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Analysis and Design of PEB Warehouse

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Keywords: Stress analysis, Load Cases, Pre-Engineered Building, Loading, etc.

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Lastly, I extend my gratitude to all the individuals, resources, and institutions that have in any way contributed to the successful completion of this Course Project. Thank you all for being an integral part of this journey.

I. INTRODUCTION

This thesis is about analysis and design of a warehouse (Location: Chennai) using Pre-Engineered Building concept, which is widely used in modern-day construction. We will be using Staad Pro software to analysis and design building components like columns, rafters, bracing members, etc. Design will be carried with reference to Indian standard codes. The major load action on the building will be Dead load, Live load, Wind load & Seismic load. Using Staad pro analysis will be carried out and stresses of the member will be calculated. Based on the stress levels, members will be reshaped for better usage and cost effectiveness. The analysis and design cover most of the thesis and is the most important part of any project. There were many dependent steps involved in the analysis part.

A. Pre-Engineered Building

These are steel buildings which are created / manufactured using various methods at factory and assembled at site. In simple words, as per the requirement building components are prefabricated at one place and erected at another place.

Table 1 – Advantages & Disadvantages of PEB

Advantages	Disadvantages
Reduced Construction time	Low Thermal Resistivity
Less Manpower at Site	Low Fire Resistance
Reduction in Cost	Sensitive to corrosion
Flexibility in Design	Finishing Details May Take Time
Scope for Future Expansion	Limited Architectural Freedom
Low Maintenance	Technical Expertise
Seismic Resistance	

Structural analysis is necessary as the reliability of the structure is investigated for all the requirements and loadings on the structure. Structural analysis is important since it finds out the critical components that need special attention or the special concept. Furthermore, understanding the design of the structure in more detail helps.

B. Warehouse
A structure or room for the storage of merchandise or commodities. A warehouse is a commercial space vital in the supply chain that is used to store finished goods and raw materials and is widely used in industries such as manufacturing and distribution. Warehouses are used for storing goods for an extended period and are typically equipped with storage areas, loading docks, conveyors, and other material-handling equipment.

II. STRUCTURAL COMPONENTS

A. Rigid Frame
The frame with members interconnected by predominantly rigid connection, which resist movements induced at the joints of member. Its members can resist bending moments, shear and axial loads.

B. Post & Beam Frame
These frames are also known as braced frames. The frame with members interconnected by pin-like manner. Its members can resist shear and axial loads.

C. Canopy
The steel structure canopy is a kind of engineering facility used to keep out rain and high-altitude falling objects. It is mainly located at the entrance and exit of buildings.

D. Pulin & Girts
Roof purlins are used to support metal roofing panels. Wall girts are used for the fastening of metal siding. They are available in stainless steel or galvanized.

E. Bracing Member
The members in a braced frame are generally made of structural steel, which can work effectively both in tension and compression.

F. Wall Framed Opening
A wall framed opening is a section of a wall that's been created to accommodate a door, window, or shutter. The framing members and flashing that surround the opening are also known as a framed opening.

G. Wall Light
The translucent panel is an economical method to allow natural light into a metal building. The translucent panels are field installed with self-drilling screws to a roof or wall panel. Generally, if located at roof these are called skylight and if in a wall it is called wall light.

H. Cage Ladder
A cage on an access ladder serves several important purposes, including Fall Protection: The cage acts as a safeguard against falls, especially in situations where the ladder's height is significant. It provides a barrier that can prevent a person from falling off the ladder or losing their balance.

I. Roof Monitor
A roof monitor is a raised section that runs along the ridge of a double-pitched roof, with its own roof that runs parallel to the main roof. The long sides of the monitor often have clerestory windows or louvers to allow light and ventilation.

III. PROBLEM STATEMENT
The Client wanted to construct a warehouse extended over the existing property and the architect proposed to go for steel shed. The proposal was for the warehouse which is rectangular in shape and steel columns supported on the ground by considering RCC foundation for the design and Consulting Engineers were approached to undertake a feasibility scheme to design the necessary support to the shed (roof and columns), ensuring the safety and durability of the structure.he key issue is predicting the effects of wind on the building, as typical wind design regulations make little reference to the layout provided by Architect. Wind will apply uplift or down pressure on any solid object depending on what kind of conditions.

We designed a rigid frame which transfers the loads effectively to the base of the support. The supports at base are designed properly, taking care of all the required specifications, making the warehouse structure more useful for a longer period during the period.

Figure – 1 Warehouse Picture

IV. MODELLING

A. *Model Details*

Table 2 – Modelling details for the structure

Description	Specifications
Type of Structure	Multi Span
Plan Dimensions	42 m x 33 m C/C of Steel Column
Height	5.0 m Clear Height
Brick Wall	1.25m
Column Base Level	+0.2m from FFL
Location	Chennai
Roof/ Wall Sheeting	0.50mm Color Coated Galvalume Sheet
Wall light	1Nos. per Bay
Roof Access	1Nos. of Cage Ladder
Ventilator	Roof Monitor at Ridge

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V. CONSULTANT/ ARCHITECT DRAWING

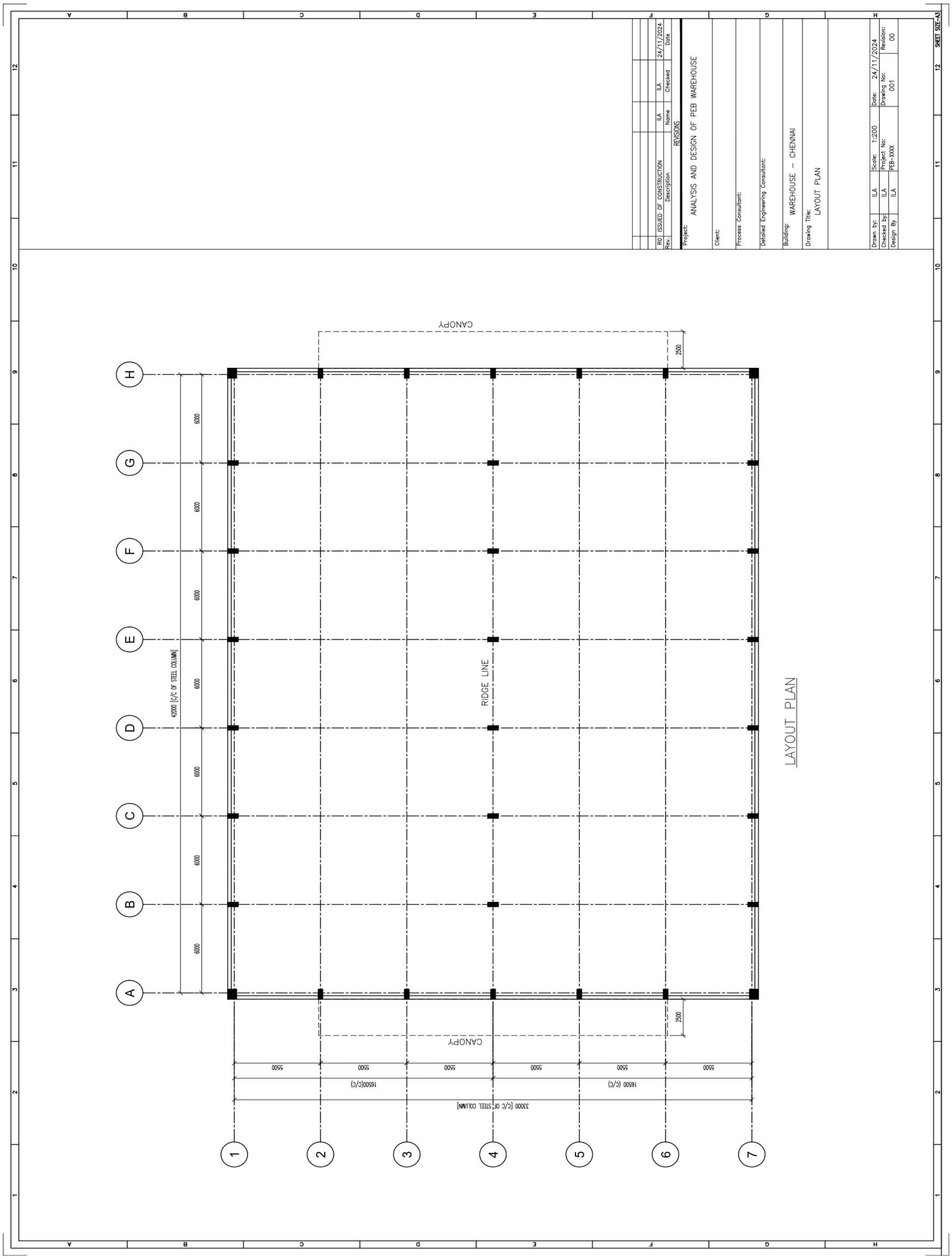

Figure – 2 Customer/ Architect Layout

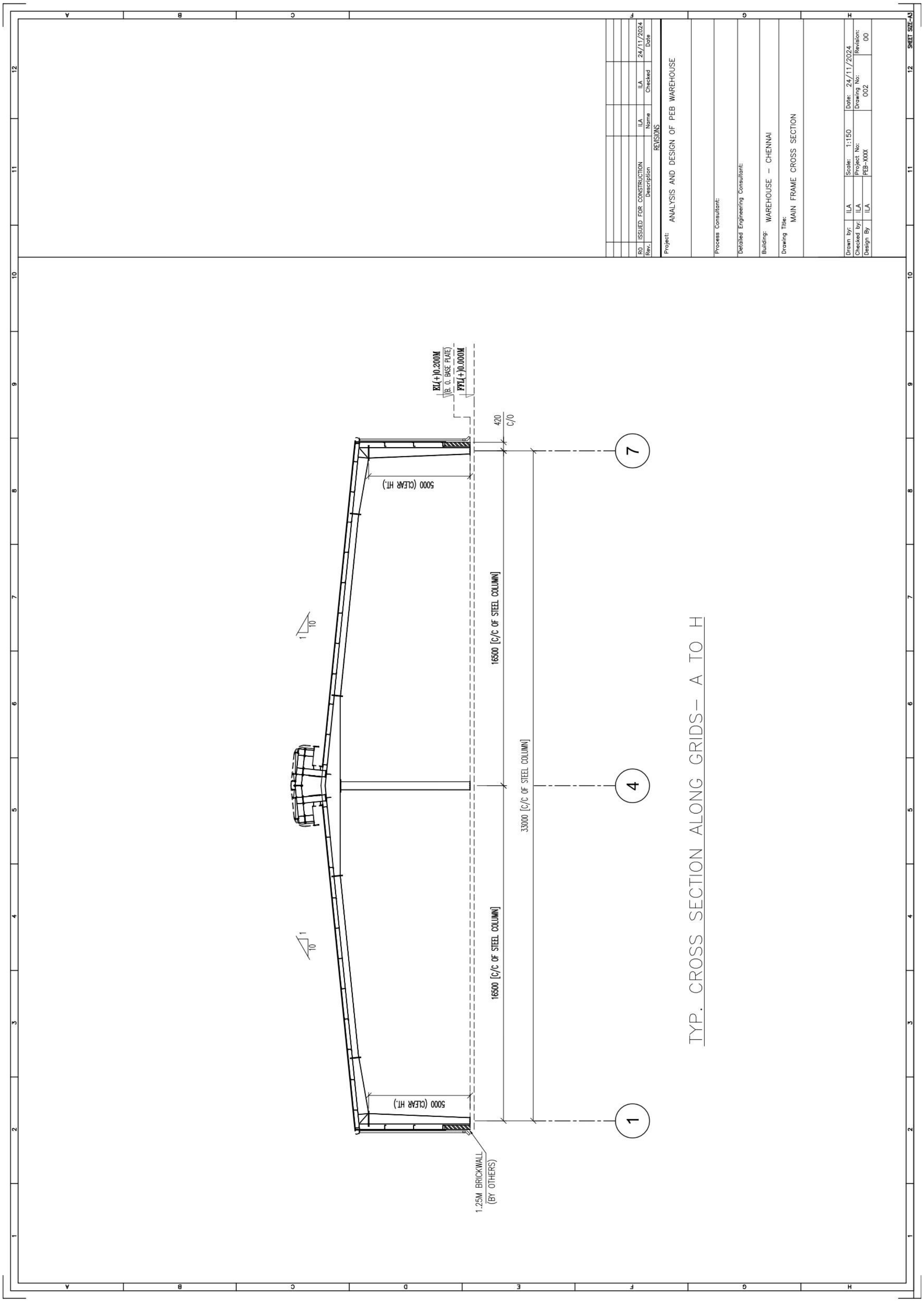

Figure – 3 Customer/ Architect Cross-section

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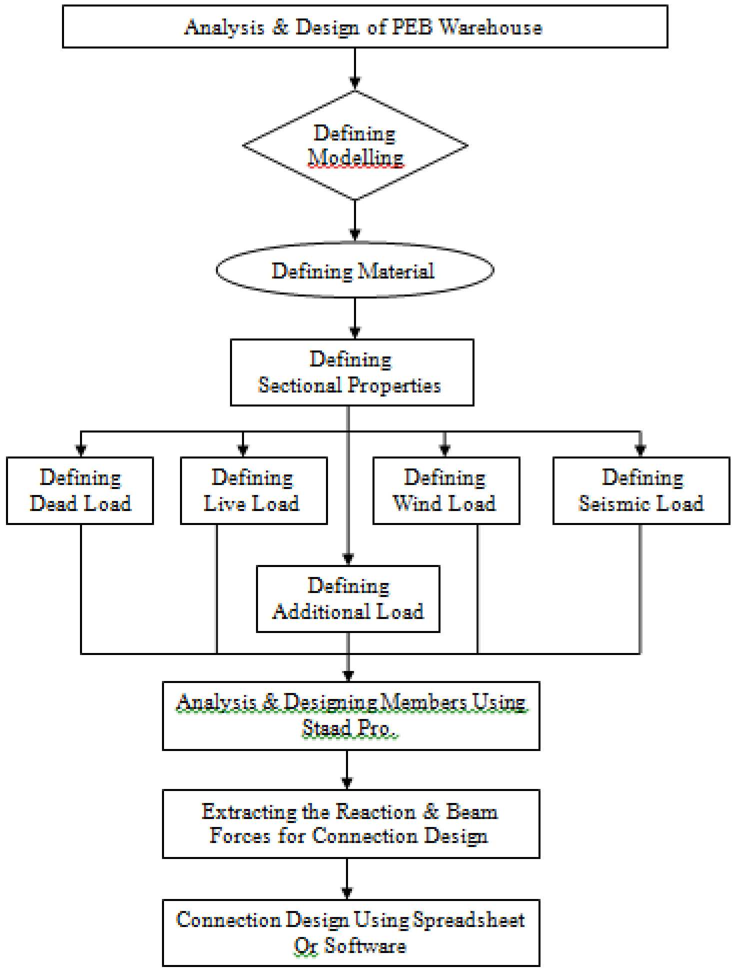
VI. MODELLING FLOW CHAT



