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A Case Study of Low Span Pre-Engineered Industrial Building

[1] R. D. Shambharkar, [2] P. A. Deotale, [3] Vivek Naik, [4] D. K. Parbat
[1] Assistant Professor, DBACER, [2] Assistant Professor, DBACER,

Abstract: **-- The design of an industrial building is governed mainly by functional requirements and the need for economy of construction. The main dimensions will be directed by the particular operational activities involved, but the structural designer's input on optimum spans and the selection of suitable cross-sections profile can have an important bearing on achieving overall economy. An aspect where the structural designer can make a more direct contribution is in lengthwise dimensions i.e. the bay lengths of the building. Here a balance must be struck between larger bays involving fewer, heavier main components such as columns, trusses, purlins, crane beams, etc. and smaller bays with a large number of these items at lower unit mass. An important consideration in this regard is the cost of foundations since a reduction in the number of columns will always result in lower foundation costs. In this a Case Study of Low Span Pre-Engineered Industrial building located at Vijayawada, used for the Cement Godown having building width of 15m, length of 50m and height of 5m. The minimum total weight of PEIB was found to be 22.51 Tonnes and optimized cost of this building was estimated as 12.54 Lakhs at the spacing of 7.14m as against conventional having a total weight of 34.4 Tonnes and cost of 14.448 Lakhs at the bay spacing of 6.25m.Therefore material saved by using PEIB was 11.73 Tonnes (32.85%) & net money saved by using PEIB was 1.91 Lakhs.**

I. INTRODUCTION

The design of industrial building is governed mainly by functional requirements and the need for economy of construction. In cross-sections these buildings will range from single or multi-bay structures of larger span when intended for use as ware houses or aircraft hangers to smaller span buildings as required for factories, assembly plants, maintenance facilities, packing plants etc. The main dimensions will be directed by the particular operational activities involved, but the structural designer's input on optimum spans and the selection of suitable cross-sections profile can have an important bearing on achieving overall economy. In small span Pre-engineered Industrial building span range is kept between 10m-18m. The available profiles of slopes for low span industrial building are 1:10, 1:12 &1:20.

1 Case Study of Low Span Pre-Engineered Industrial building

Location: Vijayawada
Utility: Cement Godown
Building width: 15m
Building Length: 50m
Eave height: 5m
C/C of Main frames: 6.25m
Maximum spacing of Purlin & Girt: 1.5m
Slope of Roof: 1:12
Structural material yield stress: 345MPa except bracing rods & Base plate. Area covered: 15mx50m

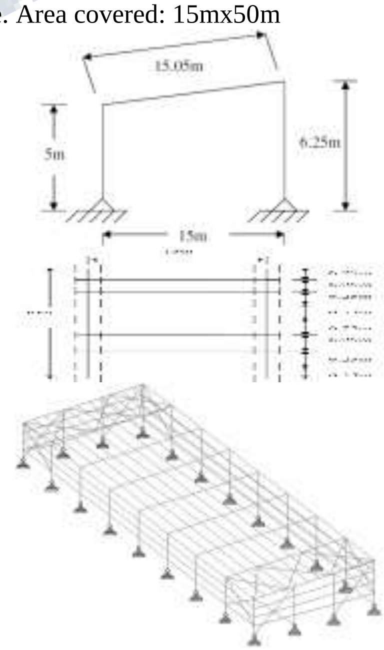

Figure-1.1 Plan and Elevation and 3D View of low Span PEIB

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The following codes have been used for the design of members

Basic codes:

IS: 800-1984: Code of practice for general construction in steel.

IS: 801-1975: Code of practice for use of Cold-formed light gauge steel structural member's in general building construction.

Other codes:

IS: 875(Part2)-1987: Imposed loads

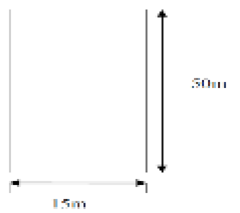
IS: 875(Part3)-1987: Wind loads

BS: 5950(part5)-1998: Code of practice for design of Cold-formed thin gauge sections.

"Metal building systems Manual-1996" Metal buildings manufacturers association.

1 Loading

Imposed Load



As per Table 2 of IS: 875(Part2) UDL on roof measured on plan area for slope less than10^0 = 75 kg/m^2

Wind Load

According to Clause 5.3 of IS: 875(Part3)

$V_z = V_b K_1 K_2 K_3$

Location assumed – Vijay Wada

Basic Wind speed V_b = 50 m/s

K_1= 1.0 i.e. Risk co-efficient from table 1 of IS: 875 (Part3)

K_2= 0.88 from table 2 of IS: 875 (Part3)

K_3=1.0 Topography factor

BUILDING PLAN

Design Wind speed V_z = 50x1x0.88x1 =

Design Wind Pressure $P_z = 0.6V_z^2 = 0.6x44^2 = 1161.6\ N/m^2$

Calculation of external pressure coefficient *"C_{pe}"*

i). Roof

Referring to Table 6 of IS: 875(Part3)

Here h = 5m; w =15m

Roof Angle ≅5°

a) Wind across Length of Building

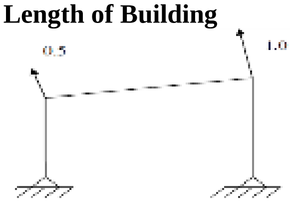


Figure 1.2 (a) Wind across Length of Building

b) Wind Along length of building

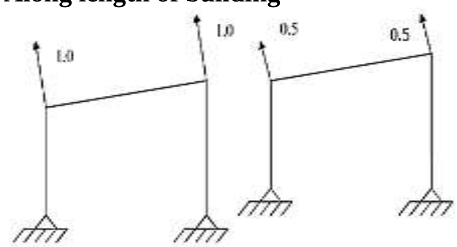


Figure 1.3 (a) Wind along Length of Building

ii) Walls

Referring to Table 4 of IS: 875(Part3)

$$\frac{h}{w} = \frac{5}{15} = \frac{1}{3} < \frac{1}{2}$$

$$\frac{l}{w} = \frac{50}{15} = 3.33 < 4$$

a) Wind across and along Length of Building

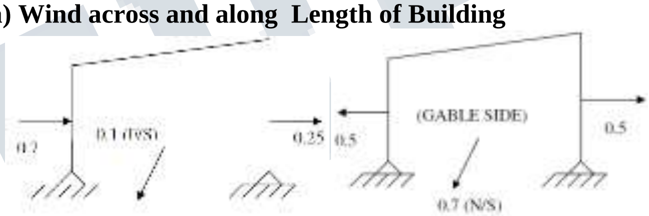


Fig 1.4 (a) Wind across Length of Building. (b) Wind along Length of Building

Considering openings to be <5% of Total Area.

