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Design of Pre-Stressed Precast Concrete Truss and a Comparative Study with Steel Truss

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ABSTRACT

Structural steel is typically and widely used to design roofs for warehouses and airplanes Hangars. Design considerations for roof support must take into account cost-effectiveness, speed of construction, structural capacity, aesthetic appearance, fire resistance, and structural Integrity during construction and after completion. Designing using steel has been the only practice when it comes to long-spanned roofs. The ease of handling and erection, geometric flexibility, and lightweight of the structural steel components are all advantages of steel structures. Concrete has not been introduced into the field of roof supports for warehouses and airplanes hangars because of the difficulty of the construction and design of concrete components without being extremely heavyweight and expensive. e.Construct, USA, LLC has offered a state-of-art concrete-truss girder system that has been built and designed for a cement storage facility in Sharjah, UAE.,

In this paper, a precast pre-stressed concrete truss is proposed as an alternative to steel trusses for span of 30m without intermediate supports. As the concrete is weak in resisting tensile stresses, design of solely reinforced concrete truss is very difficult. In order to resist the high axial tension in bottom chord, the concrete truss need to pre-stressed.. Spacing of truss governs the weight of truss system. If the spacing increases, the load on the truss system increases and member dimensions of truss also increases. Also, the length of roof slab increases as the spacing increases which leads to generate large bending moments in roof slab. To resist the bending moments the dimensions need to be increased which in turn increase the dead weight of the ent ire system. The truss is analysed in STAAD and design can be done

I. INTRODUCTION

1.1 ROOF SYSTEMS

Concrete is an artificial stone and its excellence resistance to compression resembles the properties of natural stone. It is a pseudo fluid for a part of its early age. It's Strength and other properties can be regulated to some extent during its manufacture. The key to achieving a strong, durable concrete rests in the careful proportioning and mixing of the ingredients. Portland cement's chemistry comes to life in the presence of water. Cement and water form a paste that coats each particle of stone and sand the aggregates. Through a chemical reaction called hydration, the cement paste hardens and gains strength.

1.2 DIMESIONS OF THE TRUSS

The height of reinforced concrete truss at mid span is in the range of 1/7 to 1/9 of its span length. The span of the trusses range from 18 to 30 and more . The width of the various compression and tension members is kept constant at 200 to 350 mm depending upon the span of the truss. The depth of top bottom members which are in compression generally is in the range of 200 to 300 mm. The bottom tie member should be of sufficient size to house the pre-tensioned wires or post tensioned cables. The depth is around 200 mm for spans of 15 m increasing to 300 mm for spans of 30m. The depth of diagonal web members which are in compression and tension generally vary in the narrow range from 100 to 150 mm.

1.3 ANALYSIS OF TRUSS

Generally truss members are assumed to be joined together so as to transfer only the axial forces and not moments and shears from one member to the adjacent members (they are regarded as being pinned joints). The loads are assumed to be acting only at the nodes of the trusses. The trusses may be provided over a single span, simply supported over the two end supports, in which case they are usually statically determinate. Such trusses can be analysed manually by the method of joints or by the method of sections. Computer programs are also available for the analysis of trusses.

From the analysis based on pinned joint assumption, one obtains only the axial forces in the different members of the trusses. However, in actual design the members of the trusses are joined together by more than one bolt or by welding, either directly or through larger size end gussets. Further, some of the members, particularly chord members, may be continuous over many nodes. Generally such joints enforce not only compatibility of translation but also compatibility of rotation of members meeting at the joint. As a result, the members of the trusses experience bending moment in addition to axial force. This may not be negligible, particularly at the eaves points of pitched roof trusses, where the depth is small and in trusses with members having a smaller slenderness ratio (i.e. stocky members). Further, the loads may be applied in between the nodes of the trusses, causing bending of the members. Such stresses are referred to as secondary stresses. The secondary bending stresses can be caused also by the eccentric connection of members at the joints. The analysis of trusses for the secondary moments and hence the secondary stresses can be carried out by an indeterminate structural analysis, usually using computer software.

The magnitude of the secondary stresses due to joint rigidity depends upon the stiffness of the joint and the stiffness of the members meeting at the joint. Normally the secondary stresses in roof trusses may be disregarded, if the slenderness ratio of the chord members is greater than 50 and that of the web members is greater than 100. The secondary stresses cannot be neglected when they are induced due to application of loads on members in between nodes and when the members are joined eccentrically. Further the secondary stresses due to the rigidity of the joints cannot be disregarded in the case of bridge trusses due to the higher stiffness of the members and the effect of secondary stresses on fatigue strength of members. In bridge trusses, often misfit is designed into the fabrication of the joints to create pre-stress during fabrication opposite in nature to the secondary stresses and thus help improve the fatigue performance of the truss members at their joints.

