# Comparative Study of Pre Engineered and Conventional Steel Building

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**Abstract –**

Pre Engineered Steel Buildings (PEB) fulfils the requirements of industrial steel structures along with reduced time and cost as compared to Conventional Steel Buildings (CSB). This methodology is versatile not only due to its quality predesigning and prefabrication, but also due to its light weight and economical construction. The present work presents the comparative study and design of conventional steel frames and Pre Engineered Buildings (PEB).

In this work, an industrial building of length 60 m, with roofing system as conventional steel truss and pre engineered steel truss of span 20 m, at 6 m along the length and 6 m , 8 m & 6m bay width at end frames has been considered. Slope of roof truss is taken as 5.71°. The eave height of these structures has been taken as 10 m. These structures have been analyzed and designed by using STAAD Pro V8i to compare the PEB and conventional steel truss. Design is based on IS: 800-2007. Steel with yield stress 250 MPa is considered. Loads considered in analysis are dead load, live load, wind load and seismic loads along with the various combinations as specified in IS: 800-2007. Dead load is taken based on IS: 875 (Part 1)-1987. Live load is taken based on IS: 875(Part-2)-1987. Wind load is taken based on IS: 875 (Part 3)-1987. It is assumed that the structure is located in Visakhapatnam city. Analysis results observed for base reactions and weight of the steel are compared for these three structures.

**Index Terms—**Conventional steel Buildings, Pre Engineered Steel Buildings,Seismic forces, Staad Pro V8i

## 1. INTRODUCTION

India is the second fastest growing economy in the world and a lot of it is attributed to its construction industry which figures just next to agriculture in its economic contribution to the nation. In its steadfast development, the construction industry has discovered, invented and developed a number of technologies, systems and products; one of them being the concept of Pre-engineered Buildings (PEB). As opposed to being on-site fabricated, PEBs are delivered as a complete finished product to the site from a single supplier with a basic structural steel framework with attached factory finished cladding and roofing components. The structure is erected on the site by bolting the various building components together as per specifications. PEBs are developed using potential design software. The onset of technological advancement enabling 3D modelling and detailing of the proposed structure and coordination has revolutionized conventional building construction.

PEBs have hit the construction market in a major way owing to the many benefits they possess. They exemplify the rising global construction, technology and while they oppose the practice of conventional building construction they simultaneously have taken it to a higher level too. Worldwide, they are a much used concept with studies revealing that 60% of the non-residential low-rise buildings in USA are pre-engineered; for India the concept has been gaining momentum and the scope of growth is guaranteed looking at India's huge infrastructural requirements. Studies already validate that India has the fastest growing market in the PEB construction segment. The scope of using PEBs ranges from showrooms, low height commercial complexes, industrial building and workshops, stadia, schools, bridges, fuel stations to aircraft hangars, exhibition centers, railway stations and metro applications. While we are still to see PEBs being used in residences in India, one can see their optimal use in warehouses, industrial sheds sports facilities etc. The Delhi Airport and the metro projects of Delhi, Bangalore and Mumbai are also examples of PEB applications.

**Applications**

Pre-Engineered Building concept have wide applications including warehouses, factories, offices, workshops, gas stations, showrooms, vehicle parking sheds, aircraft hangars, metro stations, schools, recreational buildings, indoor stadium roofs, outdoor stadium canopies, railway platform shelters, bridges, auditoriums, etc. PEB structures can also be designed as re-locatable structures.

a) **Industrial:**
Factories, Workshops, Warehouses, Cold stores, Car parking sheds, Slaughter houses, Bulk product storage, harbors.

b) **Commercial:**
Showrooms, Distribution centers, Supermarkets, Fast food restaurants, Offices, Labor camps, Service stations, Shopping centers.

c) **Institutional:** Schools, Exhibition halls, Hospitals, Theaters/auditoriums, Sports halls.

d) **Recreational:**Gymnasiums, swimming pool enclosures, Indoor tennis courts.

e) **Aviation& Military:** Aircraft hangars, Administration buildings, Residential barracks,Support facilities.

f) **Agriculture:** Poultry buildings, Dairy farms, Greenhouses, Grain storage, Animal confinement.

