ANALYSIS AND DESIGN OF STEEL TRUSS BRIDGE FOR MASS RAPID TRANSIT SYSTEM

A Project Report

Submitted by

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MASTER OF TECHNOLOGY

In

STRUCTURAL ENGINEERING

Under the guidance of

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Candidate's Declaration

I do hereby certify that the work presented in this report entitled "**Analysis and design of steel truss bridge for mass rapid transit system"** in partial fulfillment of curriculum of final semester of Master of Technology in Structural Engineering, submitted in the department of civil engineering, DTU is an authentic record of my work under the supervision of Dr. Ashok Kumar Gupta, Professor in department of civil engineering. I have not submitted this matter for the award of any other degree or diploma.

Date: 25 July, 2016

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This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

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Faiyaz Khan

ABSTRACT

Keywords: *Metro viaduct, Arch truss, Pratt truss, through type, Stringer, Cross Girder.*

Trusses are used in bridges to transfer the gravity load of moving vehicles to supporting piers. Among the various types of bridges plate girder bridges, truss bridges and box girder bridges are more commonly used. Recently Indian Railways has either constructed several world class bridges or they are in the process of Construction. Howrah Bridge, also known as Rabindra Setu, is to be looked at as an early classical steel bridge in India. . Some of the trusses that are used in steel bridges Truss Girders, lattice girders or open web girders are efficient and economical structural systems, since the members experience essentially axial forces and hence the material is fully utilized. Members of the truss girder bridges can be classified as chord members and web members. Generally, the chord members resist overall bending moment in the form of direct tension and compression and web members carry the shear force in the form of direct tension or compression. Due to their efficiency, truss bridges are built over wide range of spans. Truss bridges compete against plate girders for shorter spans, against box girders for medium spans and cable-stayed bridges for long spans.

In this research, a study has been carried out to find out the efficient and economical option best suited for obligatory situations in metro viaduct. In the first part of this report, background of the study and literature review of the previous study is presented. Initially two type of steel bridges are adopted in this study, first one is Pratt truss bridge and another one is Arch truss bridge. The present work is carried out considering metro viaduct in Delhi which is in zone 4 region. Members of truss bridge are designed manually. STAAD.Pro.V8i software is used throughout this study for the structural modeling and analysis of bridge. Weight of steel truss bridge is calculated and comparison is done on the basis of it.

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Since inception of Indian Railways in 1853 the Railway Engineers has history of more than 250 years of construction and maintenance of railway bridges. During the long journey they had achieved several heights and continuing to excellence. Recently Indian Railways has either constructed several world class bridges or they are in the process of construction. A technical review of design and construction of recent bridges (viz. Bogibeel, Chenab and New Jubilee Bridges) through light on the recent technological advancements attained by the Indian Railways in the field of bridge engineering. Advantages of structural steel over other construction materials are its strength and ductility. It has a higher strength to cost ratio in tension and a slightly lower strength to cost ratio in compression when compared with concrete. The stiffness to weight ratio of steel is much higher than that of concrete. Thus it has become the obvious choice for long span bridges as steel is more efficient and economic. Among the various types of bridges plate girder bridges, truss bridges and box girder bridges are more commonly used.

Structural steel has been the natural solution for long span bridges since 1890, when the Firth of Forth cantilever bridge, the world's major steel bridge at that time was completed. Steel is indeed suitable for most span ranges, but particularly for longer spans. Howrah Bridge, also known as Rabindra Setu, is to be looked at as an early classical steel bridge in India. This cantilever bridge was built in 1943. It is *97 m* high and *705 m* long. This engineering marvel is still serving the nation, deriding all the myths that people have about steel.

1.2 OBJECTIVES OF THE STUDY

Following are the main objectives of the present study:

a) To understand the behavior of structural action of steel truss bridge which have been adopted in the analysis and design for MRTS available in literature.

b) To perform the analysis and design of Pratt truss and Arch truss bridges for metro viaduct.

c) To find out the efficient and economical option best suited for obligatory situations in metro viaduct

1.3 SCOPE OF THE STUDY

The present work is about the study of steel truss bridge for obligatory situations in metro viaduct. Two type of steel bridges are adopted in this study, first one is Pratt truss bridge and another one is Arch truss bridge. The present work is carried out considering metro viaduct in Delhi which is in zone 4 region. Members of truss bridge such as bottom chord, top chord, diagonals, verticals, stingers, cross girder etc are designed manually. STAAD.Pro.V8i software is used throughout this study for the structural modeling and analysis of bridge.

1.4 METHODOLOGY

a) A thorough literature review to understand the behavior of structural action of steel truss bridge.

b) Analysis of Pratt and Arch truss bridges with geometrical and loading details given in Design Basis report of DMRC.

c) Members of truss bridge such as bottom chord, top chord, diagonals, verticals, stingers, cross girder etc are designed manually and finally reaching at a conclusion that which of the option is efficient, economical and best suited for obligatory situations in metro viaduct.

1.5 ORGANISATION OF THESIS

This thesis is divided into five chapters. This first introductory chapter presents the background; objectives; scope; methodology and research significance of the project. In the second chapter, a literature review on the behavior of steel truss bridge is reported. Focus is placed on structural behavior of the components of truss bridge. This chapter also includes the previous researches on the steel truss bridge. Chapter 3 presents structural modeling of the models of truss ,their analysis and design calculations. Chapter 4 presents the analysis and design results and different interpretations of the results. Finally in the last chapter, the work carried out is reviewed. The findings from the study are reported.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

The literature available on the analysis and design of steel truss bridge is very limited; however we can get a number of published literatures on the analysis of different truss bridges. It becomes a bit tedious to analyze the steel truss bridge and design the members of bridge manually. In addition, literature on the obligatory situations in metro viaduct are very limited. Thus the literature survey is presented here in two main areas: (i) steel truss bridge and (ii) the obligatory situations in metro viaduct .

2.2 TRUSS BRIDGE

Trusses are used in bridges to transfer the gravity load of moving vehicles to supporting piers. Depending upon the site conditions and the span length of the bridge, the truss may be either through type or deck type. In the through type, the carriage way is supported at the bottom chord of trusses. In the deck type bridge, the carriage way is supported at the top chord of trusses. Usually, the structural framing supporting the carriage way is designed such that the loads from the carriage way are transferred to the nodal points of the vertical bridge trusses. Some of the trusses that are used in steel bridges Truss Girders, lattice girders or open web girders are efficient and economical structural systems, since the members experience essentially axial forces and hence the material is fully utilized. Members of the truss girder bridges can be classified as chord members and web members. Generally, the chord members resist overall bending moment in the form of direct tension and compression and web members carry the shear force in the form of direct tension or compression. Due to their efficiency, truss bridges are built over wide range of spans. Truss bridges compete against plate girders for shorter spans, against box girders for medium spans and cable-stayed bridges for long spans. Some of the most commonly used trusses suitable for both road and rail bridges are shown below.

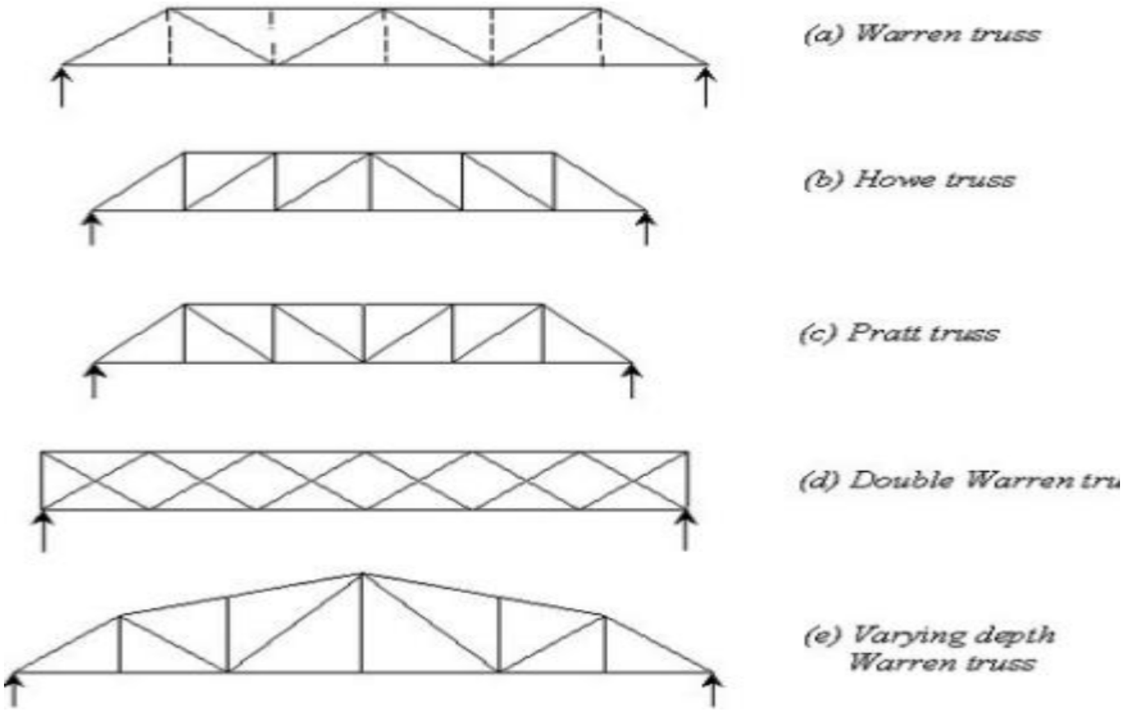


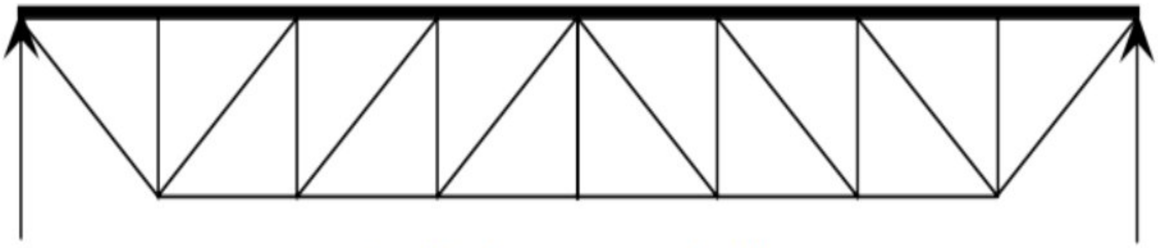
Fig. 2.1 some commonly used steel truss bridges.

For short and medium spans it is economical to use parallel chord trusses such as Warren truss, Pratt truss, Howe truss, etc. to minimize fabrication an erection costs. Especially for shorter spans the warren truss is more economical as it requires less material than either the Pratt or Howe trusses. However, for longer spans, a greater depth is required at the centre and variable depth trusses are adopted for economy. In case of truss bridges that are continuous over many supports, the depth of the truss is usually larger at the supports and smaller at midspan.

As far as configuration of trusses is concerned, an even number of bays should be chosen in Pratt and modified Warren trusses to avoid a central bay with crossed diagonals. The diagonals should be at an angle between 50° and 60° to the horizontal. Secondary stresses can be avoided by ensuring that the centroidal axes of all intersecting members meet at a single point, in both vertical and horizontal planes. However, this is not always possible, for example when cross girders are deeper than the bottom chord then bracing members can be attached to only one flange of the chords.

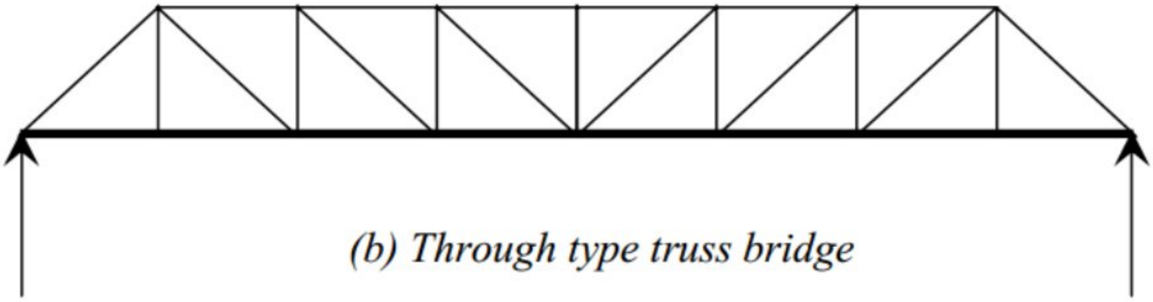
Depending upon the site conditions and the span length of the bridge, the truss may be of the "**deck type**", "**through type**" or "**semi-through type**". These are described below with respect to truss bridges:

(i) Deck Type Bridge **-** The carriageway rests on the top of the main load carrying members. In The deck type plate girder bridge, the roadway or railway is placed on the top flanges. In the deck type truss girder bridge, the roadway or railway is placed at the top chord level as shown in Fig. 2.2(a)



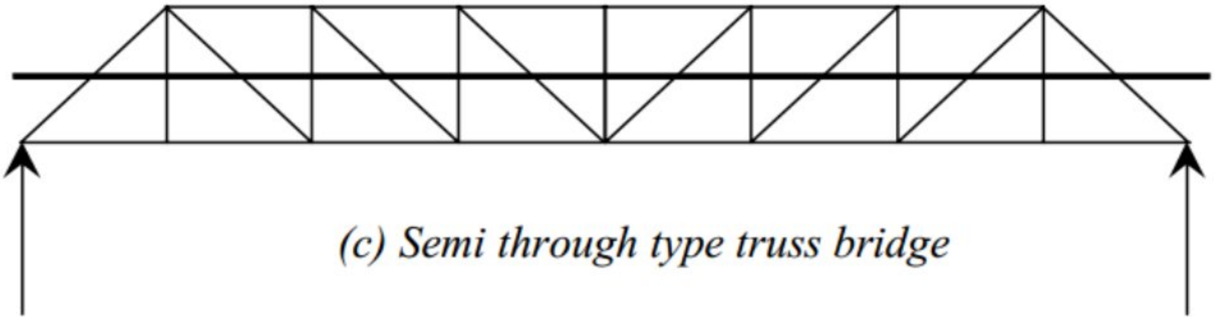
(a) Deck type truss bridge

(ii) Through Type Bridge - The carriageway rests at the bottom level of the main load carrying members [Fig.2.2(b)]. In the through type plate girder bridge, the roadway or railway is placed at the level of bottom flanges. In the through type truss girder bridge, the roadway or railway is placed at the bottom chord level.



(b) Through type truss bridge

(iii) Semi through Type Bridge - The deck lies in between the top and the bottom of the main load carrying members. The bracing of the top flange or top chord under compression is not done and part of the load carrying system project above the floor level as shown in Fig. 2.2(*c*). The lateral restraint in the system is obtained usually by the U-frame action of the verticals and cross beam acting together.



(c) Semi through type truss bridge

2.3 COMPONENTS OF STEEL TRUSS BRIDGE

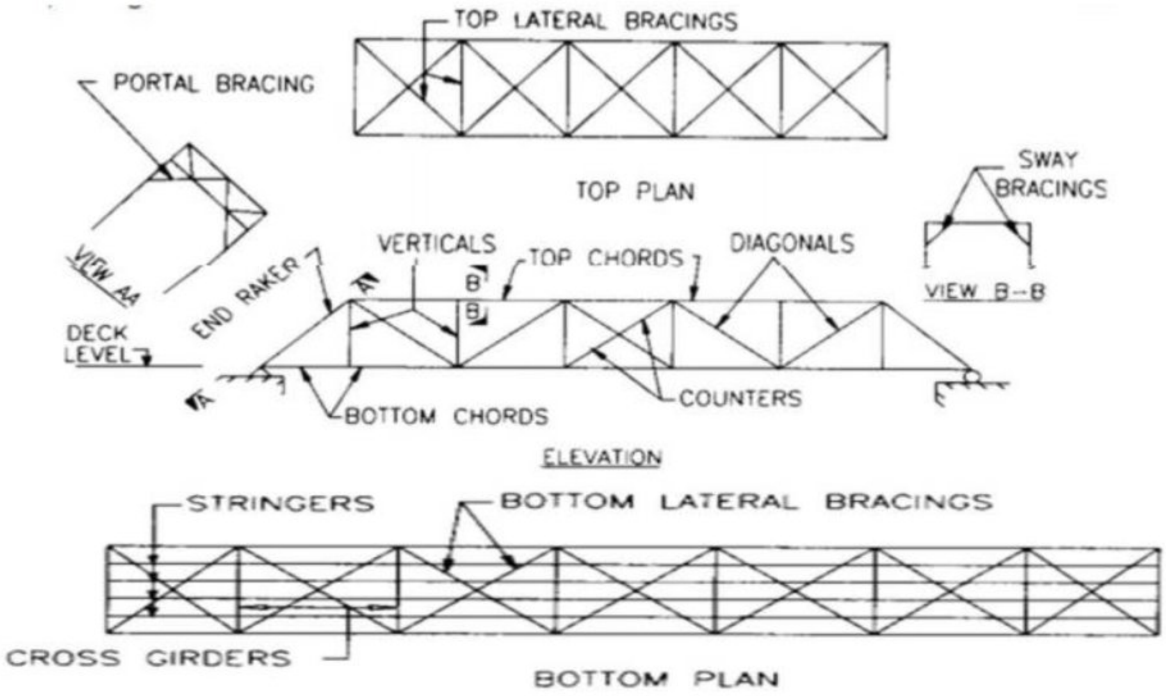


Fig.2.3 various components of steel truss

2.3.1 TOP CHORD

These are the members which are in compression. They need special attention while proportioning and detailing.