1.4 CONFIGURATION OF TRUSS

Pitched roof truss

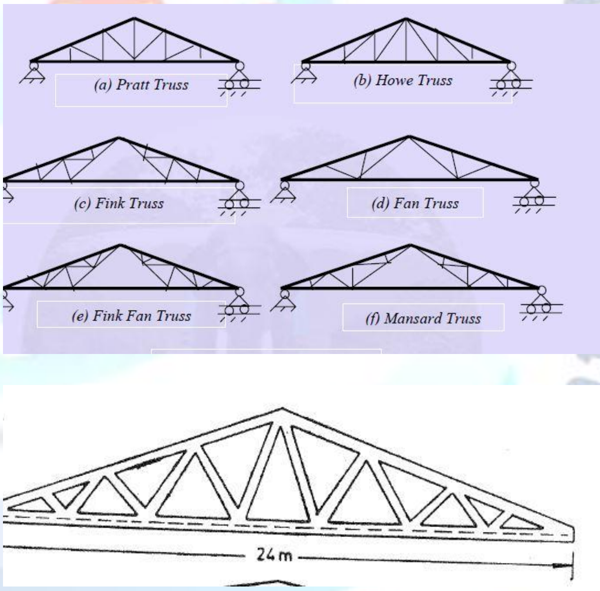


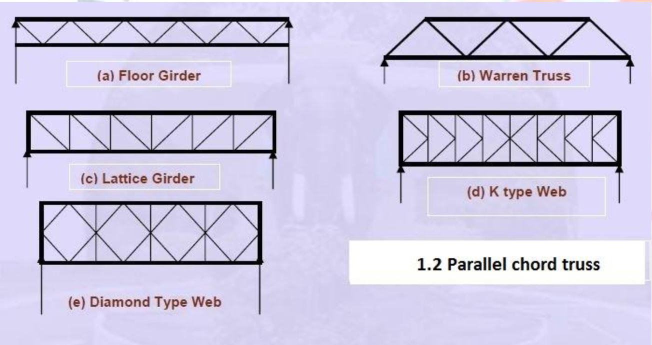
Figure 1.1: Roof Trusses

Most common types of roof trusses are pitched roof trusses wherein the top chord is provided with a slope in order to facilitate natural drainage of rain water and clearance of dust/snow accumulation. These trusses have a greater depth at the mid-span. Due to this even though the overall bending effect is larger at mid-span, the chord member and web member stresses are smaller closer to the mid-span and larger closer to the supports. The typical span to maximum depth ratios of pitched roof trusses are in the range of 4 to 8, the larger ratio being economical in longer spans. Pitched roof trusses may have different configurations. In Pratt trusses web members are arranged in such away that under gravity load the longer diagonal members are under tension and the shorter vertical members experience compression. This allows for efficient design, since the short members are under compression. However, the wind uplift may cause reversal of stresses in these members and nullify this benefit. The converse of the Pratt is the Howe truss. This is commonly used in light roofing so that the longer diagonals experience tension under reversal of stresses due to wind load.

Fan trusses are used when the rafter members of the roof trusses have to be sub-divided into odd number of panels. A combination of fink and fan can also be used to some advantage in some specific situations requiring appropriate number of panels.

Mansard trusses are variation of fink trusses, which have shorter leading diagonals even in very long span trusses, unlike the fink and fan type trusses.

1.4.2 Parallel chord truss

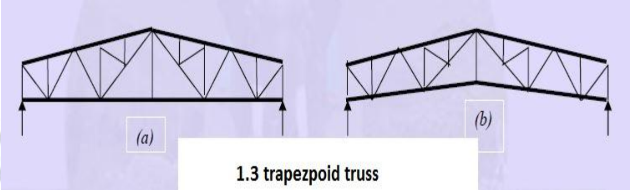


1.2 Parallel chord truss

The parallel chord trusses are used to support North Light roof trusses in industrial buildings as well as in intermediate span bridges. Parallel chord trusses are also used as pre-fabricated floor joists, beams and girders in multi-storey buildings. Warren configuration is frequently used in the case of parallel chord trusses. The advantage of parallel chord trusses is that they use webs of the same lengths and thus reduce fabrication costs for very long spans. Modified Warren is used with additional vertical, introduced in order to reduce the unsupported length of compression chord members. The saw tooth north light roofing systems use parallel chord lattice girders to support the north light trusses and transfer the load to the end columns

The economical span to depth ratio of the parallel chord trusses is in the range of 12 to 24. The total span is subdivided into a number of panels such that the individual panel lengths are appropriate (6m to 9 m) for the stringer beams, transferring the carriage way load to the nodes of the trusses and the inclination of the web members are around 45 degrees. In the case of very deep and very shallow trusses it may become necessary to use K and diamond patterns for web members to achieve appropriate inclination of the web members.

1.4.3 Trapezoidal Trusses



1.3 trapezpoid truss

In case of very long span length pitched roof, trusses having trapezoidal configuration, with depth at the ends are used .This configuration reduces the axial forces in the chord members adjacent to the supports. The secondary bending effects in these members are also reduced. The trapezoidal configurations having the sloping bottom chord can be economical in very long span trusses (spans > 30 m), since they tend to reduce the web member length and the chord members tend to have nearly constant forces over the span length. It has been found that bottom chord slope equal to nearly half as much as the rafter slope tends to give close to optimum design.

1.5 Truss Members

The members of trusses are made of either rolled steel sections or built-up sections depending upon the span length, intensity of loading, etc. Rolled steel angles, tee sections, hollow circular and rectangular structural tubes are used in the case of roof trusses in industrial buildings. In long span roof trusses and short span bridges heavier rolled steel sections, such as channels, I sections are used . Members built-up using I sections, channels, angles and plates are used in the case of long span bridge trusses.

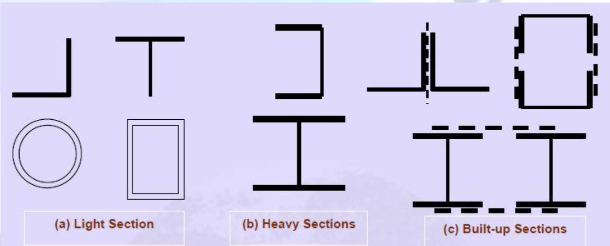


Figure1.4 Truss members

Accesses to surface, for inspection, cleaning and repainting during service, are important considerations in the choice of the built-up member configuration. Surfaces exposed to the environments, but not accessible for maintenance are vulnerable to severe corrosion during life, thus reducing the durability of the structure. In highly corrosive environments fully closed welded box sections, and circular hollow sections are used to reduce the maintenance cost and improve the durability of the structure.

1.6 Truss Connections

Members of trusses can be joined by riveting, bolting or welding. Due to involved procedure and highly skilled labor requirement, riveting is not common these days. High strength friction grip (HSFG) bolting and welding have become more common. Shorter span trusses are usually fabricated in shops and can be completely welded and transported to site as one unit. Longer span trusses can be prefabricated in segments by welding in shop. These segments can be assembled by bolting or welding at site. This results in a much better quality of the fabricated structure.

Truss connections form a high proportion of the total truss cost. Therefore it may not always be economical to select member sections, which are efficient but cannot be connected economically. Trusses may be single plane trusses in which the members are connected on the same side of the gusset plates or double plane trusses in which the members are connected on both sides of the gusset plates.

It may not always be possible to design connection in which the centroidal axes of the member sections are coincident. Small eccentricities may be unavoidable and the gusset plates should be strong enough to resist or transmit forces arising in such cases without buckling .The bolts should also be designed to resist moments arising due to in-plane eccentricities. If out-of-plane instability is foreseen, use splice plates for continuity of out-of plane stiffness.

