steel Handbook* by B.N. Sridhara through "University Press (India) Pvt. Ltd.
- 5) *Design of steel Structures* by Arya & Ajmani
- 6) *Design of steel structures* by Negi
- 7) *Steel designer's manual* by C.S. Gray and others
- 8) *Design of steel structures* by A.S. Arya and J.L. Ajmani
- 9) *Design of steel structures* by L.S. Negi
- 10) Reference to IS Codes

Indian standards for material and other properties of rivets and bolts:

 - i) **Rivets**
 - a) IS 1929 - 82- Hot forged rivets for general purposes
 - b) IS 2155-82-Cold forged rivets
 - c) IS 1149-82 - High tension steel rivet bars
 - ii) **Bolts**
 - a) IS 1363 - 84 - Parts I - III Hexagon grade C bolts
 - b) IS 3640 - 82 - Hexagon fit bolts
 - c) IS 1364 - 85 - Hexagon grades B&A bolts
 - d) IS 3757 - 85 - High strength friction grip bolts.
 - e) IS 1367-84- Parts I to VIII - Technical supply conditions for threaded fasteners
 - f) IS 1364-85Hexagon grades B&A Bolts.
 - iii) **Nuts**
 - IS 6623 - 85 - High strength friction grip nuts
 - IS 1364-79 - Parts I to XVIII Hexagon grades A & B nuts & lock nuts
 - IS 1367-85 - Parts I - VIII- Technical supply conditions parts for threaded fasteners
 - iv) **Washers**
 - IS 5370 - 69 - Plain washers
 - IS 2016 - 67 - Plain washers
 - IS 5369 - 75 - General requirements for plain washers and lock washers
 - IS 6610 - 72 - Heavy washers
 - IS 6649 - 85 - High strength friction grip washers
 - IS 3063 - 72 - spring washers.
- 11) *Indian Standards and Handbooks* , Bureau of Indian Standards, New Delhi.
I.S. 800, 813, 816, 822, 823, 806, 1161,
- 12) *Structural Welding*, V.K. Chavan, Dhanpat Rai & Sons, Delhi - 1975
- 13) *Theory and Practice of Arc Welding*, R.J. Sacks, Van Nostrand Co.Affiliated East West Press Pvt. Ltd., New Delhi.

- 14) *Design of Steel Structures*, Breslin, Lin & Scals, Wiley Eastern Pvt. Ltd.
- 15) *Design of Steel Structures*, Arya & Ajmani, Nem chand & Bros. Roorkee, 1975
- 16) *Design of Modern Steel Structures*, L.E. Grinter, Eurasia Publishing House Pvt. Ltd., New Delhi 1969.

NOTES