Internal pressure coefficient $C_{pe} = \pm 0.2$

2 Purlin design

a) Dead Load

Unit wt. /m of sheeting @ 0.06 kN/m^2 = 0.06 x 1.5 = 0.09 kN/m

Unit wt. /m or Self wt. of Purlin = 0.07 kN/m

Total Dead Load per meter on each Purlin = 0.16 kN/m (↓)

a) Imposed Load

Imposed Load intensity on Purlin = 0.75 kN/m^2

Total Imposed Load per meter on each Purlin=0.75x1.5=1.125 kN/m (↓)

c) Wind Load

Maximum Wind Load per meter on each Purlin = (1+0.2) x 1.161 x 1.5 = 2.09 kN/m(↑)

Since the Slope is very small, all the forces are assumed to act vertically w. r. t. Purlin section

Load Combination1- Dead Load + Imposed Load

Total Load per meter on Each Purlin = 0.16+1.125 =1.285KN/m (↓)

Load Combination2- Dead Load + Wind Load

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Total Load per meter on Each Purlin = 0.16 - 2.09=1.93KN/m (↑)
Hence" Load Combination 2" is more critical
Considering the Purlin as Continuous members with overlap of 285 mm on either side for splicing purpose as shown in figure below:

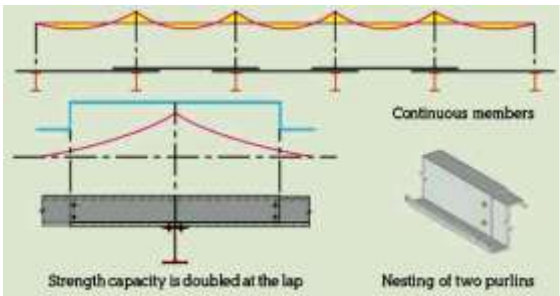
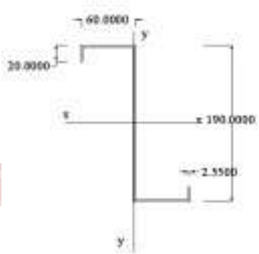


Figure 1.5 Overlapping of Purlin Sections

According to STAAD analysis, Max. Span Moment= 5.84 KN-m
Max. Negative Moment near support at a distance of 285mm from support = 5.959 KN-m.



Property of the Z-section selected is as shown below:

Table 2.1 Property of the Z-section

Area	8.37sq.cm
Thickness	2.55mm
Weight/meter	6.57Kg/m
I_{xx}	439cm^4
I_{yy}	58.8cm^4

Fig 2.1 Z- Sections

Z_{xx}	46.21cm^3
Z_{yy}	10.01cm^3

Checking the above section based on Section 9 of BS: 5950 Part5-1998

1. Overall Depth < 100 t

190mm < 100x2.55
190 < 255 (O.K)

2. Overall Width of Compression Flange/ Thickness i.e. B/t ≤ 35

60/2.55 = 23.52mm < 35mm (O.K.)

3. Width of Lip > B/5

20mm > 60/5
20mm > 12mm (O.K)

4. Total Width over both Flange>L/60

60x2 > 6250/60
120 > 104.17(O.K)

5. Zxx provided > WL/1800cm^3

Where W=D.L + L.L = 1.285kN/m
46.21cm^3 > 1.285 x 6.25 x 6250/1800
46.21 cm^3 > 27.89 cm^3 (O.K)

Checking the above section based on IS: 801-1975.

W/t = (60-2x5.55)/2.55 = 19.17, considering inside bending radius = 3mm

1) Minimum Overall depth required as per clause no.5.2.2.1 of IS: 801-1975.

$D = 2.8t^6\sqrt{(w/t)^2 - 281200/f_y}$ But not less than 4.8t

Here, f_y = 3450 kg/cm^2

$$D = 2.8\text{x}0.255^6\sqrt{(19.17)^2 - 281200/3450} = 1.83\text{cm}$$
$$= 18.3\text{m} < 20mm$$

4.8t = 4.8x2.55 = 12.24mm < 18.3mm (O.K)

2) Calculation for Laterally unbraced beams

Calculation of effective design width of compressive element as per clauseno.5.2.2.1 of IS: 801-1975

$= \left(\frac{w}{t}\right)_{\text{lim}} = \frac{1435}{\sqrt{f}} = 41.4$ Where f = 0.75 x 1600 = 1200 kg/cm^2

w = 41.4 x 2.55 = 105.57 mm > (r 60 – 2 x 5.55) i.e. 48.9 mm
Hence full flange is effective in compression.
Referring to clause no.6.3 (b) of IS: 801-1975.