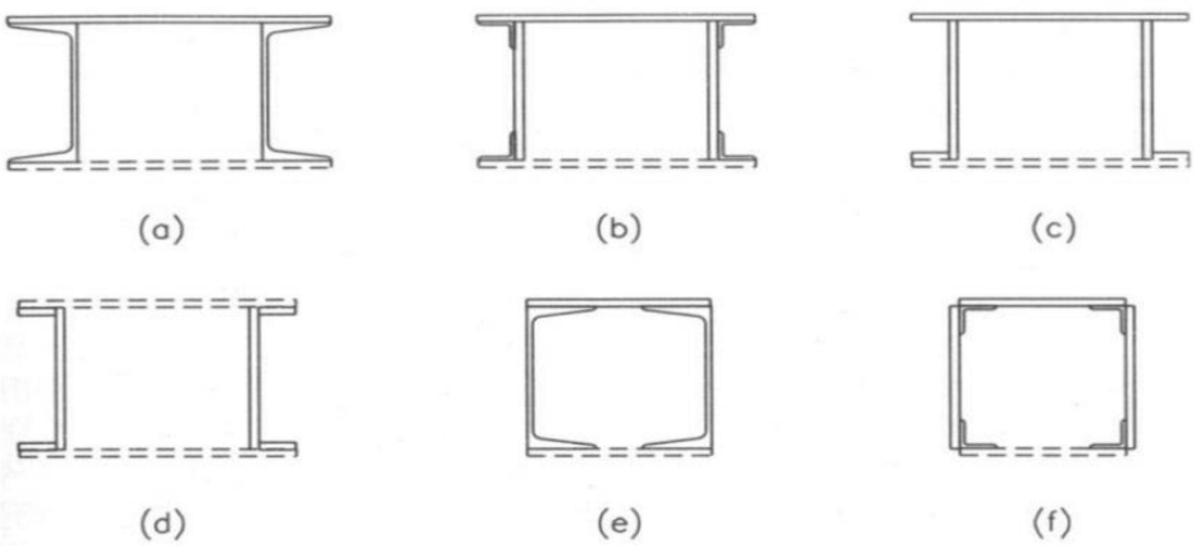


Fig.2.4 common cross sections for top chord

2.3.2 BOTTOM CHORD

These are the members which are in the tension. Some of the common cross section shown in figure2.5

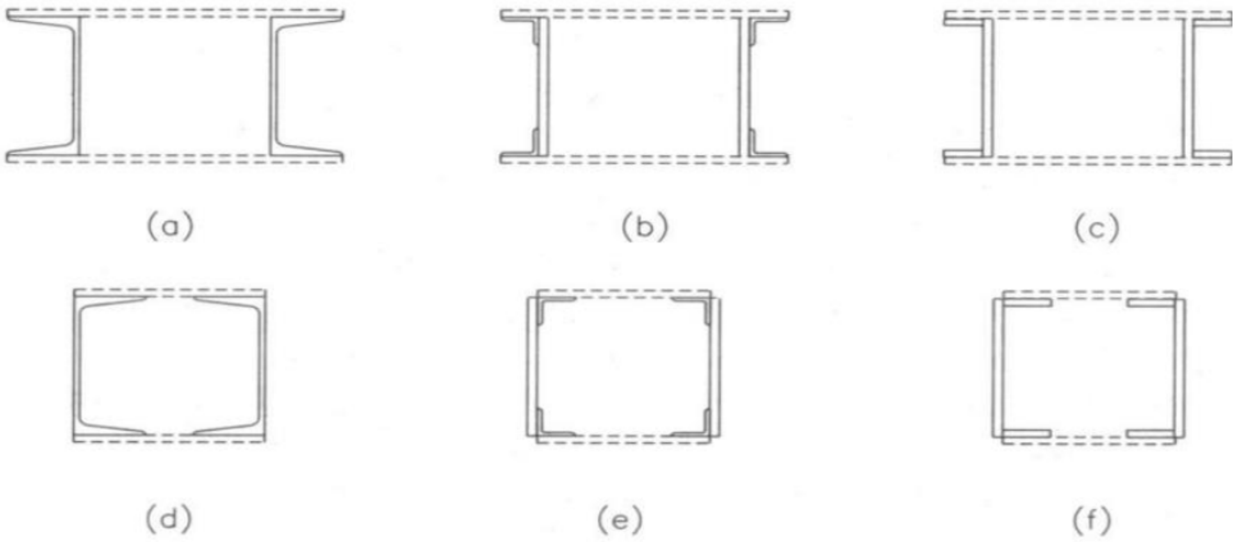


Fig.2.5 Common cross-sections of bottom chords

2.3.3 WEB MEMBERS

These are the members which could be diagonals and verticals and subjected to tension or compression which depends on the type of truss and loading. Vertical members working at compression are termed “post” and those in tension are called “hangers”.

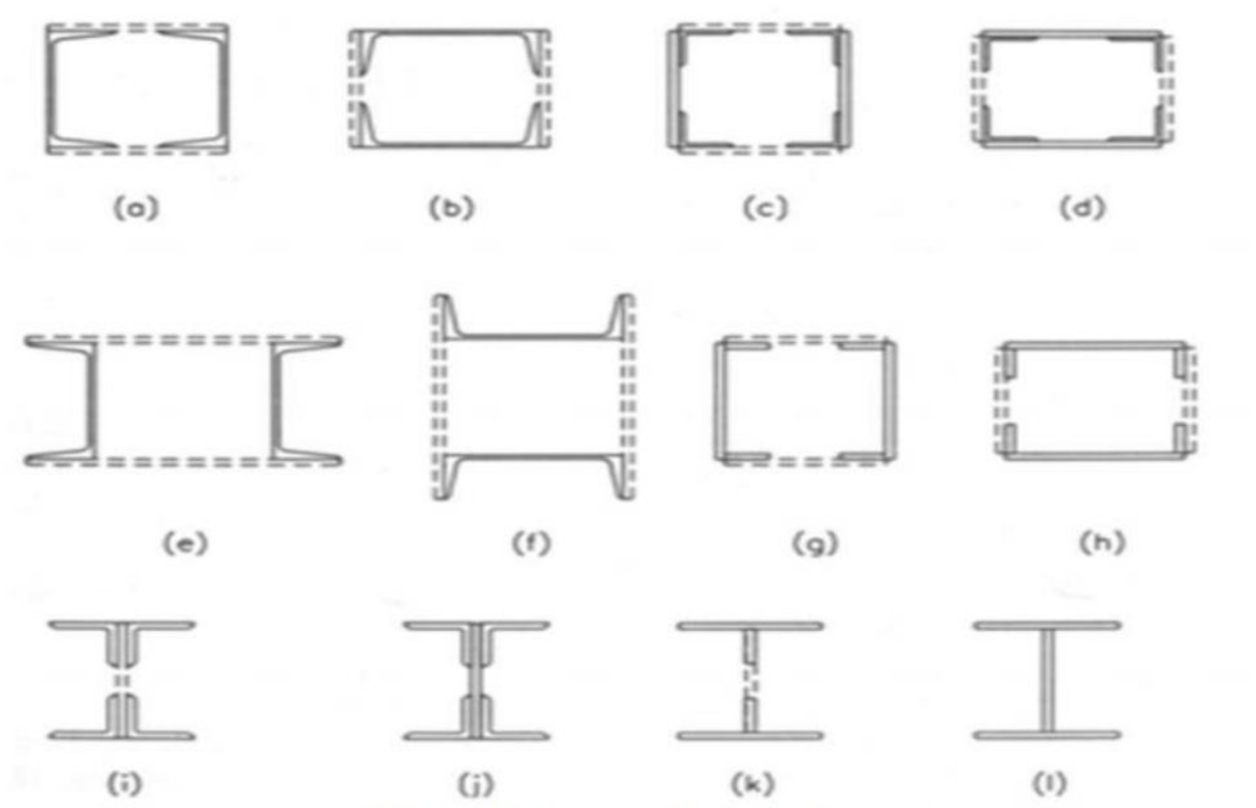


Fig. 2.6 Typical cross sections of web members

2.3.4 END POSTS OR RAKERS

These are the members which are located at the ends of a truss to carry lateral and longitudinal forces from the top chord level to the bridge bearings. For this purpose portal bracings are fixed onto them at the upper level.

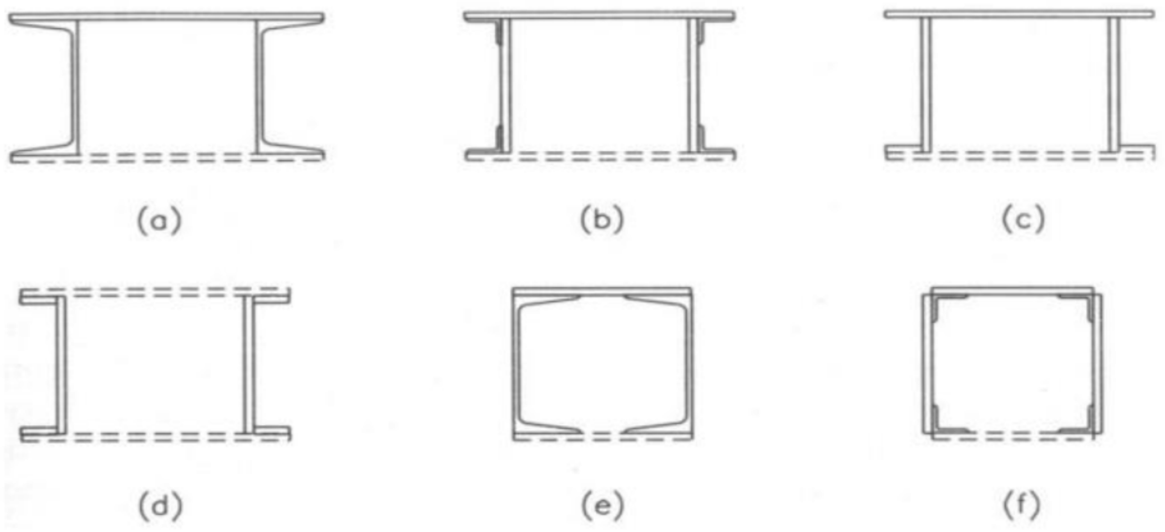


Fig. 2.7 Typical cross sections of end reacker members

2.3.5 BRACING SYSTEM

These are considered as secondary members, but in fact, vital for the successful performance of the primary members. Bracings are designed to resist two types of forces lateral and longitudinal forces. Lateral forces are those which acting transverse to the axis of the bridge. Longitudinal forces are those which acting along the axis of the bridge. Bracing may be lateral, sway or cross and portal type.

2.3.5.1 Lateral bracing: Placed between the top chords and bottom chords of a pair of trusses

(a) TRADITIONAL SYSTEM

(b) DIAMOND SYSTEM

Fig. 2.8 Typical cross sections of lateral bracing systems

2.3.5.2 Sway bracing or cross bracing:

These are placed between trusses. They are provided for distributing the transverse loads to the lateral system, also for providing torsion rigidity to the truss frame.

Fig. 2.9 sway bracing

2.3.5.3 Portal bracings

They are located at the end posts or reakers and provide end supports to the top lateral bracing system.

Fig. 2.10 portal bracing

2.4 LOADS ON BRIDGES

The following are the various loads to be considered for the purpose of computing stresses, wherever they are applicable.

(1)Dead load
(2) Live load.
(3) Impact effect.
(4) Forces due to curvature and eccentricity of Track.
(5) Temperature effect.
(6)Resistance of expansion bearings to movements
(7) Longitudinal force.
(8) Racking force.
(9) Forces on parapets.
(10) Wind pressure effect.
(l1) Forces and effects due to earthquake.
(12) Erection forces and effects.
(13) Derailment loads

2.4.1DEAD LOAD

The dead load on a bridge consists of the weight of all its structural parts and all the fixtures and services like deck surfacing, kerbs, parapets, lighting and signing devices, gas and water mains, electricity and telephone cables. The weight of the structural parts has to be guessed at the first instance and subsequently confirmed after the structural design is complete.

2.4.2 LIVE LOADS

Bridge design standards of different countries specify the design loads which are meant to reflect the worst loading that can be caused on the bridge by traffic permitted and expected to pass over it. The relationship between bridge design loads and the regulations governing the weights and sizes of vehicles is thus obvious, but other factors like traffic volume and mixture of heavy and light vehicles are also relevant.

2.4.3 LONGITUDINAL FORCES

Where a structure carries railway track, provision as under shall be made for the longitudinal loads arising from any one or more of the following causes:

(a) the tractive effort of the driving wheels of locomotives;

(b) the braking force resulting from the application of the brakes to all braked wheels;

(c) resistance to the movement of the bearings due to change of temperature and deformation of the bridge girder. Roller, PTFE or elastomeric bearings may preferably be provided to minimize the longitudinal force arising on this account.

(d) Forces due to continuation of LWR/CWR over the bridge

2.4.4 IMPACT LOAD

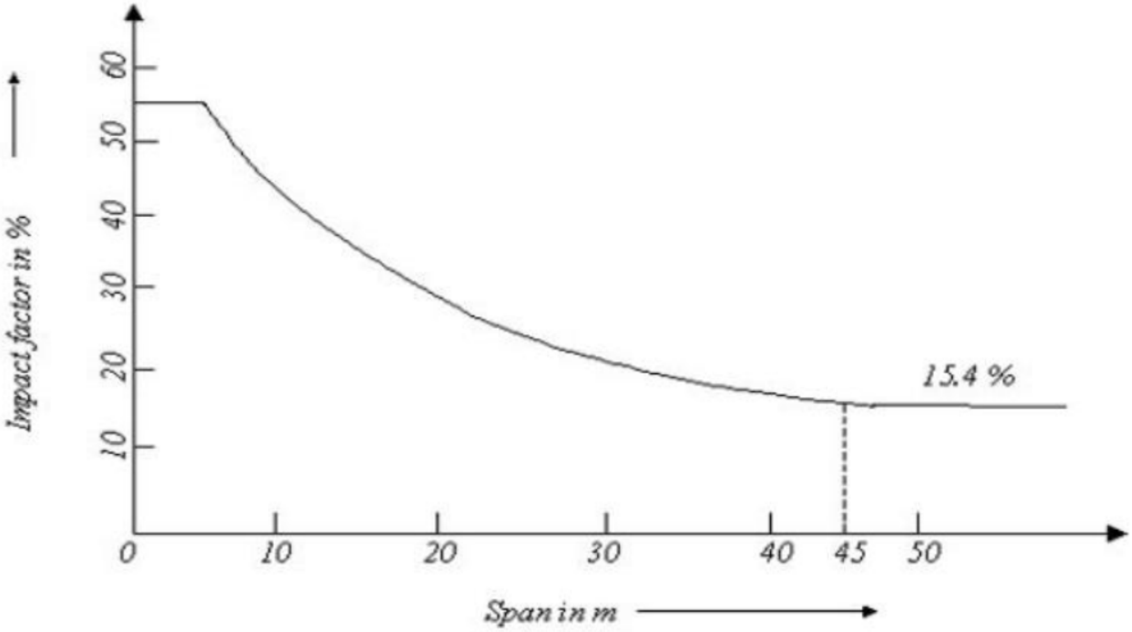
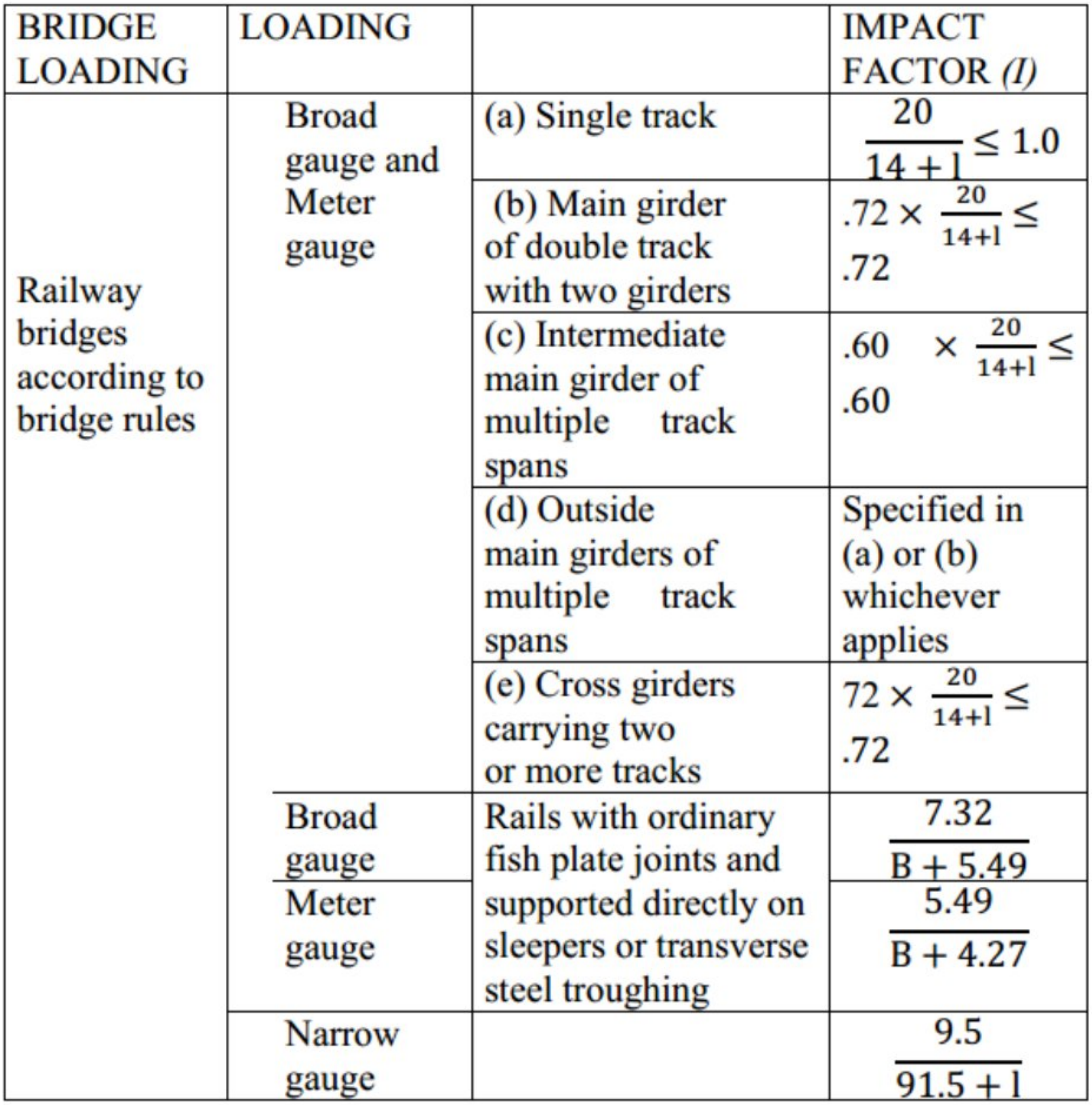


Fig.2.11 Impact percentage curve for highway bridges for IRC class A and IRC class B loading

The dynamic effect caused due to vertical oscillation and periodical shifting of the live load from one wheel to another when the locomotive is moving is known as impact load. The impact load is determined as a product of impact factor, I, and the live load. The impact factors are specified by different authorities for different types of bridges. The impact factors for different bridge for different type of moving loads is given in the table 1. Note that, in the above table l is loaded length in m and B is spacing of main girders in m.