**Scope of present study**

This study mainly focuses on to compare the structural steel weight of Conventional Steel Building and Pre-engineered Steel Building using IS 800-2007. These structures are assumed to be located in Visakhapatnam. These buildings are analyzed and designed by using STAAD Pro V8i Software. In this study compare the how axial forces and moments are varies from type II structure to type III structure.

**Objectives of study**

Objectives identified for the present study have listed as follows.

1. To compare the steel take-off of the conventional steel building and Pre engineered building.
2. To compare the member end forces of Pre engineered building and Conventional steel building.
3. To compare the joint displacements of Pre engineered building and Conventional steel building.

## 2. METHODOLOGY

In the present study two types of steel buildings designated as Type I and Type II are considered for the analysis, design and computing structural steel weight.

***Type I*** is Conventional Steel Building of length 60 m and span 20 m. Bay lengths are maintained at an interval of 6 m along length and 6 m, 8 m and 6m along span only for end frames. The height of the truss is taken as a minimum pitch that is $\frac{1}{6}^{\text{th}}$ of span. So slope of roof is taken as 5.71˚and covered with GI sheet. The spacing of purlins is maintained as 1.5 m. The eave height of the building has been taken as 10 m in which 3 m from ground level is used for brick work and remaining 7 m is used for cladding.

***Type II*** is Pre Engineered Steel Building of length 60 m and span 20 m. Bay lengths are maintained at an interval of 6 m along length and 6 m , 8 m and 6m along span only for end frames. The height of the truss is taken as a minimum pitch that is $\frac{1}{6}^{\text{th}}$ of span. So slope of roof is taken as 5.71˚and covered with GI sheet. The spacing of purlins is maintained as 1.5 m. The eave height of the building has been taken as 10 m in which 3 m from ground level is used for brick work and remaining 7 m is used as cladding.

These two structures have been analyzed and designed by using STAAD Pro V8i to compare the structural steel weight for both PEB and conventional steel building.

**Analysis:-**

CSB frame of the structure considered in the study is as shown in Figure1

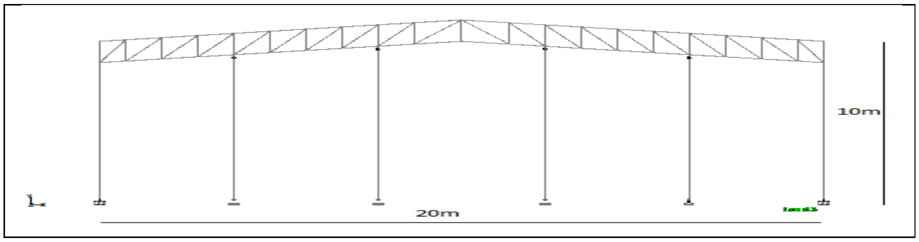


**Figure1:** Elevation of conventional steel truss with slope 5.71˚

**Pre-Engineered Buildings**

The typical PEB frame of the structure considered for the present study is shown in Figure 2.

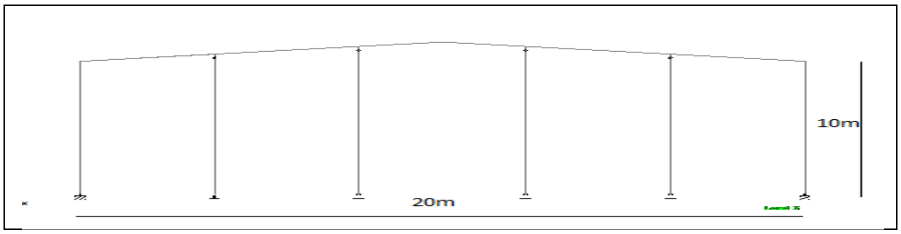


**Figure2:** Elevation of PEB with slope 5.71˚

**Table 1** Structure Parameters

| | |
|---|---|
| Type of building | Industrial building |
| Type of structure | Single story industrial structure |
| Location | Visakhapatnam |
| Area of building | 1200 $m^2$ |
| Eave height | 10 m |
| Span width | 20 m |
| Number of bays | 10 No's |
| Single bay length | 6 m |
| Total length | 60 m |
| Support condition | Fixed<br>(Main columns)<br>Pinned<br>(End wall columns) |
| Roof slope | 5.71˚ |

**Loadings considered in the parametric study**

Primary loads acting on the structure have been considered as dead load, Live load, earthquake load and wind load. The load calculation for the structure can be carried out in accordance with IS: 875 – 1987 (Part-1, 2, and 3) and IS: 1893 - 2000. For this structure wind load is critical than earthquake load. Hence, load combinations of dead load, live load, earthquake load and wind load are incorporated for design.

