Design And Implementation Of PEB (pre-engineering building) Under AWS D1.1.

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Abstract

Pre-engineered buildings have become quite popular in the last few years. The main advantages are speed of construction and good control over quality. However there is not much information on its economy. There are several parameters like the inclination of the gable, spans, bay spacing, which control the cost of the structure. In the present project the above parameters are varied systematically and in each case the gable frame designed for the common loads DL, LL, EQ, and WL. The quantity in each case is obtained and finally the structure which regulates the lowest quantity of steel is recommended. The pre-engineered steel building system construction has great advantages to the single storey buildings, practical and efficient alternative to conventional buildings, the System representing one central model within multiple disciplines. Pre-engineered building creates and maintains in real time multidimensional, data rich views through a project support is currently being implemented by Bocad software packages for design and engineering.

Keywords – *pre-engineering building, purlins, roof trusses, gantry girders.*

INTRODUCTION

Steel industry is growing rapidly in almost all the parts of the world. The use of steel structures is not only economical but also Eco-friendly at the time when there is a threat of global warming. Here, “economical” word is stated considering time and cost. Time being the most important aspect, steel structures (Pre-fabricated) is built in very short period and one such example is Pre Engineered Buildings (PEB). Pre-engineered buildings are nothing but steel buildings in which excess steel is avoided by tapering the sections as per the bending moment’s requirement. One may think about its possibility, but it’s a fact many people are not aware about Pre Engineered Buildings. If we go for regular steel structures, time frame will be more, and also cost will be more, and both together i.e. time and cost, makes it uneconomical. Thus in pre-engineered buildings, the total design is done in the factory, and as per the design, members are pre-fabricated and then transported to the site where they are erected in a time less than 6 to 8 weeks.
The structural performance of these buildings is well understood and, for the most part, adequate code provisions are currently in place to ensure satisfactory behaviour in high winds. Steel structures also have much better strength-to-weight ratios than RCC and they also can be easily dismantled. Pre Engineered Buildings have bolted connections and hence can also be reused after dismantling. Thus, pre-engineered buildings can be shifted and/or expanded as per the requirements in future. In this paper we will discuss the various advantages of pre-engineered buildings and also, with the help of three examples, a comparison will be made between pre-engineered buildings and conventional steel structures.

Literature Review

J.Jayavelmurugan et.al studied that Buildings & houses are one of the oldest construction activities of human beings. The construction technology has advanced since the beginning from primitive construction technology to the present concept of modern house buildings. The present construction methodology for buildings calls for the best aesthetic look, high quality & fast construction, cost effective & innovative touch.
Apurv Rajendra Thorat et.al studied that In the present study Pre-engineered Buildings are designed and studied in accordance with Kirby Technical Specification which is based on ASCE-07.Two examples have been

taken for the study. Comparison of Pre Engineered Buildings (PEB) with bracings and Pre Engineered Buildings (PEB) without bracings is done in two examples. Later Pre Engineered Buildings (PEB) is analyzed for Dynamic loads using El-centro specified ground motion.

Shrunkhal V Bhagatkar et. al studied that Steel industry is growing rapidly in almost all the parts of the world. The use of steel structures is not only economical but also eco friendly at the time when there is a threat of global warming. Time being the most important aspect, steel structures (Pre fabricated) is built in very short period and one such example is Pre Engineered Buildings (PEB).This review from the past experiences presents the results of experimental and analytical studies done on Pre Engineered Building. Results show that these structures are economic, reduces construction cost and time, energy efficient and flexibility of expansion.

D.Rakesh et. al studied that Now a day there is a vital change in the steel industry, majorly in the industrial structures the usage of Conventional steel building and Pre-Engineered building is more. Conventional steel building and Pre-Engineered building concept is a new conception of single storey industrial building construction. This methodology is versatile not only due to its quality pre-designing and prefabrication, but also due to its light weight and economical construction.

B K Raghu Prasad et. al studied that Pre-engineered buildings have become quite popular in the last few years. The main advantages are speed of construction and good control over quality. However there is not much information on its economy. There are several parameters like the inclination of the gable, spans, bay spacing, which control the cost of the structure. In the present paper the above parameters are varied systematically and in each case the gable frame designed for the common loads DL, LL, EQ, and WL. The quantity in each case is obtained and finally the structure which regulates the lowest quantity of steel is recommended.