VII. STAAD ANALYSIS

Project Warehouse 3D_Frame.STD

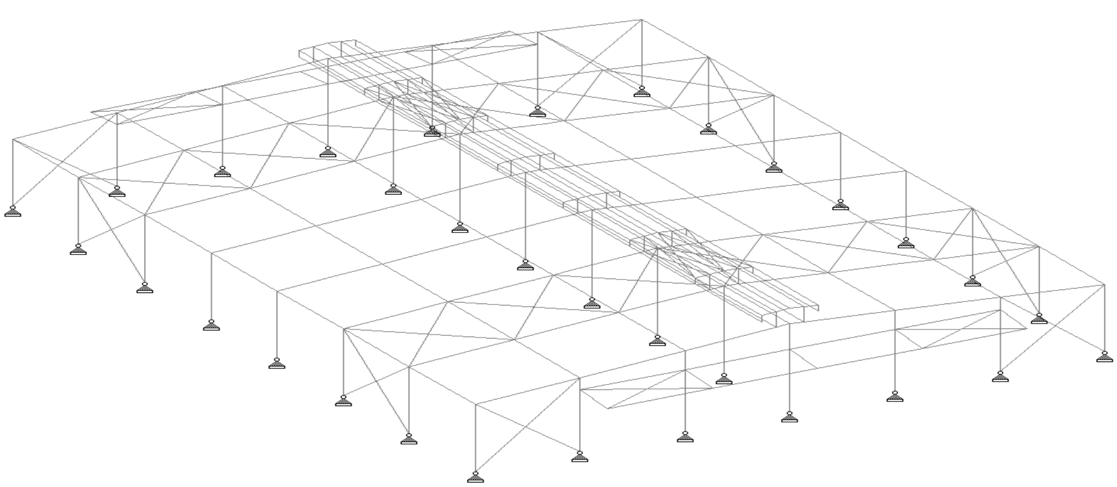


Figure – 4 Staad Pro model of the structure

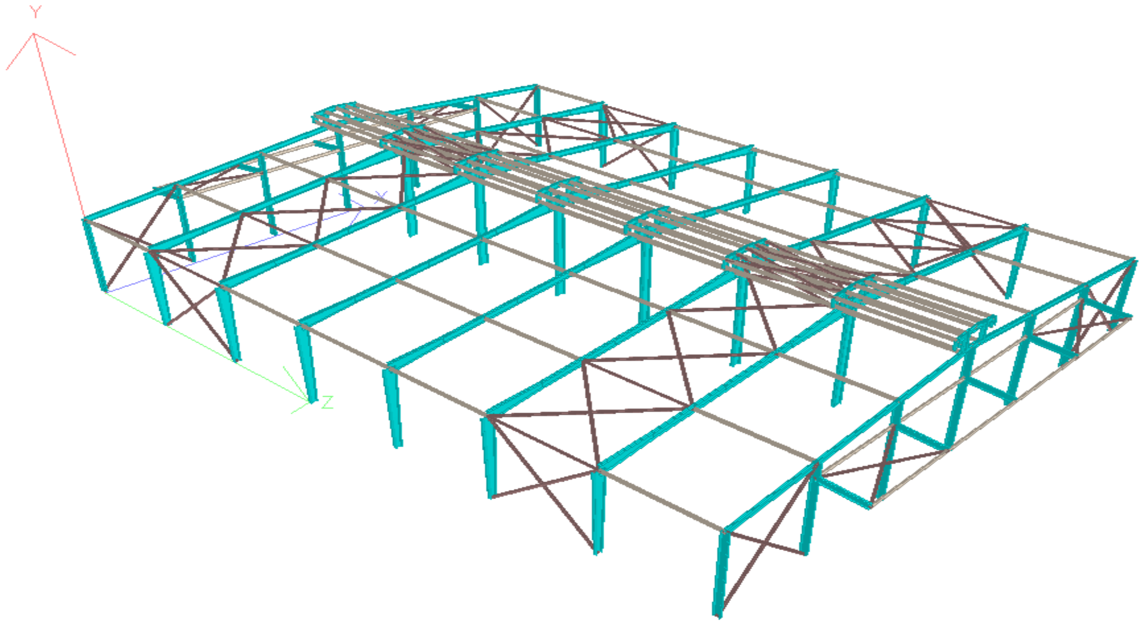


Figure – 5 Staad Pro rendered model

VIII. ANALYSIS OF COLUMN & RAFTER

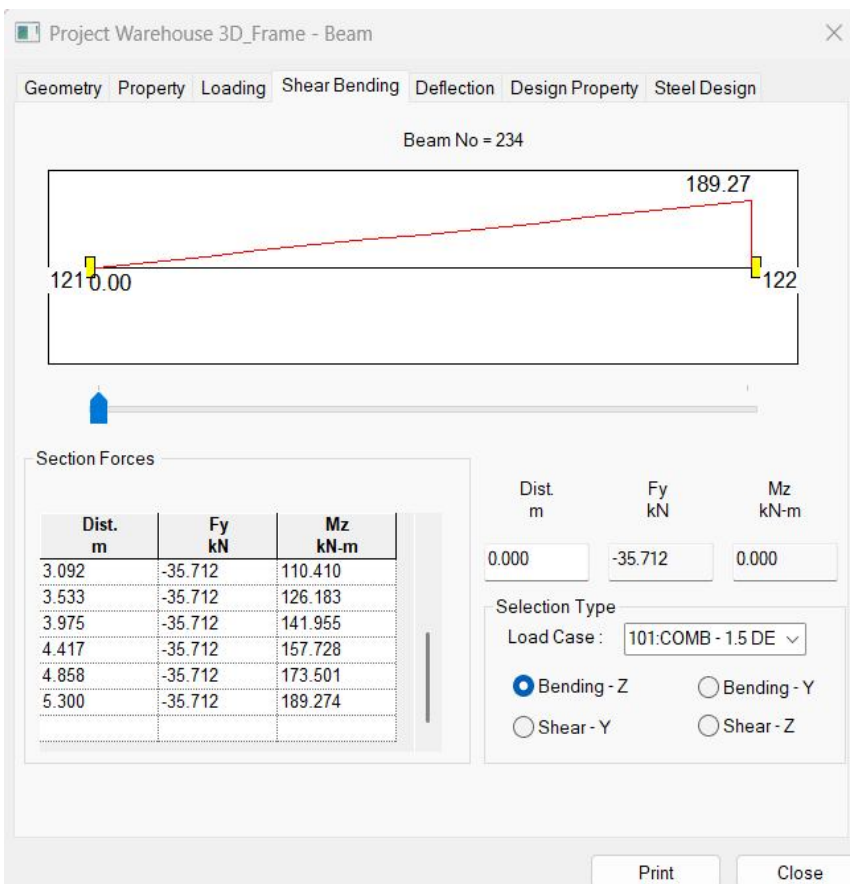


Figure – 6 BMD for the column member

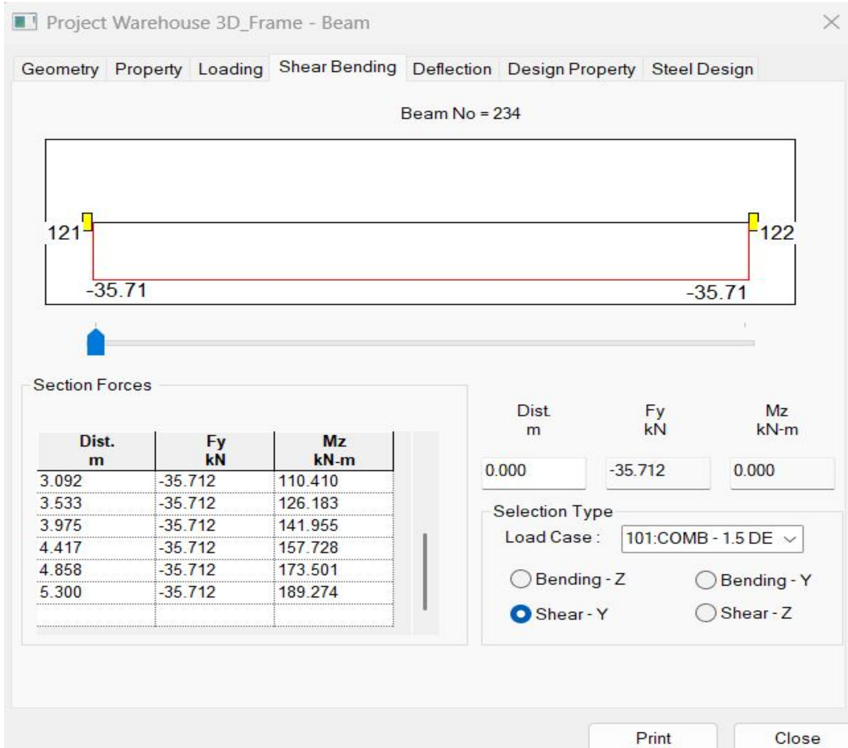


Figure – 7 SFD for the column member

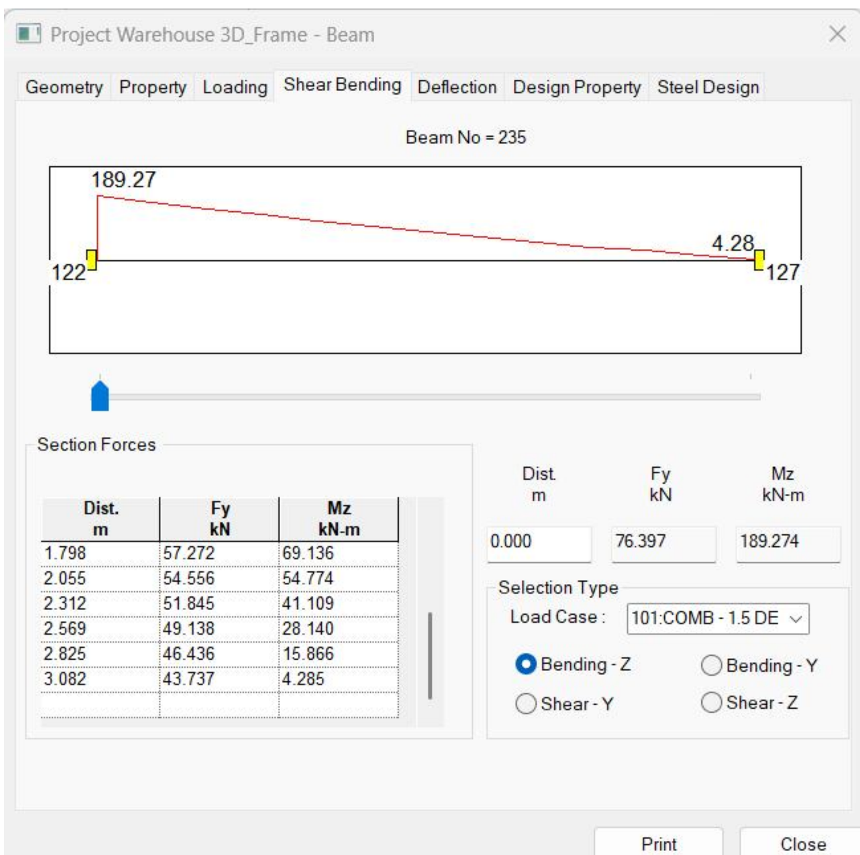


Figure – 8 BMD for the rafter member

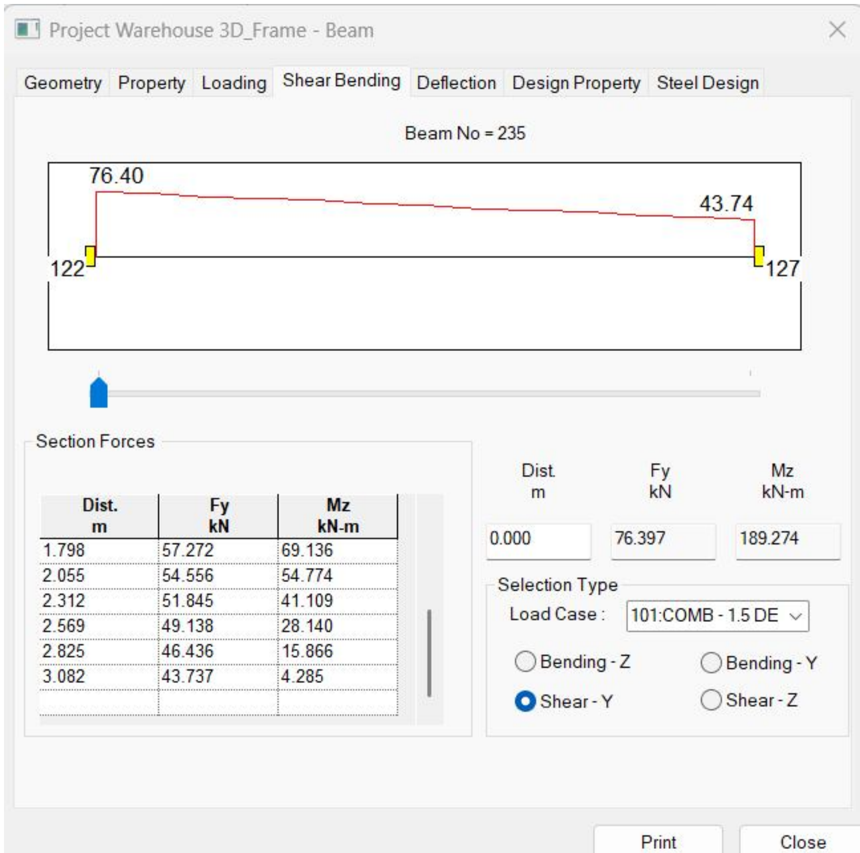


Figure – 9 SFD for the rafter member

IX. MATERIAL SPECIFICATIONS

Table – 3 Material Specifications

Sl. No.	Materials		Specifications
1	3 - Plate Welded Sections		ASTM A572Gr50 and IS2062 E350A.
	Built-Up Sections		IS2062E350 Grade
2	Hot Rolled Sections	*Beams, Angles*	ASTM A36 Gr.36 or IS 2062 – 2011 Gr A
		Rods	IS 2062 E250A or SAE 1018 with minimum 250MPa yield strength.
		Pipes	IS 1161 – 1998 and IS 806 – 1968 (240 MPa)
3	Cold Formed sections		ASTM A570 Gr 50 (painted), min. yield 340MPa
4	Anchor Bolts		IS:5624 (minimum 240 MPa) Galvanized 900 GSM, material MS confirming IS:2062
5	High Strength Bolts		ASTM A325M or IS 1367 Part 3 – 2002 Gr. 8.8 min.
6	Machine or Mild Steel		IS1367 Part 3 -2002 class 4.6, Grade-B of IS1367 Part-2 & IS-5624
7	Nuts & washers		Grade 8.8 as per IS: 1367 and shall be hot dip galvanized