**Dead load**

Dead load on the roof trusses in single storey industrial buildings consists of dead load of claddings and dead load of purlins, self weight of the trusses in addition to the weight of bracings etc. Further, additional special dead loads such as truss supported hoist dead loads, special ducting and ventilator weight etc. As the clear span length (column free span length) increases, the self weight of the moment resisting gable frame increases drastically. In such cases roof trusses are more economical.

The dead load distributed over the roof is found to be in kN/m excluding the self weight. This load is applied as uniformly distributed load over the purlins while designing the structure by PEB and CSB concept.

**Live load**

The live load on roof trusses consist of the gravitational load due to erection and servicing as well as dust load etc. and the intensity is taken as per IS:875-1987 (Part-2). Additional special live loads such as snow loads in very cold climates, crane live loads in trusses supporting monorails may have to be considered.

The load assumed to be produced by the intended use or occupancy of a building including the weight of movable partitions , distributed, concentrated loads, load due to impact and vibration, and dust load but excluding wind, seismic, snow and other loads due to temperature changes, creep, shrinkage, differential settlement, etc.

According to IS : 875 (Part 2) – 1987, for roof with no access provided, the live load can be taken as 0.75 kN/m$^2$ with a reduction of 0.02 kN/m$^2$ for every one degree above 10 degrees of roof slope. Total uniform live load acting on the purlin of the CSB and PEB structures are found to be in kN/m.

**Earthquake Load**

Since earthquake load on a building depends on the mass of the building, earthquake loads usually do not govern the design of light industrial steel buildings. Wind loads usually govern. However, in the case of industrial buildings with a large mass located at the roof, the earthquake load may govern the design. These loads are calculated as per IS:1893-2000.

**Wind load**

Wind load on the roof trusses, unless the roof slope is too high, would be usually uplift force perpendicular to the roof, due to suction effect of the wind blowing over the roof. Hence the wind load on roof truss usually acts opposite to the gravity load, and its magnitude can be larger than gravity loads.

The horizontal and vertical bracings used primarily to resist wind and other lateral loads in trusses. These bracings minimize the differential deflection between the different frames due to lateral loads in industrial buildings. They also provide lateral support to columns in small and tall buildings, thus increasing the buckling strength.

Wind load is calculated as per IS: 875 (Part 3) – 1987. The basic wind speed for the location of the building is found to be 50 m/s from the code. The wind load over the roof can be provided as uniformly distributed load acting outward over the purlins of PEB, and as uniformly distributed load acting outward over the CSB purlins. For side walls, the wind load is applied as uniformly distributed loads acting inward or outward to the walls according to the wind case.

**Basic wind Speed ($V_b$)**

Basic wind speed is based on peak gust velocity averaged over a short time interval of about 3 seconds and corresponds to mean heights above ground level in an open terrain (Category 2). Basic wind speeds have been worked out for a **50** year return period.

**Design Wind Speed ($V_z$)**

The basic wind speed ($V_b$) for any site shall be modified to include the following effects to get design wind velocity at any height ($V_z$) for the chosen structure.

a) Risk level
b) Terrain roughness, height and size of structure
c) Local topography

It can be mathematically expressed as follows.

$V_z = V_b \times k_1 \times k_2 \times k_3$

where

$V_z$ = design wind speed at any height z in m/s
$V_b$ = basic wind speed in m/s
$k_1$ = probability factor (risk coefficient)
$k_2$ = terrain, height and structure size factor and

$k_3$ = topography factor.

**Design wind pressure ($P_z$)**

The design wind pressure at any height above mean ground level shell be obtained by the following relationship between wind pressure and wind velocity.