COMPONENT OF AN INDUSTRIALBUILDING

The elements of industrial buildings are listed below.

1) Purlins
2) Sag rods
3) Principal Rafters
4) Roof Truss
5) Gantry Girders
6) Bracket
7) Column and Column base
8) Girt Rods
9) Bracings

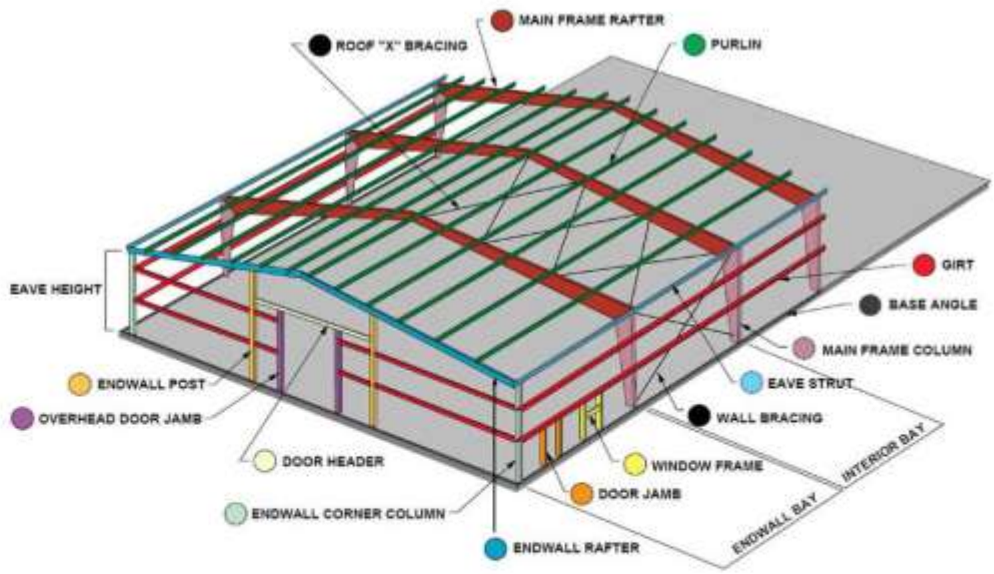

Fig 1: Various component of Industrial building

LOAD CALCULATIONS

A. Dead Load
Dead load is calculated According to IS: 875 (Part 1) –1987[15].

Dead Load on Conventional Steel Building:

Weight of the G.I sheeting = 0.129kN/m2
Weight of fixings = 0.024kN/m2
Weight of services = 0.1 kN/m2
Total weight = 0.256 kN/m2
Spacing of the purlin = 1.35 m
Total weight on purlins = 0.256 × 1.35 = 0.345 kN/m

Dead Load on Pre-Engineered Building:

Weight of the G.I sheeting = 0.129kN/m2
Weight of fixings = 0.024kN/m2
Weight of services = 0.1 kN/m2
Total weight is = 0.256 kN/m2
Total weight on purlins = 0.256 × 1.26 = 0.322 kN/m

B. Live Load:
The Live load is calculated according to IS: 875 (Part 2) –1987 [16].
Live Load on Conventional Steel Building:

Live load on the sloping roof is = 750 – 20(α -10) N/m2
Where α = 21.8˚, Therefore live load = 0.514 kN/m2
Live load on purlins = 0.514 × 1.33 = 0.683kN/m
Live Load on Pre-Engineered Building:

Live load on purlins = 0.683kN/m2
Therefore live load on purlins at 1.26 spacing
= 0.683 × 1.26 = 0.861kN/m
C. Earthquake Load:
Earthquake loads are calculated as per IS: 1893-2000 [17].
Earthquake Load on Conventional Steel Building:
Dead load = 0.256 kN/m2
Live load = 0.128 kN/m2 (25٪of reduction as per IS 1893-2002)
Total load = DL+LL = 0.384 kN/m2
Bay width of the building is 4 m
Therefore earthquake load on rafter = 0.384 × 4 = 1.538kN/m.