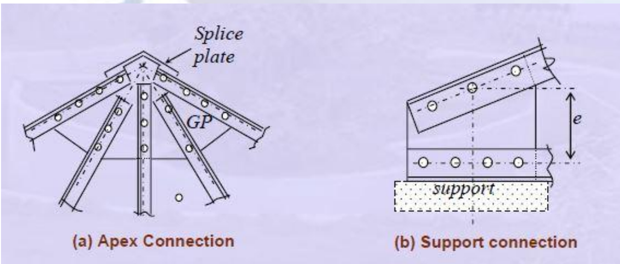


Figure 1.5 Truss Connections

If the rafter and tie members are T sections, angle diagonals can be directly connected to the web of T by welding or bolting. Frequently, the connections between the members of the truss cannot be made directly, due to inadequate space to accommodate the joint length. In such cases, gusset plates are used to accomplish such connections. The size, shape and the thickness of the gusset plate depend upon the size of the member being joined, number and

size of bolt or length of weld required, and the force to be transmitted. The thickness of the gusset is in the range of 8 mm to 12 mm in the case of roof trusses and it can be as high as 22 mm in the case of bridge trusses. The design of gussets is usually by rule of thumb. In short span (8 – 12 m) roof trusses, the member forces are smaller, hence the thickness of gussets are lesser (6 or 8mm) and for longer span lengths (> 30 m) the thickness of gussets are larger (12mm).

1.7 Design of Truss

Factors that affect the design of members and the connections in trusses are discussed as following.

1.7.1 Instability considerations

While trusses are stiff in their plane they are very weak out of plane. In order to stabilize the trusses against out- of- plane buckling and to carry any accidental out of plane load, as well as lateral loads such as wind/earthquake loads, the trusses are to be properly braced out –of –plane. The instability of compression members, such as compression chord, which have a long unsupported length out-of-plane of the truss, may also require lateral bracing.

Compression members of the trusses have to be checked for their buckling strength about the critical axis of the member. This buckling may be in plane or out-of-plane of the truss or about an oblique axis as in the case of single angle sections. All the members of a roof truss usually do not reach their limit states of collapse simultaneously. Further, the connections between the members usually have certain rigidity. Depending on the restraint to the members under compression by the adjacent members and the rigidity of the joint, the effective length of the member for calculating the buckling strength maybe less than the centre-to-centre length of the joints. The design codes suggest an effective length factor between 0.7 and 1.0 for the in-plane buckling of the member depending upon this restraint and 1.0 for the out of plane buckling.

In the case of roof trusses, a member normally under tension due to gravity loads (dead and live loads) may experience stress reversal into compression due to dead load and wind load combination. Similarly the web members of the bridge truss may undergo stress reversal during the passage of the moving loads on the deck. Such stress reversals and the instability due to the stress

reversal should be considered in design. The design standard (IS:
800) imposes restrictions on the maximum slenderness ratio, (l/r).

1.8 Economy of truss

Trusses consume a lot less material compared to beams to span the same length and transfer moderate to heavy loads. However, the labor requirement for fabrication and erection of trusses is higher and hence the relative economy is dictated by different factors. In India these
Considerations are likely to favor the trusses even more because of the lower labor cost. In order to fully utilize the economy of the trusses the designers should ascertain the following:

- Method of fabrication and erection to be followed, facility for shop fabrication available, transportation restrictions, field assembly facilities.
- Preferred practices and past experience.
- Availability of materials and sections to be used in fabrication.
- Erection technique to be followed and erection stresses.
- Method of connection preferred by the contractor and client (bolting, welding or riveting).
- Choice of as rolled or fabricated sections.
- Simple design with maximum repetition and minimum inventory of material.

1.9 General about Design

Steel trusses are widely used as roofs for warehouses and airplanes hangars. Examples are provided in Figure 1.6 and Figure 1.7. Design considerations for roof support must take into account cost-effectiveness, speed of construction, structural capacity, aesthetic appearance, fire resistance, and structural integrity during construction and after completion. Steel trusses have been the only practice when it comes to long-spanned roofs due to the ease of handling and erection, geometric flexibility, and lightweight of members.

Fig 1.6 steel truss warehouse roof

Fig 1.7 steel truss used as roof support

Apart from the advantages, steel construction is having disadvantages such as Low fire-resistance, Corrosion, High maintenance cost, High material cost. All these disadvantages are addressed when comparing steel sections with concrete ones. Concrete has very low to no maintenance cost, high fire-resistance, and low material cost. However concrete has not been used in the field of roof supports due to the difficulty of the construction and heavy weight members. Especially for long span trusses, concrete has not been used due to heavy member weight and aesthetics. In order to design the truss members to resist the tensile stresses developed due to loads and to support roofs for long spans, pre stressing need to be introduced in design of concrete trusses.

1.10 *Post Tensioning*

Post-tensioning is a technique of reinforcing concrete with high-strength steel strands typically referred to as tendons. Post-tensioning applications include parking structures, office and apartment buildings, slabs-on-ground, sports stadiums, bridges rock and soil anchors, and water-tanks. In post-tensioning, first concrete members are cast by incorporating ducts or grooves for housing the tendons. When the concrete attains sufficient strength, the high-tensile wires are tensioned by means of jack bearing on the end of the face of the member and anchored by wedge or nuts. The forces are transmitted to the concrete by means of end anchorage. The space between the duct and tendons is generally grouted after the tensioning operation.

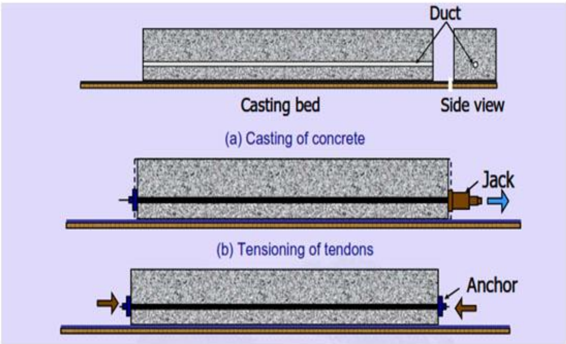


Fig 1. 8 Post tensioning

II. LITERATURE REVIEW

2.1 Introduction

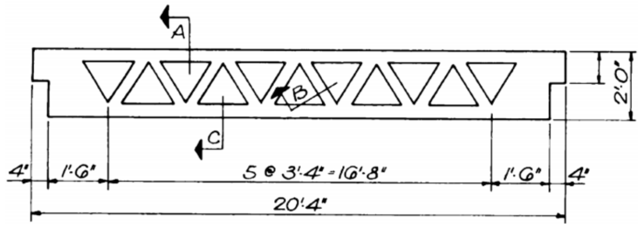
This chapter deals with literature review concerning with the work done by different authors with regard to precast pre-stressed concrete truss. Few of them are mentioned below:

2.1 Literature survey

Pradeep H. Shah and Howard R. May(1977) in their article, described the planning and design considerations in the prefabrication and pre-stressing of the Vierendeel trusses for Rock Island Parking Structure was built using Vierendeel trusses. Vierendeel trusses are trusses having rigid joints and no diagonals. The trusses used were nearly 12 ft deep and had a clear span of 32 ft and 16 in. x 22 in. cross-section dimensions for the top, bottom, and vertical members. The vertical members were post-tensioned as they resist tension.