$P_z = 0.6\,V_z^2$

**Wind pressures and forces on buildings/structures**

**General -** The wind shall be calculated for:

a) The building as a whole,
b) Individual structural elements as roofs and walls, and
c) Individual cladding units including glazing and their fixings.

**Pressure coefficients**

The pressure coefficients are always given for a particular surface or part of the surface of a building. The wind load acting normal to a surface is obtained by multiplying the area of that surface or its appropriate portion by the pressure coefficient ($C_p$) and the design wind pressure at the height of the surface from the ground.

Average values of pressure coefficients are given for critical wind directions in one or more quadrants. In order to determine the maximum wind load on the building, the total load should be calculated for each of the critical directions shown from all quadrants. Where considerable variation of pressure occurs over a surface, it has been subdivided and mean pressure coefficients given for each of its several parts.

In addition, areas of high local suction (negative pressure concentration) frequently occurring near the edges of walls and roofs are separately shown. Coefficients for the local effects should only be used for calculation of forces on these local areas affecting roof sheeting, glass panels, and individual cladding units including their fixtures. They should not be used for calculating force on entire structural elements such as roof, walls or structure as a whole.

**Wind load on individual members**

When calculating the wind load on individual structural elements such as roofs and walls, and individual cladding units and their fittings, it is essential to take account of the pressure difference between opposite faces of such elements or units. For clad structures, it is, therefore, necessary to know the internal pressure as well as the external pressure. Then the wind load, F, acting in a direction normal to the individual structural element or cladding unit is:

$F = (C_{pe} - C_{pi})\, A\, P_z$Where

$C_{pe}$ = external pressure coefficient,
$C_{pi}$ = internal pressure- coefficient,
A = surface area of structural or cladding unit, and
$P_z$ = Design wind pressure

**External pressure coefficients ($C_{pe}$)**

Walls:

Pitched roofs of rectangular clad buildings: The average external pressure coefficients and pressure concentration coefficients for pitched roofs of rectangular clad building shall be as given in Table 3.5. Where no pressure concentration coefficients are given, the average coefficients shall apply.

**Internal Pressure Coefficients ($C_{pi}$)**

Internal air pressure in a building depends upon the degree of permeability of cladding to the flow of air. The internal air pressure may be positive or negative depending on the direction of flow of air in relation to openings in the buildings.

In the case of buildings where the claddings permit the flow of air with openings not more than about 5 percent of the wall area but where there are no large openings, it is necessary to consider the possibility of the internal pressure being positive or negative. Two design conditions shall be examined, one with an internal pressure coefficient of +0.5 and another with an internal pressure coefficient of -0.5.

The internal pressure coefficients algebraically added to the external pressure coefficient and the analysis which indicates greater distress of the member shall be adopted. In most situations a simple inspection of the sign of external pressure will at once indicate the proper sign of the internal pressure coefficient to be taken for design.

**Buildings with medium and large openings**

Buildings with medium and large openings may also exhibit either positive or negative internal pressure depending upon the direction of wind. Buildings with medium openings between about 5 to 20 percent of wall area shall be examined for an internal pressure coefficient of +0.5 and later with an internal pressure coefficient of -0.5 and the analysis which produces greater distress of the members shall be adopted. Buildings with large openings, that is, openings larger than 20 percent of the wall area shall be examined once with an internal pressure
coefficient of +0.7 and again with an internal pressure coefficient of -0.7,and the analysis which produces greater distress on the members shall be adopted.

In this work the building is assumed with medium openings between about 5 to 20 percent of wall area shall be examined for an internal pressure coefficient of +0.5 and later with an internal pressure coefficient of -0.5