Earthquake Load on Pre-Engineered Building:
Dead load = 0.256 kN/m2
Live load = 0.209 kN/m2 (25٪of reduction as per is 1893-2002
Total load = 0.465 kN/m2
Bay width of the building is 4 m
Therefore earthquake load on rafter = 0.444 × 4 = 1.86 kN/m.

D. Wind Load
Wind load is calculated as per IS: 875 (Part 3) – 1987
Basic Wind speed V_b = 39 m/sec
Risk Coefficient K_1 = 1
Terrain, Height and Structure size factor K_2 = 1
Topography factor K_3 = 1
Design Wind Speed $V_z = V_b K_1 K_2 K_3$ = 39 m/sec
Design Wind Pressure $P = 0.06\ V_z^2 = 1.5\ kN/m^2$
The Internal Coefficients are taken as +0.5 and -0.5. Wind Load on individual members are then calculated by

$$F = (Cpe – Cpi) \times A \times P$$

Where, Cpe – External Coefficient
Cpi – Internal Coefficient
A – Surface Area in m^2
P – Design Wind Pressure in kN/m^2

Solution:

Assuming pitch = $\frac{1}{4}$

$\therefore$ Rise = $\frac{1}{4}$ of span

$\therefore$ Rise = $\frac{12}{4}$ = 3 m

Inclination of rafter with horizontal θ

$\tan\theta = \frac{3}{4}$

$\therefore \theta = 26.56°$

Length of rafter = $\sqrt{3^2 + 4^2}$ = 4.47 m

Length rafter = $\frac{4.47}{3}$ = 1.49 m

Calculate of panel point loads.

Dead load calculation:

(1) Weight of covering material = 112.7 N/m^2
On plan area = $\frac{112.7}{\cos\ 26.56}$ = 126 N/mm^2
W_s = 126 N/m^2

(2) Weight of purlin assumed = w_p = 80 N/m^2
W_p = 126 N/m^2

(3) Weight of bracing assumed = w_b = 15 N/m^2
W_b = 15 N/m^2

(4) Weight of truss $= W_t = \left(\frac{span}{3} + 5\right) \times 10$
$= \left(\frac{8}{3} + 5\right) \times 10 = 76.67\ N/m^2$
W_t = 76.67 N/m^2

Total dead load on plan area
D.L = 297.67 N/m^2 Say 300 N/m^2
$\therefore$ Total D.L = $300 \times 4 \times 4$ = 4800 N.

W_t = D.L. on each intermediate panel
$= \frac{4800\ N}{3}$ = 1600 N

D.L. on each panel $\left(\frac{W_D}{2}\right)$
$= \frac{1600}{2}$ = 800 N

Live load calculation:

As slope of roof = 26.56° assuming not provided
L.L. = 750 – 20 (26.56 – 10)
= 418.8 N/m^2 for purlin

Total for live truss load = $\frac{2}{3} \times 418.8 \times 4 \times 4$
= 4467.2 N

W_L = L.L. on each intermediate panel point
$= \frac{4467.2}{3}$
= 1489.06 N Say1500 N

L.L. on each panel point
$\left(\frac{W_L}{2}\right) = \frac{1500}{2}$= 750N.

Wind load calculation:

Wind load in roof by static method is given by
F = (C_{pe} – C_{pi}) $\times A \times P_d$
C_{pe} = external air pressure coefficient
C_{pi} = internal air pressure coefficient
A = surface area under consideration
P_d = Design wind pressure
Computation of design wind pressure
$P_d = 0.6 (V_z)^2$
where, V_z is the design wind speed in m/s
$V_z = V_b \times k_1 \times k_2 \times k_3$
Where, V_b = basic wind speed in m/s
k_1 = the probability factor or risk factor
k_2 = Terrain, height, structure size factor.
k_3 = topography factor
= 1 for upwind slope $\theta < 3°$
= 1 to 1.36 for upwind slope $\theta > 3°$
Hence k_1 = 1 for general building and structure having life 50 years.
k_2 = 0.80 for category 4 and class A
[Category 4 = Terrain with numerous closely spaced larger and high obstruction it includes well developed industrial complexes.]
Class A = Structure and their components having greatest horizontal or vertical dimension less than 20 m.
K_1 = 1.0
V_b = 39 m/s for Ahmednager
$\therefore V_z = V_b \times k_1 \times k_2 \times k_3$
$= 39 \times 1 \times 0.8 \times 1$
$\therefore$ Design wind pressure P_d
$= 0.6 (V_z)^2$
$= 0.6 (31.2)^2$
$\therefore P_d = 584.06$ N/m^2
Wind normal to ridge wind angle = 0°
Building height ratio = $\frac{h}{w}$
$= \frac{6.2}{8} = 0.775$
$\frac{1}{2} < \frac{h}{w} < \frac{3}{2}$ for Roof angle C_{pe} Leeward windward