Figure 2.1 Vierendeel trusses used on the façade for the Rock Island Parking structure

William t. Carroll et. Al.(1978) in their paper presented two case studies to compare the cost of a totally pre-stressed concrete truss structure with steel truss. A prototype that had a clear span of 20 ft 4 in. and a depth of 2 ft, for a span to depth ratio of 10. The other prototype had 60 ft 10 in. and 8 ft 6 in. for span and depth, respectively, yielding a span to depth ratio of 7. The smaller trusses that were suggested in the paper consisted of only diagonal members and no verticals and the bigger ones had two verticals near to the center of the trusses. Figure 2.2 and Figure 2.3 show the two prototypes of the trusses researched



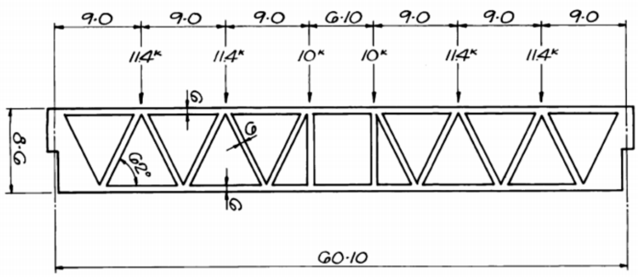


Fig 2. 2 Concrete truss prototype-II (Carroll and al. 1978)

All the top, bottom, and diagonal members were pre-stressed. The paper stated that due to the friction losses at the hold down devices, the pre-stressing in the diagonals was found to be only 35% of the initial pre-stressing. The paper stated that the members cracked at an early stage of loading, which was due to the reason that the diagonals were not fully pre-stressed. The research stated that using concrete trusses would lower the price to almost half when compared with steel trusses.

M. DeSutter(2007) invented system which is a precast, pre-stressed concrete truss to use as a load bearing Wall in building construction. The system is called “ER-Post” system. The purpose of the system was to provide a column-free space for the condominiums. Figure 2.4, DeSutter done post tensioning for Vierendeel trusses to design the precast/pre- stressed trusses with a depth of 13.5 ft, span of 67.33 ft, and span to depth ratio of 5.

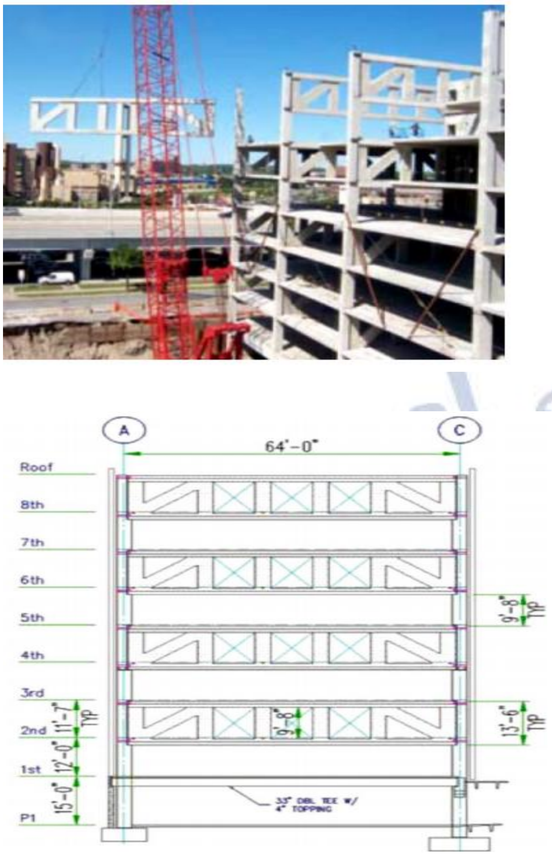


Fig 2. 3 Erection of ER-POST trusses (Trygestad and DeSutter 2007)

Peter S. Samir (2013) In his paper, proposed a precast concrete truss for span of 160 ft. as an alternative to steel truss. The proposed design was an evolution of the system that was developed by e.Construct USA, LLC(2010) and was used in the construction of a cement storage facility at United Arab of Emirates. The proposed truss design consists of two segments that were formed using standard bridge girder forms with block-outs in the web that resulted in having diagonals and vertical sections. The two segments were connected using a wet joint and post-tensioned longitudinally as shown in Fig 2.7. A Finite Element Analysis for the truss-girder system was conducted to investigate stresses at truss connections and the wet joint. A 30 –foot long truss specimen was tested to investigate the constructability of the truss and the structural capacity of the diagonals, verticals, and connections. That research developed a system with span of 160ft and span to depth ratio of 27. This study stated that the results matched with the actual behavior of truss. This study recommended to provide smooth curved corners instead of sharp ones to avoid stress concentrations.

Fig 2. 4 The truss designed by e.Construct USA, LLC (the view is center to end)

Fig 2. 5 The trusses resting on the temporary supports before post-tensioning

III. MODELING

3.1 General

Truss is modeled in STAAD. The truss elements between joints are treated as rigid and capable of transferring moment and shear except vertical members.

3.2 Geometric details

Table 3. 1 Geometric details of Truss

S.No.	Particulars	Data
1	Span of truss	30m
2	Length of shed	54m
3	Spacing of trusses	6m
4	Roofing	Concrete panels
5	Grade of Concrete	M35
6	Grade of Steel	Fe500

3.3 Loads considered

3.3.1 Dead load

Thickness of roof slab=75mm

Unit-weight of concrete = 25Kn/m^3

Dead load = 25X0.075 = 1.875Kn/m^2

Dead load per meter run for 6m spacing = 1.875X6 = 11.25Kn/m

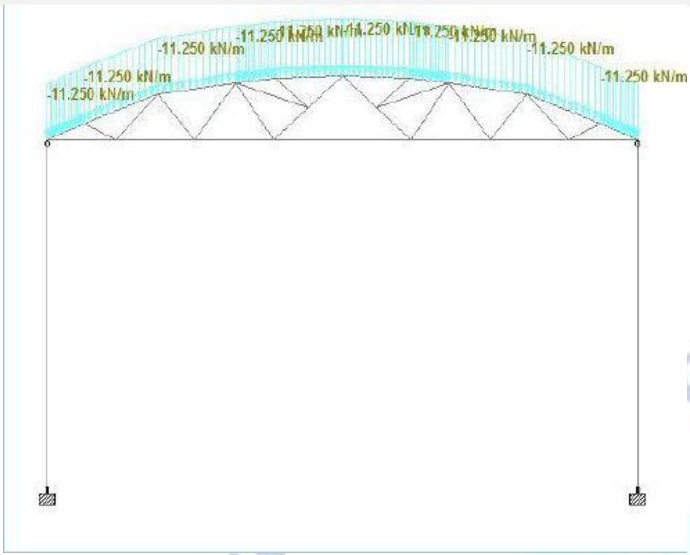


Fig 3. 1 **Dead load on Roof truss**

3.3.2 Live load

According to Indian Standards IS: 875(Part 2)-1987 clause 4.1, for Flat, Curved or Sloped roofs with slopes up to and including 10 degrees if access is not provided except for maintenance Live load is taken as 0.75Kn/m²