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X. LOAD CALCULATIONS

Following Basic Loads shall be considered in Design of Structure and its elements

- Dead Loads (DL)
- Imposed Loads or Live Loads (LL)
- Wind loads (WL)
- Earthquake Loads (EQ)

A. *Dead Load Calculation*

Dead Load comprises of the weight of all permanent construction including frames, columns, beams, walls, roofing elements, cladding elements, sheeting and other steel elements permanently attached to building or structure. In general, dead loads for the materials used in Construction will be evaluated as per IS: 875-1987(Part-1) - "Weight of Building Materials" or as per the manufacturer's literature. The following unit weights shall be used for the materials listed.

Table 4 – Unit weight of Materials

Material	Unit weight	Unit
Structural Steel	78.50	kN/m^3
RCC Structure	25.00	kN/m^3
Brick Wall including plaster	21.00	kN/m^3

Self-weight of elements which are modelled in analysis programs shall be computed automatically. For other elements computation shall be carried out as per listed unit weights.

Figure – 10 Dead Load Calculation

DEAD LOAD CALCULATION

Width of the building (W) -	33.84	m
Building Tributory (T) -	6	m
No of Purlins in One Bay (N) -	24	Nos
Purlin Section -	200Z1.5	
Purlin Lapping -	Cont.\385 Lap	
Roof Sheeting -	0.50 mm Thk. CCGL	

Weight of Purlins -

$$\frac{N \times \text{Unit Wt. of Purlins} \times \text{Length of Purlin Including Lap}}{W \times T}$$

Weight of Purlins -	3.25	Kg/m^2
Weight of Panel -	4.49	Kg/m^2
Total Dead Load on Roof -	**7.74**	Kg/m^2
	or	
	0.08	kN/m^2
	=	
	0.10	kN/m^2

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B. Live Load

Imposed Load or otherwise Live Load is assessed based on the occupancy type and use of floor.
Considered Non-Accessible Roof – 0.75kN/m^2 (IS875-Part2_Table-2)

C. Wind Load Calculation

IS 875 (Part 3): 2015 is used to determine wind loads.

- Identify the basic wind speed (V) for your location from the wind map provided in the standard.
- Determine the importance factor (I) and the exposure factor (K) based on the building's characteristics.
- As per IS875-2015 guidance, Cyclone factor considered for Chennai Location.
- Calculate the wind pressure (Pd)
- Once you have wind pressure, determine the design wind force (F_d) using the formula:
- The effective area is calculated based on the projected area of the structure perpendicular to the wind direction.

Figure – 11 to 15 Wind Load Calculation

WIND LOAD CALCULATION

AS PER IS : 875 (PART 3) - 2015

User Input

Basic Wind Speed =	50	m/ sec	
Building Eave Height (H) =	10	m	Building Mean Height (H) = 10.846 m
Roof Slope =	1	: 10	*5.71 Degrees*
Length of the building (L) =	42.84	m	
Width of the building (W) =	33.84	m	
Class of Structure =	1	*All general buildings and structures*	
Probablity Factor (k_1) =	1.00		------------ *IS875(Part-3)-2015, Clause 6.3.1, Table -1*
Terrain Category =	2	*Open terrain with well scattered obstructions having heights generally between 1.5 to 10m*	
Terrain Factor (k_2) =	1.000		------------ *IS875(Part-3)-2015, Clause 6.3.2.2, Table -2*
Topography Factor (k_3) =	1.00		------------ *IS875(Part-3)-2015, Clause 6.3.3.1*
Cyclone Zone =	Yes		
Cyclone Factor (k4) =	1.15		------------ *IS875(Part-3)-2015, Clause 6.3.4*
Design Wind Pressure (p_z) =	**1.984**	KN/m^2	
$P_z = 0.6 \times (V_b \times K1 \times K2 \times K3 \times K4)^2$		**KN/m^2**	------------ *IS875(Part-3)-2015, Clause 7.2*

For Frame

Wind Directionality Factor (kd) =	1.00	------------ *IS875(Part-3)-2015, Clause 7.2.1*
Max. Frame Tributary=	6.00	
Effective Frame Area =	33.96	
Area Averaging Factor (ka) =	0.89	------------ *IS875(Part-3)-2015, Clause 7.2.2*
Combination Factor (kc) =	0.90	------------ *IS875(Part-3)-2015, Clause 7.3.3.13*

Design Wind Pressure (p_d) = 1.586 KN/m^2

$P_d = kd \times ka \times kc \times P_z$ **KN/m^2** ------------ *IS875(Part-3)-2015, Clause 7.2*

Not Less Than

Design Wind Pressure (p_d) = 1.389 KN/m^2 **KN/m^2**

$P_d = 0.7 \times P_z$ **KN/m^2** ------------ *IS875(Part-3)-2015, Clause 7.2*

Therefore Design Wind Pressure (p_d) = **1.586** **KN/m^2**

For Sheeting & Coldform

Wind Directionality Factor (kd) = 1.00 ------------ *IS875(Part-3)-2015, Clause 7.2.1*

Max. Purlin / Girts Tributary= 1.50

Effective Purlin Area = 9.00

Area Averaging Factor (ka) = 1.00 ------------ *IS875(Part-3)-2015, Clause 7.2.2*

Combination Factor (kc) = 0.90 ------------ *IS875(Part-3)-2015, Clause 7.2.3.3.13*

Design Wind Pressure (p_d) = 1.786 KN/m^2

$P_d = kd \times ka \times kc \times P_z$ **KN/m^2** ------------ *IS875(Part-3)-2015, Clause 7.2*

Not Less Than

Design Wind Pressure (p_d) = 1.389 KN/m^2 **KN/m^2**

$P_d = 0.7 \times P_z$ **KN/m^2** ------------ *IS875(Part-3)-2015, Clause 7.2*

Therefore Design Wind Pressure (p_d) = **1.786** **KN/m^2**

Permeability Condition = **Low Permeability** *Opening Area - Below 5%*

Internal Press.Co-efficient Cpi= **±0.2** ------------ *IS875(Part-3)-2015, Clause 7.3.2*

Wind Load on Individual members F = **$(C_{pe} - C_{pi}) \times A\, P_d$** **KN** ------------ *IS875(Part-3)-2015, Clause 7.3.1*

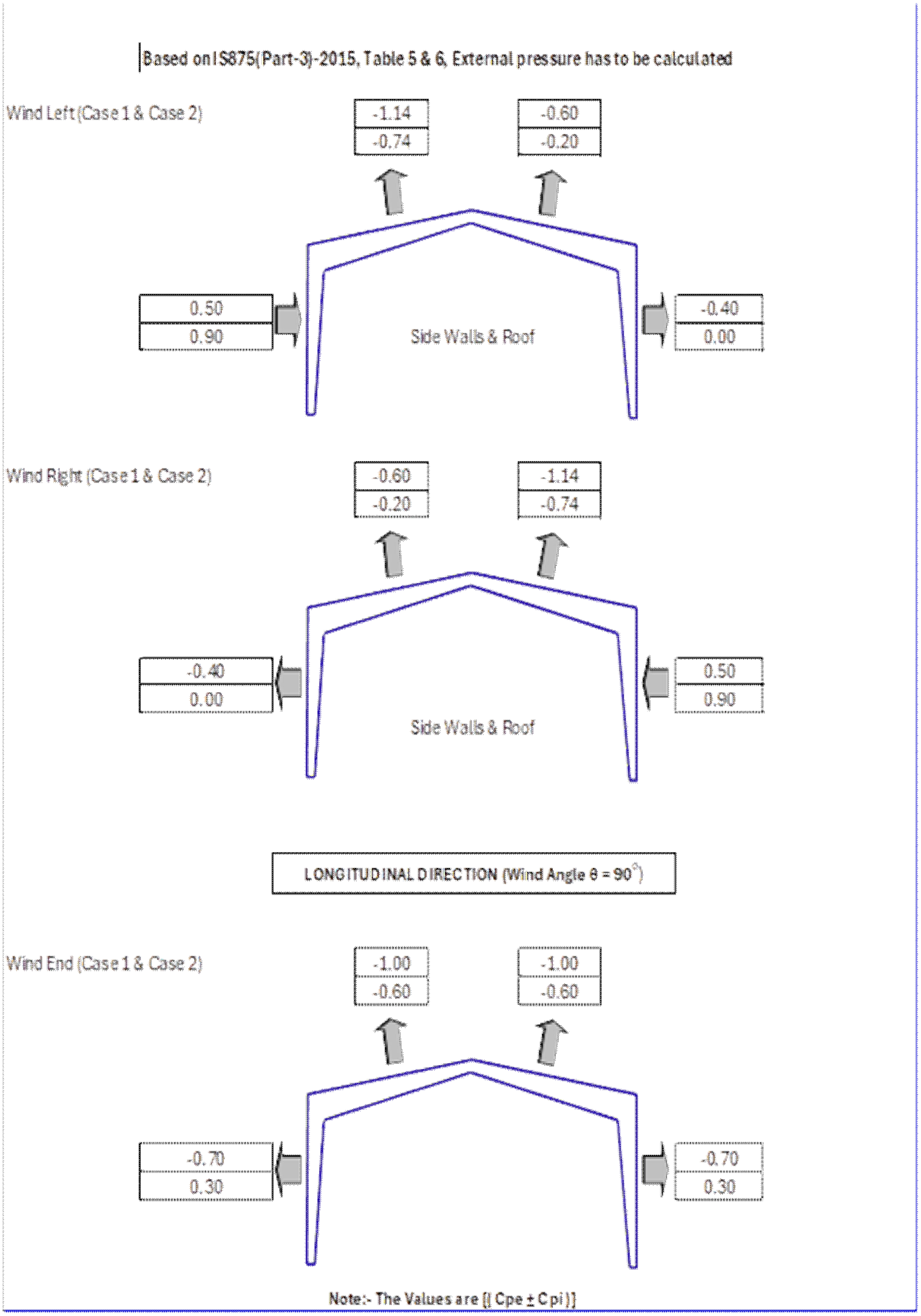
Where,

C_{pi} = *Internal Pressure Co-efficient*

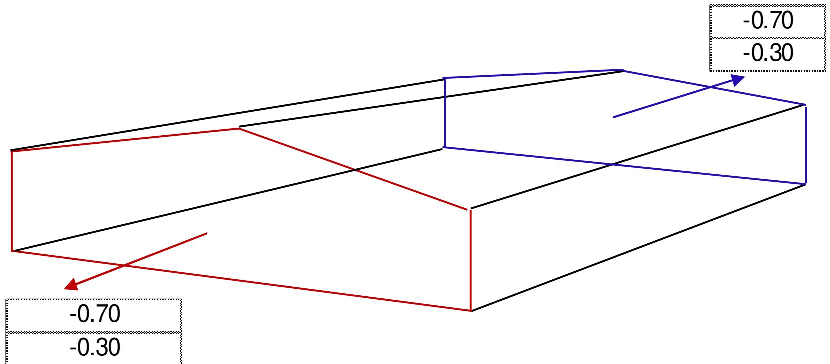
C_{pe} = *External Pressure Co-efficient*

A = *Surface area of structural element or cladding unit*

P_z = *Design Wind Pressure*

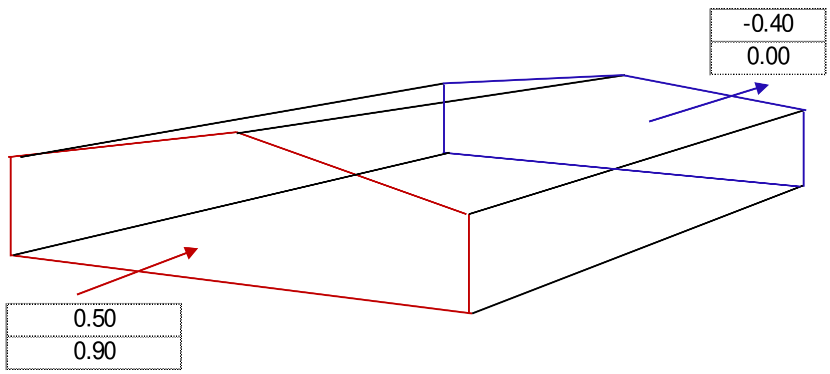

Based on IS875(Part-3)-2015, Table 5 & 6, External pressure has to be calculated
Wind Left (Case 1 & Case 2)
-1.14
-0.74
-0.60
-0.20
0.50
0.90
Side Walls & Roof
-0.40
0.00
Wind Right (Case 1 & Case 2)
-0.60
-0.20
-1.14
-0.74
-0.40
0.00
Side Walls & Roof
0.50
0.90
LONGITUDINAL DIRECTION (Wind Angle θ = 90°)
Wind End (Case 1 & Case 2)
-1.00
-0.60
-1.00
-0.60
-0.70
0.30
-0.70
0.30
Note:- The Values are [(Cpe ± Cpi)]