20°	-0.y	-0.5
30°	-0.2	-0.5

For roof angle 26.56°

$$0.7 - \left(\frac{0.7 - 0.2}{30 - 20}\right) \times 6.56 = 0.372$$

$\therefore$ For θ = 26.56°

C_{pe} = - 0.372 for windward side
C_{pe} = - 0.5 for windward side
Internal air pressure coefficient C_{pi}:
For normal permeability windward side = $\pm$ 0.2
Leeward side = $\pm$ 0.2
Wind parallel to ridge wind angle = 90°
External air pressure co-efficient C_{pe}
On both slopes for $\frac{1}{4}$ of length of building = - 0.8
On both slopes for mid $\frac{1}{4}$ of length of building
For roof angle 20° = - 0.6
For roof angle 30° = - 0.8
$\therefore$for roof angle 26.56° = - 0.73
$0.6 + \left(\frac{0.8-0.6}{10}\right) \times 6.56 = 0.73$
Internal air pressure for normal permeability

On both slope for total length of building $C_{pi} = \pm 0.2$
$P_d = 584.06\ N/m^2$
Total pressure = (C_{pe} - C_{pi})
$C_{pi} = \pm 0.2$ $C_{pi} = -0.2$

Wind normal to ridge:
Windward side $C_{pe} = -0.372 - 334.08\ N/m^2 - 100.46\ N/m^2$
Leward side $C_{pe} = -0.5 - 408.8\ N/m^2 - 175.22\ N/m^2$

Wind parallel to ridge:
$C_{pe} = -0.8 - 584.06\ N/m^2 - 350.44\ N/m^2$
Maximum wind pressure = $-584.06\ N/m^2$
i.e. uplift on both slope.
Total wind load = area subjected to wind × pressure intensity pressure
$= 4.47 \times 4 \times 584.06 = 10448$ N
Wind load on each intermediate panel point w_w
$= \frac{10448}{3}$ = 3482.7 N Say 3500 N
Wind load on each panel point
$= \frac{w_w}{2}$ = 1741.3 N Say 1750 N

Design of Purlins:
Spacing of purlins = 1.49 m
Weight of G.I. sheet (20 G) = $112.7\ N/m^2$
Load in purlins per meter length
- (i) weight of sheeting = 112.7 × 1.49 = 167.923 N/m
- (ii) weight of purlin assumed = 100.00 N/m
  Total D.L. = 267.923 N/m
- (iii) imposed load = 418.8 × 1.49 cos 26.56
  = 558.16 N/m
- (iv) wind load = - 584.06 × 1.49 = - 870.25 N/m

Load combination:
D.L. + L.L. = 267.923 + 558.16
= 826.00 N/m
D.L. + L.L. = 267.923 + (-870.25)
= - 602.32 N/m
Since increase in permissible stresses is 33.33% when wind load is considered; D.L. may be considered33.33 % less effective.
$\therefore \frac{-602.32}{1.33} = -452.88$ N/m
As (D.L.+ L.L.) > (D.L. + W.L.)
i.e. 826.00 N/m > 452.88 N/m
Combination of |(D.L + L.L) is critical
Maximum bending moment = $\frac{wl^2}{10} = \frac{826 \times 4^2}{10}$
= 1321.6 N.m
= 1321.6×10^3 N.mm
For an angle purlin Z_x required = $\frac{M}{\sigma_{bc}} = \frac{1321.6 \times 10^3}{165}$
Z_x required = $8 \times 10^3\ mm^3$
Minimum depth required = $\frac{L}{45} = \frac{4000}{45}$ = 88.88 mm
Minimum width required = $\frac{L}{60} = \frac{4000}{60}$ = 66.67 mm
Provide 100 × 75 × 6 mm ISA giving $Z_x = 14.4 \times 10^3\ mm^3$
Weight =80 N/m

Design of members:
The forces calculated in the members of the roof truss are very small. Assuming minimum single angle 50 × 50 × 6 for principal rafter and tie, check the capacity of these members.
Assuming 16 mm bolt.