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

Where, L = Unbraced Length of the member
= 2.08m (considering sag rods at spacing 2.08m)
I_{yc} = Moment of inertia of the compression portion of a section about the gravity axis of the entire section parallel to the web
$I_{y/2}$ = 58.8/ 2 = 29.4 cm^4
S_{xc} = Z_{xx}= 46.21 cm^3
d = Depth of the section =190mm

$$\frac{L^2 S_{xc}}{dI_{yc}} = \frac{208^2\text{x}46.21}{19\text{x}29.4} = 3579 \quad \text{(A)}$$

$$\frac{0.18\pi^2 EC_b}{fy} = 1030 \quad \text{(B)}$$

$$\frac{0.9\pi^2 EC_b}{fy} = 5150 \quad \text{(C)}$$

Where, E = 2 x 10^6 kg/cm^2
f_y = 3450 kg/cm^2
(A) > (B)
(A) < (C)

Hence $F_b = \frac{2}{3}f_y - \frac{f_y{}^2}{2.7\pi^2 EC_b}\left(\frac{L^2 \text{ x } S_{xc}}{d \text{ x } I_{yc}}\right)$

$$F_b = \frac{2}{3}\text{ x }3450 - \frac{3450^2}{2.7\pi^2\text{x}2.\text{x}10^2\text{x}1}\text{ x }3579$$
$$= 1500.7 \text{ kg/cm}^2$$

$F_b = 150.07 \text{ N/mm}^2$

Referring to Clause no. 6.1 of IS: 801-1975
F_b= (Basic design stress) = 0.6 F_y = 0.6 x 3450 =2070kg/cm^2

F = 207 N/mm^2
Hence, $F_b = 150.07\ N/mm^2$ is the governing value
Since here Wind load condition is critical,
F_{bperm} = 1.33 x 150.07 = 199.59 Mpa

$$F_{b_{act}} = \frac{M}{Z_{xx}} = \frac{5.84 \text{ x } 10^6}{46.21 \text{ x } 10^3}$$

$$F_{b_{act}} = 126.37\text{Mpa} < 199.59Mpa\ (safe)$$

Check for Deflection according to M. B. M. A (Due to Live load only)

a) Permissible Deflection due to Imposed load on Purlin as per MBMA = span/240.

$$\delta_{perm} = \frac{6250}{240} = 26.04 \text{ mm}$$

Load due to Imposed load only = 1.125 x 6.25=7.03 kN.
b) Calculated Deflection due to Imposed load on Purlin

$$\delta_{cal} = \frac{5\text{xwl}^4}{384\text{EI}} = \frac{5\text{xWl}^3}{384\text{EI}} = \frac{5\text{x}7.03\text{x}10^3\text{x}6250^3}{384\text{x}2\text{x}10^5\text{x}439\text{x}10^4} = 25.46\text{mm} < 26.04mm$$

Hence found safe when checked for deflection.
Calculation for Shear Stress in web
Referring to Clause .no. 6.4 of IS: 801-1975
h = 190 - (2x2.55) = 178.9mm
h / t = 70.15

$$\frac{4590}{\sqrt{F_y}} = 78.14 > 70.15$$

Hence Maximum Average Permissible shear stress, $F_v = \frac{1275\sqrt{F_y}}{\frac{h}{t}}$

$$F_v = \frac{1275\sqrt{1380}}{70.15} = 675.18 \text{ kg/cm}^2$$

Maximum F_y = 0.4 F_y = 0.4 x 3450 = 1380 kg/cm^2
F_v = 67.51 N/mm^2
Actual Shear stress for Dead Load + Wind Load = 1.93 kN/m

$$\text{Actual Shear stress} = \frac{1.93\text{x}10^3\text{x}6.25}{178.9\text{x}2.55} = 26.45 \text{ N/mm}^2$$

= 26.45 Mpa < 67.51 x1.33
= 26.45 Mpa < 89.78 Mpa

Check for Combined Shear & Bending Stress in Web

Referring to Clause No. 6.4.2 & 6.4.3 of IS: 801-1975

$$F_{bw} = \frac{36560000}{\left(\frac{h}{t}\right)^2} = \frac{36560000}{(70.15)^2} = 7429.3 \text{ kg/cm2}$$

F_{bw} = 742.93 N/mm^2
F = 0.6 x 33450 = 2070 kg/cm^2 = 207 N/mm^2

$$\sqrt{(f_{bw}/F_{bw})^2 + (f_v/F_v)^2} \le 1.0$$

$$\sqrt{(126.37/199.59)^2 + (26.45/89.8)^2} = 0.698 \le 1.0 \text{ Hence Safe}$$

3 Side Girt design

Assuming Side Girt spacing = 1.5m
Span of Side Girt = 6.25m

a) Dead Load

Unit weight per meter run of Sheeting @ 0.06 kN/m^2 = 0.06 x 1.5 = 0.09 kN/m
Unit weight per meter run for self-weight of Side Girt = 0.07 kN/m
Total Dead Load per meter on each Girt = 0.16 kN/m

b) Wind Load

Maximum Suction Wind Load per meter = (0.5+0.2) x 1.161x1.5=1.21kN/m.
Maximum Pressure Wind Load per meter = (0.7+0.2) x1.161x1.5=1.56 kN/m.
Considering the Side Girt as Continuous members with overlap of 285mm on either side for splicing purpose

$M_y = \frac{0.16 \text{ x } 6.25^2}{10}$ Due to Dead Load only
M_y = 0.625 kNm

- Gravity load due to Dead Load+ Wind Load = 1.93 KN/m
- Horizontal Load due to Wind Load = 1.56 KN/m
- Maximum Negative moment = 5.96 KNm

$M_x = \left(\frac{1.56}{1.93}\text{x}5.96\right)$ Due to Maximum Wind Load only
M_x = 4.82kNm
Sag rods needs to be placed at 1/3 of Span i.e. 2.08m.

Checking the adequacy of Girt section as per BS: 5950-Part 5

Using the same Section as used for Purlin i.e. Lipped "Z" Section (20mmx60mmx190mmx2.55mmthick) @ 6.57 Kg/m
1) Overall Depth < 100 t
190mm< 100x2.55
190< 255 (O.K)
2) Overall Depth > L/45
190mm > 138.89mm (O.K)
3) Total width of both flanges > L/60
= 60x2 > 6250 /60
=120 > 104.17(O.K)
4) Overall Width of Compression Flange/ Thickness i.e. B/t £ 35
= 60 / 2.55 = 23.52 < 35 (O.K)
5) Width of Lip > B/5
20 > 60 /5
20 > 12 (O.K)
6) Unity check as per BS: 5950 Part-5
Checking for Design expression for "Z" sheeting rails as per Table 13 of BS: 5950 Part-5

Consider Span as Continuous & two rows of Vertical supports at 1/3 of Span.

$$\frac{W_w L}{2250 Z_x} + \frac{W_d L}{16880 Z_y} \leq 1.0$$

Where,

W_w = 1.56x6.25 = 9.75 kN Wind Load on Girt causing Tension on inside face.