TRANSVERSE DIRECTION (Wind Angle θ = 0°)



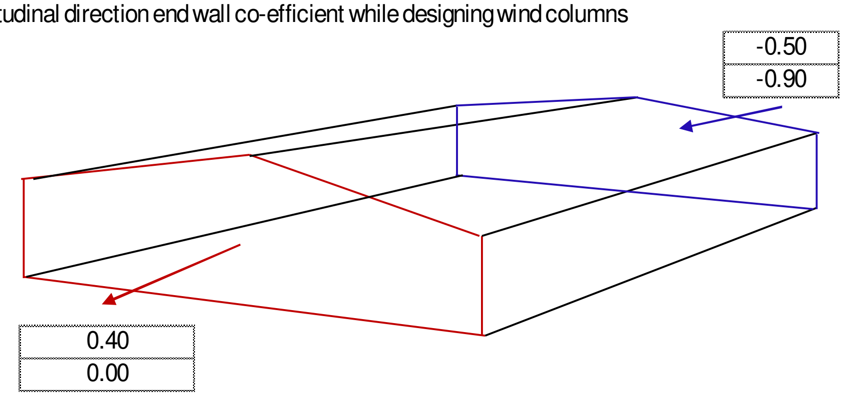
End Walls

LONGITUDINAL DIRECTION (Wind Angle θ = 90°)



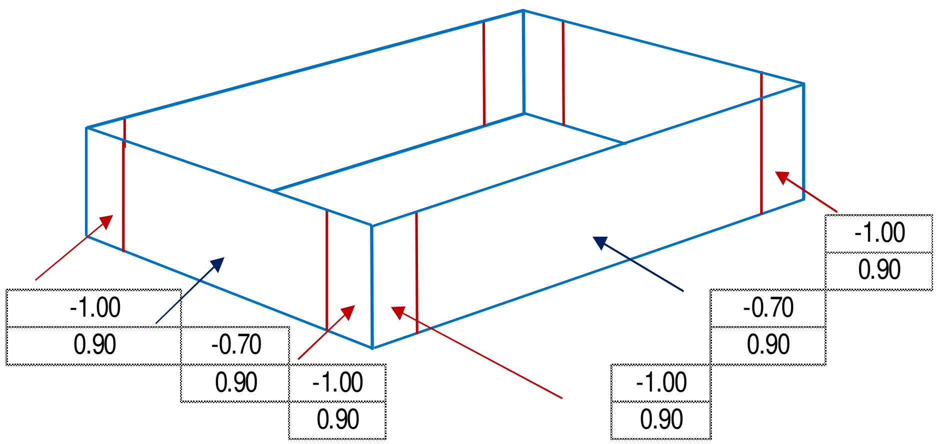
End Walls

Note:- Use longitudinal direction end wall co-efficient while designing wind columns



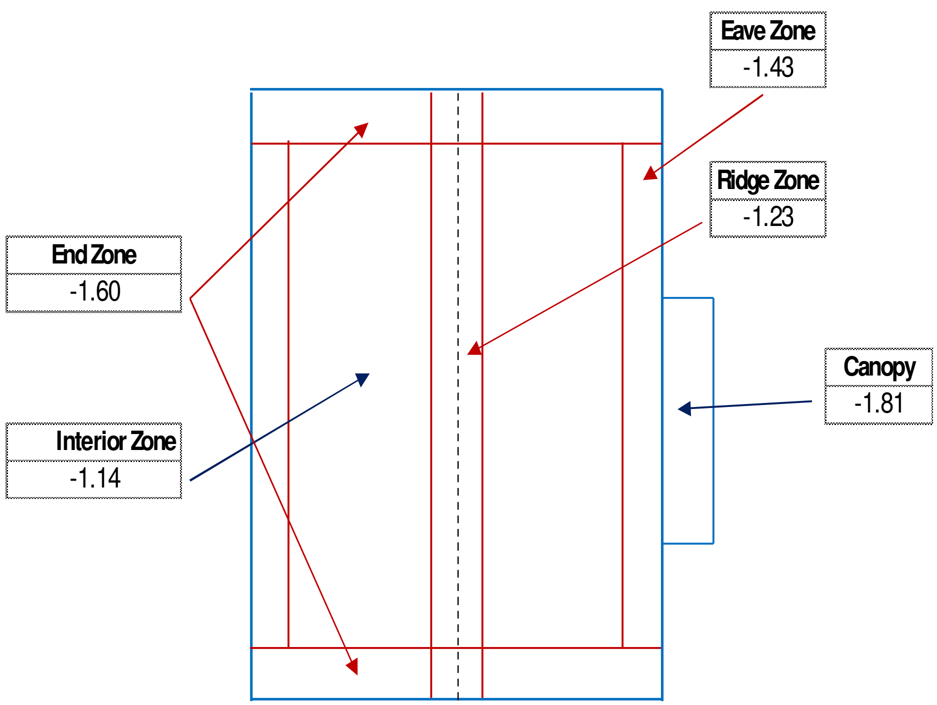
End Walls

For Design of Girts and Wall Panels



Corner zone distance = 8.46 m

For Design of Purlins and Roof Panels



Corner zone distance = 5.056 m

D. Seismic Load Calculation

The following parameters shall be considered as per IS: 1893-2016 (Part-1)

Seismic Zone = III

Seismic Zone Factor Z = 0.16

Structure Importance Factor (Table -8, IS1893-2016 (Part-1))

I = 1

Response Reduction Factor (Table-23, Chapter-12 – IS800-2007)

R = 3 (For OMF)

Response Reduction Factor R = 4 (For OCBF)

*As this a hybrid structure, we have considered R = 4 In Staad (Conservative Side)

Damping factor (Steel) = 5 %

Soil type for Spectral Acceleration Co-efficient (As Per Tender) = Soft

Fundamental period of vibration in seconds, T shall be estimated by Eigen value analysis using analysis model.

Design Horizontal Seismic Coefficient

Design Seismic Base Shear $V_b = A_h \times W$

Where,

"W" is the seismic weight of the building

"A_h" is Design Horizontal Seismic Coefficient

"Sa/g" is Average response acceleration co-efficient

For computing design seismic forces, following factors shall be considered

- 100% Dead load of structure, collateral loads,

E. Serviceability Requirements

Deflection limits followed as per table 6 of IS800-2007.

Table 5 – Deflection Limitation

Column (Lateral)	Lateral Deflection-Height/150
Rafter (Vertical)	Vertical deflection-Span/180
Girt	Vertical deflection-Span/150
Purlin	Vertical deflection-Span/150
Cantilever Canopy beam	Vertical deflection-Span/120

XI. ANALYSIS AND DESIGN CONSIDERATIONS

1) The lateral stability of the building is provided through the frame action of the rigid frame Structure.
2) The longitudinal stability of the building is provided through the system of cross bracing.
3) The sidewall girts are by pass beams (Continuous) supported at frame column location and span the bay spacing of the building.
4) The end wall girts are by pass beams (Continuous) supported at end wall column locations.
5) All columns are pinned to the base.
6) End frame frames are considered as non-expandable (Post & Beam).

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XII. LOAD COMBINATION

Figure – 16 Load Combination as per IS 800-2007

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	1.2 1.2	1.05 0.53	0.6 1.2	—	1.0	0.8	0.8	0.8
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**.

XIII. CONNECTION DESIGN - SAMPLE

A. Anchor Bolt And Base Plate Design

Material Properties

Concrete:

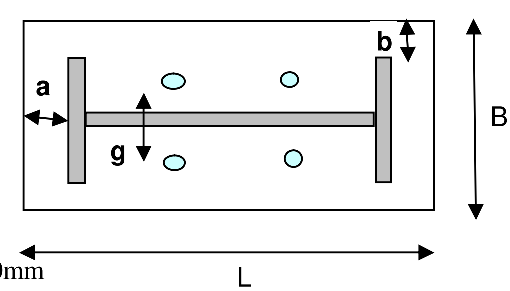
Grade of concrete = M25 = 25.0Mpa
Maximum Bearing Pressure = (0.45 F_{ck}) = 11.25Mpa IS 800:2007; Cl. 7.4.1

Steel Section:

Yield Stress F_y = 345Mpa
Ultimate Stress F_u = 490Mpa

Anchor Bolts: Grade 4.6
Ultimate Tensile Stress of Bolt f_{ub} = 400Mpa IS 1367 (Part 3) : 2002, Table 3
Yield Stress of Bolt f_{yb} = 240Mpa IS 1367 (Part 3) : 2002, Table 3
Total number of Anchor bolt N = 4Nos.

COLUMN SECTION DETAILS
Web Depth d_w = 300mm
Web Thickness t_w = 5mm
Flange Width B_f = 175mm
Flange Thickness t_f = 10mm



BASE PLATE DETAILS – Assume Dimensions
Length of Base Plate L = 350mm
Width of Base Plate B = 200mm
Cantilever along Length a or c = 15mm
Cantilever along Width b or c = 12.5mm

CONNECTION DETAILS – Assume

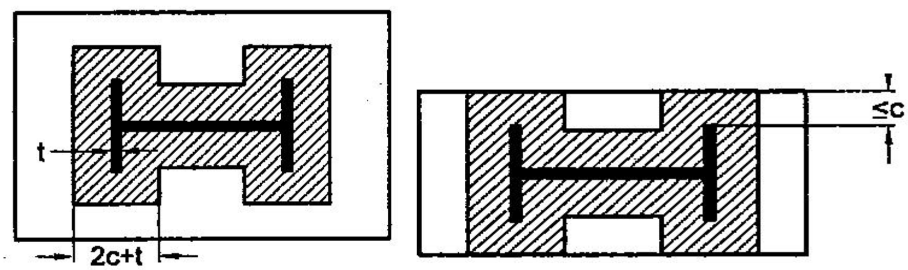
Diameter of Bolts to be used		d_b	=	20mm	
Diameter of Hole	d_h	=	(d+6)	=	26mm
Spacing/Pitch of Bolts Provided	g	=	100mm	$\geq 2.5*d_b$	
Spacing/Pitch of Bolts Provided	p	=	100mm		

1) Loads From Staad Output

LOAD COMBINATION	VERTICAL (KN)	SHEAR ALONG MAJOR AXIS(KN)
(DL + LL)	93.0	96.0
(DL + WL)	-159.0	96.0

2) Check For Bearing Pressure Under Base Plate

AS PER IS 800:2007; Cl. 7.4.3



Effective area of base Plate Aeff = $24100mm^2$

BEARING PRESSURE UNDER BASE PLATE

W = (P / A_{eff})

W = $3.86N/mm^2$ < $11.25N/mm^2$

[HENCE SAFE]

3) Check For Thickness Of Base Plate For Verticle Pressure

Max. Intensity of the Pressure Under Base Plate w = $3.86N/mm^2$

$$t_s = \sqrt{2.5\,w\,(a^2 - 0.3b^2)\gamma_{m0} / f_y} \quad > t_f$$

used $(a^2\text{-}0.3b^2)$ = c^2 **IS 800:2007; Cl. 7.4.3.1**

Thickness of Base Plate Required t_s = 2.63mm

4) Check For Thickness Of Base Plate For Uplift

Spacing of Bolts Provided along width g = 100mm

Spacing of Bolts Provided along length p = 100mm

Distance from bolt to flange A = 100mm

Maximum Tension Force acting on each Bolt T = 39750N

Uplift force in web per bolt row $P_w = \dfrac{2T}{1+\left(\frac{0.5g}{A}\right)^3}$

P_w = 70667N

Plate moment due to the uplift force in web M = $PL/8$

M = $2P_w\,(g\text{-}0.5d_b)/8$,

Where $P=2P_w$ & $L = (g\text{-}0.5d_b)$

M = 1590000N-MM

Thickness of Base plate required due to Uplift t_s = $\sqrt{(6 * M * Y_{m0})/(1.2 * f_y * d_w)}$

Where d_w = depth of web

t_s = 9mm

Hence, Provide Thickness of Base Plate t_s = 16mm

[HENCE SAFE]

5) Check For Bolts Subjected To Combined Shear And Tension

Gross Area of Bolts (A_{sb}) A_{sb} = $314mm^2$

Net Tensile/Shear Stress Area of Bolts (A_n) A_s = $245mm^2$

Partial safety factor for material Y_{mo} = 1.1

Partial safety factor for bolt Y_{mb} = 1.25

Factored shear force acting on each bolt V_{sb} = V/N

Where N = Nos. of Bolts

V_{sb} = 24kN

Factored shear force acting on each bolt T_b = T/N

Where N = Nos. of Bolts

T_b = 39.75kN

Nominal tensile capacity of bolt T_{nb} = $0.9 f_{ub} A_n < f_{yb} A_{sb} ({Y_{mb}}/{Y_{m0}})$

$0.9 f_{ub} A_n$ = 88.22kN

$f_{yb} A_{sb} ({Y_{mb}}/{Y_{m0}})$ = 85.68kN

T_{nb} = 85.68kN

Design Tensile capacity of bolt T_{db} = ${T_{nb}}/{Y_{mb}}$

T_{db} = 68.54kN

Nominal Shear capacity of bolt V_{nsb} = $\frac{f_u}{\sqrt{3}}(n_u A_{nb} + n_s A_{sb})$

Where n_n = 1, n_s = 0

= 56.59kN

Design Shear capacity of bolt V_{dsb} = ${V_{nsb}}/{Y_{mb}}$

V_{dsb} = 45.27kN

Design shear capacity of bolt V_{db} = 45.27kN

Design Tensile capacity of bolt T_{db} = 68.54kN

Factored shear force acting on each bolt V_{sb} = 24.00kN

Factored Tensile force acting on each bolt T_s = 39.75kN

[HENCE SAFE]

A Bolt subjected to Combined Shear & Tension shall satisfy $\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1.0$

IS 800:2007; Cl. 10.3.6

$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 = 0.62 < 1$$

[HENCE SAFE]