Strength of bolt in
Single shear = 0.462 f_u ($n_n A_{nb}$)
$= 0.462 \times 400\ (1 \times 157)$
$= 29.01 \times 10^3$N = 29 kN. … (1)
In double shear = 2×29 = 58.8 kN.
Strength inn bearing (8 mm thick gusset plate)
$= 2 \times d \times t \times f_p$
$= 104.96 \times 10^3$N = 104.96 kN. … (2)
∴ Bolt value is minimum of Equation) 1) and (2)
i.e. 29 kN for single shear
58 kN for double shear
Provide minimum 2 bolts, which are more than sufficient.
Checking the assumed section for tie member
Design force == 23.25 kN Tensile
Check for compressive force = 15.63 kN.
ISA $50 \times 50 \times 6$ mm
Area of each angle = A_g = 568 m^2
$C_{xx} = C_{yy}$ = 14.5
Mm
$r_{xx} = r_{yy}$ = 15.1 mm r_{vv} = 9.6 mm
Area of connected leg = $A_{go} = \left(50 - 18 - \frac{6}{2}\right)$
= 174 mm^2
Area of outstanding leg = $A_{go} = \left(50 - \frac{6}{2}\right) \times 6$
= 282 mm^2
$A_n = 2(174 + 282) = 912$ mm^2

(i) Design strength due to yielding of gross section, T_{dg}
$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{2 \times 568 \times 250}{1.10} = 258181.8$ N

(ii) Design strength due to rupture of critical section, T_{dn}
$T_{dn} = \alpha A_n \frac{f_u}{\gamma_{m1}}$ … α = 0.6 for two bolts.
$= 0.6 \times 912 \times \frac{410}{1.25} = 179481.6$ N
∴T_{dn} = 179.48 kN

(iii) Strength due to block shear T_{db}

Assuming gauge distanceg = 28 mm (from Table 2.4.2) and end distance = e = 1.7 × diameter of hole = 1.7 × 18 = 30.6 say 35 mm and pitch p = 2.5 × diameter of bolt = 2.5 × 16 = 40 mm.
$A_{vg} = L_{vg} \times$ t $= 76 \times 6 = 450$ mm^2
A_{vn} = {L_{vg} - [(number of bolts – 0.5)] $\times t$}
= {75 – [(2 – 0.5)18]} × 6
= 288 mm^2
$A_{tg} = L_{tg} \times t = 22 \times 6$= 132 mm^2
= (L_{tg}– 0.5 d_h) $\times t$ = (22 – 0.5 × 18 } × 6
= 78 mm^2
$T_{db_1} = A_{vg} f_y / (\sqrt{3} \times \gamma_{mo}) + 0.9\ A_{tn} f_u / \gamma_{m1}$
$= 450 \times \frac{250}{\sqrt{3} \times 1.1} + 0.9 \times 78 \times 410/1.25$
= 59047.18+23025.6
= 82072.78 N = 82.07 kN
For two bolts, T_{db_1} = 164.14 kN
$T_{db_2} = 0.9\ A_{vg} f_y / (\sqrt{3} \times \gamma_{m1}) + 0.9\ A_{tg} f_u / \gamma_{mo}$
$= 0.9 \times 288 \times 410/\sqrt{3} \times 1.25) + 132 \times 250/1.10$
= 49084.9 + 30000
= 79084.9 N = 79.08 kN
For two bolts, T_{db_2} = 158.16 kN
∴ T_{db} = lesser of T_{db_1} and T_{db_2} i.e. 158.16 kN.
The design tensile strength of angle = lesser of T_{dg}, T_{dh} and T_{db} i.e. lesser of (258.18 kN, 179.48 kN) = 158.16 kN>> required 23.25 kN.