W_d= 0.16x6.25 = 1 or 2 kN whichever is greater.

$$\frac{9.75x6250}{2250x46.21} + \frac{2.6250}{16880x10.01} = 0.58 + 0.073 = 0.65 \leq 1.0 \text{ Hence Safe}$$

4 End wall Column design

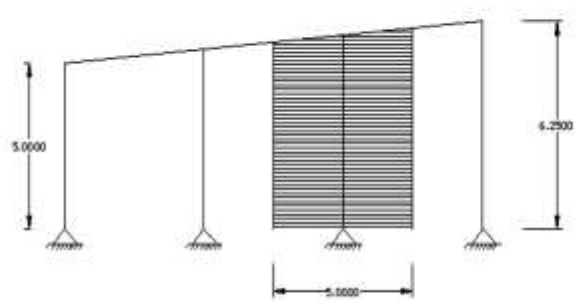


Figure-4.1 Gable End Wind Column

Loadings:

Bending Moment on end wall Column due to Wind load from Gable end side Axial

Compressive Load due to Self weight of side sheeting, girt etc.

Consider End wall Column spacing 5m c/c

Dead Load

Assume Self weight due to side sheeting and Girt = 0.16 kN/m.

Load at each node (junction with side Girt) on end wall Column 0.16x5 =0.8kN Axial

Compressive Load on end wall column due to Side sheeting & Girt =0.8x3=2.4 kN.

Where 3 is the number of Girts

Assume Self Weight of Column = 1.5kN.

Maximum Length of End wall Column =5.84m.

Total Axial Compressive Load = 1.5+2.4=3.9kN.

Wind Load

Wind Load on End wall Column due to Wind influence area = 5x5.84x0.9x1.161=30.52kN

Consider End wall Column pinned at both the ends.

$$M_{x_{max}} = \frac{30.52x5.84}{8} = 22.28 \text{ kNm}$$

Shear Force at ends (supports) due to Wind Load=30.52/2=15.26 kN

The properties of the single section are as listed below

Table 4.1 The properties of the single section

Area in cm^2	Weight (kg/m)	I_{xx} cm^4	I_{yy} cm^4	Z_{xx} cm^3	Z_{yy} cm^3	r_{xx}	r_{yy}
14.2	11.2	704	119.4	78.2	22.2	7.04	2.89

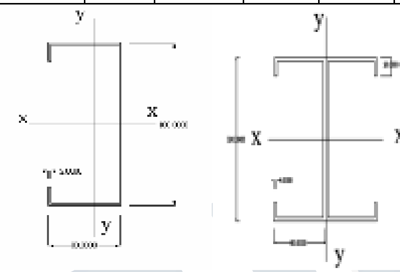


Figure 4.2 Combined Sections Back to Back

The properties of the combined section are as listed below:

Table 4.2 The properties of the combined section

Area in cm^2	Weight (kg/m)	I_{xx} cm^4	I_{yy} cm^4	Z_{xx} cm^3	Z_{yy} cm^3	r_{xx}	r_{yy}
28.4	22.4	1408	432	156.4	54	7.04	3.9

As per clause no. 6.6.3 of IS: 801-1975

$$\frac{KL}{r_x} = \frac{1x584}{7.04} = 82.95 < 200$$

$$\frac{KL}{r_y} = \frac{1x150}{3.9} = 38.46 < 200$$

Considering Compression Flange restrained at 1.5m spacing c/c

Finding out "F_{a1}" i.e. permissible average compressive stress as per Clause no.6.6.1.1of IS: 801-1975

$$C_c = \sqrt{\frac{2\pi^2 E}{F_y}} = \sqrt{\frac{2\pi^2 x2x10^6}{3450}} = 107$$

Calculation of Effective design width of Compression element as per Clauseno.5.2.1.1 of IS: 801-1975.

$$\left(\frac{w}{t}\right)_{lim} = \frac{1435}{\sqrt{f}} = \frac{1435}{\sqrt{20}}$$

Considering Actual Stress in Compressive element f = 20kg/cm^2

$$\left(\frac{w}{t}\right)_{lim} = 321 \text{ or } w = 321x4 = 1284$$

Q=1.0

Where Q = Effective Design Area / Full or Gross Area.

Steel thickness used is 4mm > 2.29mm.

$$\frac{KL}{r} = 82.95 < C_c = 107$$

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As $\frac{KL}{r_x} < C_c$ according to Clause no. 6.6.1.1. (b) Of IS: 801-1975.

$$F_{a_1} = \frac{\left(1 - \frac{(KL/r)^2}{2(C_c)^2}\right) F_y}{\frac{5}{3} + \frac{3}{8} x \frac{(KL/r)}{C_c} - \frac{(KL/r)^3}{8(C_c)^3}}$$

$$F_{a_1} = \frac{\left(1 - \frac{(82.95)^2}{2(107)^2}\right) x\ 3450}{\frac{5}{3} + \frac{3}{8} x \frac{(82.95)}{107} - \frac{(82.95)^3}{8(107)^3}} = 1276.39 \frac{kg}{cm^2}$$

$$= 127.64\ N/mm^2$$

Permissible Axial Compressive under Wind Load = 127.64 x 1.33 =169.76 Mpa.