6) Calculation Of Anchor Bolt Length

Anchor Length required = T/(π*d*ζbd*N) **IS 456:2000, Cl. 26.2.1**

T = Total Tension in the Bolt

D = Diameter of Bolt

ζbd = Design Bond Strength of Concrete

N = No. of Bolts

For M25 Grade of Conctre, Bond Strength ζbd = 1.4 **IS 456:2000, Cl. 26.2.1.1**

Anchor Length required = 452mm

Anchor Length of Dia Bolt = 600mm

[HENCE SAFE]

7) Therfore provide 16mm thick base plate & 4nos. Of 20mm dia. Anchor bolt

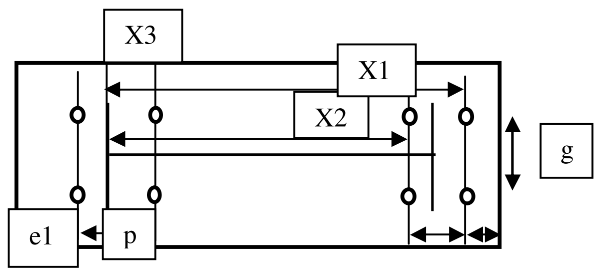
Design Of Knee Connection – Column To Rafter

LOAD FROM STAAD ANALYSIS

MOMENT (M) = 34kN-m

TENSION (P) = 127kN

SHEAR FORCE (S) = 73kN



BEAM SIZE

Web Depth = 600mm

Web Thk. = 6mm

Flange Width = 175mm

Flange Thk. = 8mm

COLUMN SIZE

Web Depth = 600mm

Web Thk. = 5mm

Flange Width = 175mm

Flange Thk. = 10mm

CONNECTION PLATE SIZE

Assume

Length = 820mm

Width = 250mm

Thickness = 25mm

Yield Stress = 345Mpa

USE OF HIGH TENSILE BOLTS:

Assume

Bolt Dia. = 27mm

No. of Bolts = 8Nos.

e1 = 50mm

e2 = 70mm

p/g = 110mm

LOADING PER BOLT

Shear per Bolt = S/Nos. of Bolts = 73/8

= 9.13kN

Tension per Bolt

Due to Axial Bolt= P/Nos. of Bolts = 127/8

= 15.88kN

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<u>Effective Lever Arm Calculation</u>

X1 = 670mm, $X1^2$ = 0.45m
X2 = 560mm, $X2^2$ = 0.31m
X3 = 55mm, $X3^2$ = 0.00m

Effective lever arm = $\frac{X_1}{\sum X^2}$ = 0.88m

Maximum Tension in extreme Bolt = $\frac{M * X_1}{\sum X^2}$ = 304.6kN

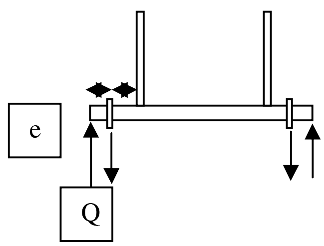
Number of rows = 2
Tension due to moment in each extreme Bolt = 152.288kN
Tension due to Axial Force = 15.88kN
Tension in each extreme Bolt (Moment + Axial Force) = 168.2kN

Prying Force Q = $\frac{l_v}{2l_e}\left[T_e - \frac{\beta \eta f_0 b_e t^4}{27 l_e l_v^2}\right]$

Where

Distance from the bolt centre line to toe of fillet weld l_v = 45mm

$l_e = 1.1t\sqrt{\frac{\beta f_0}{f_y}}$



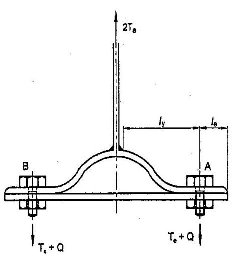


Fig. 16 Combined Prying Force and Tension

Distance between prying force and bolt centre line l_{e1} = 50mm
End distance l_{e2} = 50mm
Minimum of either of above two values l_{e2} = 50mm
2 for non pre tensioned bolt and 1 for pretensioned bolt β = 2
η = 1.5
Proof stress f_0 = 0.56kN/mm^2
Effective width of flange per pair of bolts b_e = 125mm
Assumed Fillet weld size = 6mm
Q = 62.61kN
Total Tension in each extreme Bolt (Tension + Prying Force) = 231kN
Shear in one bolt = 9.1kN

Design shear force for slip resistance (V_{dsf}) = $\frac{\mu_f * n_e * K_h * (F_0 - F * T_f)}{\Upsilon_{mf}}$

Where,

Coefficient of friction μ_f = 0.55

No. of effective interface n_e = 1.00

1 for clearance hole

0.85 for short, slotted holes K_h = 1.0

0.7 for long slotted holes

Partial slip factor for slip resistance γ_f = 1.25

Minimum bolt tension (proof load) F_0 = $0.7 * f_{ub} * A_n$

2 if external load is repetitive F = 2.0

1.7 if external load is non repetitive

Total external tension in each bolt T_f = 230.77kN

Design shear force for slip resistance (V_{dsf}) = 0.0kN < 9.13

(Connection will govern in bearing)

CHECK FOR BOLTS SUBJECTED TO COMBINED SHEAR AND TENSION

Gross Area of Bolts (A_{sb}) A_{sb} = 573mm^2

Net Tensile/Shear Stress Area of Bolts (A_n) A_n = 447mm^2

Partial safety factor for material Υ_{mo} = 1.1

Partial safety factor for bolt Υ_{mb} = 1.25

Ultimate Tensile Stress of Bolt f_{ub} = 800MPa

Yield Stress of Bolt f_{yb} = 640MPa

No. of Shear Plane with thread intercepting = 1

No. of Shear Plane without thread intercepting = 0

Ultimate Tensile Stress of PLATE f_u = 450MPa

Nominal tensile capacity of bolt T_{nb} = $0.9 f_{ub} A_n < f_{yb} A_{sb} (\Upsilon_{mb}/\Upsilon_{m0})$

$0.9 f_{ub} A_n$ = 321.5kN

$f_{ub} A_n A_{sb} (\Upsilon_{mb}/\Upsilon_{m0})$ = 416.4kN

T_{nb} = 321.5kN

Design Tensile capacity of bolt T_{db} = T_{nb}/Υ_{mb}

T_{db} = 257.2kN

Nominal Shear capacity of bolt V_{nsb} = $\frac{f_u}{\sqrt{3}}(n_u A_{nb} + n_s A_{sb})$

Where $n_n = 1$, $n_s = 0$

= 206.3kN

Design Shear capacity of bolt V_{dsb} = V_{nsb}/Υ_{mb}

V_{dsb} = 165.02kN

Distance between extreme rows of bolt l_j = 726 > 405

Reduction for Long Joint required

Reduction Factor for shear β_{ij} = 0.941,

where $0.75 \leq \beta_{ij} \leq 1.0$

Design shear capacity of bolt after the reduction V_{dsb} = 155.21kN

Reduction Factor for Bearing K_b = 0.602

K_b=min of $e/3d_0, p/3d_0-0.25, f_{ub}/f_u, 1$

Design Bearing Strength of the Bolt V_{dpb} = 365.964kN

Design shear capacity of bolt V_{db} = 155.21kN

Design Tensile capacity of bolt T_{db} = 257.20kN

Factored shear force acting on each bolt V_{sb} = 9.13kN
Factored Tensile force acting on each bolt T_b = 230.77kN
[HENCE SAFE]

A Bolt subjected to Combined Shear & Tension shall satisfy $\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1.0$

IS 800:2007; Cl. 10.3.6

$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 = 0.81 < 1$$

[HENCE SAFE]

CALCULATION OF THICKNESS OF CONNECTION PLATE

Bending Moment in the end plate at flange force

$T \times l_v - Q(l_e + l_v)$ = 4.46kN-m

At Bolt Line Q x le = 3.10kN-m

Design Bending Moment = 4.46kN-m

Thickness of Plate Required t = $\sqrt{\frac{4*Moment*1.1}{F_y*b_o}}$

t = 21.3mm < 25mm

[HENCE SAFE]

CALCULATION OF COLUMN WEB THICKNESS ADEQUACY CHECK

Total depth of beam = 616mm
Total depth of column = 620mm
Required web thickness = 5.02mm
[HENCE SAFE]

8) *Therfore provide 25mm thick connection plate & 8nos. Of 27mm dia. Connection bolt*

Design Of Pinned End Plate Connection

LOAD FROM STAAD ANALYSIS

TENSION (P) = 38kN
SHEAR FORCE (S) = 19kN

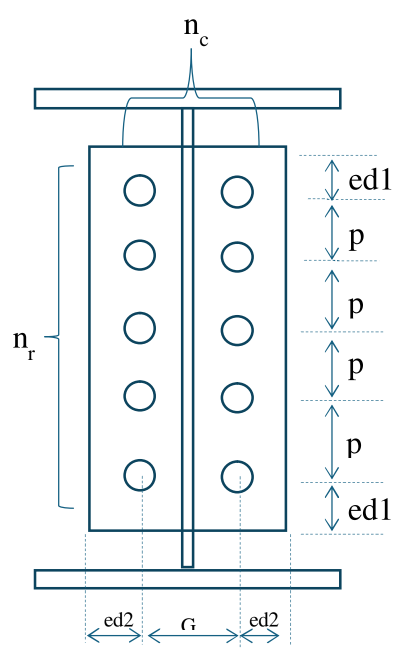
BEAM SIZE

Web Depth = 300mm
Web Thk. = 5mm
Flange Width = 130mm
Flange Thk. = 6mm

CONNECTION PLATE SIZE

Assume

Length = 320mm
Width = 200mm
Thickness = 12mm
Yield Stress = 345Mpa



<u>USE OF HIGH TENSILE BOLTS:</u>

Assume

Bolt Dia. = 16mm

No. of Bolts = 4Nos.

ed1 = 40mm

ed2 = 40mm

p/g = 90mm

LOADING PER BOLT

Shear per Bolt = S/Nos. of Bolts = 19/4

= 4.75kN

Tension per Bolt

Due to Axial Bolt= P/Nos. of Bolts = 78/4

= 9.5kN

Tension due to Axial Force = 9.5kN

Tension in each extreme Bolt (Moment + Axial Force) = 9.5kN

Prying Force Q = $\frac{l_v}{2l_e}\left[T_e - \frac{\beta\eta f_0 b_e t^4}{27 l_e l_v^2}\right]$

Where

Distance from the bolt centre line to toe of fillet weld l_v = 8.5mm

l_e = $1.1t\sqrt{\frac{\beta f_0}{f_y}}$

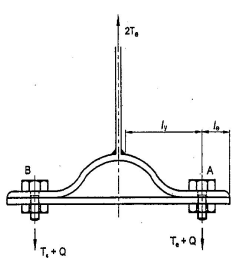
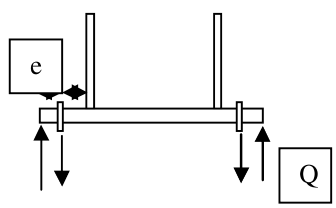


Fig. 16 Combined Prying Force and Tension



Distance between prying force and bolt centre line l_{e1} = 24mm

End distance l_{e2} = 40mm

Minimum of either of above two values l_{e2} = 24mm

2 for non pre tensioned bolt and 1 for pretensioned bolt β = 2

η = 1.5

Proof stress f_0 = 0.56kN/mm^2

Effective width of flange per pair of bolts b_e = 100mm

Assumed Fillet weld size = 6mm

Q = -11.72kN (-Ve)

= 0

Total Tension in each extreme Bolt (Tension + Prying Force) = 9.5kN

Shear in one bolt = 4.75kN

Design shear force for slip resistance (V_{dsf}) = $\frac{\mu_f * n_e * K_h * (F_0 - F * T_f)}{\Upsilon_{mf}}$

Where,

Coefficient of friction μ_f = 0.55

No. of effective interface n_e = 1.00

1 for clearance hole

0.85 for short, slotted holes K_h = 1.0

0.7 for long slotted holes

Partial slip factor for slip resistance y_f = 1.25

Minimum bolt tension (proof load) F_0 = $0.7 * f_{ub} * A_n$

= 87.82kN

2 if external load is repetitive F = 2.0

1.7 if external load is non repetitive

Total external tension in each bolt T_f = 9.5kN

Design shear force for slip resistance (V_{dsf}) = 30.3kN > 4.75

(Connection will govern in bearing)

Bearing capacity of plate/bolt = $\frac{2.5 * k_b * d * t * f_u}{\Upsilon_{mb}}$

= 122.7 > 4.75

(This check is not required as the connection as the resultant frictional force is more than shear)

<u>CHECK FOR BOLTS SUBJECTED TO COMBINED SHEAR AND TENSION</u>

Gross Area of Bolts (A_{sb}) A_{sb} = $201mm^2$

Net Tensile/Shear Stress Area of Bolts (A_n) A_n = $157mm^2$

Partial safety factor for material Υ_{mo} = 1.1

Partial safety factor for bolt Υ_{mb} = 1.25

Ultimate Tensile Stress of Bolt f_{ub} = 800MPa

Yield Stress of Bolt f_{yb} = 640MPa

No. of Shear Plane with thread intercepting = 1

No. of Shear Plane without thread intercepting = 0

Ultimate Tensile Stress of PLATE f_u = 450MPa

Nominal tensile capacity of bolt T_{nb} = $0.9 f_{ub} A_n < f_{yb} A_{sb} (\Upsilon_{mb} / \Upsilon_{m0})$

$0.9 f_{ub} A_n$ = 112.9kN

$f_{ub} A_n A_{sb} (\Upsilon_{mb} / \Upsilon_{m0})$ = 146.2kN

T_{nb} = 112.9kN

Design Tensile capacity of bolt T_{db} = T_{nb} / Υ_{mb}

T_{db} = 90.3kN

Nominal Shear capacity of bolt V_{nsb} = $\frac{f_u}{\sqrt{3}} (n_u A_{nb} + n_s A_{sb})$