BRIDGE LOADING	LOADING		IMPACT FACTOR *(I)*
Railway bridges according to bridge rules	Broad gauge and Meter gauge	(a) Single track	$\frac{20}{14+l} \le 1.0$
		(b) Main girder of double track with two girders	$.72 \times \frac{20}{14+l} \le .72$
		(c) Intermediate main girder of multiple track spans	$.60 \times \frac{20}{14+l} \le .60$
		(d) Outside main girders of multiple track spans	Specified in (a) or (b) whichever applies
		(e) Cross girders carrying two or more tracks	$72 \times \frac{20}{14+l} \le .72$
	Broad gauge	Rails with ordinary fish plate joints and supported directly on sleepers or transverse steel troughing	$\frac{7.32}{B+5.49}$
	Meter gauge		$\frac{5.49}{B+4.27}$
	Narrow gauge		$\frac{9.5}{91.5+l}$

2.4.5 THERMAL FORCES

The free expansion or contraction of a structure due to changes in temperature may be restrained by its form of construction. Where any portion of the structure is not free to expand or contract under the variation of temperature, allowance should be made for the stresses resulting from this condition. The coefficient of thermal expansion or contraction for steel is 11.7 x 10-6/.

2.4.6 WIND LOAD

Wind load on a bridge may act

(1) Horizontally, transverse to the direction of span

(2) Horizontally, along the direction of span

(3) Vertically upwards, causing uplift

(4) Wind load on vehicles

Wind load effect is not generally significant in short-span bridges; for medium spans, the design of sub-structure is affected by wind loading; the super structure design is affected by wind only in long spans. For the purpose of the design, wind loadings are adopted from the maps and tables given in IS: 875 (Part III).

2.4.7 RACKING FORCES

This is a lateral force produced due to the lateral movement of rolling stocks in railway bridges.

Lateral bracing of the loaded deck of railway spans shall be designed to resist, in addition to the wind and centrifugal loads, a lateral load due to racking force of 6.0 kN/m treated as moving load. This lateral load need not be taken into account when calculating stresses in chords or flanges of main girders.

2.4.8 SEISMIC LOAD

If a bridge is situated in an earthquake prone region, the earthquake or seismic forces are given due consideration in structural design. Earthquakes cause vertical and horizontal forces in the structure that will be proportional to the weight of the structure. Both horizontal and vertical components have to be taken into account for design of bridge structures. IS:1893 – 1984 may be referred to for the actual design loads.

2.4.9. FORCE DUE TO CURVATURE

When a track or traffic lane on a bridge is curved allowance for centrifugal action of the moving load should be made in designing the members of the bridge. All the tracks and lanes on the structure being considered are assumed as occupied by the moving load.

2.4.10 ERECTION FORCES

There are different techniques that are used for construction of railway bridges, such as launching, pushing, cantilever method, lift and place. In composite construction the composite action is mobilised only after concrete hardens and prior to that steel section has to carry dead and construction live loads. Depending upon the technique adopted the stresses in the members of the bridge structure would vary. Such erection stresses should be accounted for in design. This may be critical, especially in the case of erection technologies used in large span bridges.

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2.5 APPLICABLE SPAN OF TRUSS BRIDGE

Truss brides are generally applied within the following range

1. Simple truss bridge is in the range of 55 meter to 85 meter span.
2. Continuous truss bridge is in the range of 60 meter to 300 meter.
3. Cantilever truss bridge is in the range of 300 meter to 510 meter span (in Japan, only one bridge has longer span than 200 meter, and that is Minato Ohashi, with 510 meter span.)

Among the 3 types of bridges, the simple truss bridge or the continuous truss bridge, either with approximate 60 meter to 100 meter span is usually applied.

2.6 CONNECTIONS

Members of trusses can be joined by riveting, bolting or welding. Due to involved procedure and highly skilled labour requirement, riveting is not common these days, except in some railway bridges in India. In railway bridges riveting may be used due to fatigue considerations. Even in such bridges, due to recent developments, 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. However, the higher cost of shop fabrication due to excise duty in contrast to lower field labour cost frequently favor field fabrication in India.

2.7 OBLIGATORY SITUATIONS

The common type of standard span superstructures, used for metro construction in India with span range of 16m to 37m are: (1) Box girder supporting 2 tracks – precast segmental. (2) U-girders supporting 2 tracks – precast segmental. (3) U-girder supporting single track - full span precast. (4) I-girder (precast slab) with in-situ slab. The sketch of the first two types is depicted in Figs.2.12 and 2.13

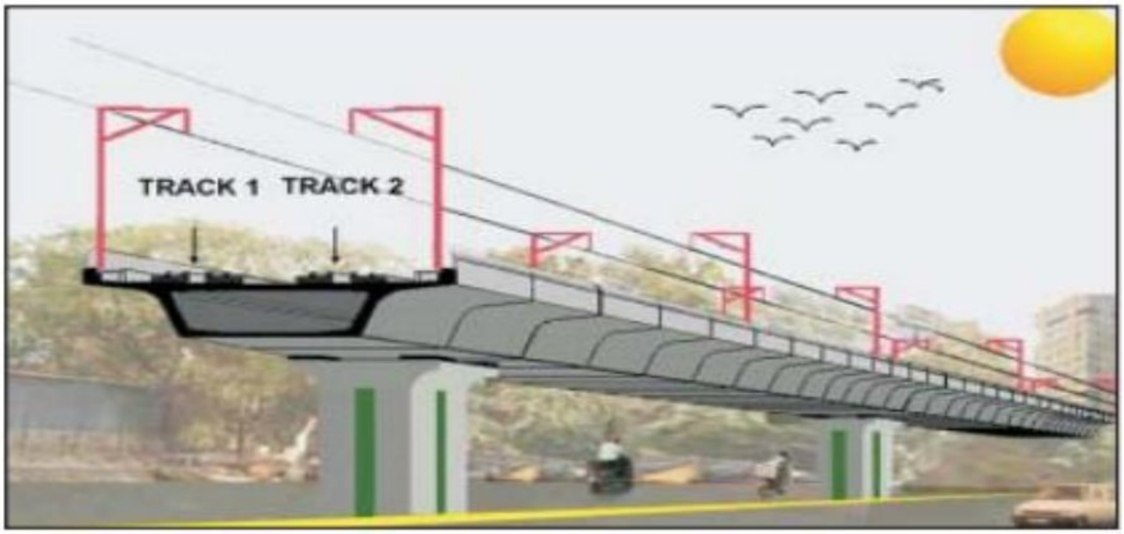


Fig.2.12 Box Girder (Precast Segmental) Supporting 2 Tracks

Fig.2.13 U-Girder (Precast Segmental) Supporting 2 Tracks

Obligatory situations occur when spans longer than about 37.0m are required wherein standard precast segments are impractical or uneconomical to use. These situations can also occur when fairly long spans are required to straddle across major obstacles and railway tracks or the alignment has sharp curvatures to avoid existing buildings. In these situations, conventional method is to use 3 span structures using cantilever construction technique. Although steel trusses launched into position have also been used effectively in such situations.

Cast-in-situ free cantilevering method at major road and railway crossings are:

a. **OUTER RING ROAD CROSSING AT MADHUBAN CHOWK**

span arrangement: 38.5m + 55.0m + 38.5m

b. **RING ROAD CROSSING**

span arrangement: 38.5m + 55.0m + 38.5

c. **ASHOK VIHAR CROSSING**

span arrangement: 33.5m + 46.2m + 33.4m

d**. PULBANGASH RAILWAY TRACKS**

span arrangement: 41.5m + 60.5m + 41.5m

CHAPTER 3
METHODOLOGY

3.1 INTRODUCTION

The study in this report is concerned with the behavior of structural action of steel truss bridge for metro viaduct and to find out the efficient and economical option best suited for obligatory situations in metro viaduct. This chapter includes the modeling of Pratt and Arch steel truss bridge in STAAD.Pro.V8i software and members are designed manually. Weight of steel truss bridge is calculated and comparison is done on the basis of it.

3.2 MODELING OF TRUSS BRIDGE

A steel truss of through type is selected for span 58.3m. The dimensions of the through truss are based on the DBR as shown in Fig.3.1 and 3.2. Structural configuration is having verticals as compression members and diagonals as tension members. Bottom Chord and Top Chord are built up box sections, verticals, diagonals, cross girders and stringers are built up I-sections. Bottom floor system consists of Cross girders, composite Stringers with deck and plan bracings. Deck slab is proposed to be 9700mm only against 10500mm carried out in viaduct portion. Top plan consists of transverse members and plan bracings. . Secondary stresses are considered as per IRS SBC code for Bottom chord, Top chord, Verticals and Diagonal members. Member weight is applied as self weight with increase for connections. Deck load, SIDL and LL are applied at stringers. Wind loads are applied at top and bottom chord joints.

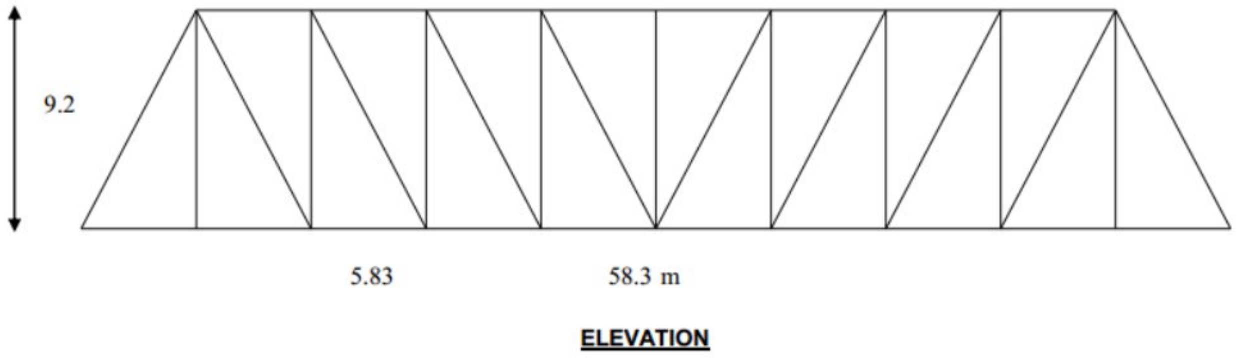


Fig.3.1 Elevation of truss

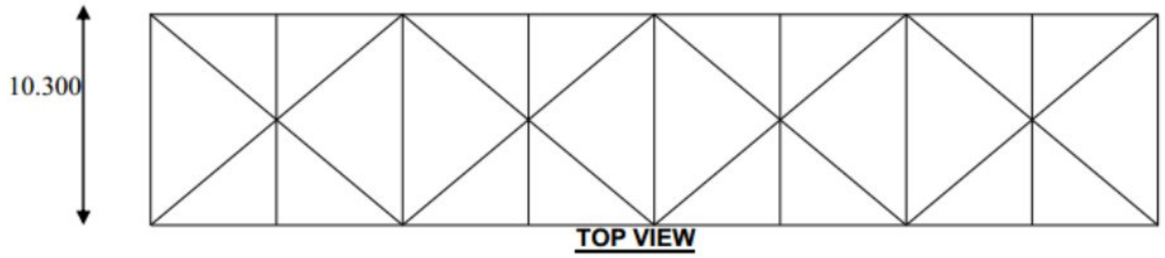


Fig.3.2 Top view of truss

3.3 INPUT DATA FOR LOAD CALCULATIONS:

3.3.1 FOR DEAD LOAD:

Increment Factor for self wt = 0.25

3.3.2 FOR SIDL:

1	Rails pads					0.30	t/m
2	Cables					0.07	t/m
3	Cable trough cell					0.74	t/m
4	Cable trays					0.01	t/m
5	Hand rail					0.08	t/m
6	Plinth					3.40	t/m
7	Light wt deck drainage concrete					0.24	t/m
8	Parapet					0.00	t/m
8	Miscellaneous (OCS, signaling)					0.40	t/m
	Total					**5.24**	t/m
	Load of walkway					0.50	t/m

3.3.3 FOR LIVE LOAD

Vertical train live load								
	All axle Loads			17	t	=	166.77	kN
	One Bogie Length			22.1	m			
	Configuration			Alt 1	Alt 2			
		a=		2.250	2.605			
		b=		2.500	2.290			
		c=		12.600	12.310			
	CG of LL from rail level		=	1.83	m			
	Impact Factor		=	1.2				
Horizontal train live load								
	Braking Load			18	% of unfactored vertical load			
	Traction Load			20	% of unfactored vertical load			
	In seismic condition, transverse Load			50	% of normal condition			

3.3.4 FOR WIND LOAD CALCULATION

Calculation of Wind Load					
	Intensity given in D.B.R	=	1.609	kN/m2	for unloaded
		=	1.5	kN/m2	(for loaded as per Bridge Rules)
Height of moving load		=	3.5	m	

3.3.5 FOR SIESMIC LOAD

Seismic Zone		IV		
Z		0.24		
R		4		
I		1.5		
Sa/g		2.5		
Minimum Ah		0.1		
a_h	=	$(z/2).(Sa/g)_h$	=	0.113
		(R/I)		
a_v	=	$(2/3)(z/2).(Sa/g)_v$	=	0.075
		(R/I)		

3.3.6 RAKING FORCE

Raking force	=	600	kg/m	(as per Br rules 2.9.1)

3.4 COMPUTATIONAL MODELS

In this study, four models are developed. Two model for each truss bridge, model one consider dead load, wind load, seismic load ,raking forces etc. and other consider live load only. In live load model, eight bogies of metro train is considered and force calculation is done for every 0.5m interval.