Live load per meter run for 6m spacing = 0.75X6= 4.5Kn/m

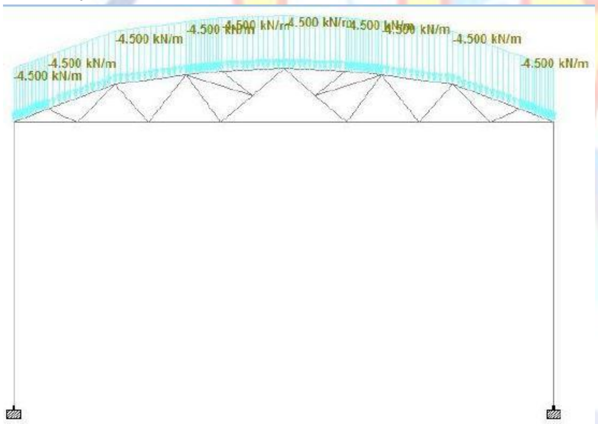


Fig 3. 2 Live load on Roof truss

3.3.3 Wind load

Basic wind speed = 55m/s

Terrain category -4

Class of the structure – class C

$$V_b = 50 \text{ m/sec}$$

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

Assuming design life of structure is for 50 years Probability factor or risk coefficient K1 = 1.0 (Table 1 clause 5.3.1)

Terrain, height and structure size factor K2= 0.98(for terrain category 2, Class B structure and building height 6m from Table 2 clause 5.3.2.2)

Topography factor K3 = 1(for plain land) (clause 5.3.3)

Design wind speed V_Z = 55x1x1x1 = 55 m/sec

Design wind pressure $P_Z = 0.6V_Z^2$ = 1.743Kn/m²

The wind load, F, acting in a direction normal to the individual structural element or cladding unit is:

$F = (C_{Pe} – C_{pi}) \times A \times P_d$ (clause 6.2.1 IS 875: part3)

where, C_{pe} = external pressure coefficient,

(for h/w = 0.216, l/w =1.8 and $\theta = 2.86^0$, for 0 deg Cpe =0.8572, 0.4)

for 90 deg Cpe = 0.8,0.8)

C_{pi} = internal pressure coefficient,(± 0.2,)

A = surface area of structural element

P_z = design wind pressure (1.743Kn/m²)

From the above formula, the calculated wind force is as follows

For wind angle of 0 deg with roof

Wind force acting on windward side = 16.58Kn/m

Wind force acting on leeward side = 9.41Kn/m

For wind angle of 90 deg with roof

Wind force acting on windward side = 15.687Kn/m

Wind force acting on leeward side = 15.687Kn/m

A. *3.4 Plan of shed*

The span for the trusses is set at 30m with a 5% slope (crowning). The building was proposed to be 54m longitudinally with 9 spaces between truss-girder lines, resulting in 6m spacing between every other truss-girder for Plan A and with 6 spaces between truss-girder lines, resulting in 6m spacing between every other truss-girder for Plan B. Figures 3.4.1 is the plan view of the building layout representation for Plan A and Plan B. Figure 3.4.2 and 3.4.3.are an elevation view of the truss resting on columns for s/d = 20 and s/d = 25. The columns height is 6m.

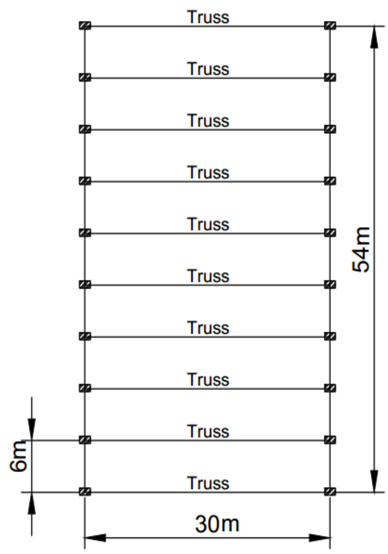


Fig 3. 3 Plan view of building layout for Plan A and Plan B

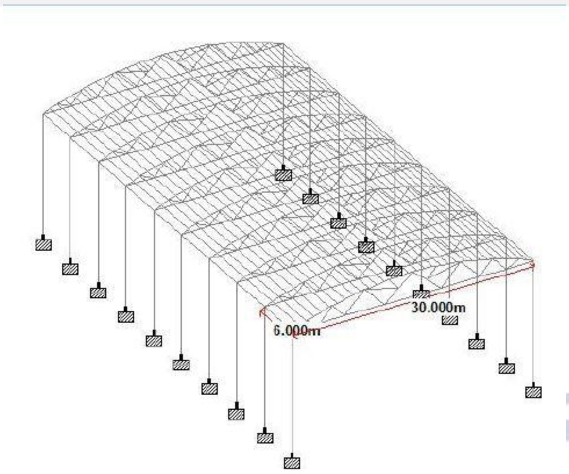
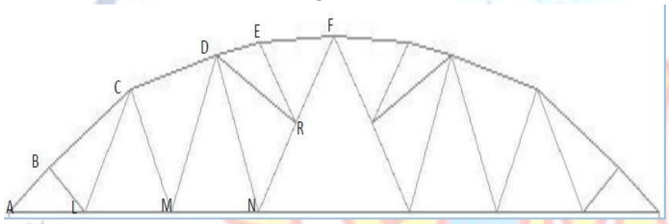


Fig 3. 4 shed having 30m span roof truss

IV. DESIGN OF TRUSS MEMBERS

4.1 Introduction

This chapter deals with the design of truss members of the following truss

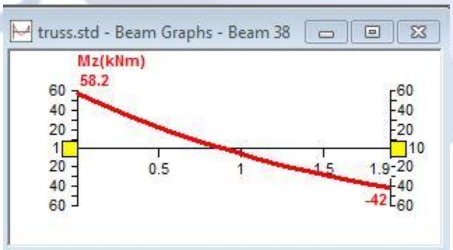


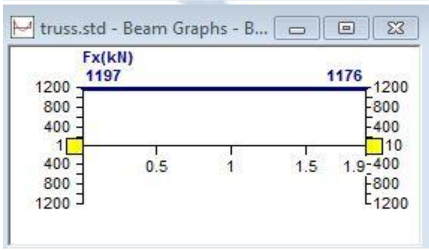
.**Fig 4. 1 Member identification of Truss**