## 3. LOAD CALCULATIONS

Span of the truss = 20 m
Length of the building = 60 m
Bay width = 6 m each
Spacing of purlins = 1.5 m
Height of the eaves = 10 m
Height of the building at ridge = 11 m
**Dead load**: (as per IS 875 (Part-1) – 1987)
Weight of the G.I sheeting = 0.131 kN/$m^2$ (class 1 G.I sheeting, thickness 1.60 mm)
Weight of fixings = 0.025 kN/$m^2$
Weight of services = 0.1 kN/$m^2$
Total weight is = 0.256 kN/$m^2$
Total weight on purlins = 0.256 × 1.507
= 0.3857 kN/$m^2$.
**Live load:** (as per IS 875 (Part-2) – 1987)
Live load on purlins = 0.750 kN/$m^2$
Therefore live load on purlins at 1.507 spacing = 0.75 × 1.507
=1.130 kN/$m^2$.
**Earthquake load:** (as per IS: 1893 (Part-1)-2002)
Dead load = 0.256 kN/$m^2$
Live load =0.1875 kN/$m^2$ (25⁄ of reduction as per is 1893 (Part-1)-2002)
Total load =0.4435 N/$m^2$
Bay width of the building is 5 m
Therefore earthquake load on rafter
= 0.4435 × 5 = 2.2175 kN/m.
**Wind load:** (As per IS 875:Part-III)
Location: Visakhapatnam.
Risk coefficient factor ($k_1$) = 1
Terrain and height facto ($k_2$) = 0.954
Topography factor ($k_3$) = 1
Design wind speed $V_z$ =$V_b \times k_1 \times k_2 \times k_3$
Where $V_b$ = basic wind speed = 50 m/s
Design wind speed $V_z$ =50×1×0.937×1
= 46.85 m/s
Design wind pressure $P_z$ = $0.6 \times V_z^2$
$P_z$ = $0.6 \times 46.85^2$
$P_z$ = 1316 N/$m^2$

**Table2:** wind load left ($C_{pi}$ = -0.5)

| Description | Roof W.W | Roof L.W | Wall A | Wall B |
|---|---|---|---|---|
| Coefficients ($C_{pe}$) | -0.93 | -0.6 | +0.7 | -0.3 |
| ($C_{pe}$-$C_{pi}$) | -1.43 | -1.1 | +0.2 | -0.8 |
| Wind load (F) kN/m | -2.25 | -1.73 | 0.30 | -1.22 |

**Table3:** wind load right ($C_{pi}$ = -0.5)

| Description | Roof W.W | Roof L.W | Wall A | Wall B |
|---|---|---|---|---|
| Coefficients ($C_{pe}$) | -0.6 | -0.93 | -0.3 | -0.7 |
| ($C_{pe}$-$C_{pi}$) | -1.1 | -1.43 | -0.8 | -1.2 |
| Wind load (F) kN/m | -1.73 | -2.25 | 1.22 | -1.84 |

**Table4:** wind load left ($C_{pi}$ = +0.5)

| Description | Roof W.W | Roof L.W | Wall A | Wall B |
|---|---|---|---|---|
| Coefficients ($C_{pe}$) | -0.6 | -0.93 | -0.3 | +0.7 |
| ($C_{pe}$-$C_{pi}$) | -0.1 | -0.43 | 0.2 | 1.2 |
| Wind load (F) kN/m | -0.15 | -0.67 | 0.30 | 1.84 |

**Table5:** wind load right ($C_{pi}$ = +0.5)

| Description | Roof W.W | Roof L.W | Wall A | Wall B |
|---|---|---|---|---|
| Coefficients ($C_{pe}$) | -0.93 | -0.6 | -0.3 | +0.7 |
| ($C_{pe}$-$C_{pi}$) | -0.43 | -0.1 | +0.2 | +1.2 |
| Wind load (F) kN/m | +0.205 | -0.15 | +0.30 | +1.84 |

**Table6:** wind load parallel ($C_{pi}$ = -0.5)

| Description | Roof W.W | Roof L.W | Wall A | Wall B |
|---|---|---|---|---|
| Coefficients ($C_{pe}$) | -0.8 | -0.4 | -0.5 | -0.5 |
| ($C_{pe}$-$C_{pi}$) | -1.3 | -0.9 | -1 | -1 |
| Wind load (F) kN/m | -2.1 | -1.45 | -1.62 | -1.62 |

**Table7:** wind load parallel ($C_{pi}$ = +0.5)

| Description | Roof W.W | Roof L.W | Wall A | Wall B |
|---|---|---|---|---|
| Coefficients ($C_{pe}$) | -0.8 | -0.4 | -0.5 | -0.5 |
| ($C_{pe}$ - $C_{pi}$) | -0.3 | 0.1 | 0 | 0 |
| Wind load (F) kN/m | -0.6 | 0.2 | 0 | 0 |

## 4. RESULTS AND DISSCUSSIONS

**Weight of steel (steel take-off)**

The weights of conventional steel building and PEB are calculated after the design. For conventional steel building the weight of the section is given in Table 8. For PEB with slope 5.71˚, the weight of sections is given in Table 9.