3.4.1 PRATT TRUSS: MODEL 1

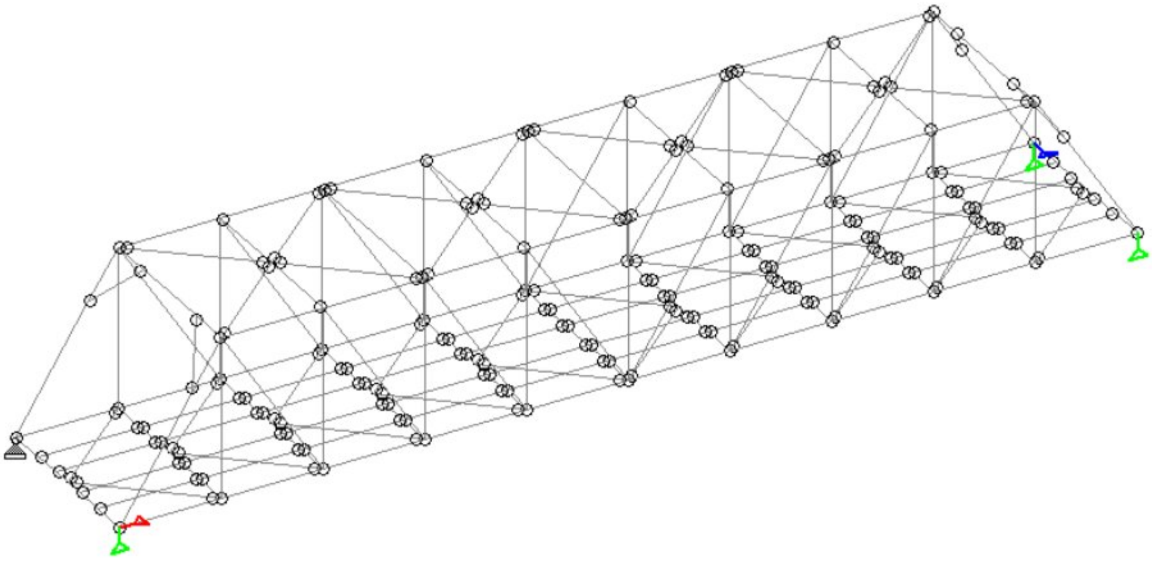


Fig. 3.3 Pratt truss model 1

3.4.2 PRATT TRUSS: MODEL 2

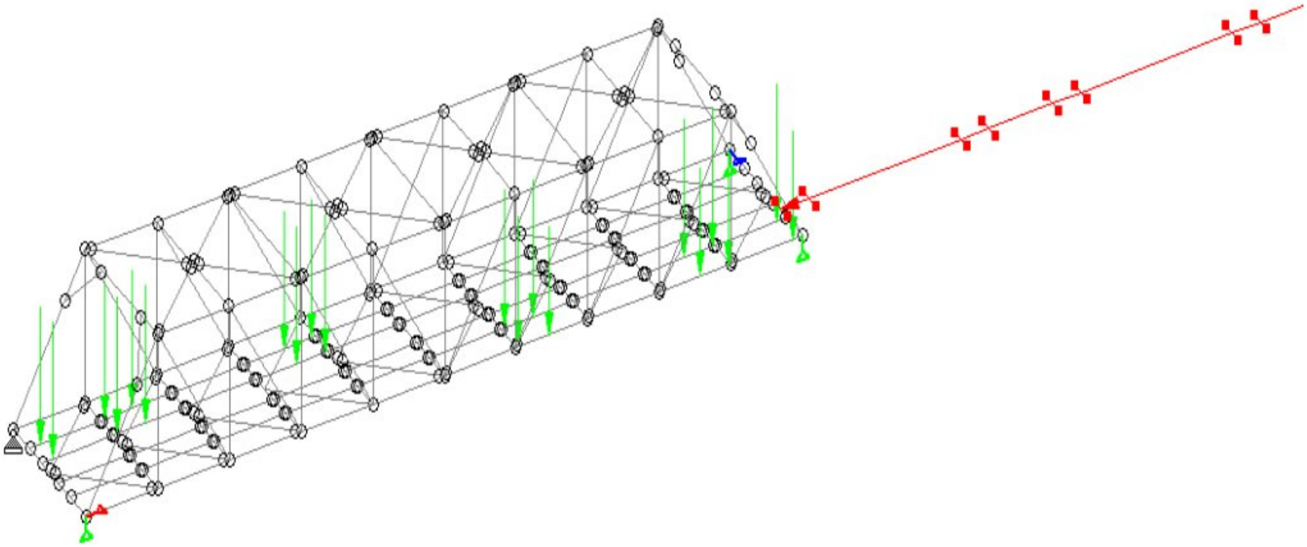


Fig. 3.4 Pratt truss model 2

3.4.3 ARCH TRUSS: MODEL 1

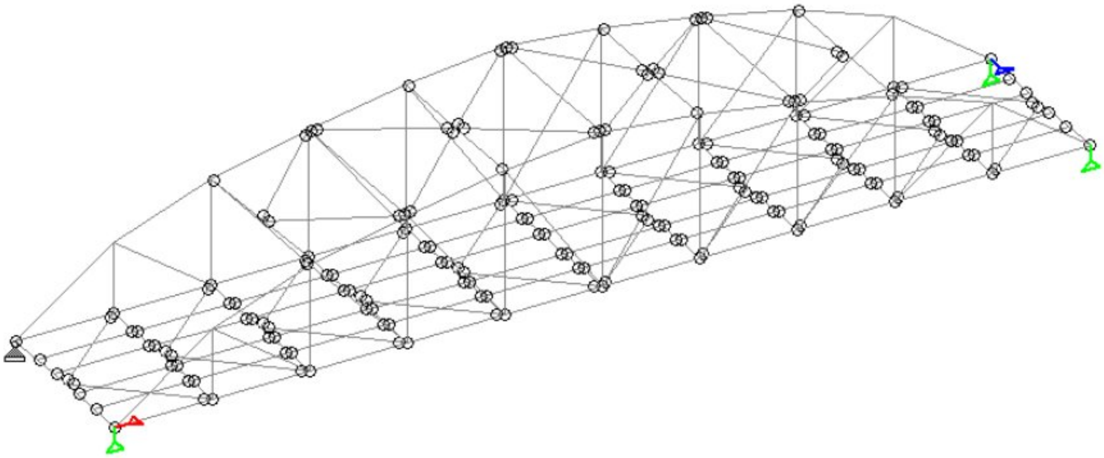


Fig. 3.5 Arch truss model 1

3.4.4 ARCH TRUSS: MODEL 2

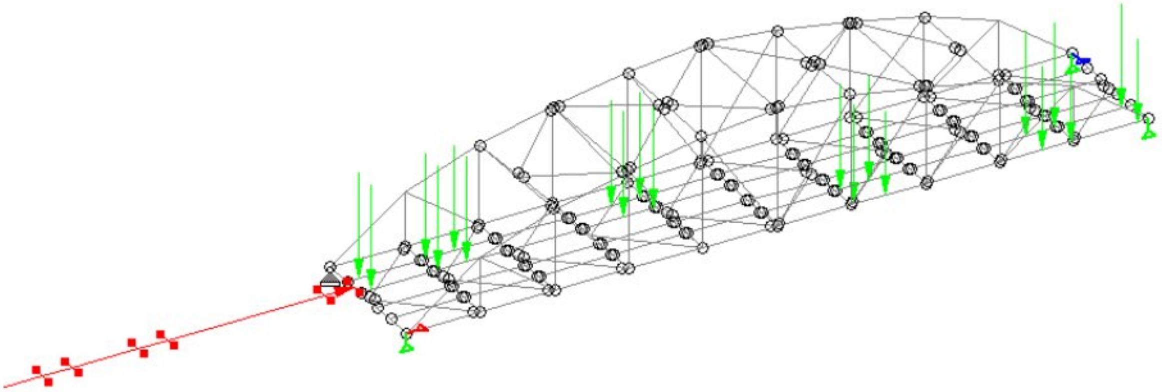


Fig. 3.6 Arch truss model 2

3.5 GENERAL DESIGN PRINCIPLES

3.5.1 Optimum depth of truss girder

The optimum value for span to depth ratio depends on the magnitude of the live load that has to be carried. The span to depth ratio of a truss girder bridge producing the greatest economy of material is that which makes the weight of chord members nearly equal to the weight of web members of truss. It will be in the region of 10, being greater for road traffic than for rail traffic. IS: 1915-1961, also prescribes same value for highway and railway bridges. As per bridge rules published by Railway board, the depth should not be greater than three times width between centres of main girders. The spacing between main truss depends upon the railway or road way clearances required.

3.5.2 Design of compression chord members

Generally, the effective length for the buckling of compression chord member in the plane of truss is not same as that for buckling out-of plane of the truss i.e. the member is weak in one plane compared to the other. The ideal compression chord will be one that has a section with radii of gyration such that the slenderness value is same in both planes. In other words, the member is just likely to buckle in plane or out of plane. These members should be kept as short as possible and consideration is given to additional bracing, if economical. Depth of the member needs to be chosen so that the plate dimensions are reasonable. Trusses with spans up to 100 m often have open section compression chords. In such cases it is desirable to arrange for the vertical posts and struts to enter inside the top chord member, thereby providing a natural diaphragm and also achieving direct connection between member thus minimizing or avoiding the need for gussets. However, packing may be needed in this case. For trusses with spans greater than about 100 m, the chords will be usually the box shaped such that the ideal disposition of material to be made from both economic and maintenance view points. For shorter spans, rolled sections or rolled hollow sections may be used.

3.5.3 Design of tension chord members

Tension members should be as compact as possible, but depths have to be large enough to provide adequate space for bolts at the gusset positions and easily attach cross beam. The width out-of-plane of the truss should be the same as that of the verticals and diagonals so that simple lapping gussets can be provided without the need for packing. It should be possible to achieve a net section about 85% of the gross section by careful arrangement of the bolts in the splices.

3.5.4. Design of vertical and diagonal member

Diagonal and vertical members are often rolled sections, particularly for the lightly loaded members, it is desirable to keep all diagonals at the same angle, even if the chords are not parallel. This arrangement prevents the truss looking over complex when viewed from an angle. In practice, however, this is usually overruled by the economies of the deck structure where a constant panel length is to be preferred.

3.5.6 Lateral bracing for truss bridges

Lateral bracing in truss bridges is provided for transmitting the longitudinal live loads and lateral loads to the bearings and also to prevent the compression chords from buckling. This is done by providing stringer bracing, bracing girders and chord lateral bracing.

3.6 DESIGN OF PRATT TRUSS

Design of Bottom Chord member							
Grade of steel	σy			=	350	N/mm^2	
Member no.				=	L0-L1		
Load combination				=	DL+LL	Combination	
(+ve Values = Compression in the Member)					DL+LL		
(-ve Values = Tension in the Member)							
Member forces -ref Table-1							
Axial Force				=	-2822.89	kN	TENSION
Moment-Y				=	-3.67	kNm	
Moment-Z				=	159.25	kNm	
Gross Area				=	35200	mm^2	
Area for Tension member				=	28160	mm^2	
Length of member				=	5.83	m	
Slenderness ratio	l/rx			=	22.16		
Slenderness ratio	l/ry			=	22.10		
σa all (T)				=	207.00	N/mm^2	
σa all (C)				=	197.0	N/mm^2	
σm all				=	219.0	N/mm^2	
Increase in permissible stresses σ_a all (T)			0.00 %	=	207.0	N/mm^2	
Increase in permissible stresses σ_m all			0.00 %	=	219.0	N/mm^2	
Increase in permissible stresses σ_a all (C)			0.00 %	=	197.1	N/mm^2	

Stresses in members for combinations:-							
σa =	P	=	-2822.89*1000		=	-100.24	N/mm^2
	A		28160				
σmy =	My	=	-3.6652*1000000		=	-0.53	N/mm^2
	Zy		6.95E+06				
σmz =	Mz	=	159.25*1000000		=	20.37	N/mm^2
	Zz		7.82E+06				
Stress ratio:-							
	σa	+	σmy	+	σmz		
	σa all		σm all		σm all		
	-100.24	+	-0.53	+	20.37		
	207.00		219.00		219.00		
	-0.48	+	0.00	+	0.09		
	0.58	<	1	SAFE			

Design of Bottom Chord member								
Grade of steel		σy			=	350	N/mm^2	
Member no.					=	L0-L1		
Load combination					=	EQ	Combination	
(+ve Values = Compression in the Member)						EQ		
(-ve Values = Tension in the Member)								
Member forces -ref Table-1								
Axial Force					=	-3355.42	kN	TENSION
Moment-Y					=	-19.35	kNm	
Moment-Z					=	169.5694	kNm	
Gross Area					=	35200	mm^2	
Area for Tension member					=	28160	mm^2	
Length of member					=	5.83	m	
Slenderness ratio		l/rx			=	22.16		
Slenderness ratio		l/ry			=	22.10		
σa all (T)					=	207.00	N/mm^2	
σa all (C)					=	197.0	N/mm^2	
σm all					=	219.0	N/mm^2	
Increase in permissible stresses σ_a all (T)				16.67%	=	241.6	N/mm^2	
Increase in permissible stresses σ_m all				16.67%	=	255.6	N/mm^2	
Increase in permissible stresses σ_a all (C)				16.67%	=	229.9	N/mm^2	

Stresses in members for combinations:-							
σa =	P	=	-3355.422*1000		=	-119.16	N/mm^2
	A		28160				
σmy =	My	=	-19.3452*1000000		=	-2.78	N/mm^2
	Zy		6.95E+06				
σmz =	Mz	=	169.5694*1000000		=	21.69	N/mm^2
	Zz		7.82E+06				
Stress ratio:-							
	σa	+	σmy	+	σmz		
	σa all		σm all		σm all		
	-119.16	+	-2.78	+	21.69		
	241.60		255.60		255.60		
	-0.49	+	-0.01	+	0.08		
	0.59	<	1	SAFE			

Design of top Chord member							
Grade of steel	σy			=	350	N/mm^2	
Member no.				=	U4-U5		
Load combination				=	EQ	Combination	
(+ve Values = Compression in the Member)					EQ		
(-ve Values = Tension in the Member)							
Axial Force				=	6573.43	kN	COMPRESSION
Moment-Y				=	88.95	kNm	
Moment-Z				=	-127.851	kNm	
Gross Area				=	50400	mm^2	
Area for Tension member				=	40320	mm^2	
Length of member				=	5.83	M	
Slenderness ratio	l/rx			=	23.02		
Slenderness ratio	l/ry			=	23.57		
σa all (T)				=	207.00	N/mm^2	
σa all (C)				=	196.1	N/mm^2	
σm all				=	219.0	N/mm^2	
Increase in permissible stresses σ_a all (T)			16.67%	=	241.6	N/mm^2	
Increase in permissible stresses σ_m all			16.67%	=	255.6	N/mm^2	
Increase in permissible stresses σ_a all (C)			16.67%	=	228.8	N/mm^2	

Stresses in members for combinations:-							
σa =	P	=	6573.4284*1000		=	130.43	N/mm^2
	A		50400				
σmy =	My	=	88.9546*1000000		=	12.80	N/mm^2
	Zy		6.95E+06				
σmz =	Mz	=	-127.8508*1000000		=	-16.36	N/mm^2
	Zz		7.82E+06				
Stress ratio:-							
	σa	+	σmy	+	σmz		
	σa all		σm all		σm all		
	130.43	+	12.80	+	-16.36		
	228.80		255.60		255.60		
	0.57	+	0.05	+	-0.06		
	0.68	<	1	SAFE			

Design of end reacker member							
Grade of steel	σy			=	350	N/mm^2	
Member no.				=	L0-U1		
Load combination				=	DL+LL	Combination	
(+ve Values = Compression in the Member)					DL+LL		
(-ve Values = Tension in the Member)							
Member forces -ref Table-1							
Axial Force				=	4522.70	kN	COMPRESSION
Moment-Y				=	0.83	kNm	
Moment-Z				=	-100.999	kNm	
Gross Area				=	43200	mm^2	
Area for Tension member				=	34560	mm^2	
Length of member				=	10.89	m	
Slenderness ratio	l/rx			=	41.83		
Slenderness ratio	l/ry			=	44.79		
σa all (T)				=	207.00	N/mm^2	
σa all (C)				=	178.7	N/mm^2	
σm all				=	219.0	N/mm^2	
Increase in permissible stresses σ_a all (T)			0.00%	=	207.0	N/mm^2	
Increase in permissible stresses σ_m all			0.00%	=	219.0	N/mm^2	
Increase in permissible stresses σ_a all (C)			0.00%	=	178.7	N/mm^2	

Stresses in members for combinations:-							
σa =	P	=	4522.7*1000		=	104.69	N/mm^2
	A		43200				
σmy =	My	=	0.833*1000000		=	0.12	N/mm^2
	Zy		6.95E+06				
σmz =	Mz	=	-100.9988*1000000		=	-12.92	N/mm^2
	Zz		7.82E+06				
Stress ratio:-							
	σa	+	σmy	+	σmz		
	σa all		σm all		σm all		
	104.69	+	0.12	+	-12.92		
	178.70		219.00		219.00		
	0.59	+	0.00	+	-0.06		
	0.65	<	1	SAFE			

Design of diagonal member							
Grade of steel	σy			=	350	N/mm^2	
Member no.				=	L1-L2		
Load combination				=	DL+LL	Combination	
(+ve Values = Compression in the Member)					DL+LL		
(-ve Values = Tension in the Member)							
Member forces -ref Table-1							
Axial Force				=	-3410.57	kN	TENSION
Moment-Y				=	1.71	kNm	
Moment-Z				=	-189.022	kNm	
Gross Area				=	29600	mm^2	
Area for Tension member				=	23680	mm^2	
Length of member				=	10.89	m	
Slenderness ratio	l/rx			=	42.49		
Slenderness ratio	l/ry			=	95.15		
σa all (T)				=	207.00	N/mm^2	
σa all (C)				=	99.7	N/mm^2	
σm all				=	219.0	N/mm^2	
Increase in permissible stresses σ_a all (T)			0.00%	=	207.0	N/mm^2	
Increase in permissible stresses σ_m all			0.00%	=	219.0	N/mm^2	
Increase in permissible stresses σ_a all (C)			0.00%	=	99.8	N/mm^2	

Stresses in members for combinations:-							
σa =	P	=	-3410.5666*1000		=	-144.03	N/mm^2
	A		23680				
σmy =	My	=	1.7052*1000000		=	0.25	N/mm^2
	Zy		6.95E+06				
σmz =	Mz	=	-189.0224*1000000		=	-24.18	N/mm^2
	Zz		7.82E+06				
Stress ratio:-							
	σa	+	σmy	+	σmz		
	σa all		σm all		σm all		
	-144.03	+	0.25	+	-24.18		
	207.00		219.00		219.00		
	-0.70	+	0.00	+	-0.11		
	0.81	<	1	SAFE			

Design of vertical member							
Grade of steel	σy			=	350	N/mm^2	
Member no.				=	L2-U2		
Load combination				=	DL+LL	Combination	
(+ve Values = Compression in the Member)					DL+LL		
(-ve Values = Tension in the Member)							
Member forces -ref Table-1							
Axial Force				=	2069.13	kN	COMPRESSION
Moment-Y				=	5.15	kNm	
Moment-Z				=	-60.1426	kNm	
Gross Area				=	26720	mm^2	
Area for Tension member				=	21376	mm^2	
Length of member				=	9.2	m	
Slenderness ratio	l/rx			=	35.00		
Slenderness ratio	l/ry			=	77.14		
σa all (T)				=	207.00	N/mm^2	
σa all (C)				=	128.6	N/mm^2	
σm all				=	219.0	N/mm^2	
Increase in permissible stresses σ_a all (T)			0.00%	=	207.0	N/mm^2	
Increase in permissible stresses σ_m all			0.00%	=	219.0	N/mm^2	
Increase in permissible stresses σ_a all (C)			0.00%	=	128.7	N/mm^2	