4.2 Design of top chord members

4.2.1 Design of top chord member AB

The dimensions of the top chord member AB are:
Length = 1.901 m
Dimension = 350X400
Compression Load (taken from staad) = 1200 KN
Moment (taken from staad) = 59 KNm

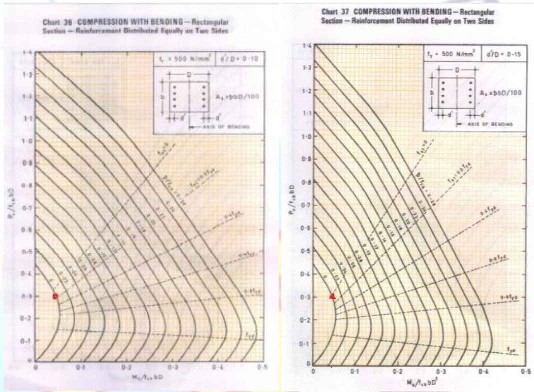




4.2.1a **Bending moment diagram**
4.2.1b **Shear force diagram**

b = 350
D= 400
Pu = 1200 KN
Mu = 59 KNm
L =1.901 m
Fck = 30 N/mm^2
Fy = 500 N/mm^2
Le = 0.65x1.901 =1.23565
Le/D = 3.089125
Pu/(fckbd) = 0.28571429
Mu/(fck b d^2) = 0.03511905
Effective cover d' = 50 mm
d'/ D = 0.125

For Fe500 and d'/D = 0.125, referring to the charts of SP-16 the p/fck value to be derived based on Pu/(fck b d) and Mu/(fck b d^2) where p is the percentage of steel to be provided .



4.2.1C **SP16 charts**

d' /D	p/fck
5.	0
0.125	-----
0.15	0

The percentage of steel value coming from charts is 0. So, the minimum percentage of steel has to be provided.
Minimum percentage of steel = 0.8x350x400/100 = 1120 mm^2.
Hence provide a main reinforcement of 4-20 mm + 4- 16 mm.
Provide a shear reinforcement of 8mm @ 200 c/c

4.2.2 Design of Top Chord member BC:

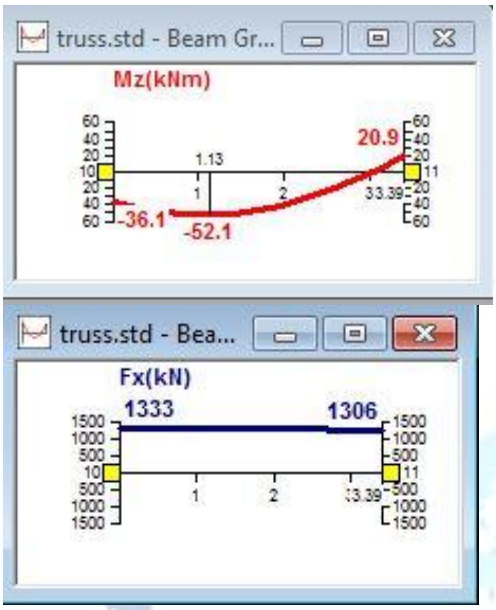
The dimensions of the top chord member BC are:

Length = 3.387 m

Dimension = 350X400

Compression Load (taken from staad) = 1330 KN

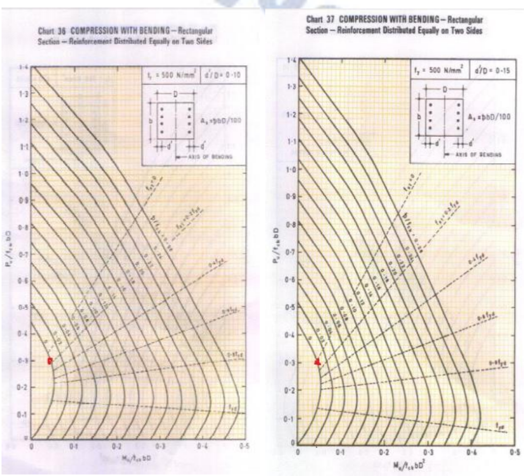
Moment (taken from staad) = 53 KNm



4.2.2.**aBendingmoment**
4.2.2.b Axial force

b = 350
D= 400
Pu = 1330 KN
Mu = 53 KNm
L =3.387 m
Fck = 30 N/mm^2
Fy = 500 N/mm^2
Le = 0.65x3.387 =2.20155
Le/D = 5.503875
Pu/(fckbd) = 0.31666667
Mu/(fck b d^2) = 0.03154762
Effective cover d' = 50 mm
d'/ D = 0.125

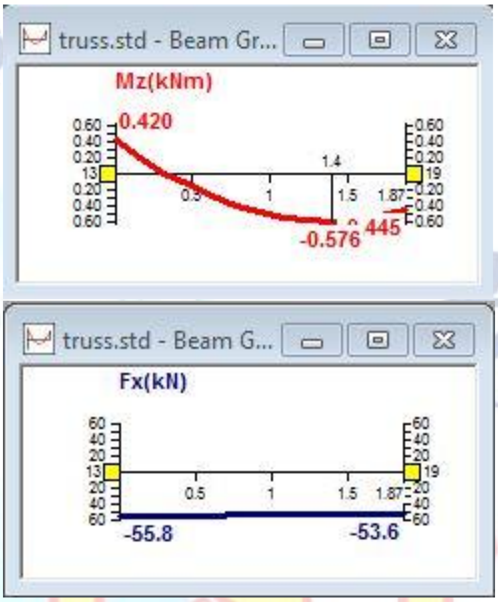
For Fe500 and d'/D = 0.125, referring to the charts of SP-16 the p/fck value to be derived based on Pu/(fck b d) and Mu/(fck b d^2) where p is the percentage of steel to be provided



4.2.2.C **SP16 charts**

Design of Top Chord member ER:
The dimensions of the top chord member ER are:

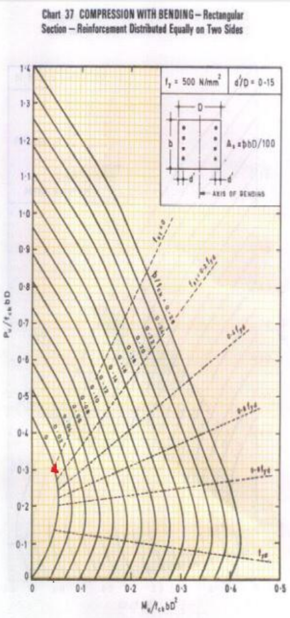
Length = 1.865 m
Dimension = 150x300
Compression Load (taken from staad) = 54 KN
Moment (taken from staad) = 1KNm

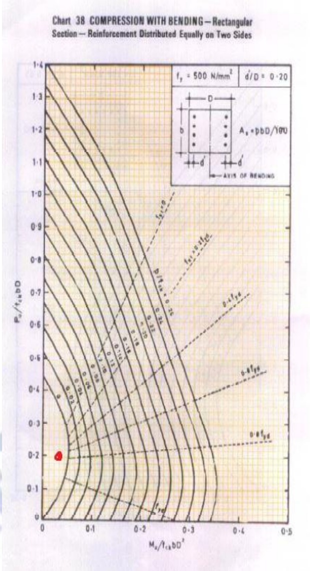


4.2.12.a.**Bendingmoment** **4.2.12.b Axial force**

b = 150
D= 300
Pu = 54KN
Mu = 1 KNm
L =1.865 m
Fck = 30 N/mm^2
Fy = 500 N/mm^2
Le = 0.65x1.865= 1.21225
Le/D = 4.04083333
Pu/(fckbd) = 0.040
Mu/(fck b d^2) = 0.00246114
Effective cover d' = 50 mm
d'/ D = 0.16666667

For Fe500 and d'/D = 0.166666667, referring to the charts of SP-16 the p/fck value to be derived based on Pu/(fck b d) and Mu/(fck b d^2) where p is the percentage of steel to be provided
.





4.2.12.c SP 16 charts

d' /D p/fck

0.15

0.16666667 -----

0.20 0

The percentage of steel value coming from charts is 0. So, the minimum percentage of steel has to be provided.

Minimum percentage of steel = 0.8x150x300/100 = 360 mm².

Hence provide a main reinforcement of 6-12 mm.

Provide a shear reinforcement of 8mm@ 200 c/c

DEFLECTION ANALYSIS

5.1 Introduction

In general steel trusses are being used everywhere for constructing long span roofs. Pre-stressed pre-cast concrete trusses are also one of the options for the long span roofs. In this project a Pre-stressed pre-cast concrete truss has been designed for a given loading conditions. Now, a steel which is a standard one which can be provided for same loading as of Pre-stressed pre-cast concrete truss is taken and deflection analysis of both trusses can be done. For that the standard steel truss is introduced as below.

5.2. Steel Truss

As there is a availability of steel truss standards from sp38, the standard steel section is taken from sp38 instead of making the design.

The same loading given for concrete truss is applied here

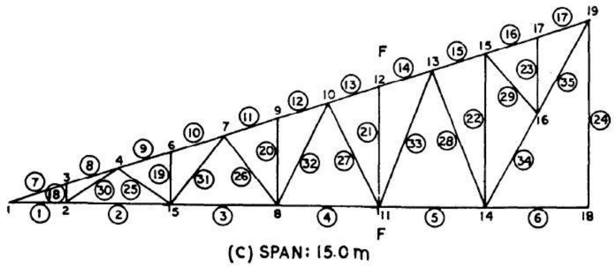


Figure 5.1: Steel truss.

The dimensions of the above ISA type section is shown below and taken from sp38 table 158 .

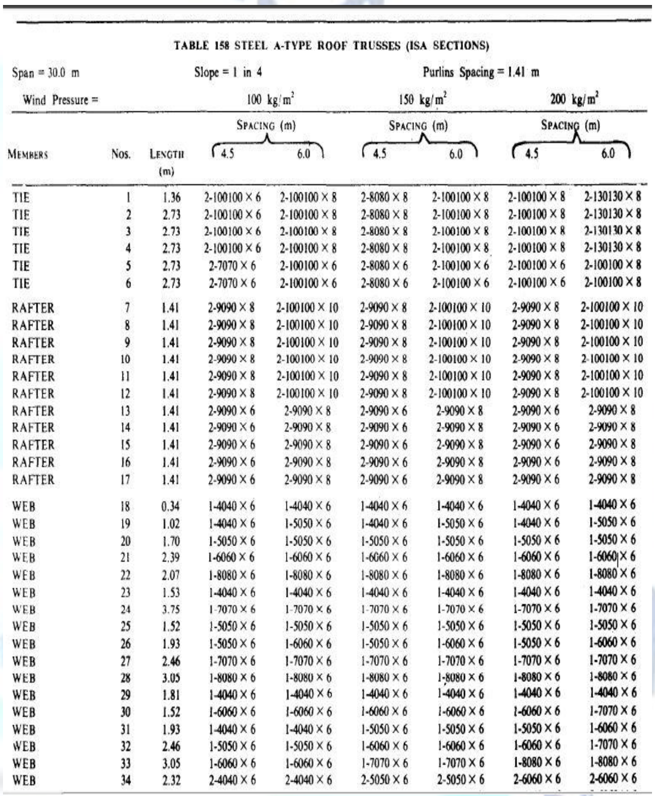


TABLE 158 STEEL A-TYPE ROOF TRUSSES (ISA SECTIONS)