Actual Axial Compressive Stress $= \frac{3900}{28.4x10^2} = 1.37$ Mpa

Calculation for permissible compressive bending stress

As per Clause no. 6.3 of IS: 801-1975

F (Basic design stress) = 0.6 Fy = 0.6x3450 =2070kg/cm^2 = *207 N/mm^2*

Under Wind Load Condition F_b = 207 x 1.33 = 275.31N/mm^2

$\left(\frac{w}{t}\right)_{lim} = \frac{1435}{\sqrt{f}} = \frac{1435}{\sqrt{1500}} = 37$ Where f = 0.75 x 2000 = 1500kg/cm^2

w = 148 mm > 66 mm Hence Full flange is effective in compression.

Referring to Clause no.6.3 (a) of IS: 801-1975.

L = 1.5m is Unbraced Length i.e. Spacing of side Girts.

$S_{xc} = Z_{xx}$ =156.4 cm^3

$$I_{yc} = \frac{I_{yy}}{2} = \frac{432}{2} = 216\ cm^4$$

$$\frac{L^2 S_{xc}}{dI_{yc}} = \frac{150^2 x156.4}{18x216} = 905 \qquad (1)$$

$$\frac{0.36\pi^2 EC_b}{F_y} = 2060 \qquad (2)$$

$$\frac{1.8\pi^2 EC_b}{F_y} = 10299 \qquad (3)$$

(1) < (2)

(1) < (3)

Hence,

F_b = 275.31 N/mm^2

$$F_{bc_{act}} = \frac{22.28x10^6}{156.4x10^3} = 142.46 < 275.31\ Mpa$$

Unity check

Referring to Clause no. 6.7.2 of IS: 801-1975,

$$\frac{f_a}{F_{a_1}} = \frac{1.37}{169.76} = 0.0080 < 0.15$$

$$\frac{f_b}{F_{b_1}} = \frac{142.46}{275.31} = 0.52$$

$$\frac{f_a}{F_{a_1}} + \frac{f_b}{F_{b_1}} < 1.0$$

$$\frac{f_a}{F_{a_1}} + \frac{f_b}{F_{b_1}} = 0.52 < 1.0\ Hence\ Safe$$

Check for Deflection

Permissible Deflection due to Wind load on Column as per MBMA

$$\delta_{allow} = \frac{span}{120} = \frac{5840}{120} = 48.67\ mm$$

Calculated Deflection due to Wind load on Column

$$\delta_{cal} = \frac{5xwl^4}{384EI} = \frac{5xWl^3}{384EI} = \frac{5x30.52x10^3 x5840^3}{384x2x10^5 x1408x10^4} = 28.1\ mm < 48.67 mm$$

Hence the Column is found safe when checked for Deflection.

5 Eave Beam design

Maximum Wind force on Gable end= (0.7 + 0.2) x 1.161 x (15 x 5 + 0.5 x 15 x 1.25) = 88.163KN.

Axial Compressive force at each corner i.e. on each Eave Beam = 88.163/4 = 22.04kN.

Wind force due to drag by Clause no.6.3.1 of IS: 875 PART 3-1987

d = 50m b = 15m h = 5m

h < b; d/h = 50/5 =10 > 4

$C_f{}' = 0.04$

F' = Frictional drag force

$$= C_f{}'(d - 4h)bP_d + C_f{}'(d - 4h)2bP_d$$

$$F' = 0.04x(50 - 4x5)x15x1.161 + 0.04x(50 - 4x5)x2x15x1.161 = 34.83\ kN$$

Therefore Frictional Drag force on each Eave Beam = 34.83/2 = 17.42 kN.

Total Maximum Axial Load on Eave beam = 22.043 + 17.42 = 39.463 kN.

The Cross-section selected as Eave beam is as shown below:

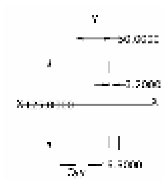


Figure 5.1 C – Section

The property of the Lipped C section is as listed below:

Table 5.1 The property of the Lipped C

Area cm^2	Weight (kg/m)	I_{xx} (cm^4)	I_{yy} (cm^4)	Z_{xx} (cm^3)	Z_{yy} (cm^3)	r_{xx} (cm)	r_{yy} (cm)	C_{yy} (cm)	Xo	J (cm^4)	Cw (cm^6)
7.81	6.13	181	27	29	8	4.8 2	1.84	1.68	4.05	0.26 65	965

Referring to Clause no.6.6.1.2 of IS: 801-1975.

ISSN (Online) 2456-1290

International Journal of Engineering Research in Mechanical and Civil Engineering (IJERMCE)
Vol 3, Issue 1, January 2018

$$\sigma_{ex} = \frac{\pi^2 E}{\left(\frac{KL}{r_x}\right)^2} = \frac{\pi^2 x2x10^6}{\left(208.33/4.82\right)^2} = 10566$$

$$r_o = \sqrt{{r_x}^2 + {r_y}^2 + {X_o}^2} = \sqrt{4.82^2 + 1.84^2 + 4.05^2}$$
$$= 6.56 \text{ cm}$$

$$\sigma_t = \frac{1}{A{r_o}^2}\left[GJ + \frac{\pi^2 EC_w}{(KL)^2}\right]$$
$$= \frac{1}{7.81x6.56^2}\left[795000x02665 + \frac{\pi^2 x2x10^6 x965}{(208.33)^2}\right] = 1936.23 \frac{kg}{cm^2}$$
$$= 193.63 N/mm^2$$

$$\beta = 1 - \left(\frac{X_o}{r_o}\right)^2 = 1 - \left(\frac{4.05}{6.56}\right)^2 = 0.62$$

$$\sigma_{TFO} = \frac{1}{2\beta}\left[(\sigma_{ex} + \sigma_t) - \sqrt{(\sigma_{ex} + \sigma_t)^2 - 4\beta\sigma_{ex}\sigma_t}\right]$$

$$\sigma_{TFO} = \frac{1}{2x0.62}\left[(10566 + 1936) - \sqrt{(10566 + 1936)^2 - 4x0.62x10566x1936}\right]$$
$$= 1796.2 kg/cm^2$$
$$\sigma_{TFO} = 179.62 N/mm^2$$

$$\sigma_{TFO} = 1796.2 \frac{kg}{cm^2} > 0.5cf_y = 0.5x3450 = 1725 \frac{kg}{cm^2}$$

$$F_{a_2} = 0.522F_y - \frac{{F_y}^2}{7.67x\sigma_{TFO}}$$

$$F_{a_2} = 0.522x3450 - \frac{3450^2}{7.67x1796.2} = 936.95$$

$$F_{a_2} = 1.33x936.95 = 1246.14 \frac{kg}{cm^2}$$

$$\left(\frac{w}{t}\right)_{lim} = \frac{1435}{\sqrt{f}} = \frac{1435}{\sqrt{750}} = 52.4$$

Where f = 0.75 x 1000 =750kg/cm^{2}is actual stress in compressive element.