Stresses in members for combinations:-							
σa =	P	=	2069.1328*1000		=	77.44	N/mm^2
	A		26720				
σmy =	My	=	5.145*1000000		=	0.74	N/mm^2
	Zy		6.95E+06				
σmz =	Mz	=	-60.1426*1000000		=	-7.69	N/mm^2
	Zz		7.82E+06				
Stress ratio:-							
	σa	+	σmy	+	σmz		
	σa all		σm all		σm all		
	77.44	+	0.74	+	-7.69		
	128.70		219.00		219.00		
	0.60	+	0.00	+	-0.04		
	0.64	<	1	SAFE			

Design of top cross girder member							
Grade of steel	σy			=	350	N/mm^2	
Member no.				=	U2-U*		
Load combination				=	DL+LL	Combination	
(+ve Values = Compression in the Member)					DL+LL		
(-ve Values = Tension in the Member)							
Member forces -ref Table-1							
Axial Force				=	-656.95	kN	TENSION
Moment-Y				=	0.01	kNm	
Moment-Z				=	0.38318	kNm	
Gross Area				=	21760	mm^2	
Area for Tension member				=	17408	mm^2	
Length of member				=	10.3	m	
Slenderness ratio	l/rx			=	47.57		
Slenderness ratio	l/ry			=	83.45		
σa all (T)				=	207.00	N/mm^2	
σa all (C)				=	118.1	N/mm^2	
σm all				=	219.0	N/mm^2	
Increase in permissible stresses σ_a all (T)			0.00%	=	207.0	N/mm^2	
Increase in permissible stresses σ_m all			0.00%	=	219.0	N/mm^2	
Increase in permissible stresses σ_a all (C)			0.00%	=	118.1	N/mm^2	

Stresses in members for combinations:-							
σa =	P	=	-656.9528*1000		=	-37.74	N/mm^2
	A		17408				
σmy =	My	=	0.0098*1000000		=	0.00	N/mm^2
	Zy		6.95E+06				
σmz =	Mz	=	0.38318*1000000		=	0.05	N/mm^2
	Zz		7.82E+06				
Stress ratio:-							
	σa	+	σmy	+	σmz		
	σa all		σm all		σm all		
	-37.74	+	0.00	+	0.05		
	207.00		219.00		219.00		
	-0.18	+	0.00	+	0.00		
	0.18	<	1	SAFE			

Design of Top lateral bracing							
Grade of steel	σy			=	350	N/mm^2	
Member no.				=	U4-U5		
				=	EQ	Combination	
(+ve Values = Compression in the Member)							
(-ve Values = Tension in the Member)							
Member forces							
Axial Force				=	584.82	kN	COMPRESSION
Moment-Y				=	0.00	kNm	
Moment-Z				=	0.00	kNm	
Additional compressive Forces (As per clause 6.17.2)				=	6203.27	kN	
2.5% x (U1-U2)				=	155.0818	kN	
Total Axial Force				=	739.91	kN	
Gross Area				=	11688	mm^2	
Area for Compression member				=	11688	mm^2	
Length of member				=	7.78	m	
Slenderness ratio	l/rx			=	45.60		
Slenderness ratio	l/ry			=	81.60		
σa all (T)				=	154.00	N/mm^2	
σa all (C)				=	121.0	N/mm^2	
σm all				=	218.75	N/mm^2	
Increase in permissible stresses σ_a all (T)			16.67%	=	179.7	N/mm^2	
Increase in permissible stresses σ_m all			16.67%	=	255.3	N/mm^2	
Increase in permissible stresses σ_a all (C)			16.67%	=	141.2	N/mm^2	

Stresses in members for combinations:-							
σa =	P	=	739.906615*1000		=	63.30	N/mm^2
	A		11688				
σmy =	My	=	0*1000000		=	0.00	N/mm^2
	Zy		5.87E+04				
σmz =	Mz	=	0*1000000		=	0.00	N/mm^2
	Zz		5.87E+04				
Stress ratio:-							
	σa	+	σmy	+	σmz		
	σa all		σm all		σm all		
	63.30	+	0.00	+	0.00		
	141.20		255.30		255.30		
	0.45	+	0.00	+	0.00		
	0.45	<	1	SAFE			

Design of Bottom lateral bracing							
Grade of steel	σy			=	350	N/mm^2	
Member no.				=	L1-L2		
				=	EQ	Combination	
(+ve Values = Compression in the Member)							
(-ve Values = Tension in the Member)							
Member forces							
Axial Force				=	-1158.58	kN	TENSION
Moment-Y				=	0.00	kNm	
Moment-Z				=	0.00	kNm	
Gross Area				=	10488	mm^2	
Area for Tension member				=	8390	mm^2	
Length of member				=	7.78	m	
Slenderness ratio	l/rx			=	61.49		
Slenderness ratio	l/ry			=	108.39		
σa all (T)				=	207.00	N/mm^2	
σa all (C)				=	82.2	N/mm^2	
σm all				=	218.75	N/mm^2	
Increase in permissible stresses σ_a all (T)			16.67%	=	241.6	N/mm^2	
Increase in permissible stresses σ_m all			16.67%	=	255.3	N/mm^2	
Increase in permissible stresses σ_a all (C)			16.67%	=	96.0	N/mm^2	

Stresses in members for combinations:-							
σa =	P	=	-1158.5756*1000		=	-138.08	N/mm^2
	A		8390.4				
σmy =	My	=	0*1000000		=	0.00	N/mm^2
	Zy		1.05E+05				
σmz =	Mz	=	0*1000000		=	0.00	N/mm^2
	Zz		1.05E+05				
Stress ratio:-							
	σa	+	σmy	+	σmz		
	σa all		σm all		σm all		
	-138.08	+	0.00	+	0.00		
	241.60		255.30		255.30		
	-0.57	+	0.00	+	0.00		
	0.57	<	1	SAFE			

Design of Portal bracing							
Grade of steel		Σy			=	350	N/mm^2
Member no.					=	L0-U1	
				Load combination		EQ	Combination
Axial Force					=	221.3036	kN
Gross Area					=	2259	mm^2
Area for Tension member					=	1807	mm^2
Length of member		Lx			=	2.2	m
Length of member		Ly			=	2.2	m
Slenderness ratio		l/rx			=	72.61	
Slenderness ratio		l/ry			=	72.61	
σa all (T)					=	207.00	N/mm^2
σa all (C)					=	136.7	N/mm^2
Increase in permissible stresses σ_a all (T)				16.67%	=	241.6	N/mm^2
Increase in permissible stresses σ_a all (C)				16.67%	=	159.5	N/mm^2
Stresses in members for combinations:-							
σa =	P	=	221.303		=	97.97	N/mm^2
	A		2259				
Stress ratio:-							
	σa	=	97.9652				
	σa all		159.5				
			0.61<	1	SAFE		

3.7 FATIGUE ANALYSIS FOR PRATT TRUSS

Fatigue Analysis	**Allowable Fatigue Forces**				**Actual Fatigue Forces**			**Check**
MEMBER	**f1**	**f2**	**fmin / fmax**	**Perm. Stress**	**Fatigue Force**	**Effective Area**	**Actual Stress**	**Safe/Unsafe**
BOTTOM CHORD								
L0-L1	-1593.83	-2885.00	0.55	185.01	-2885.00	23680.00	-121.833	SAFE
L1-L2	-1369.95	-2321.16	0.59	193.89	-2321.16	23680.00	-98.022	SAFE
L2-L3	-2458.66	-3670.90	0.67	218.10	-3670.90	23680.00	-155.021	SAFE
L3-L4	-3212.60	-4645.31	0.69	224.95	-4645.31	28800.00	-161.295	SAFE
L4-L5	-3740.50	-5297.67	0.71	230.21	-5297.67	34400.00	-154.002	SAFE
TOP CHORD								
U1-U2	2711.60	4025.56	0.67	293.32	4025.56	23680.00	169.9983	SAFE
U2-U3	3292.10	5293.50	0.62	293.32	5293.50	23680.00	223.5431	SAFE
U3-U4	4080.18	6015.86	0.68	293.32	6015.86	28800.00	208.884	SAFE
U4-U5	4254.08	6329.87	0.67	293.32	6329.87	32000.00	197.8084	SAFE
DIAGONALS								
L0-U1	3072.35	4615.00	0.67	293.32	4592.71	26880.00	170.8599	SAFE
U1-L2	-1076.58	-3480.17	0.31	141.66	-2302.22	23680.00	-97.2221	SAFE
L2-U3	-1625.77	-2519.00	0.65	210.45	-2582.03	18990.00	-135.968	SAFE
U3-L4	-1001.64	-1609.00	0.62	203.27	-1716.40	12600.00	-136.222	SAFE
L4-U5	-322.00	-699.14	0.46	165.69	-937.18	12600.00	-74.3797	SAFE

VERTICALS								
U1-L1	-453.52	-899.25	0.50	173.68	-890.64	13209.6	-67.4236	SAFE
U2-L2	2101.90	2111.36	1.00	293.32	2031.63	21376	95.04277	SAFE
U3-L3	1431.49	1455.35	0.98	293.32	1330.92	21376	62.26241	SAFE
U4-L4	656.47	656.67	1.00	293.32	454.48	13209.6	34.40504	SAFE
U5-L5	73.49	74.12	0.99	293.32	68.25	13190.4	5.174104	SAFE
TOP CROSS MEMBER								
U1-U1'	-187.00	-288.42	0.65	211.38	-313.77	17408	-18.0242	SAFE
REST	-473.00	-670.36	0.71	227.59	-682.08	12492.88	-54.5977	SAFE
BOTTOM BRACINGS								
L0-L1	-237.00	-652.38	0.36	149.60	-401.07	8390.4	-47.8009	SAFE
L1-L2	-197.00	-532.76	0.37	150.55	-390.61	8390.4	-46.5544	SAFE
L2-L3	-122.00	-367.26	0.33	145.02	-298.85	8390.4	-35.6178	SAFE
L3-L4	-42.00	-210.44	0.20	126.51	-191.58	8390.4	-22.8335	SAFE
L4-L5	-14.00	-59.88	0.23	131.19	-135.98	8390.4	-16.2066	SAFE
TOP BRACINGS								
U1-U2	411.98	404.54	0.98	293.32	426.81	9350.4	45.64621	SAFE
U2-U3	413.45	421.86	0.98	293.32	426.86	9350.4	45.65107	SAFE
U3-U4	542.57	543.44	1.00	293.32	555.04	9350.4	59.36031	SAFE
U4-U5	543.35	543.44	1.00	293.32	552.98	9350.4	59.13944	SAFE
BRACING	11.23	-48.09	-0.23	127.84	-110.76	2260	-49.01	SAFE

3.8 CROSS SECTIONAL DETAILS OF MEMBERS OF PRATT TRUSS

SECTION PROVIDED		**total area(cm^2)**	**Iyy(cm^4)**	**Izz(cm^4)**	**ryy(cm)**	**rzz(cm)**	**Net area(cm^2)**
Bottom chord							
L0-L1	BUILT UP	352	328541	326773	26	26	281.6
L1-L2	BUILT UP	352	328541	326773	26	26	281.6
L2-L3	BUILT UP	352	328541	326773	26	26	281.6
L3-L4	BUILT UP	384	375733	341173	27	26	307.2
L4-L5	BUILT UP	464	431286	359173	27	25	371.2
Top chord							
U1-U2	BUILT UP	379.2	264841	303701	24	26	303.36
U2-U3	BUILT UP	379.2	264841	303701	24	26	303.36
U3-U4	BUILT UP	424	303421	318101	25	25	339.2
U4-U5	BUILT UP	504	303421	318101	25	25	403.2
Diagonal							
L0-U1	BUILT UP	432	264841	303701	24	26	345.6
U1-L2	I SECTION	296	36883	184951	11	26	236.8
L2-U3	I SECTION	236.8	36883	184951	11	26	189.44
U3-L4	I SECTION	157.6	21341	152175	10	26	126.08
L4-U5	I SECTION	157.6	21341	152175	10	26	126.08
Verticals							
U1-L1	I SECTION	165.12	12808	101338	9	25	132.096
U2-L2	I SECTION	267.2	36872	179098	12	26	213.76
U3-L3	I SECTION	267.2	36872	179098	12	26	213.76
U4-L4	I SECTION	165.12	12808	101338	9	25	132.096
U5-L5	I SECTION	164.88	12808	101338	9	25	131.904

TOP BRACING							
	I SECTION	116.88	12805	41002	10	17	93.504
Bottom bracing							
	I SECTION	104.88	5404	16789	7	13	83.904
Top cross member							
REST	I SECTION	217.6	41682	128245	12	22	174.08
U1-U1'	I SECTION	156.16	17073	85240	10	22	124.928

3.9 **DESIGN OF ARCH TRUSS**

Design of Bottom Chord member							
Grade of steel	σy			=	350	N/mm^2	
Member no.				=	L0-L1		
Load combination				=	EQ	Combination	
(+ve Values = Compression in the Member)					EQ		
(-ve Values = Tension in the Member)							
Axial Force				=	-5902.89	kN	TENSION
Moment-Y				=	-14.48	kNm	
Moment-Z				=	449.085	kNm	
Gross Area				=	47200	mm^2	
Area for Tension member				=	37760	mm^2	
Length of member				=	5.83	m	
Slenderness ratio	l/rx			=	22.16		
Slenderness ratio	l/ry			=	22.10		
σa all (T)				=	207.00	N/mm^2	
σa all (C)				=	197.0	N/mm^2	

σm all				=	219.0	N/mm^2	
Increase in permissible stresses σ_a all (T)			16.67%	=	241.6	N/mm^2	
Increase in permissible stresses σ_m all			16.67%	=	255.6	N/mm^2	
Increase in permissible stresses σ_a all (C)			16.67%	=	229.9	N/mm^2	

Stresses in members for combinations:-							
σa =	P	=	-5902.8928*1000		=	-156.33	N/mm^2
	A		37760				
σmy =	My	=	-14.4844*1000000		=	-2.08	N/mm^2
	Zy		6.95E+06				
σmz =	Mz	=	449.085*1000000		=	57.45	N/mm^2
	Zz		7.82E+06				
Stress ratio:-							
	σa	+	σmy	+	σmz		
	σa all		σm all		σm all		
	-156.33	+	-2.08	+	57.45		
	241.60		255.60		255.60		
	-0.65	+	-0.01	+	0.22		
	0.88	<	1	SAFE			

Design of top Chord member						
Grade of steel	Σy		=	350	N/mm^2	
Member no.			=	U1-U2		
Load combination			=	EQ	Combination	
(+ve Values = Compression in the Member)				EQ		
(-ve Values = Tension in the Member)						
Axial Force			=	6791.50	kN	COMPRESSION
Moment-Y			=	13.77	kNm	
Moment-Z			=	-161.249	kNm	
Gross Area			=	39200	mm^2	
Area for Tension member			=	31360	mm^2	
Length of member			=	6.2	m	
Slenderness ratio	l/rx		=	23.81		
Slenderness ratio	l/ry		=	25.50		
σa all (T)			=	207.00	N/mm^2	
σa all (C)			=	194.7	N/mm^2	
σm all			=	219.0	N/mm^2	
Increase in permissible stresses σ_a all (T)		16.67%	=	241.6	N/mm^2	
Increase in permissible stresses σ_m all		16.67%	=	255.6	N/mm^2	
Increase in permissible stresses σ_a all (C)		16.67%	=	227.2	N/mm^2	

Stresses in members for combinations:-							
σa =	P	=	6791.498*1000		=	173.25	N/mm^2
	A		39200				
σmy =	My	=	13.769*1000000		=	1.98	N/mm^2
	Zy		6.95E+06				
σmz =	Mz	=	-161.2492*1000000		=	-20.63	N/mm^2
	Zz		7.82E+06				
Stress ratio:-							
	σa	+	σmy	+	σmz		
	σa all		σm all		σm all		
	173.25	+	1.98	+	-20.63		
	227.20		255.60		255.60		
	0.76	+	0.01	+	-0.08		
	0.85	<	1	SAFE			

Design of end reacker member						
Grade of steel	σy		=	350	N/mm^2	
Member no.			=	L0-U1		
Load combination			=	EQ	Combination	
(+ve Values = Compression in the Member)				EQ		
(-ve Values = Tension in the Member)						
Axial Force			=	6580.70	kN	COMPRESSION
Moment-Y			=	0.83	kNm	
Moment-Z			=	-74.5584	kNm	
Gross Area			=	47200	mm^2	
Area for Tension member			=	37760	mm^2	
Length of member			=	7.21	M	
Slenderness ratio	l/rx		=	27.69		
Slenderness ratio	l/ry		=	29.65		
σa all (T)			=	207.00	N/mm^2	
σa all (C)			=	191.9	N/mm^2	
σm all			=	219.0	N/mm^2	
Increase in permissible stresses σ_a all (T)		16.67%	=	241.6	N/mm^2	
Increase in permissible stresses σ_m all		16.67%	=	255.6	N/mm^2	
Increase in permissible stresses σ_a all (C)		16.67%	=	223.9	N/mm^2	

Stresses in members for combinations:-							
σa =	P	=	6580.7*1000		=	139.42	N/mm^2
	A		47200				
σmy =	My	=	0.833*1000000		=	0.12	N/mm^2
	Zy		6.95E+06				
σmz =	Mz	=	-74.5584*1000000		=	-9.54	N/mm^2
	Zz		7.82E+06				
Stress ratio:-							
	σa	+	σmy	+	σmz		
	σa all		σm all		σm all		
	139.42	+	0.12	+	-9.54		
	223.90		255.60		255.60		
	0.62	+	0.00	+	-0.04		
	0.66	<	1	SAFE			