Where $n_n = 1$, $n_s = 0$

= 72.4kN

Design Shear capacity of bolt V_{dsb} = V_{nsb} / Υ_{mb}

V_{dsb} = 57.95kN

Distance between extreme rows of bolt l_j = 90.3 > 240

Reduction for Long Joint required

Reduction Factor for shear β_{ij} = 1.0,

where $0.75 \leq \beta_{ij} \leq 1.0$

Design shear capacity of bolt after the reduction V_{dsb} = 57.95kN
Reduction Factor for Bearing K_b = 0.7407
K_b=min of $e/3d_0, p/3d_0-0.25, f_{ub}/f_u, 1$

Design Bearing Strength of the Bolt V_{dpb} = 128kN
Design shear capacity of bolt V_{db} = 57.95kN
Design Tensile capacity of bolt T_{db} = 90.30kN

Factored shear force acting on each bolt V_{sb} = 0.00kN
Factored Tensile force acting on each bolt T_b = 9.50kN
[HENCE SAFE]

A Bolt subjected to Combined Shear & Tension shall satisfy $\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1.0$

IS 800:2007; Cl. 10.3.6

$$\left(\frac{V_{sb}}{V_{db}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 = 0.01 < 1$$

[HENCE SAFE]

CALCULATION OF THICKNESS OF CONNECTION PLATE

Bending Moment in the end plate at flange force
$T \times l_v - Q(l_e+l_v)$ = 0.08kN-m
At Bolt Line $Q \times le$ = 0.00kN-m
Design Bending Moment = 0.08kN-m

Thickness of Plate Required $t = \sqrt{\frac{4*Moment*1.1}{F_y*b_0}}$
t = 3.20mm < 12mm
[HENCE SAFE]

9) *Therfore provide 12mm thick connection plate & 4nos. Of 16mm dia. Connection bolt*

Pipe Bracing Connection
Member Force = 58kN
Tension Member = ISNB 125 (L)
Length of Member (L) = 8.16m
Yield Strength = 250N/mm^2

STRENGTH DUE TO YIELDING OF GROSS SECTION

Strength due to yielding of gross section $T_g = \frac{A_g*f_y}{\Upsilon_{mo}}$ **IS 800:2007; Cl. 6.2**
Gross area of section A_g = 1910mm^2
Resistance governed by yielding Υ_{mo} = 1.1
Strength due to yielding of gross section T_g = 434.1kN > 58kN
[HENCE SAFE]

SLENDERNESS CHECK

L_{xx}/r_{xx} <= 180 **IS 800:2007; Table-3**
= 8160/47.8
L_{xx}/r_{xx} = 170.72 < 180

L_{yy}/r_{yy} <= 180
= 8160/47.8
L_{yy}/r_{yy} = 170.72 < 180
[HENCE SAFE]

CONNECTION DETAIL

Bolt Pattern = 2Rows
Bolt Dia. = 20mm Dia.
Total No. of Bolts = 4
Plate Thickness = 12mm Thk.
Weld Thickness = 3mm
Weld Length = 150mm

BOLT SHEAR CAPACITY

Shear Strength of bolt $V_{dsb} = \frac{V_{nsb}}{\Upsilon_{m1}}$ **IS 800:2007; Cl. 10.3.3**

Where $V_{nsb} = \frac{f_u}{\sqrt{3}}(n_n A_{nb} + n_s A_{ns})$

Shear Strength of one Bolt = 60.19kN
Shear Strength of 4Nos. of Bolts = 240.76kN > 58kN
[HENCE SAFE]

BEARING ON PLATE

Bearing strength of bolt on plate $V_{sb} = V_{npb}/\Upsilon_{mb}$

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

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

Bearing strength of 4Nos. of bolts on plate = 432kN > 58kN
[HENCE SAFE]

BLOCK SHEAR CHECK

Strength due to block shear $T_{db} = \frac{A_{vg} * f_y}{\sqrt{3} * \Upsilon_{m0}} + \frac{0.9 * A_{tn} * f_u}{\Upsilon_{m1}}$ **IS 800:2007; Cl. 6.4.1**

OR

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

Gross area in shear 5040mm^2
Net area in shear 3528mm^2
Gross area in tension 720mm^2
Net area in tension 504mm^2
Shear Strength due to block shear T_d = 885.771kN > 58kN
[HENCE SAFE]

WELD STRENGTH

Design weld strength $P_{wd} = L_w * t_e * \frac{f_u}{\sqrt{3} * \Upsilon_{mw}} * k_s$

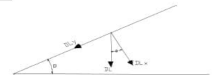
= 284.17kN > 58kN
[HENCE SAFE]

XIV. COLD-FORM DESIGN (PURLIN & GIRTS)

Figure – 17 to 26 Cold-form Design

DESIGN OF END ROOF PURLIN @6.42 M BAY SPACING

The Purlin shall be designed as 3-Span Continuous Purlin



LOAD CALCULATION :

Span of the Building	=	33.840	m		
Purlin Length	Le =	6.420			
Purlin Spacing (maximum)	Ps =	1.500	m		
Roof Slope (1:10)	X =	10		Kx =	0.995
	Y =	1		Ky =	0.100
Dead Load Intensity	DL =	10	Kg/m2		
Live Load Intensity	LL =	75	Kg/m2		
Wind Load Intensity	WL =	178.5	Kg/m2		
Total Pr. Co-eff for Wind =	Cp =	1.6			
Grade of Steel	Fya =	350	Mpa		
No of Sag Rod =		3	Nos.		

COMBINATION - I [DEAD LOAD + IMPOSE LOAD]

Total Load per metre = [(DL + LL+ CL) x Kx] = **126.87** Kg/m DOWNWARD

COMBINATION - II [DEAD LOAD + WIND LOAD]

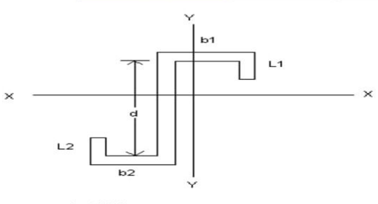
Total Load per metre = [(WL x Cp - DL x Kx)] = **310.11** Kg/m UPWARD

Design of Purlin for End Span :

Maximum Span Moment	(for full bay spacing)(DL+LL+CL)	M_{span}=	**418**	Kg-m
Maximum negative Moment near Support (for full bay spacing)(DL+LL+CL)		M_{supp}=	**523**	Kg-m
Maximum Span Moment	(for full bay spacing)(DL+WL)	M_{span}=	**1023**	Kg-m
Maximum negative Moment near Support (for full bay spacing)(DL+WL)		M_{supp}=	**1278**	Kg-m

Try with following Z-Section :-

t	d	b_1	b_2	L_1	L_2	D
2.5	195	64	67	25	25	200



Input the value of section properties in mm

X =	100.80	mm	=	**10.08**	**cm**
I_{xx} =	5579451.6	mm^4	=	**557.95**	**cm^4**
$Z1_{xx}$ top	55352.52	mm^3	=	**55.35**	**cm^3**
$Z1_{xx}$ bot	56243.63	mm^3	=	**56.24**	**cm^3**
Y =	63.45	mm	=	**6.35**	**cm**
I_{yy} =	889081.98	mm^4	=	**88.91**	**cm^4**
$Z_{yy\ right}$	14012.00	mm^3	=	**14.01**	**cm^3**
$Z_{yy\ left}$	13667.98	mm^3	=	**13.67**	**cm^3**
Area =	9.28	cm^2			
Wt/m =	7.28	Kg.			

As per IS: 801-1975 cl. No. 5.2.4

Overall Depth < 150*t

200 < **375** **OK**

Minimum overall depth required as per cl. No. 5.2.1.1 of IS : 801-1975

$=2.8\,t\,\sqrt[6]{(b1/t)^2 - 281200/Fy}$ but not less then 4.8t here Fy = 345 N/sqmm

20.63 not less then 4.8*t

20.63 > **12.00** **OK**

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Calculation for laterally unbraced Purlins

Calculation of effective design width of compression element as per cl. No. 5.2.1.1 of IS 801-1975

$(b1/t)_{lim}$ = $1435/(f)^{1/2}$

Considering f (actual stress in compression element) = 157.50 N/mm^2
= 1575.00 kgf/cm2

$\frac{w}{t} = \frac{b1-(t \times 1.5*2)-(t \times 2)}{2.50} = \frac{51.50}{2.50} = 20.60$

$\frac{1435}{\sqrt{f}} = \frac{1435}{39.69} =$ **36.16** > **20.60** **OK**

Hance full flange effective in compression.

Referring to Cl. No. 6.3 (b) of IS: 801-1975

$\frac{L^2 S_{xc}}{dI_{yc}}$

L= unbraced length of the member = 1.605 m

Iyc = moment of inertia of the compression portion of a section about the gravity axis of the entire section parallel to web = Iyy/2 44.45 cm4

Sxc= Compression Section Modulus of the entire section about major axis, Ixx/ distance to extreme fibre = Zx = 55.35 cm3

d = Depth of section = 20.00 cm

$\frac{L^2 \times Zx}{depth \times Iyc}$ = 1603.78 ——— 1

$\frac{0.18\ (Pi)^2\ E\ C_b}{Fy}$ = 1040.54 ——— 2

$\frac{0.90\ (Pi)^2\ E\ C_b}{Fy}$ = 5202.69 ——— 3

(i) is > **(ii)**
< **(iii)**

hence $Fb = \frac{2Fy}{3} - \frac{Fy^2}{2.7\ (pi)^2\ E\ Cb} \times \frac{L^2 S_{xc}}{d\ I_{yc}}$

$Fb = \frac{2 \times 3500}{3} - \frac{3500\text{^}2}{2.7x\ (pi)2 \times 2050000\ x1} \times 1603.78$

Fb = 2333.33 - 0.2242 x 1603.78

Fb = 1973.70 kg/cm^2 **OR** Fb = 197.37 N/mm^2

Refering to cl. No. 6.1 of IS 801-1975, Hance Fb = 197.37 N/mm2

Fb (actual) (Max Load Case) = (Span Moment)*10)/Zxx
= **184.73** N/mm^2 < **197.37** N/mm^2 **OK**

Fb (actual) (Max Load Case) (Support Moment*10)/Zxx
= **115.45** N/mm^2 < **197.37** N/mm^2 **OK**

Stress in Inclined Plane:

No of Sag Rod = 3 Nos

Total Load per metre = (DL+ LL+ COLL) x Ky) = 12.69 Kg/m

Maximum Span Moment over Sag Rod, M_{supp} = 2.52 Kg-m
= **252.30** Kg-cm

Developed Bending Stress σ_y = Mspan / Zy_{bot} = **18.01** **Kg/cm^2** <

Allowable Bending Stress σa = 0.6 x fy = **2100 Kg/cm2** **OK**

$\frac{\sigma_{dev,\ ver}}{\sigma_{per,ver}} + \frac{\sigma_{dev,\ hor}}{\sigma_{per,hor}}$ **0.945 <1.0** **OK**

Check for deflection.

COMBINATION - II [DEAD LOAD + WIND LOAD] = 3.10 kg/cm

Max. Deflection = 0.0065 $w_x Le^4/EI$ = **3.069** cm

Permissible deflection on purlin as per IS800-2007 = Span/ 150 42.80 mm

Actual Deflection for above cobination = **30.69** mm < **42.80** **OK**

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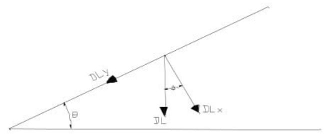
DESIGN OF INTERMEDIATE ROOF PURLIN @ 6.00 M BAY SPACING

The Purlin shall be designed as 3-Span Continuous Purlin

LOAD CALCULATION :



Span of the Building	=	33.840	m	
Purlin Length	Le =	6.770		
Purlin Spacing (maximum)	Ps =	1.500	m	
Roof Slope (1:10)	X =	10		Kx = 0.995
	Y =	1		Ky = 0.100
Dead Load Intensity	DL =	10	Kg/m2	
Live Load Intensity	LL =	75	Kg/m2	
Wind Load Intensity	WL =	179	Kg/m2	
Total Pr. Co-eff for Wind =	Cp =	1.14		Ref. Table 5, IS:875-(III)-1987
Grade of Steel	Fya =	350	Mpa	
No of Sag Rod =		2	Nos.	

COMBINATION - I [DEAD LOAD + IMPOSE LOAD]

Total Load per metre = [(DL + LL+ CL) x Kx] = **126.87** Kg/m DOWNWARD

COMBINATION - II [DEAD LOAD + WIND LOAD]

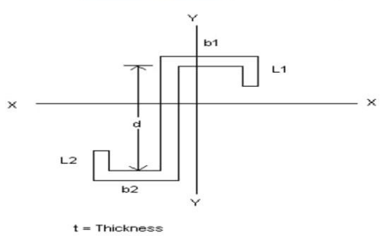
Total Load per metre = [(WL x Cp - DL x Kx)] = **217.73** Kg/m UPWARD

Design of Purlin for Intermediate Span :

Maximum Span Moment (for full bay spacing)(DL+LL+CL)	M_{span}=	**145**	Kg-m
Maximum negative Moment near Support (for full bay spacing)(DL+LL+CL)	M_{supp}=	**581**	Kg-m
Maximum Span Moment (for full bay spacing)(DL+WL)	M_{span}=	**249**	Kg-m
Maximum negative Moment near Support (for full bay spacing)(DL+WL)	M_{supp}=	**998**	Kg-m

Try with following Z-Section :-

t	d	b_1	b_2	L_1	L_2	D
1.75	197	64	67	25	25	200



Input the value of section properties in mm

X =	100.80 mm	=	**10.08**	cm
I_{xx} =	3972920.1 mm^4	=	**397.29**	cm^4
$Z1_{xx}$ top	39415.80 mm^3	=	**39.42**	cm^3
$Z1_{xx}$ bot	40047.63 mm^3	=	**40.05**	cm^3
Y =	63.83 mm	=	**6.38**	cm
I_{yy} =	645963.14 mm^4	=	**64.60**	cm^4
$Z_{yy\ right}$	10120.07 mm^3	=	**10.12**	cm^3
$Z_{yy\ left}$	9874.07 mm^3	=	**9.87**	cm^3
Area =	6.55 cm^2			
Wt/m =	5.14 Kg.			