Span = 30.0 m Slope = 1 in 4 Purlins Spacing = 1.41 m

			Wind Pressure = 100 kg/m²		150 kg/m²		200 kg/m²	
			Spacing (m)		Spacing (m)		Spacing (m)	
Members	Nos.	Length (m)	4.5	6.0	4.5	6.0	4.5	6.0
TIE	1	1.36	2-100100 × 6	2-100100 × 8	2-8080 × 8	2-100100 × 8	2-100100 × 8	2-130130 × 8
TIE	2	2.73	2-100100 × 6	2-100100 × 8	2-8080 × 8	2-100100 × 8	2-100100 × 8	2-130130 × 8
TIE	3	2.73	2-100100 × 6	2-100100 × 8	2-8080 × 8	2-100100 × 8	2-100100 × 8	2-130130 × 8
TIE	4	2.73	2-100100 × 6	2-100100 × 8	2-8080 × 8	2-100100 × 8	2-100100 × 8	2-130130 × 8
TIE	5	2.73	2-7070 × 6	2-100100 × 6	2-8080 × 6	2-100100 × 6	2-100100 × 6	2-100100 × 8
TIE	6	2.73	2-7070 × 6	2-100100 × 6	2-8080 × 6	2-100100 × 6	2-100100 × 6	2-100100 × 8
RAFTER	7	1.41	2-9090 × 8	2-100100 × 10	2-9090 × 8	2-100100 × 10	2-9090 × 8	2-100100 × 10
RAFTER	8	1.41	2-9090 × 8	2-100100 × 10	2-9090 × 8	2-100100 × 10	2-9090 × 8	2-100100 × 10
RAFTER	9	1.41	2-9090 × 8	2-100100 × 10	2-9090 × 8	2-100100 × 10	2-9090 × 8	2-100100 × 10
RAFTER	10	1.41	2-9090 × 8	2-100100 × 10	2-9090 × 8	2-100100 × 10	2-9090 × 8	2-100100 × 10
RAFTER	11	1.41	2-9090 × 8	2-100100 × 10	2-9090 × 8	2-100100 × 10	2-9090 × 8	2-100100 × 10
RAFTER	12	1.41	2-9090 × 8	2-100100 × 10	2-9090 × 8	2-100100 × 10	2-9090 × 8	2-100100 × 10
RAFTER	13	1.41	2-9090 × 6	2-9090 × 8	2-9090 × 6	2-9090 × 8	2-9090 × 6	2-9090 × 8
RAFTER	14	1.41	2-9090 × 6	2-9090 × 8	2-9090 × 6	2-9090 × 8	2-9090 × 6	2-9090 × 8
RAFTER	15	1.41	2-9090 × 6	2-9090 × 8	2-9090 × 6	2-9090 × 8	2-9090 × 6	2-9090 × 8
RAFTER	16	1.41	2-9090 × 6	2-9090 × 8	2-9090 × 6	2-9090 × 8	2-9090 × 6	2-9090 × 8
RAFTER	17	1.41	2-9090 × 6	2-9090 × 8	2-9090 × 6	2-9090 × 8	2-9090 × 6	2-9090 × 8
WEB	18	0.34	1-4040 × 6	1-4040 × 6	1-4040 × 6	1-4040 × 6	1-4040 × 6	1-4040 × 6
WEB	19	1.02	1-4040 × 6	1-5050 × 6	1-4040 × 6	1-5050 × 6	1-4040 × 6	1-5050 × 6
WEB	20	1.70	1-5050 × 6	1-5050 × 6	1-5050 × 6	1-5050 × 6	1-5050 × 6	1-5050 × 6
WEB	21	2.39	1-6060 × 6	1-6060 × 6	1-6060 × 6	1-6060 × 6	1-6060 × 6	1-6060 × 6
WEB	22	2.07	1-8080 × 6	1-8080 × 6	1-8080 × 6	1-8080 × 6	1-8080 × 6	1-8080 × 6
WEB	23	1.53	1-4040 × 6	1-4040 × 6	1-4040 × 6	1-4040 × 6	1-4040 × 6	1-4040 × 6
WEB	24	3.75	1-7070 × 6	1-7070 × 6	1-7070 × 6	1-7070 × 6	1-7070 × 6	1-7070 × 6
WEB	25	1.52	1-5050 × 6	1-5050 × 6	1-5050 × 6	1-5050 × 6	1-5050 × 6	1-5050 × 6
WEB	26	1.93	1-5050 × 6	1-6060 × 6	1-5050 × 6	1-6060 × 6	1-5050 × 6	1-6060 × 6
WEB	27	2.46	1-7070 × 6	1-7070 × 6	1-7070 × 6	1-7070 × 6	1-7070 × 6	1-7070 × 6
WEB	28	3.05	1-8080 × 6	1-8080 × 6	1-8080 × 6	1-8080 × 6	1-8080 × 6	1-8080 × 6
WEB	29	1.81	1-4040 × 6	1-4040 × 6	1-4040 × 6	1-4040 × 6	1-4040 × 6	1-4040 × 6
WEB	30	1.52	1-6060 × 6	1-6060 × 6	1-6060 × 6	1-6060 × 6	1-6060 × 6	1-7070 × 6
WEB	31	1.93	1-4040 × 6	1-4040 × 6	1-5050 × 6	1-5050 × 6	1-5050 × 6	1-6060 × 6
WEB	32	2.46	1-5050 × 6	1-5050 × 6	1-6060 × 6	1-6060 × 6	1-6060 × 6	1-7070 × 6
WEB	33	3.05	1-6060 × 6	1-6060 × 6	1-7070 × 6	1-7070 × 6	1-8080 × 6	1-8080 × 6
WEB	34	2.32	2-4040 × 6	2-4040 × 6	2-5050 × 6	2-5050 × 6	2-6060 × 6	2-6060 × 6

Table 5.1 Dimensions of ISA section

The analysis results of ISA type section is shown below. This results are taken from sp38, table

5.3 Deflection Analysis

The deflection results of steel truss: (taken from staad)

Bottom chord member 1: 0.984 mm

Bottom chord member 2: 3.247 mm

Bottom chord member 3: 1.018 mm

Bottom chord member 4: 1.047 mm

Bottom chord member 5: 1.099 mm

Bottom chord member 6: 0.802 mm

Bottom chord member 7: 0.780 mm

Bottom chord member 8: 1.1 mm

Bottom chord member 9: 1.095 mm

Bottom chord member 10: 1.019 mm

Bottom chord member 11: 3.247 mm
Bottom chord member 12: 0.984 mm

Maximum deflection noted in the steel truss is <u>3.247 mm.</u>

The deflection results of pre-stressed concrete truss: (taken from staad)

Bottom chord member AL: 0.919 mm
Bottom chord member LM: 0.909 mm
Bottom chord member MN: 1.628 mm
Bottom chord member NO: 1.457 mm
Bottom chord member OP: 1.628 mm
Bottom chord member PQ: 0.909 mm
Bottom chord member QR: 0.918 mm

Maximum deflection noted in the pre-stressed concrete truss is <u>1.628 mm</u>

- ➔ Comparing both results it can be concluded that pre-stressed concrete truss experiencing the lesser deflection than the steel truss.

V. CONCLUSIONS

- ➔ Hence the designs of pre-stressed pre-cast concrete truss members have done.
- ➔ Deflection analysis done for pre-stressed pre-cast concrete truss and steel truss.
- ➔ Deflection of steel truss is more than pre-stressed pre-cast concrete truss.
- ➔ Precast pre-stressed concrete member is recommended to use in Marine Areas.
- ➔ Precast pre-stressed concrete member is also recommended to use where there is a chance of fire accidents.

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