W = 52. 4 x 3. 2 = 167.67mm > 114.6 mm. Hence full flange and web are effective in compression.

Actual Axial compressive stress under Wind Load

$$39.46kN = \frac{39.46x10^3}{7.81x10^2} = 50.52 \frac{N}{mm^2} < 124.61 \frac{N}{mm^2}$$

Hence above section was found safe for axial compressive Stress.

Finding out F_{bperm} as per Clause no.6.3. (a) of IS: 801-1975.

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

$I_{yc} = I_y/2 = 27/2$ cm^4
$S_{xc} = Z_{xx} = 29$ cm^3
d = 125mm = 12.5cm
L = 208.33cm

$$\frac{L^2 S_{xc}}{dI_{yc}} = 7458.6 \quad (1)$$

$$\frac{0.36\pi^2 EC_b}{F_y} = 2060 \quad (2)$$

$$\frac{1.8\pi^2 EC_b}{F_y} = 10299 \quad (3)$$

Eq. (1) > Eq. (2)
Eq. (1) < Eq. (3)

$$F_b = \frac{2}{3}f_y - \frac{{f_y}^2}{2.7\pi^2 EC_b}\left(\frac{L^2 \text{ x } S_{xc}}{d \text{ x } I_{yc}}\right)$$

$$F_b = \frac{2}{3}x3450 - \frac{3450^2}{5.4\pi^2 x2.x10^6 x1}x7458.6 = 1467.6 \text{ kg/cm}^2$$

$$F_{bperm} = 146.71x1.33 = 195.12Mpa$$

Considering Eave beam as simply supported

$$\text{Wind Load on Eave beam} = \frac{(0.7 + 0.2)x1.161x1.5x6.25}{2}$$
$$= 4.89kN$$

$$M_x = \frac{4.89x6.25}{8} = 3.82kNm$$

$$\text{Actual Bending Stress} = \frac{3.82x10^6}{29x10^3} = 131.74 \text{ Mpa}$$

Total Compressive Stress = 131.74+50.52= 182.26 Mpa < 0.6 x 345 x 1.33 (O.K)

Total Compressive Stress=182.26 Mpa < 275.31 Mpa; Hence Safe

6 Bracings design

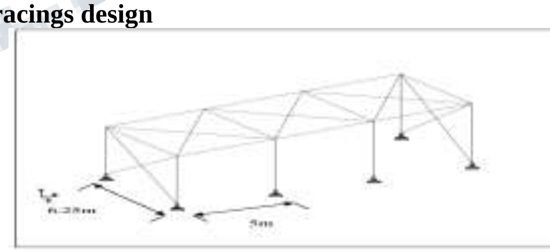


Figure 6.1 Bracing System

Total Maximum Axial Load on Eave beam = 39.458 KN.

$$\theta = \tan^{-1}\left(\frac{6.25}{6.25}\right) = 45^o$$

$$\text{Force in Vertical Bracing member} = \frac{39.458}{\cos\theta} = \frac{39.458}{\cos 45}$$
$$= 55.8kN$$

Using “X” Bracings at both ends Fy = 250 Mpa

$$Area_{reqd} = \frac{55.8x10^3}{2x0.6x250x1.33} = 139.84 \text{ mm}^2$$

Using 20mm Diameter Rod,

ISSN (Online) 2456-1290

International Journal of Engineering Research in Mechanical and Civil Engineering
(IJERMCE)
Vol 3, Issue 1, January 2018

$$\text{Area}_{\text{provided}} = \frac{0.8\text{x}\pi\text{x}20^2}{4} = 251.32\text{mm}^2 > 139.84\text{mm}^2 \text{ Hence Ok}$$

Using 20mm dia Rod @ 2.47 Kg/m

6.1 Main frame design

Load Calculation

a) Dead Load:

Dead weight (Roof Sheeting & Purlins) on frame is considered as 0.1 7kN/m^2

Hence, Loads on rafter as U.D.L = 0.17 x 6.25 = 1.0625 KN/m (↓)

Nodal Loads on Column at Girt Points = 0.17 x 1.5 x 6.25 = 1.59 KN (↓)

Total Load transferred by Girt & Sheeting on Column = 1.59 x 3 = 4.78 kN

Where 3 is the no. of Girts.

b) Service Load:

Service Load on the Rafter is considered as 0.1 kN/m^2

Hence, Loads on rafter as U.D.L = 0.1 x 6.25 = 0.625 kN/m(↓)

c) Imposed Load:

As θ<10°Live Load = 0.75kN/m^2

Hence, Loads on rafter as U.D.L = 0.75 x 6.25 = 4.6875 kN/m(↓)

d) Wind Load:

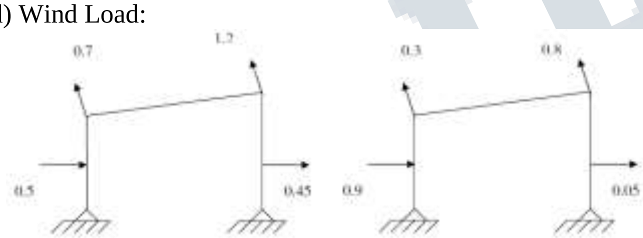


Figure 6.1 (a) Wind across Length of Building

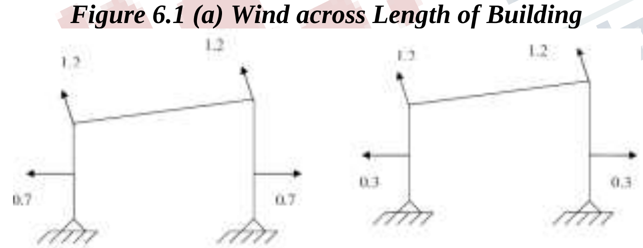
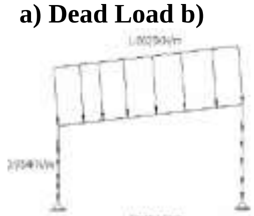


Figure 6.2 (b) Wind along Length of Building

Loads considered:

a) Dead Load b) Live Load



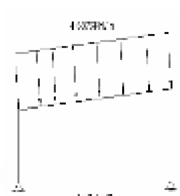
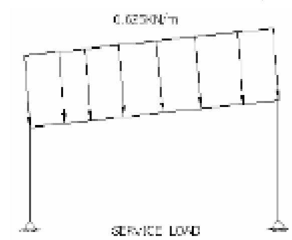


Figure-6.3(a) *Figure-6.3(b)*

c) Service Load **d) Wind Load -1 (windward)**



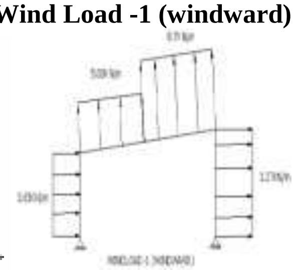
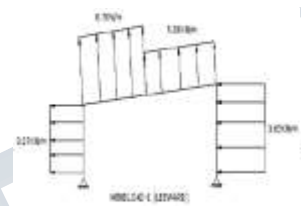


Figure-6.3(c) *Figure-6.3 (d)*

e) Wind Load -1(Leeward) *f) Wind Load-2(Windward)*



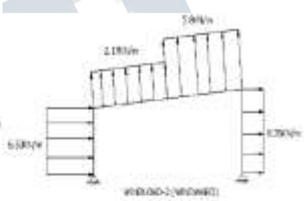
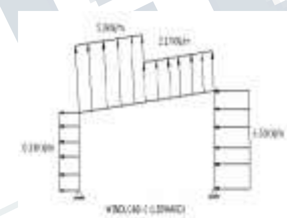


Figure-6.3 (e) *Figure-6.3 (f)*

g) Wind Load-2 (Leeward) **h) Wind Load-3**



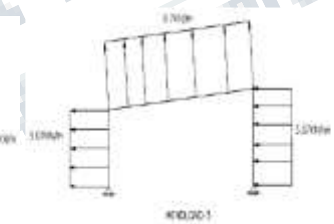


Figure-6.3 (g) *Figure-6.3 (h)*

i) Wind Load-4

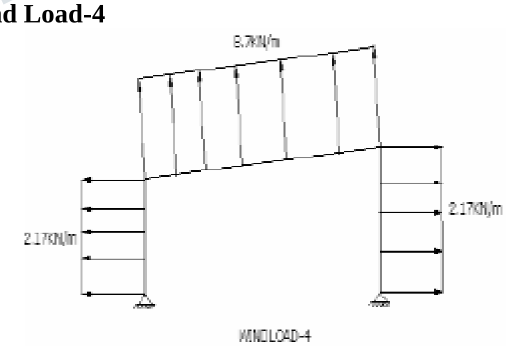


Figure-6.3 (i)

Load Combination

The STAAD analysis has been carried out considering the following combination of Individual load cases:

1) D.L + L.L +S.L
2) D.L +W.L1 (windward)
3) D.L + W.L1 (leeward)
4) D.L + W.L2 (windward)
5) D.L + W.L2 (leeward)
6) D.L+W.L3
7) D.L+W.L4

ISSN (Online) 2456-1290

International Journal of Engineering Research in Mechanical and Civil Engineering (IJERMCE)

Vol 3, Issue 1, January 2018

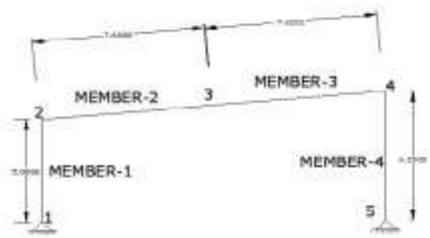


Figure 6.4 PEIB Frame

The Maximum Designed Axial force, Shear force and Bending moment for the Four members is as shown below:

Table 6.1 Maximum Designed Axial force, Shear force and Bending moment

Member no.	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	55.5 (C)	30.4	104.4
2	32.4 (T)	49.9	106
3	32.7 (T)	58.3	149
4	62 (C)	37.5	149

The Frame with configuration of all four members is as shown below:

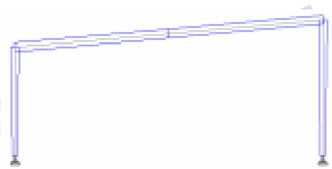


Figure 6.2 2-D View of Frame

The Depth of flange, web thickness and flange width are kept constant throughout the
Pre-engineered Lean to portal frame

Table 6.2 Properties of Members of The Frame

Member no.	Depth of Section at Start Node	Depth of Section at End Node	thickness of web	Width of flange	thickness of flange
1	250 mm	400 mm	6 mm	200 mm	10 mm
2	400 mm	400 mm	6 mm	200 mm	10 mm
3	400 mm	400 mm	6 mm	200 mm	10 mm
4	400 mm	250 mm	6 mm	200 mm	10 mm

Figure 6.3 - 3-D View of Lean to Frame

Checks To Be Performed For Frame

1) Unity Check for combined action of axial tension and bending stress.

By performing above checks the four members were found to be safe.

2) Check for Deflection.