Design of diagonal member						
Grade of steel	σy		=	350	N/mm^2	
Member no.			=	L1-L2		
Load combination			=	EQ	Combination	
(+ve Values = Compression in the Member)				EQ		
(-ve Values = Tension in the Member)						
Member forces -ref Table-1						
Axial Force			=	-1544.97	kN	TENSION
Moment-Y			=	-1.62	kNm	
Moment-Z			=	-17.64	kNm	
Gross Area			=	13760	mm^2	
Area for Tension member			=	11008	mm^2	
Length of member			=	7.21	m	
Slenderness ratio	l/rx		=	28.13		
Slenderness ratio	l/ry		=	63.00		
σa all (T)			=	207.00	N/mm^2	
σa all (C)			=	153.8	N/mm^2	
σm all			=	219.0	N/mm^2	
Increase in permissible stresses σ_a all (T)		16.67%	=	241.6	N/mm^2	
Increase in permissible stresses σ_m all		16.67%	=	255.6	N/mm^2	
Increase in permissible stresses σ_a all (C)		16.67%	=	179.5	N/mm^2	

Stresses in members for combinations:-							
σa =	P	=	-1544.97*1000		=	-140.35	N/mm^2
	A		11008				
σmy =	My	=	-1.617*1000000		=	-0.23	N/mm^2
	Zy		6.95E+06				
σmz =	Mz	=	-17.64*1000000		=	-2.26	N/mm^2
	Zz		7.82E+06				
Stress ratio:-							
	σa	+	σmy	+	σmz		
	σa all		σm all		σm all		
	-140.35	+	-0.23	+	-2.26		
	241.60		255.60		255.60		
	-0.58	+	0.00	+	-0.01		
	0.59	<	1	SAFE			

Design of vertical member						
Grade of steel	σy		=	350	N/mm^2	
Member no.			=	L5-U5		
Load combination			=	EQ	Combination	
(+ve Values = Compression in the Member)				EQ		
(-ve Values = Tension in the Member)						
Member forces -ref Table-1						
Axial Force			=	1278.02	kN	COMPRESSION
Moment-Y			=	-61.70	kNm	
Moment-Z			=	-1.4308	kNm	
Gross Area			=	16488	mm^2	
Area for Tension member			=	13190	mm^2	
Length of member			=	9.2	m	
Slenderness ratio	l/rx		=	37.11		
Slenderness ratio	l/ry		=	104.38		
σa all (T)			=	207.00	N/mm^2	
σa all (C)			=	87.0	N/mm^2	
σm all			=	219.0	N/mm^2	
Increase in permissible stresses σ_a all (T)			=	241.6	N/mm^2	
Increase in permissible stresses σ_m all			=	255.6	N/mm^2	
Increase in permissible stresses σ_a all (C)			=	101.5	N/mm^2	

Stresses in members for combinations:-							
σa =	P	=	1278.018*1000		=	77.51	N/mm^2
	A		16488				
σmy =	My	=	-61.7008*1000000		=	-8.88	N/mm^2
	Zy		6.95E+06				
σmz =	Mz	=	-1.4308*1000000		=	-0.18	N/mm^2
	Zz		7.82E+06				
Stress ratio:-							
	σa	+	σmy	+	σmz		
	σa all		σm all		σm all		
	77.51	+	-8.88	+	-0.18		
	101.50		255.60		255.60		
	0.76	+	-0.03	+	0.00		
	0.80	<	1	SAFE			

Design of top cross girder member						
Grade of steel	σy		=	350	N/mm^2	
Member no.			=	U2-U*		
Load combination			=	EQ	Combination	
(+ve Values = Compression in the Member)				EQ		
(-ve Values = Tension in the Member)						
Axial Force			=	-742.19	kN	TENSION
Moment-Y			=	0.01	kNm	
Moment-Z			=	27.44	kNm	
Gross Area			=	15488	mm^2	
Area for Tension member			=	12390	mm^2	
Length of member			=	10.3	m	
Slenderness ratio	l/rx		=	47.57		
Slenderness ratio	l/ry		=	83.45		
σa all (T)			=	207.00	N/mm^2	
σa all (C)			=	118.1	N/mm^2	
σm all			=	219.0	N/mm^2	
Increase in permissible stresses σ_a all (T)		16.67%	=	241.6	N/mm^2	
Increase in permissible stresses σ_m all		16.67%	=	255.6	N/mm^2	
Increase in permissible stresses σ_a all (C)		16.67%	=	137.8	N/mm^2	

Stresses in members for combinations:-							
σa =	P	=	-742.1932*1000		=	-59.90	N/mm^2
	A		12390.4				
σmy =	My	=	0.0098*1000000		=	0.00	N/mm^2
	Zy		6.95E+06				
σmz =	Mz	=	27.44*1000000		=	3.51	N/mm^2
	Zz		7.82E+06				
Stress ratio:-							
	σa	+	σmy	+	σmz		
	σa all		σm all		σm all		
	-59.90	+	0.00	+	3.51		
	241.60		255.60		255.60		
	-0.25	+	0.00	+	0.01		
	0.26	<	1	SAFE			

Design of Top lateral bracing						
Grade of steel	σy		=	350	N/mm^2	
Member no.			=	U4-U5		
			=	EQ	Combination	
(+ve Values = Compression in the Member)						
(-ve Values = Tension in the Member)						
Axial Force			=	646.81	kN	COMPRESSION
Moment-Y			=	0.00	kNm	
Moment-Z			=	0.00	kNm	
Additional compressive Forces (As per clause 6.17.2)			=	6204.84	kN	
2.5% x (U1-U2)			=	155.121	kN	
Total Axial Force			=	801.93	kN	
Gross Area			=	11688	mm^2	
Area for Compression member			=	11688	mm^2	
Length of member			=	7.78	m	
Slenderness ratio	l/rx		=	45.60		
Slenderness ratio	l/ry		=	81.60		
σa all (T)			=	154.00	N/mm^2	
σa all (C)			=	121.0	N/mm^2	
σm all			=	218.75	N/mm^2	
Increase in permissible stresses σ_a all (T)		16.67%	=	179.7	N/mm^2	
Increase in permissible stresses σ_m all		16.67%	=	255.3	N/mm^2	
Increase in permissible stresses σ_a all (C)		16.67%	=	141.2	N/mm^2	

Stresses in members for combinations:-							
σa =	P	=	801.930815*1000		=	68.61	N/mm^2
	A		11688				
σmy =	My	=	0*1000000		=	0.00	N/mm^2
	Zy		5.87E+04				
σmz =	Mz	=	0*1000000		=	0.00	N/mm^2
	Zz		5.87E+04				
Stress ratio:-							
	σa	+	σmy	+	σmz		
	σa all		σm all		σm all		
	68.61	+	0.00	+	0.00		
	141.20		255.30		255.30		
	0.49	+	0.00	+	0.00		
	0.49	<	1	SAFE			

Design of Bottom lateral bracing-						
Grade of steel	σy		=	350	N/mm^2	
Member no.			=	L1-L2		
			=	EQ	Combination	
(+ve Values = Compression in the Member)						
(-ve Values = Tension in the Member)						
Member forces						
Axial Force			=	-1319.97	kN	TENSION
Moment-Y			=	0.00	kNm	
Moment-Z			=	0.00	kNm	
Gross Area			=	10488	mm^2	
Area for Tension member			=	8390	mm^2	
Length of member			=	7.78	m	
Slenderness ratio	l/rx		=	61.49		
Slenderness ratio	l/ry		=	108.39		
σa all (T)			=	207.00	N/mm^2	
σa all (C)			=	82.2	N/mm^2	
σm all			=	218.75	N/mm^2	
Increase in permissible stresses σ_a all (T)		16.67%	=	241.6	N/mm^2	
Increase in permissible stresses σ_m all		16.67%	=	255.3	N/mm^2	
Increase in permissible stresses σ_a all (C)		16.67%	=	96.0	N/mm^2	

Stresses in members for combinations:-							
σa =	P	=	-1319.9718*1000		=	-157.32	N/mm^2
	A		8390.4				
σmy =	My	=	0*1000000		=	0.00	N/mm^2
	Zy		1.05E+05				
σmz =	Mz	=	0*1000000		=	0.00	N/mm^2
	Zz		1.05E+05				
Stress ratio:-							
	σa	+	σmy	+	σmz		
	σa all		σm all		σm all		
	-157.32	+	0.00	+	0.00		
	241.60		255.30		255.30		
	-0.65	+	0.00	+	0.00		
	0.65	<	1	SAFE			

Design of portal bracing							
Grade of steel		σy			=	350	N/mm^2
Member no.					=	L0-U1	
				Load combination		DL+LL	Combination
Axial Force (C)					=	47.1282	kN
Gross Area					=	2259	mm^2
Area for Tension member					=	1807	mm^2
Length of member		lx			=	2.2	m
Length of member		ly			=	2.2	m
Slenderness ratio		l/rx			=	72.61	
Slenderness ratio		l/ry			=	72.61	
σa all (T)					=	207.00	N/mm^2
σa all (C)					=	136.7	N/mm^2
Increase in permissible stresses σ$_a$ all (T)				0.00%	=	207.0	N/mm^2
Increase in permissible stresses σ$_a$ all (C)				0.00%	=	136.7	N/mm^2
Stresses in members for combinations:-							
σa =	P	=	47.1282		=	20.86	N/mm^2
	A		2259				
Stress ratio:-							
	σa	=	20.86242	=	0.15		
	σa all		136.7				
			<	1	SAFE		

3.10 FATIGUE ANALYSIS FOR ARCH TRUSS

Fatigue Analysis	**Allowable Fatigue Forces**				**Actual Fatigue Forces**			**Check**
	f1	**f2**	**fmin / fmax**	**Perm. Stress**	**Fatigue Force**	**Effective Area**	**Actual Stress**	**Safe/Unsafe**
BOTTOM CHORD								
L0-L1	-3277	-5352	0.61	200	-5352	37760	-142	SAFE
L1-L2	-2968	-4667	0.64	207	-4667	34560	-135	SAFE
L2-L3	-3513	-5210	0.67	220	-5210	34560	-151	SAFE
L3-L4	-3805	-5475	0.69	226	-5475	38400	-143	SAFE
L4-L5	-3825	-5364	0.71	233	-5364	41600	-129	SAFE
TOP CHORD								
U1-U2	4399	6549	0.67	293	6549	31360	209	SAFE
U2-U3	4654	6856	0.68	293	6856	34560	198	SAFE
U3-U4	4358	6465	0.67	293	6465	38400	168	SAFE
U4-U5	4236	6331	0.67	293	6331	41600	152	SAFE
DIAGONALS								
L0-U1	4488	6365	0.71	293	6008	37760	159	SAFE
U1-L2	-932	-1520	0.61	200	-2158	11008	-196	SAFE
L2-U3	-591	-1033	0.57	190	-1547	18990	-81	SAFE
U3-L4	-78	-362	0.22	129	-793	9856	-80	SAFE
L4-U5	-141	-440	0.32	143	-756	9856	-77	SAFE

VERTICALS								
U1-L1	-398	-809	0.49	171	-824	10253	-80	SAFE
U2-L2	23	289	0.08	206	671	10349	65	SAFE
U3-L3	54	350	0.15	242	455	10349	44	SAFE
U4-L4	403	773	0.52	293	517	13210	39	SAFE
U5-L5	790	1225	0.64	293	796	13190	60	SAFE
TOP CROSS MEMBER								
REST	-500	-714	0.70	228	-709	11213	-63	SAFE
BOTTOM BRACINGS								
L0-L1	-340	-804	0.42	159	-504	8390	-60	SAFE
L1-L2	-233	-585	0.40	155	-427	8390	-51	SAFE
L2-L3	-119	-404	0.29	140	-296	8390	-35	SAFE
L3-L4	-27	-242	0.11	119	-177	8390	-21	SAFE
L4-L5	-49	-95	0.52	176	-171	8390	-20	SAFE
TOP BRACINGS								
U2-U3	3	422	0.01	180	138	9350	15	SAFE
U3-U4	410	543	0.75	293	579	9350	62	SAFE
U4-U5	408	569	0.72	293	575	9350	61	SAFE

3.11 **CROSS SECTIONAL DETAILS OF MEMBERS OF ARCH TRUSS**

SECTION DETAILS		total area (cm^2)	Iyy(cm^4)	Izz(cm^4)	ryy(cm)	rzz(cm)	Net area(cm^2)
Bottom chord							
L0-L1	BUILT UP	472	328541.3	326773.3	26.4	26.3	377.6
L1-L2	BUILT UP	432	328541.3	326773.3	26.4	26.3	345.6
L2-L3	BUILT UP	432	328541.3	326773.3	26.4	26.3	345.6
L3-L4	BUILT UP	480	375733.3	341173.3	26.9	25.6	384
L4-L5	BUILT UP	520	431286.5	359173.3	27.3	24.9	416
Top chord							
U1-U2	BUILT UP	392	264840.5	303701.3	24.3	26.0	313.6
U2-U3	BUILT UP	432	264840.5	303701.3	24.3	26.0	345.6
U3-U4	BUILT UP	480	303421.3	318101.3	24.7	25.3	384
U4-U5	BUILT UP	520	303421.3	318101.3	24.7	25.3	416
Diagonal							
L0-U1	BUILT UP	472	264840.5	303701.3	24.3	26.0	377.6
U1-L2	I SECTION	137.6	36883.1	184951.5	11.4	25.6	110.08
L2-U3	I SECTION	137.6	36883.1	184951.5	11.4	25.6	110.08
U3-L4	I SECTION	123.2	21341.4	152174.9	9.7	25.9	98.56
L4-U5	I SECTION	123.2	21341.4	152174.9	9.7	25.9	98.56

VERTICALS							
U1-L1	I SECTION	128.16	12808.3	101338.5	8.8	24.8	102.528
U2-L2	I SECTION	129.36	36872.1	179097.6	11.9	26.3	103.488
U3-L3	I SECTION	129.36	36872.1	179097.6	11.9	26.3	103.488
U4-L4	I SECTION	165.12	12808.3	101338.5	8.8	24.8	132.096
U5-L5	I SECTION	164.88	12808.3	101338.5	8.8	24.8	131.904
TOP BRACING							
	I SECTION	116.88	12805.4	41001.9	9.5	17.1	93.504
BOTTOM BRACING							
	I SECTION	104.88	5403.9	16789.0	7.2	12.7	83.904
Top cross member							
REST	I SECTION	154.88	41682.4	128244.8	12.3	21.7	123.904
U1-U1'	I SECTION	140.16	17073.4	85239.5	9.6	21.5	112.128

3.12 DESIGN OF STRINGER

3.12.1 Loading

Stringer is designed for its self weight, superimposed dead load from the rails, deck load as well as the railway live load. The coefficient of dynamic augmentation is estimated as per the Bridge rules that is based on Clause 2.4.1.1(a) which states the loaded length is taken as 1.5 x actual span. The loaded length for the estimation of the load is the span.

Span, L =5.830m (Simply supported)

Loaded length for the estimation of CDA = 1.5*span of stringer

Hence, Loaded length for CDA = 8.745 m and CDA =0.693

3.12.2 Assumed Section for Stringer:

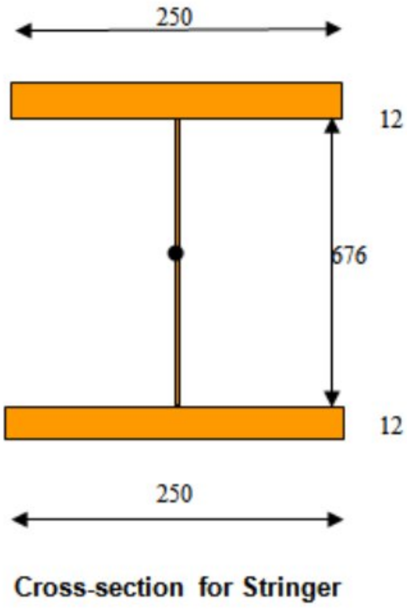
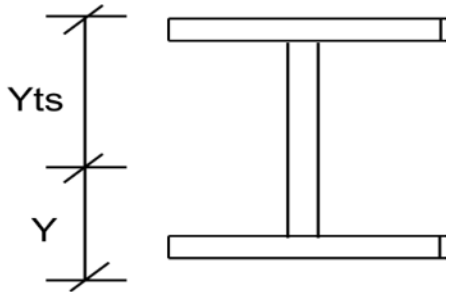


Fig3.7 Cross section of stringer

		Width (mm)	Thickness (mm)	Area (mm^2)	
Top plate		250	12	3000.00	
Vertical plate		676	12	8112.00	
Bottom plate		250	12	3000.00	
			Total Area =	14112	mm^2
			Weight =	0.65	t
Total Depth, D =		700.00	mm		

3.12.3 Composite sectional properties for stringer

(a) Steel Girder:-



C.G from bottom:

Description		**Area**	**Y**	**AY**	
		cm2	cm	cm3	
Top plate		30.00	69.40	2082.00	
Web plate		81.12	35.00	2839.20	
Bottom plate-I		30.00	0.60	18.00	
Bottom plate-II		0.00	0.00	0.00	
		141.12		4939.20	
		Y=	35.00	cm	
Ybs=	35.00	cm	Yts=	35.00	cm

Moment of inertia of steel girder

Description	**Area**	**d**	**Ad^2**	**I (self)**
	cm2	cm	cm4	cm4
Top plate	30.00	34.40	35500.80	3.60
Web plate	81.12	0.00	0.00	30891.58
Bottom pl-I	30.00	34.40	35500.80	3.60
Bottom pl-II	0.00	34.40	0.00	0.00
	141.12		71001.60	30898.78
Ina = Iself + Ad^2	=	101900.38	cm^4	
Zbs = Ina/Ybs	=	2911.43936	cm^3	
Zts = Ina/Yts	=	2911.43936	cm^3	

(b) Concrete Section:-

C.G. from bottom of the haunch:-

Description		**Area**	**Y**	**AY**
		cm2	cm	cm3
Rectangular haunch		175.00	3.50	612.50
Slab		2356.00	10.00	23560.00
total		2531.00		24172.50
		Yc from top=	10.45	cm
		Yc from bottom=	9.55	cm