**Weight of the steel for the conventional steel building**

In order to calculate the steel weight of conventional steel building the following member properties are used (Table 8). Hot rolled "I" sections are assigned for columns. For the top chord, bottom chord and bracings Indian standard double angles are used. For purlins, girts and eave strut ISMC (Indian Standard Medium Channels) are used. Now using the above parameters the lengths and weights are calculated accordingly.

Table 8**:** Steel take-off for conventional steel building

| Profile | Length (m) | Weight (t) |
|---|---|---|
| Tapered Member no: 1 | 42.16 | 1.41 |
| Tapered Member no: 2 | 220 | 92.61 |
| ST ISMC 400 | 220.89 | 10.82 |
| ST 250CS80 x 5 | 120 | 1.9 |
| ST 290ZS75 x 3.15 | 960 | 10.9 |
| ST 300ZS75 x 3.15 | 976 | 11.39 |
| ST ISMB600 | 220 | 34.78 |
| ST ISA75x75x5 | 180.11 | 5.42 |
| Prismatic Steel | 328.61 | 1.14 |
| | **Total** | **170.37** |

**Weight of the steel for the PEB**

In order to calculate steel weight of PEB the following member properties are used (Table 9). For columns and rafters tapered "I" sections are assigned. For the purlins cold formed "Z" sections are used. For the girts cold formed "C" sections are used. For bracings Indian standard double angle sections are used. Now using the above parameters the lengths and weights are calculated accordingly.

Table 9: Steel take-off for the PEB

| Profile | Length(m) | Weight (t) |
|---|---|---|
| Tapered MembNo:1 | 26.51 | 8.41 |
| Tapered MembNo:2 | 218.83 | 69.59 |
| Tapered MembNo:3 | 132.89 | 33.47 |
| Tapered MembNo:4 | 8.47 | 2.16 |
| Tapered MembNo:5 | 26.51 | 7.18 |
| Tapered MembNo:6 | 26.51 | 7.79 |
| Tapered MembNo:7 | 42.16 | 1.41 |
| ST 250CS80x3.15 | 120 | 1.23 |
| ST 250ZS75X2.55 | 960 | 8.9 |
| ST 260ZSx75x2 | 257.17 | 1.75 |
| ST 300ZSx75x2.55 | 360 | 3.41 |
| | **Total** | **145.3** |

**Comparison of member forces and joint displacements for CSB vs PEB**

Comparison of member forces and joint displacements at ridge and haunch portions with critical load and at same location was as mentioned below

From the comparison it is observed that

1. At ridge and haunch of the building Maximum bending moment for PEB is more than CSB by 6.19 % and 12.45 % this is due to the entire force will concentrated in PEB where as in CSB it is distributed.
2. At ridge of the building axial force for CSB is more than PEB by 2.2 %, at haunch of the building PEB is more than CSB by 27% this is due to the entire force has to transfer at haunch like beam action in PEB where as in CSB it is like a truss arrangement.
3. At ridge and haunch of the building shear force for PEB is more than CSB by 16.5 % and 22.53 % this is due to the entire force will concentrated in PEB where as in CSB it is distributed.
4. At ridge the vertical displacement for PEB is 33mm where as for CSB it is 28mm. Both the displacements are within the allowable limit.
5. At haunch the horizontal displacement for PEB and CSB is 8mm.Both the displacements are within the allowable limit

## 5. CONCLUSIONS

After analysing, the following are the conclusions of Pre-Engineered steel Building when compared with Conventional Steel Buildings

1. The total steel take-off for PEB is 16% of the conventional steel building.
2. It is observed than maximum moment will be high for PEB than CSB
3. It is observed than maximum shear force will be high for PEB than CSB
4. For PEB the axial force at haunch is higher than CSB
5. For CSB the axial force at ridge is higher than PEB

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