As per MBMA Maximum permissible Horizontal Deflection due to Wind Load only for the frame = h/60 to h/ 100.

Maximum permissible Horizontal Deflection due to Wind Load only for the frame = 5000/60 to 5000/ 100 = 83.33mm to 50mm.

As per MBMA Maximum permissible Vertical Deflection due to Live Load only for the frame = span/240 = 15000/ 240 = 62.5mm.

The frame was checked for horizontal Deflection at nodes "2" &"4" for Wind Load and was found Safe.

The frame was also checked for Vertical Deflection at node "3" for Live Load and was found safe.

Weight of Members of Lean To Frame

The weight of all members in a" LEAN TO FRAME" is as shown below:

Table 6.3 Weight of Member

Member no.	Profile	Length	Weight of Member in Tonnes
1	Tapered	5 m	0.2239
2	Linear	7.53 m	0.3631
3	Linear	7.53 m	0.3631
4	Tapered	6.25 m	0.2799
Total Weight of all Members			**1.23**

The Final Weight of Lean to frame including Stiffeners, Splice plates will be1.15 times the total weight of all members.

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ISSN (Online) 2456-1290

International Journal of Engineering Research in Mechanical and Civil Engineering (IJERMCE)

Vol 3, Issue 1, January 2018

Therefore Final Weight of the" LEAN TOFRAME" will be 1.15x 1.23 = 1.414 Tonnes.

Connections

Detailed Design of different Connections like Main frame base Plate, Gable end Column base plate, Splice plate connection, Bolt design etc. are not covered to limit the size of the problem. Bolt used for the Connections is "HTFG" i.e. High tension friction grip bolts.

Code used for Connection design is "IS- 3757".

The table below gives the members adopted for the design of PEIB with Low span and frame spacing 6.25m.

Table 6.4 Members adopted for the design of PEIB

Sr. No.	Items	Section/Size	Material
1	Main frame	Built up I-section	345 Mpa
2	Purlin	Lipped Z 20x60x190x2.55	345 Mpa
3	Side Girt	Lipped Z 20x60x190x2.55	345 Mpa
4	Girt End wall Column Side	Lipped Z 20x60x190x2.55	345 Mpa
5	Eave beam	Lipped C 20x50x125x3.2	345 Mpa
6	End wall Column Combined	Lipped C 20x80x180x4	345 Mpa
7	Bracing	20mm dia rod	250 Mpa
8	Sag Rod Roof	10mmx1.7m	250 Mpa
9	Sag Rod Sides	10mmx1.7m	250 Mpa
10	Base Plate Main frame	16mmx250mmx350mm	250 Mpa
11	Base Plate End wall Column	10mmx180mmx210mm	250 Mpa
12	Anchor plate Main frame	8mmx90mmx90mm	250 Mpa
13	Anchor plate End wall Column	6mmx80mmx80mm	250 Mpa
14	Foundation Bolt Main frame	4 no.'s M16 dia 500mm long	250 Mpa
15	Foundation Bolt End wall Column	2 no.'s M16 dia 500mm long	250 Mpa

Table 4.12 Total Weight of conventional Industrial building

Sr. No	Member description	QUANTITY	Designed conventional Angle/channel/beam section		
			Section	Unit wt. (Kg)	Total wt. (Kg)
1	Main frame	9	I-section with same size	636 Kg	5724Kg
2	Purlins	12	ISJC-175with one 12 ϕ sag rod at mid span of each span	596 Kg	7152 Kg
3	Side girts	6	ISMC-150 with one 12 ϕ sag rod at mid span of each span	836 Kg	5016 Kg
4	Gable end girts	6	ISMC-150	246 Kg	1476 Kg
5	Main columns	18	ISMB-300	221 Kg	3978 Kg
6	Gable end wind column	4	ISHB-250	222 Kg	888 Kg
7	Eaves beam	2	ISMB-250	1865 Kg	3730 Kg
8	Tie level bracing	24	ISA-70x70x6	50.4 Kg	1209 Kg
9	Tie runners	2	ISA-65x65x6	290 Kg	580 Kg
10	Gable end tie level bracing	12	ISA-80x80x6	58.4 Kg	701Kg
11	Gable end side bracing	16	ISA-65x65x6	39 Kg	625 Kg
12	Rafter bracing	8	ISA-100x100x6	92 Kg	736 Kg
13	Gable end column bracing	16	ISA-55x55x6	28 Kg	448 Kg
14	Addl. Gusset plates		20% of bracings (#8,10,11,12&13)+5 % of rest item = 3719×0.2+27661×0.05=2126.85Kg		2127 Kg
TOTAL					34390 Kg

Table 4.13 Results for Low Span PEIB Having Lean to Frame

Sr. no.	Parameter and items	Frame Spacing 6.25m
1	Slope (V:H)	1:12
2	Spacing	6.25M
3	No. of frames	9

4	built up tapered- I sections	12.73T
5	Lipped z section purlins	3.93T
6	Side girt	2.86T
7	End wall girt	0.88T
8	Eaves beam	0.62T
9	End wall column	0.5T
10	Vertical bracing	0.167T
11	Roof bracing	0.238T
12	Sag rod	0.45T
13	Base plate main frame	0.198T
14	Base plate end wall column	0.012T
15	Foundation bolt main frame	0.0567T
16	Foundation bolt end wall column	0.007T
17	Anchor plate main frame	0.009T
18	Anchor plate end wall column	0.004T
19	**Total weight of materials in tonnes**	**22.67T**
20	**Total cost of materials in lakhs**	**12.59**

II. CONCLUSION

The minimum total weight of PEIB having area of 15m x 50m and Eave height of 5m was found to be 22.67 Tonnes and optimized cost of this building was estimated as 12.59 Lakhs at spacing of 6.25 m as against conventional having total weight of 34.4 Tonnes and cost of 14.448 Lakhs at bay spacing of 6.25m.Therefore material saved by using PEIB was 11.73 Tonnes (32.85%) & net money saved by using PEIB was 1.91 Lakhs.

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