Moment of inertia of concrete section alone:-

Description		Area	**d**	**Ad^2**		**I (self)**
		cm2	cm	cm4		cm4
Rectangular haunch		175.00	6.05	6406.65		714.58
Triangular haunch		0.00	9.55	0.00		0.00
Slab		2356.00	0.45	475.88		78533.33
		2531.00		6882.53		79247.92
Ina = Iself + Ad^2		=	86130.44	cm^4		

Composite section with K=1 and modular ratio= 6.31

Gross area of concrete slab with haunch =						2531.00	cm^2
Transformed area of concrete in terms of steel =						401.26	cm^2
Moment of inertia (MI) of concrete in terms of steel =						13654.83	cm^4
		Ybc=	67.96	cm			
		Ytc=	2.04	cm			
		Ycc=	29.04	cm			
Moment of inertia of the composite section				Ina,c =		407545.892	cm^4
Zbc = 407546 / 67.96						5996.9	cm^3
Ztc = 407546 / 2.04						199682.3	cm^3
Zcc (in terms of concrete) = 407546 x 6.31 / 29.04 =						88519	cm^3

Composite section with K=2.5

Transformed area of concrete in terms of steel Ac=						160.50	cm^2
Transformed moment of inertia of concrete steel=						5461.93055	cm^4
Distance of neutral axis from composite section when K=2.5 :							
		Ybc =	58.71	cm			
		Ytc=	11.29	cm			
		Ycc =	38.29	cm			
Moment of inertia of the composite section				Ina,c =		311107	cm^4
Zbc = 311107 / 58.71						5299.3	cm^3
Ztc = 311107 / 11.29						27548.0	cm^3
Zcc (in terms of concrete) = 311107 x 6.31x2.5 / 38.29 =						128114	cm^3

Check Arrangement of Section

Permissible web depth / thickness = 60.00

Web depth / thickness of web = 56.33 < 60.00

Hence, no intermediate stiffeners are required

Outstand of Flange

The outstand of the compression flange, , is limited to=12t

Permissible outstand = 12 * 12 144.00 mm

Outstand of flange = 0.5 * (250 - 12) 119.00 mm

Hence safe

3.12.4 Permissible Stresses (IRS SBC)

Type of Stress			Tension (kg/mm^2)			Comp (kg/mm^2)	
(1) Basic permissible stress			22.5			22.5	
(2) Permissible stress in fatigue for class D connection			13.861			29.022	
f_{min} / f_{max} =		0.269					
Design permissible stresses			13.861			22.500	
Bending tensile stress					=	13.861	kg/mm^2
Bending compression stress					=	22.500	kg/mm^2
Shear stress(Avg.)					=	12.7	kg/mm^2
d1/t =	56.3						

3.12.5 Bending Moment & Shear Force

Moments in member(Mton.m)									
Member	DL	SIDL	Live Load	CDA @	LL	Seismic	Wind	max of (5,6)	1+2+3+4
	1	2	3	4	3+4	5	6	7	8
5	10.30	5.06	24.70	17.11	41.81	2.07	3.4	3.40	57.16
6	10.30	5.06	24.70	17.11	41.81	2.07	3.4	3.40	57.16

Shear force (Mton)									
Member	DL	SIDL	Live Load	CDA @	LL	Seismic	Wind	max of (5,6)	
	1	2	3	4	3+4	5	6		
1	7.05	3.47	20.40	14.13	34.53	1.42	2.769	2.77	
20	7.05	3.47	20.40	14.13	34.53	1.42	2.769	2.77	

3.12.6 Stress Check

S.N.	Loading	Tension (f_{bc})	Compression (f_{tc})	Conc. Compr. (f_{cc})	Shear
		kg/mm^2	kg/mm^2	kg/mm^2	kg/mm^2
1.	D.L. of girder + slab (steel section)	3.538	3.54		0.87
2.	SIDL (Long term)	0.95	0.18	0.00	0.43
3.	Live Load (Short term)	6.97	0.21	0.05	4.26
	Total Actual Stresses	11.46	3.93	0.05	5.55
	Allowable Stress	13.86	22.50	1.53	12.70
	Actual to allowable ratio	**0.83**	**0.17**	**0.03**	**0.44**
4	Max of Wind /Seismic (Short term)	0.567	0.017	0.00	0.34
	Total Actual Stresses	12.03	3.95	0.06	5.89
	Allowable Stress (16.66% more)	16.17	26.25	1.53	14.82
	Actual to allowable ratio	**0.74**	**0.15**	**0.04**	**0.40**

3.12.7 Check for Deflection:

D =	$\underline{5*W*L^4}$	=	$\underline{5*M*L^2}$	(assuming UDL)	
	384EI		48EI		
E =	Modulus of elasticity		=	20387.4	kg/mm^2
I =	Moment of inertia		=	1.02E+09	mm^4
M =	Moment				
L =	Span		=	5830	mm

S.N.	Loading	Moment of Inertia	Moment	Deflection
		mm^4	t.m	mm
1.	D.L. of girder + slab (steel section)	1.02E+09	10.30	1.76
2.	SIDL (Long term)	3.11E+09	5.06	0.28
3.	Live Load (Short term)	4.08E+09	41.81	1.78
	Total Deflection			3.82
	Allowable Deflection =	L/600 =	9.717 mm	
	Deflection =	=	3.819 mm	***Safe***

3.13 DESIGN OF CROSS GIRDER

3.13.1
Loading

The cross girder is designed for its selfweight, load of the stringer, deck load as well as the railway live load. The coefficient of dynamic augmentation is estimated as per the Bridge rules that is based on Cl 2.4.1.1(a) which states the loaded length is taken as 2.5 x spacing of cross girders. Furthermore the CDA is reduced to 72% or a max of 72% as the bridge carries two tracks as per Cl 2.4.1.1(e). The load on the cross girder is estimated based on the loaded length being twice the spacing between the cross girders.

Span, L =	10.300	m
Spacing of cross girders =	5.830	m
Loaded length for load calculations =	11.660	m
Spacing of stringer=	1.760	m

Estimation of CDA is based on loaded length of 2.5xspan

Hence, Loaded length for CDA =	14.575	m
Hence, CDA =	0.539	
CDA for double track, hence CDA =	0.388	

3.13.2 Cross sectional property of cross girder

		Width (mm)	Thickness (mm)	Area (mm^2)	
	Top plate	500	20	10000	
	Vertical plate	1400	20	28000	
	Bottom plate	550	30	16500	
			Total	54500	mm^2
			No. of Cross girders =	11	
			Total Weight =	48.47	t
	Total depth, D =	1450	mm		

Sectional Properties			z			I
NA from top yt =	806.19	mm	Z_t =	2.2E+07	mm^3	Ixx=17646226643mm4
NA from bottom yb =	643.81	mm	Z_b =	2.74E+07	mm^3	Iyy=625204166mm4

3.13.3 Permissible stress

Type of stress		Tension (kg/mm^2)	Comp (kg/mm^2)
(1) Basic permissible stress		22.5	22.5
(2) Permissible stress in fatigue		16.735	29.90
f_{min} / f_{max} =	0.386		
Design permissible stresses		16.735	22.5
Bending tensile stress =		16.735	
Bending compression stress =		22.5	
Shear stress =		12.7	
d1/t =	70		

	Bending Moments(Mton)							
Member	DL	SIDL	Live Load	CDA @	Total	Seismic	Wind	1+2+3+4 + max (5,6)
	1	2	3	4	1+2+3+4			
319	85.00	42.00	145.80	56.56	329.36	12.30	3.50	341.66

Actual Stresses for Bending							
Normal case	(DL+SIDL+Live Load)						
Bending stress in tension =	M / Z_b =	0.012017	t/mm^2	=	12	kg/mm^2	
			Stress ratio	=	0.77		***Safe***
Bending stress in compression =	M / Z_t =	0.015047	t/mm^2	=	15.047	kg/mm^2	
			Stress ratio	=	0.71		***Safe***
Seismic/Wind case	(DL+SIDL+Live Load+Seismic)						
Bending stress in tension =	M / Z_b =	1.25E-02		=	12	kg/mm^2	
			Stress ratio	=	0.66		***Safe***
Bending stress in compression =	M / Z_t =	1.56E-02		=	15.609	kg/mm^2	
			Stress ratio	=	0.63		***Safe***

	Shear Force (ton)							
Member	DL	SIDL	Live Load	CDA @	Total	Seismic	Wind	1+2+3+4 + max (5,6)
	1	2	3	4	1+2+3+4			
301	30.50	13.60	44.50	17.26	105.86	4.11	3.10	109.97

Normal case					
Net area for shear			=	14000	mm2
Shear Force			=	105.864	t
Shear stress	Shear Force / Area of Web		=	7.56	kg/mm^2
		Stress ratio =		0.5952	***Safe***
Seismic case					
Shear Force			=	109.974	t
Shear stress	Shear Force / Area of Web		=	7.85	kg/mm^2
		Stress ratio =		0.61	***Safe***

Check for Deflection:						
E =	Modulus of elasticity		=	2.04E+04	kg/mm^2	
I =	Moment of inertia		=	1.76E+10	mm^4 =	
L =	Span		=	10300.00	mm	
Allowable deflection , d_{allo} =			10300 / 600 =		17.17	mm

	Deflections from Staad of			
Load	Cross Girder	Bot. Chord	Relative Def.	
	mm	mm	mm	
DL	47.20	43.00	4.2	
SIDL	18.11	16.03	2.1	
LL	26.14	20.01	6.1	
Total			12.4	**safe**

CHAPTER 4
RESULTS AND DISCUSSIONS

4.1 INTRODUCTON

The truss models are analyzed using computer software STAAD.ProV8i. This chapter include parameters which are being studied such as nature of forces and their magnitude develop in members such as diagonals, verticals and top chord bottom chord, etc. Weight of each member is calculated and finally weight of steel truss bridge is obtained in both cases. On the basis of weight of truss, both models are compared. This chapter includes all the results of four models.

4.2 MODEL-1 PRATT TRUSS

4.2.1 AXIAL FORCE IN TOP CHORD, BOTTOM CHORD, END REAKER DUE TO DEAD LOAD

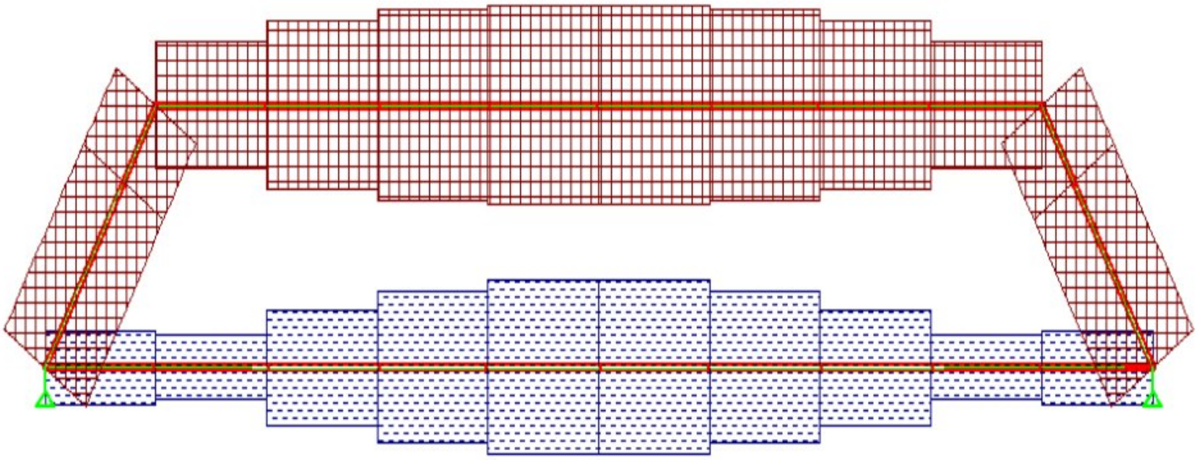


Fig.4.1 axial force due to dead load

Here, it can be seen that top chord members are in compression and bottom chord are in tension and end reaker members are also in compression. Axial forces are increasing in members which are in center in both top chord as well as bottom chord.

4.2.2 AXIAL FORCE IN END REAKER, VERTICAL AND DIAGONALS.

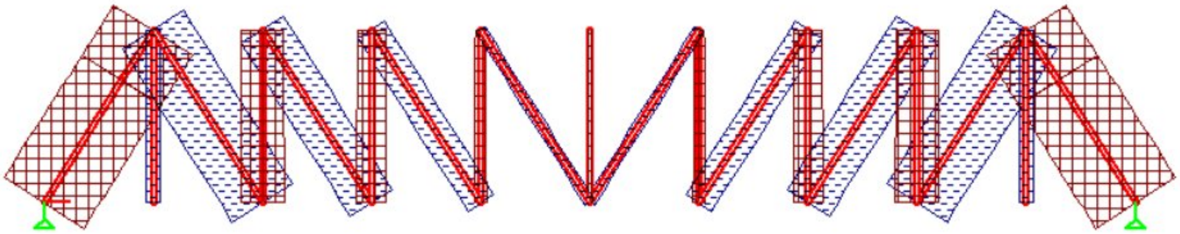


Fig.4.2 axial force due to dead load

Diagonals members are in tension and vertical members are in compression. Diagonal members and verticals which are near support carry more axial force than center members.

4.2.3 AXIAL FORCE IN TOP LATERAL BRACINGS

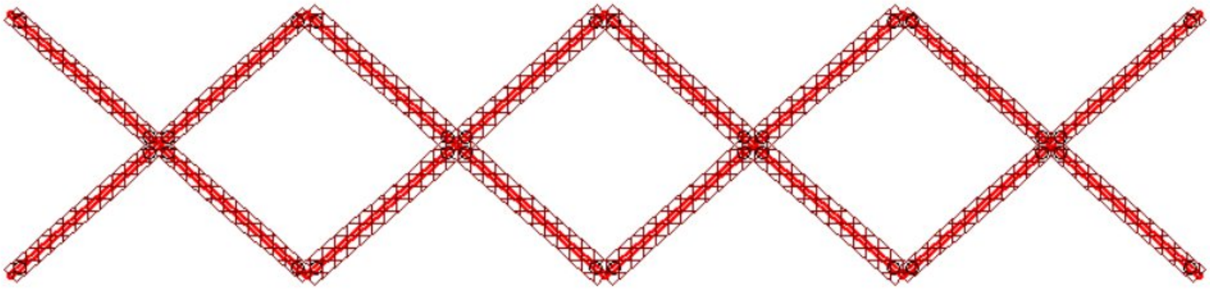


Fig.4.3 axial force due to dead load

Axial forces in top lateral bracings are compressive in nature and increasing towards center.

4.2.4 AXIAL FORCE IN BOTTOM LATERAL BRACING

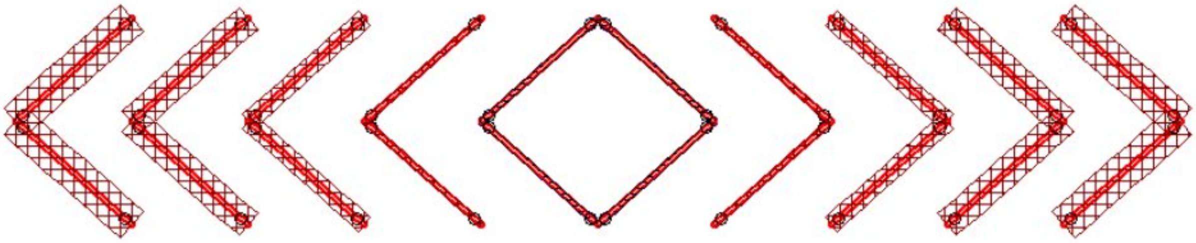


Fig.4.4 axial force due to dead load

Axial forces in bottom lateral bracings are compressive in nature and increasing towards support.

4.2.5 SHEAR FORCE VARIATION IN STRINGER

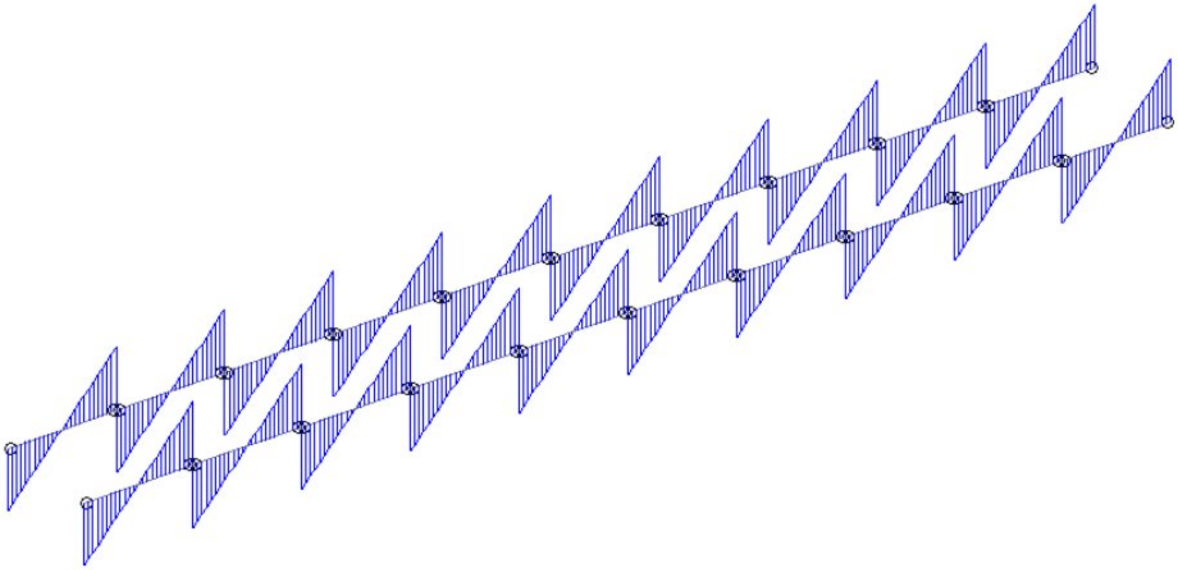


Fig.4.5 shear force due to dead load

It can be seen from figure that shear force variation is triangular in nature for stringer beam.