As per IS: 801-1975 cl. No. 5.2.4

Overall Depth	<	150*t	
200	<	**262.5**	**OK**

Minimum overall depth required as per cl. No. 5.2.1.1 of IS : 801-1975

$=2.8\,t\ \sqrt[6]{(b1/t)^2 - 281200/Fy}$ but not less then 4.8t here Fy = 345 N/sqmm

16.26		not less then 4.8*t	
16.26	>	**8.40**	**OK**

Calculation for laterally unbraced Purlins

Calculation of effective design width of compression element as per cl. No. 5.2.1.1 of IS 801-1975

$(b1/t)_{lim} = 1435/(f)^{1/2}$

Considering f (actual stress in compression element) = 157.50 N/mm^2
= 1575.00 kgf/cm2

$\frac{w}{t} = \frac{b1-(t \times 1.5*2)-(t \times 2)}{1.75} = \frac{55.25}{1.75} = 31.57$

$\frac{1435}{\sqrt{f}} = \frac{1435}{39.69} =$ **36.16** > **31.57** **OK**

Hance full flange effective in compression.

Referring to Cl. No. 6.3 (b) of IS: 801-1975

$\frac{L^2 S_{xc}}{dI_{yc}}$

L= unbraced length of the member = 1.693 m

Iyc = moment of inertia of the compression portion of a section about the gravity axis of the entire section parallel to web = Iyy/2 32.30 cm4

Sxc= Compression Section Modulus of the entire section about major axis, Ixx/ distance to extreme fibre = Zx = 39.42 cm3

d = Depth of section = 20.00 cm

$\frac{L^2 \times Zx}{\text{depth} \times Iyc} = 1747.91$ —— 1

$\frac{0.18\,(Pi)^2\,E\,C_b}{Fy} = 1040.54$ —— 2

$\frac{0.90\,(Pi)^2\,E\,C_b}{Fy} = 5202.69$ —— 3

(i) is > **(ii)**
< **(iii)**

hence $Fb = \frac{2Fy}{3} - \frac{Fy^2}{2.7\,(pi)^2\,E\,Cb} \times \frac{L^2 S_{xc}}{d\,I_{yc}}$

$Fb = \frac{2 \times 3500}{3} - \frac{3500^2}{2.7x\,(pi)2 \times 2050000\,x1} \times 1747.91$

Fb = 2333.33 - 0.2242 x 1747.91

Fb = 1941.38 kg/cm^2 **OR** Fb = 194.14 N/mm^2

Refering to cl. No. 6.1 of IS 801-1975, Hance Fb = 194.14 N/mm2

Fb (actual) (Max Load Case) = (Span Moment)*10)/Zxx
= **63.30** N/mm^2 < **194.14** N/mm^2 **OK**

Fb (actual) (Max Load Case) (Support Moment*10)/Zxx
= **126.59** N/mm^2 < **194.14** N/mm^2 **OK**

Stress in Inclined Plane:

No of Sag Rod = 3 Nos

Total Load per metre = (DL+ LL+ COLL) x Ky) = 12.69 Kg/m

Maximum Span Moment over Sag Rod, M_{supp} = **2.81** Kg-m
= **280.56** Kg-cm

Developed Bending Stress σ_y = Mspan / Zy_{bot} = **27.72** Kg/cm^2 <

Allowable Bending Stress σa = 0.6 x fy = **2100** **Kg/cm2** **OK**

$\frac{\sigma_{dev,\,ver}}{\sigma_{per,ver}} + \frac{\sigma_{dev,\,hor}}{\sigma_{per,hor}}$ **0.339** **<1.0** **OK**

Check for deflection.

COMBINATION - II [DEAD LOAD + WIND LOAD] = 2.18 kg/cm

Max. Deflection = $0.0026\,w_xLe^4/EI$ = **1.497** cm

Permissible deflection on purlin as per IS800-2007 = Span/ 150 45.13 mm

Actual Deflection for above cobination = **14.97** mm < **45.13** **OK**

DESIGN OF END SIDE WALL GIRT @6.42 M BAY SPACING

The Girt shall be designed as 3-Span Continuous Girt

LOAD CALCULATION :

Span of the Building	=	33.840	m
Girt Length	Le =	6.420	m
Girt Spacing (maximum)	Ps =	1.430	m
Dead Load Intensity	DL =	10	Kg/m2
Wind Load Intensity	WL =	178.5	Kg/m2
Total Pr. Co-eff for Wind =	Cp =	1	
Grade of Steel	Fy =	350	Mpa
No of Sag Rod =		2	Nos

Ref. Table 5, IS:875-(III)-1987

FOR DEAD LOAD -

Total Load per metre, W_{dl} = [DL] x Ps = **14.30** Kg/m DOWNWARD

Moment at Span, Mspan = **11.79** Kg-m

Moment at Support, Msupp = **14.73** Kg-m

FOR WIND LOAD -

FOR END GIRTS:

Total Load per metre, Wwl = [(WL x Cp) x Ps] = **191.44** Kg/m HORIZONTAL

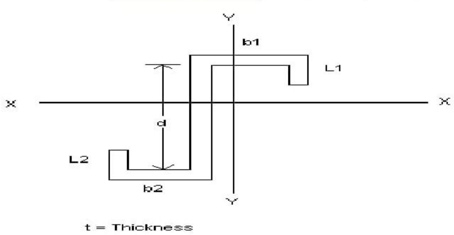
Moment at Span, Mspan = **631.2** Kg-m

Moment at Support, Msupp = **789.1** Kg-m

Design of Girt for End Span :

Try with following Z-Section :-

t	d	b_1	b_2	L_1	L_2	D
1.75	197	64	67	25	25	200



X =	100.80 mm	=	**10.08**	cm
I_{xx} =	3972920.1 mm^4	=	**397.29**	cm^4
$Z1_{xx}$ top	39415.80 mm^3	=	**39.42**	cm^3
$Z1_{xx}$ bot	40047.63 mm^3	=	**40.05**	cm^3
Y =	63.83 mm	=	**6.38**	cm
I_{yy} =	645963.14 mm^4	=	**64.60**	cm^4
$Z_{yy\ right}$	10120.07 mm^3	=	**10.12**	cm^3
$Z_{yy\ left}$	9874.07 mm^3	=	**9.87**	cm^3
Area =	6.55 cm^2			
Wt/m =	5.14 Kg.			

Input the value of section properties in mm

As per IS: 801-1975 cl. No. 5.2.4

Overall Depth < 150*t

200 < **262.5** **OK**

Minimum overall depth required as per cl. No. 5.2.1.1 of IS : 801-1975

$=2.8\,t\ \sqrt[6]{(b1/t)^2 - 281200/Fy}$ but not less then 4.8t here Fy = 345 N/sqmm

16.26 not less then 4.8*t

16.26 > **8.40** **OK**

Calculation for laterally unbraced Girts

Calculation of effective design width of compression element as per cl. No. 5.2.1.1 of IS 801-1975

$(b1/t)_{lim}$ = $1435/(f)^{1/2}$

Considering f (actual stress in compression element) = 157.50 N/mm^2

= 1575.00 kgf/cm2

$\frac{w}{t}$ = $\frac{b1-(t \times 1.5*2)-(t \times 2)}{1.75}$ = $\frac{55.25}{1.75}$ = 31.57

$\frac{1435}{\sqrt{f}}$ = $\frac{1435}{39.69}$ = **36.16** > **31.57** **OK**

Hance full flange effective in compression.

Referring to Cl. No. 6.3 (b) of IS: 801-1975

$\frac{L^2 S_{xc}}{dI_{yc}}$

L= unbraced length of the member = 2.14 m

Iyc = moment of inertia of the compression portion of a section about the gravity axis of the entire section parallel to web = Iyy/2 32.30 cm4

Sxc= Compression Section Modulus of the entire section about major axis, Ixx/ distance to extreme fibre = Zx = 39.42 cm3

d = Depth of section = 20.00 cm

$\frac{L^2 \times Zx}{\text{depth} \times Iyc}$ = 2794.41 — 1

$\frac{0.18\,(Pi)^2\,E\,C_b}{Fy}$ = 1040.54 — 2

$\frac{0.90\,(Pi)^2\,E\,C_b}{Fy}$ = 5202.69 — 3

(i) is > **(ii)**
< **(iii)**

hence Fb = $\frac{2Fy}{3} - \frac{Fy^2}{2.7\,(pi)^2\,E\,Cb} \times \frac{L^2 S_{xc}}{d\,I_{yc}}$

Fb = $\frac{2 \times 3500}{3} - \frac{3500\text{^}2}{2.7x\,(pi)2 \times 2050000\,x1} \times 2794.41$

Fb = 2333.33 - 0.2242 x 2794.41

Fb = 1706.71 kg/cm^2 OR Fb = 170.67 N/mm^2

Refering to cl. No. 6.1 of IS 801-1975, Hance Fb = 170.68 N/mm2

Fb (permissible) = 170.67 N/mm^2

Fb (actual) (Max Load Case) = (Span Moment)*10)/Zxx
= **160.15** N/mm^2 < **170.67** N/mm^2 **OK**

Fb (actual) (Max Load Case) = (Support Moment*10)/Zxx
= **100.09** N/mm^2 < **170.67** N/mm^2 **OK**

Check for deflection.

Wind load (WL) = 1.91 kg/cm

Max. Deflection = 0.0065 $w_x Le^4/EI$ = **2.66** cm

Permissible deflection on purlin as per Tender = Span/150 for DL+IL = 42.80 mm

Actual Deflection for above load = **26.60** mm < **42.80** **OK**

DESIGN OF INTERMEDIATE SIDE WALL GIRT @ 6.00 M BAY SPACING

The Girt shall be designed as 3-Span Continuous Girt

LOAD CALCULATION :

Span of the Building	=	33.840	m	
Girt Length	Le =	6.770	m	
Girt Spacing (maximum)	Ps =	1.500	m	
Dead Load Intensity	DL =	10	Kg/m2	
Wind Load Intensity	WL =	179	Kg/m2	
Total Pr. Co-eff for Wind =	Cp =	0.9		Ref. Table 5, IS:875-(III)-1987
Grade of Steel	Fy =	350	Mpa	
No of Sag Rod =		2	Nos	

FOR DEAD LOAD -

Total Load per metre, W_{dl} = [DL] x Ps =	**15.00**	Kg/m	DOWNWARD
Moment at Span, Mspan =	**4.30**	Kg-m	
Moment at Support, Msupp =	**17.19**	Kg-m	

FOR WIND LOAD -

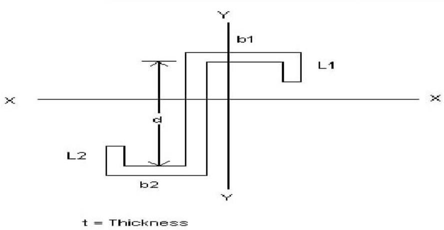
FOR END GIRTS:

Total Load per metre, Wwl = [(WL x Cp) x Ps] =	**180.73**	Kg/m	HORIZONTAL
Moment at Span, Mspan =	**207.1**	Kg-m	
Moment at Support, Msupp =	**828.3**	Kg-m	

Design of Girt for Intermediate Span :

Try with following Z-Section :-

t	d	b_1	b_2	L_1	L_2	D
1.75	197	64	69.5	25	25	200



Input the value of section properties in mm

X =	101.45	mm	=	**10.14**	**cm**
I_{xx} =	4014940.0	mm^4	=	**401.49**	**cm^4**
$Z1_{xx}$ top	39576.32	mm^3	=	**39.58**	**cm^3**
$Z1_{xx}$ bot	40739.32	mm^3	=	**40.74**	**cm^3**
Y =	64.43	mm	=	**6.44**	**cm**
I_{yy} =	678562.18	mm^4	=	**67.86**	**cm^4**
$Z_{yy\ right}$	10532.27	mm^3	=	**10.53**	**cm^3**
$Z_{yy\ left}$	10079.20	mm^3	=	**10.08**	**cm^3**
Area =	6.59	cm^2			
Wt/m =	5.17	Kg.			

As per IS: 801-1975 cl. No. 5.2.4

Overall Depth	<	150*t	
200	<	**262.5**	**OK**

Minimum overall depth required as per cl. No. 5.2.1.1 of IS : 801-1975

$=2.8\,t\ \sqrt[6]{(b1/t)^2 - 281200/Fy}$ but not less then 4.8t here Fy = 345 N/sqmm

16.26		not less then 4.8*t	
16.26	>	**8.40**	**OK**

Calculation for laterally unbraced Girts

Calculation of effective design width of compression element as per cl. No. 5.2.1.1 of IS 801-1975

$(b1/t)_{lim}$ = $1435/(f)^{1/2}$

Considering f (actual stress in compression element) = 157.50 N/mm^2

= 1575.00 kgf/cm2

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$\frac{w}{t} = \frac{b1-(t \times 1.5*2)-(t \times 2)}{1.75} = \frac{55.25}{1.75} = 31.57$

$\frac{1435}{\sqrt{f}} = \frac{1435}{39.69} =$ **36.16** > **31.57** **OK**

Hance full flange effective in compression.

Referring to Cl. No. 6.3 (b) of IS: 801-1975

$\frac{L^2 S_{xc}}{dI_{yc}}$

L= unbraced length of the member = 2.26 m

Iyc = moment of inertia of the compression portion of a section about the gravity axis of the entire section parallel to web = Iyy/2 33.93 cm4

Sxc= Compression Section Modulus of the entire section about major axis, Ixx/ distance to extreme fibre = Zx = 39.58 cm3

d = Depth of section = 20.00 cm

$\frac{L^2 \times Zx}{\text{depth} \times Iyc}$ = 2970.16 —— 1

$\frac{0.18\,(Pi)^2\,E\,C_b}{Fy}$ = 1040.54 —— 2

$\frac{0.90\,(Pi)^2\,E\,C_b}{Fy}$ = 5202.69 —— 3

(i) is > **(ii)**
< **(iii)**

hence $Fb = \frac{2Fy}{3} - \frac{Fy^2}{2.7\,(pi)^2\,E\,Cb} \times \frac{L^2 S_{xc}}{d\,I_{yc}}$

$Fb = \frac{2 \times 3500}{3} - \frac{3500\text{^}2}{2.7x\,(pi)2 \times 2050000\,x1} \times 2970.16$

Fb = 2333.33 - 0.2242 x 2970.16

Fb = 1667.29 kg/cm^2 OR Fb = 166.73 N/mm^2

Refering to cl. No. 6.1 of IS 801-1975, Hance Fb = 166.73 N/mm2

Fb (permissible) = 166.73 N/mm^2

Fb (actual) (Max Load Case) = (Span Moment)*10)/Zxx

= **52.33** N/mm^2 < **166.73** N/mm^2 **OK**

Fb (actual) (Max Load Case) = (Support Moment*10)/Zxx

= **104.65** N/mm^2 < **166.73** N/mm^2 **OK**

Check for deflection.