Check for compression, 15.63 kN :
Length of member = 1.4 m.
At two bolts are used at each end, effective length = KL
K = 0.85
∴ Slenderness ratio $\lambda_1 = \frac{KL}{r_{min}} = \frac{0.85\times1400}{15.1}$
= 78.80 < 180 … OK
For buckling class ‘C’ from table 4.5 c f_{cd}⇒ ? for f_y 250
λ_1 = 70 f_{cd_1} = 152
λ_2 = 80 f_{cd_2} = 136
$f_{cd} = f_{cd_1} - \left[\frac{f_{cd_1}-f_{cd_2}}{\lambda_2-\lambda_1}(\lambda-\lambda_1)\right]$
$= 152 - \left[\frac{152-136}{80-70}(78.8-70)\right]$
= 137.9 N/mm^2
∴ $P_d = A \times f_{cd} = 2 \times 568 \times 137.9 = 156677\ N$
∴ Provide 2 ISA 50 × 50 × 6 for main tie and for principle rafter.
Checking the adequacy of a single angle 50 × 50 × 6 mm for main strut, main sling, minor strut and major sling.
Maximum tension = 8.83 kN.
Maximum tension = 7.76 kN.
Length of member = 2 cm.
Area of angle = 568 mm^2 r_{vv} = 9.6 mm
Assuming 16 mm bolt – 2 No.
Area of connected leg = $A_{nc} = \left(50 - 18 - \frac{6}{2}\right) \times 6$
= 174 mm^2
Area of outstanding leg = $A_{go} = \left(50 - \frac{6}{2}\right) \times 6$ = 282 mm^2
$A_n = A_{nc} + A_{go}$ = 174 + 282
= 456 mm^2

(i) Design strength due to yielding of gross section, T_{dg}.
$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} = \frac{568\times250}{1.10}$ = 129090.9 N
∴ T_{dg} = 129.09 kN

(ii) Design strength due to rupture of critical section, T_{dn}.
$T_{dn} = \frac{\alpha A_g f_u}{\gamma_{m1}} = 0.6 \times \frac{456\times410}{1.25}$
∴T_{dn} = 89.74 kN

(iii) Strength due to block shear, T_{dg}.
T_{db_1} = 82.07 kN. Refer Equations (3) & (4)
T_{db_2} = 79.08 kN. Refer Equations (3) & (4)
T_{db} = 8279.08 kN.

∴Design strength of angle = lesser of T_{dg}, T_{dn} and T_{db}
= 79.08 kN>> required 7.76 kN. … OK
Compressive strength of the angle.
Slenderness ratio $\lambda = \frac{KL}{r_{min}}$ … K = 0.85 for two bolts
$= \frac{0.85\times2000}{9.6}$
= 177.08 < 180
For buckling class ‘C’ from table 4.5 C f_{cd}= ?forf_y = 250 Mpa
$\lambda_1 = 170 f_{cd_1}$= 47.8
$\lambda_2 = 180 f_{cd_2}$= 43.8
$F_{cd} = f_{cd_1} - \left[\frac{(f_{cd_1}-f_{cd_2})}{\lambda_2-\lambda_1}(\lambda-\lambda_1)\right]$
$= 47.8 - \left[\frac{(47.8-43.4)}{180-170}(177.08-170)\right]$
= 44.68 N/mm^2
∴ $P_d = A \times f_{cd}$ = 568 × 44.68
= 25380.96 N
= 25.38 kN.>> required 8.83 kN
∴ Provide single angle 50 × 50 × 6 mm for main strut, main sling, minor strut and minor sling.

CONCLUSION

Choosing steel to design a Pre-engineered steel structures building is to choose a material which offers low cost, strength, durability, design flexibility, adaptability and recyclability. Steel is the basic material that is used in the Materials that are used for Pre-engineered steel building. It negates from regional sources. It also means choosing reliable industrial products which come in a huge range of shapes and colors, it means rapid site installation and less energy consumption. It means choosing to commit to the principles of sustainability. Infinitely recyclable, steel is the material that reflects the imperatives of sustainable development.

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