Wind load (WL) = 1.81 kg/cm

Max. Deflection = 0.0026 wxLe4/EI = = **1.23** cm

Permissible deflection on purlin as per Tender = Span/150 for DL+IL = 45.13 mm

Actual Deflection for above load = **12.29** mm < **45.13** **OK**

XV. CONCLUSION

In this paper we have effectively noticed that PEB structures can be easily designed effortlessly using software and simple calculations for connection design. By using Cold-form sections for sheeting support, dead load on the structure can be reduced. By using simpler profiles like "I", "C", "Z", etc. sections PEB structures can be constructed fastly which end up in energy saving and cost effective against conventional steel structures. There are many choices that can be made in structural configuration of PEB.

REFERENCES

[1] N. Subramanian,2010 "Steel Structures Design and Practice" oxford University press.
[2] The relevant Standard/Codes used for the design for various elements and components of the building are given below

Table 6 – IS Code used in Building Design

S. No	Code	Description
1.	IS:875(Part-1)-1987	Code of Practice for Design Loads (other than earthquake) for buildings and structures – Unit weights of buildings materials and stored material.
2.	IS:875(Part-2)-1987	Code of Practice for Design Loads (other than earthquake) for buildings and structures – Imposed loads.
3.	IS:875(Part-3)-2015	Code of Practice for Design Loads (other than earthquake) for buildings and structures – Wind loads.
4.	IS:1893(Part-1)-2016	Criteria for Earthquake Resistant Design of Structures-General Provisions and Buildings
5.	IS:1893(Part-4)-2015	Criteria for Earthquake Resistant Design of Structures-Industrial Structures including Stack-Like Structure
6.	IS: 800-2007	Code of Practice for General Construction in Steel
7.	IS:2062-2011	Hot rolled low, medium and high tensile Structural Steel.
8.	IS: 1161-1998	Specification for Steel tubes for Structural Purposes.
9.	IS:4923-1997	Hollow Steel Sections for Structural use
10.	IS:808-1989	Dimensions for hot rolled steel beams, columns, channels and angle sections
11.	IS: 801-1975	Code of practice for Cold-formed Light gauge steel structural members in General Building Construction
12.	SP	Special Publications of Bureau of Indian Standards

Annexure – A

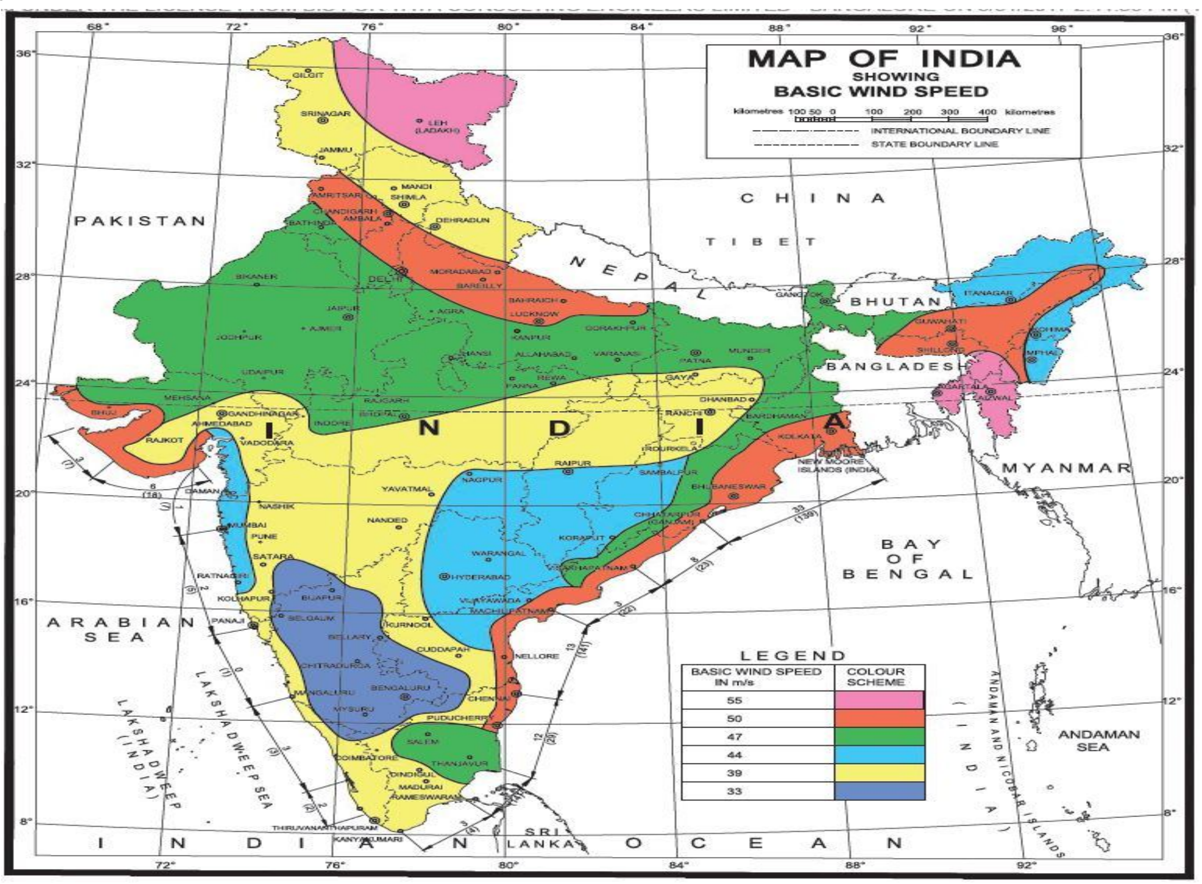


Figure 27 – India Wind speed map As per IS 875 Part-3 (2015)

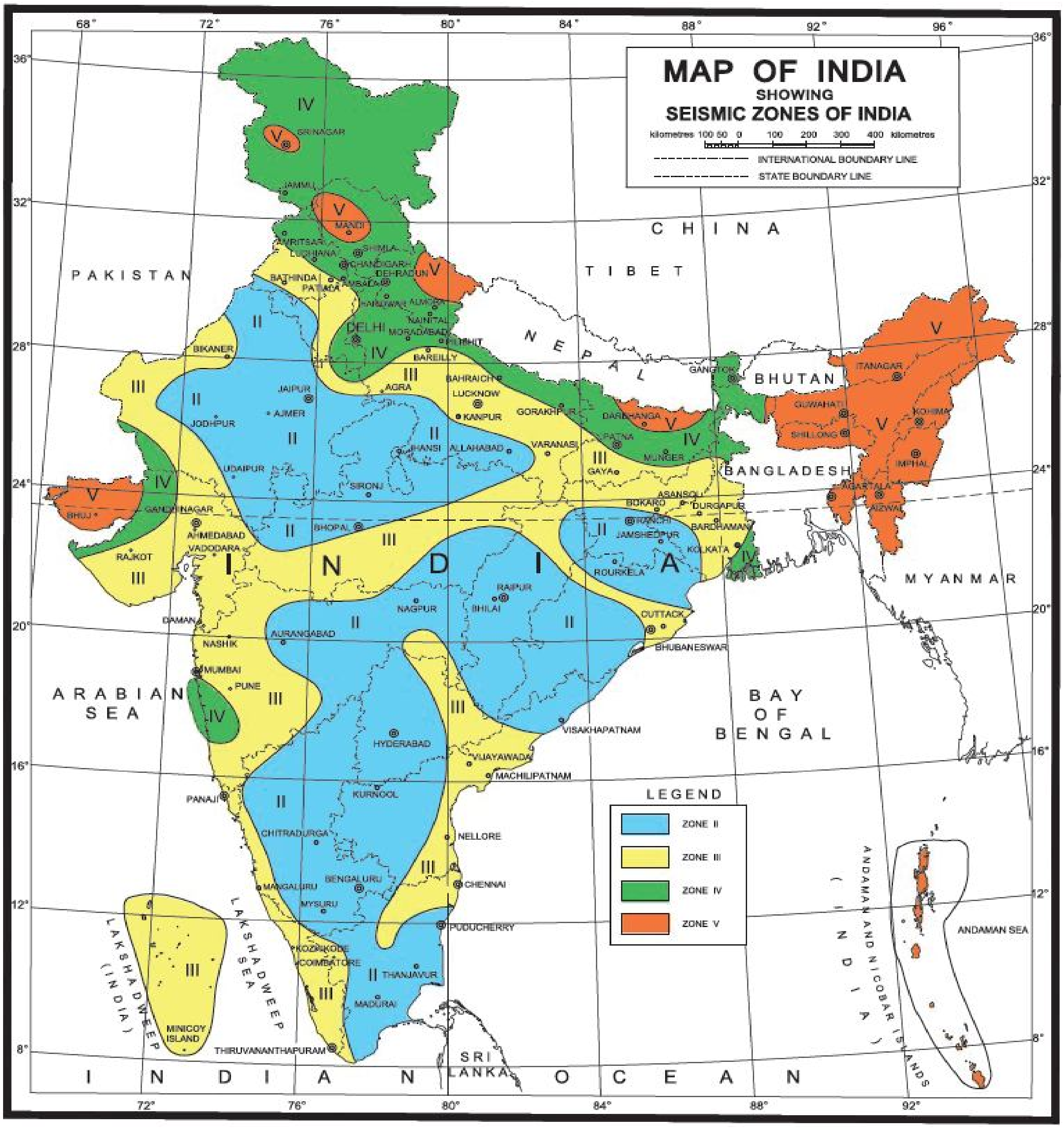
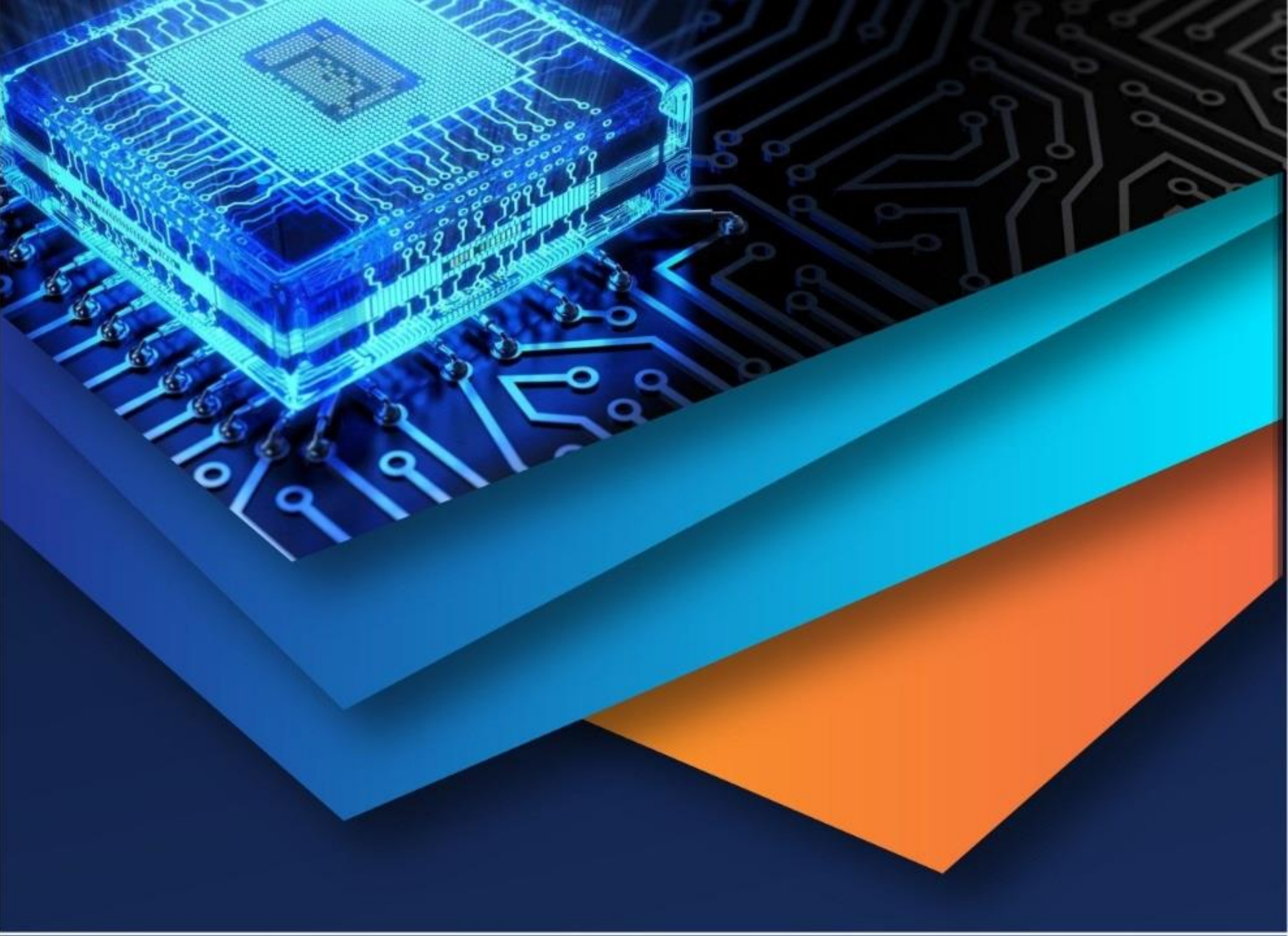


Figure 28 – Inida Seismic zone Map As per 1893 Part-1 (2016)

Annexure – B

Figure –General Arrangement Drawing



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