4.2.6 BENDING MOMENT VARIATION IN STRINGER

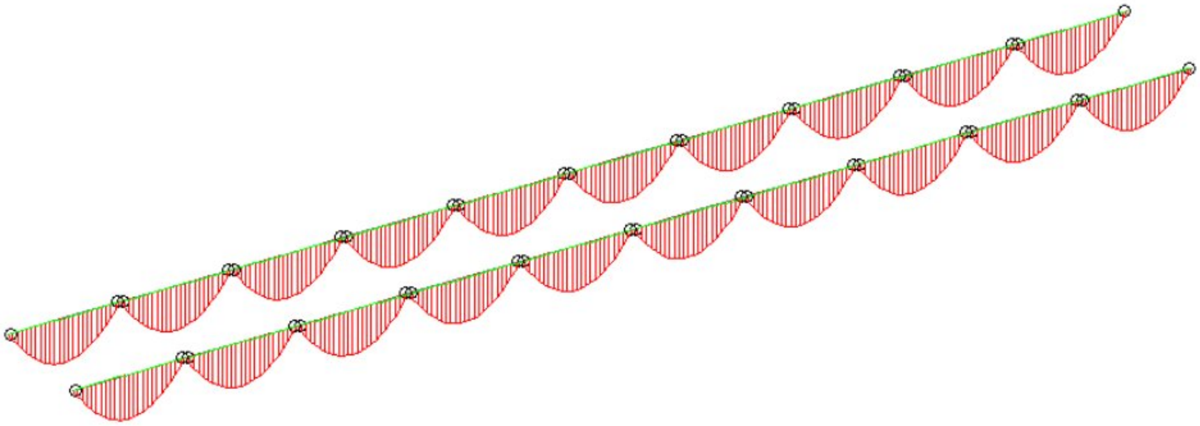


Fig.4.6 bending moment due to dead load

Bending moment variation is parabolic due to dead load.

4.2.7 SHEAR FORCE IN CROSS GIRDER

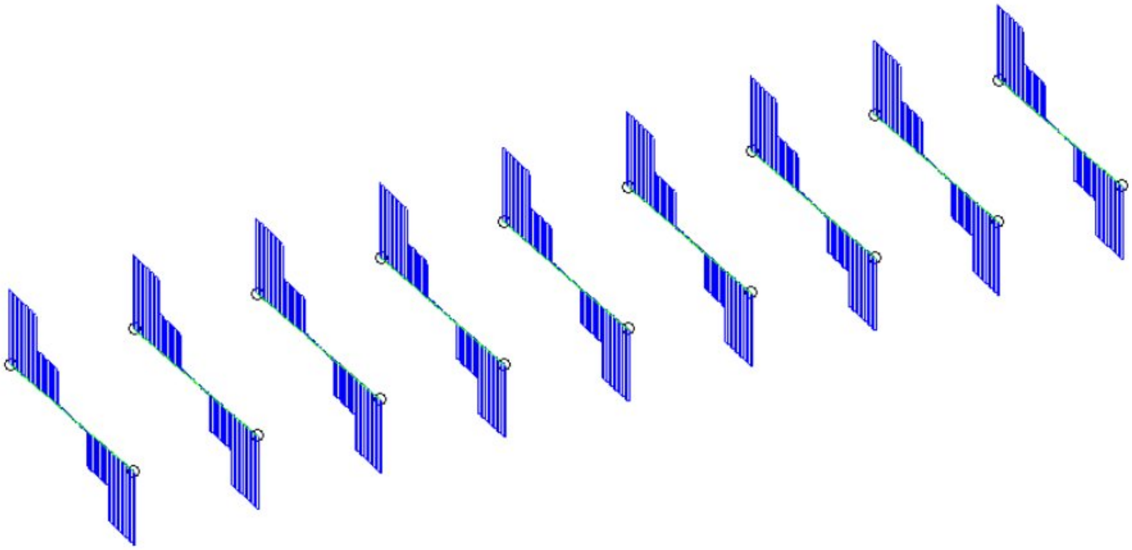


Fig.4.7 shear force due to dead load

Shear force variation is rectangular and decreasing at location of stringer beam in cross girder.

4.2.8 BENDING MOMENT VARIATION IN CROSS GIRDER

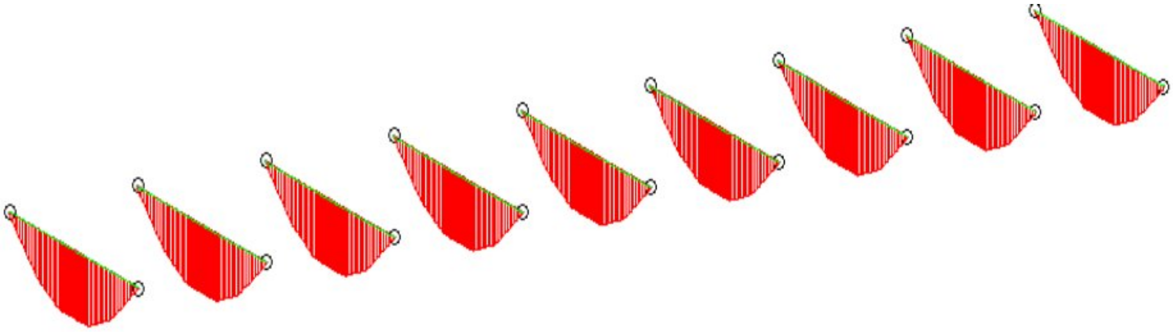


Fig.4.8 bending moment due to dead load

Bending moment variation is straight line having four kinks at the location of stringer beam.

4.2.8 DEFORMED SHAPE OF TRUSS

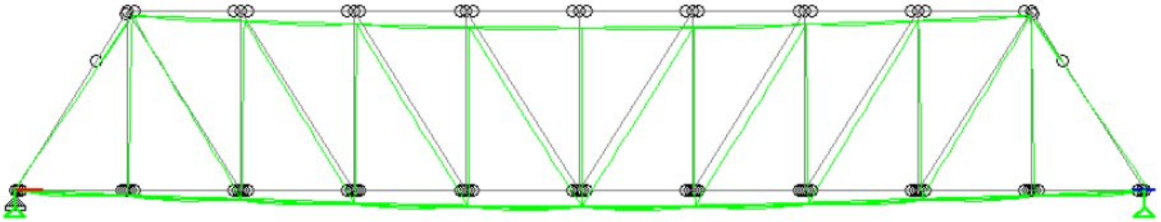


Fig.4.9 deformed shape due to dead load

It can be seen from figure that deformation of truss is similar like deformation of simply supported beam.

4.2.9 AXIAL FORCE DUE TO LIVE LOAD

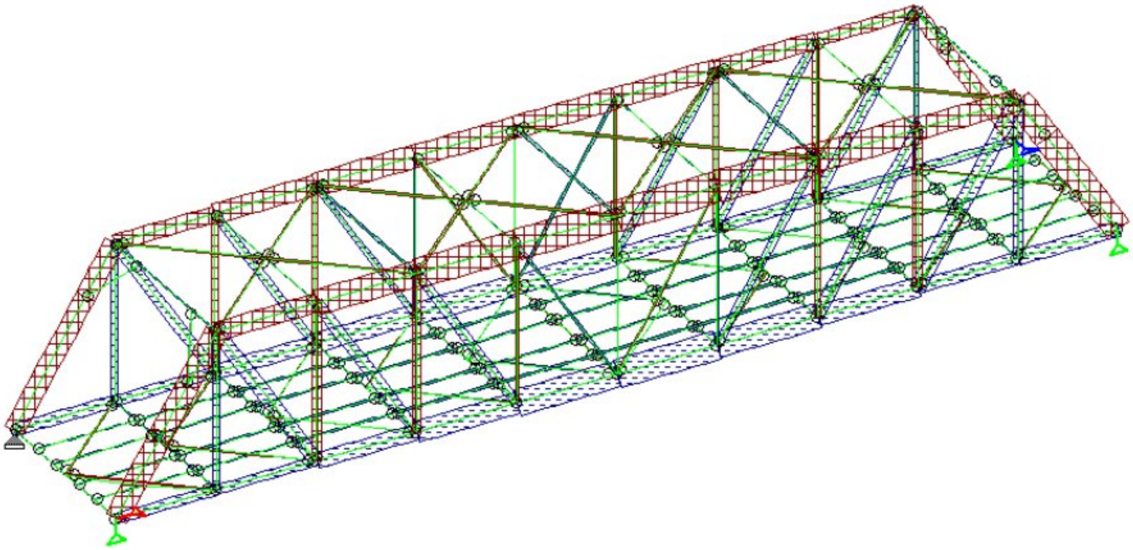


Fig.4.10 axial force due to live load

4.3 MODEL-2 ARCH TRUSS

4.3.1 AXIAL FORCE IN TOP CHORD, BOTTOM CHORD, END REAKER DUE TO DEAD LOAD

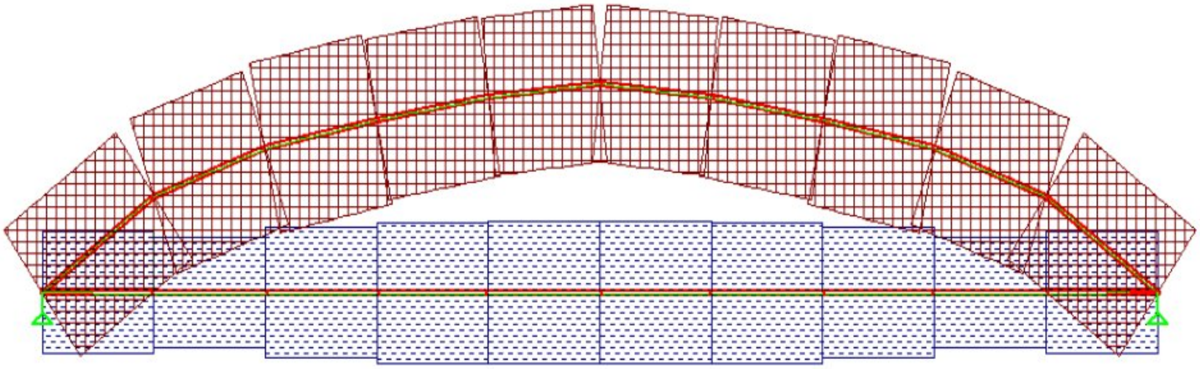


Fig.4.11 axial force due to dead load

Here, it can be seen that top chord members are in compression and bottom chord are in tension and end reaker members are also in compression. Axial forces are increasing in members which are near support in top chord.

4.3.2 AXIAL FORCE IN END REAKER, VERTICAL AND DIAGONALS.

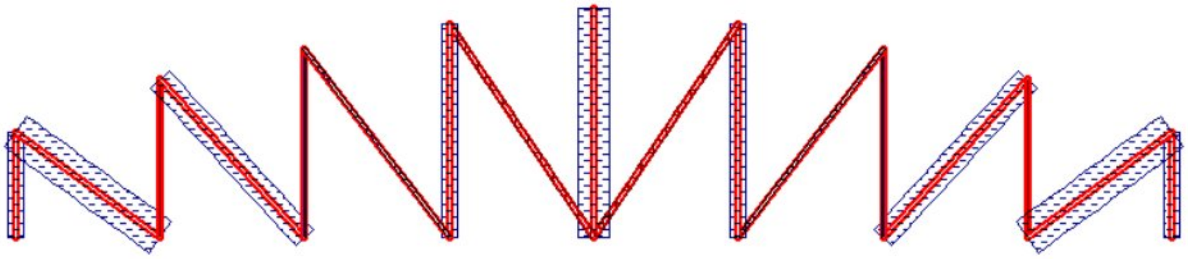


Fig.4.12 axial force due to dead load

Diagonals and verticals members are in tension. Diagonal members which are near support carry more axial force than center members. But in case of vertical members, near center carry more axial force.

4.3.3 AXIAL FORCE IN TOP LATERAL BRACINGS

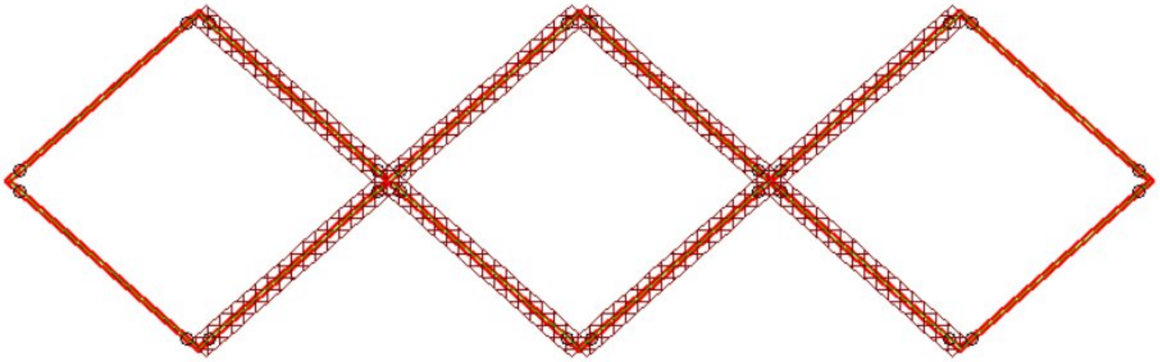


Fig.4.13 axial force due to dead load

Axial forces in top lateral bracings are compressive in nature and increasing towards center.

4.3.4 AXIAL FORCE IN BOTTOM LATERAL BRACING

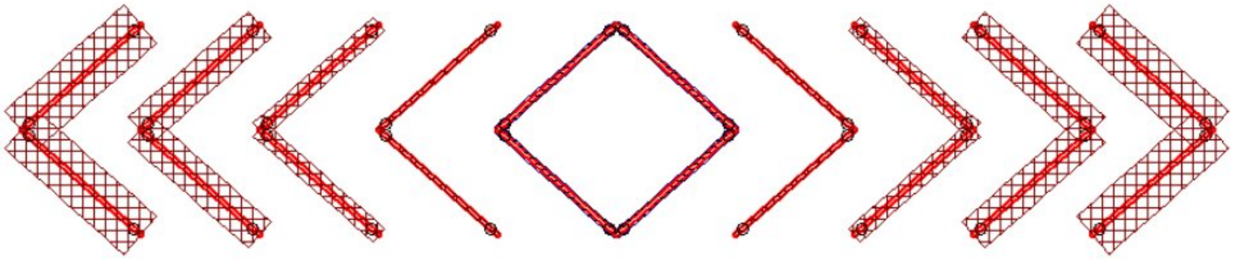


Fig.4.14 axial force due to dead load

Axial forces in bottom lateral bracings are compressive in nature and increasing towards support

4.3.5 SHEAR FORCE VARIATION IN STRINGER

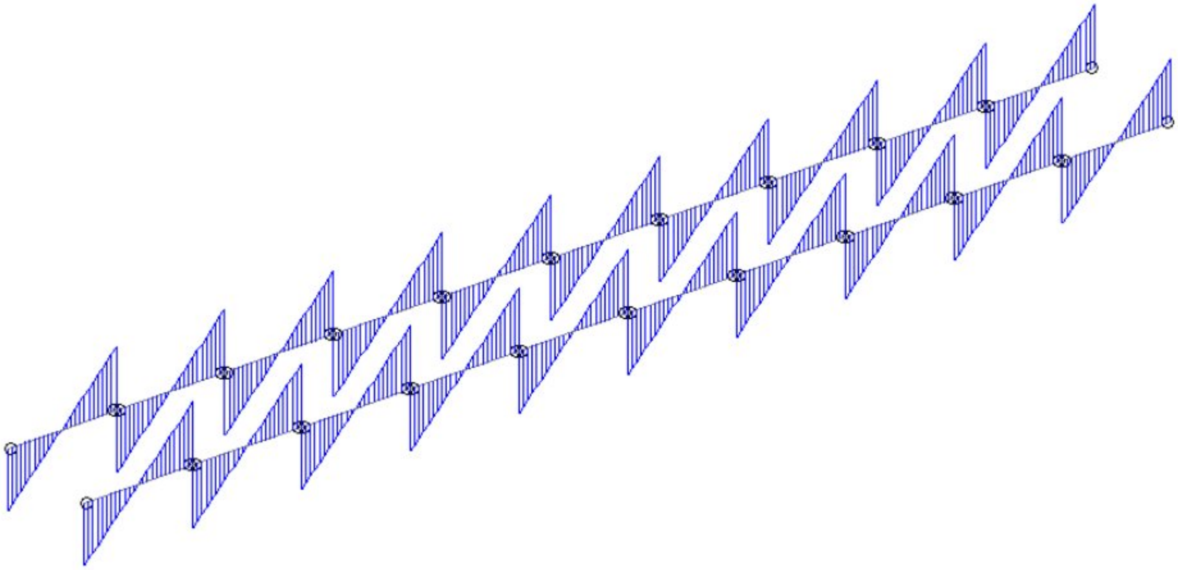


Fig.4.15 shear force due to dead load

It can be seen from figure that shear force variation is triangular in nature for stringer beam

4.3.6 BENDING MOMENT VARIATION IN STRINGER

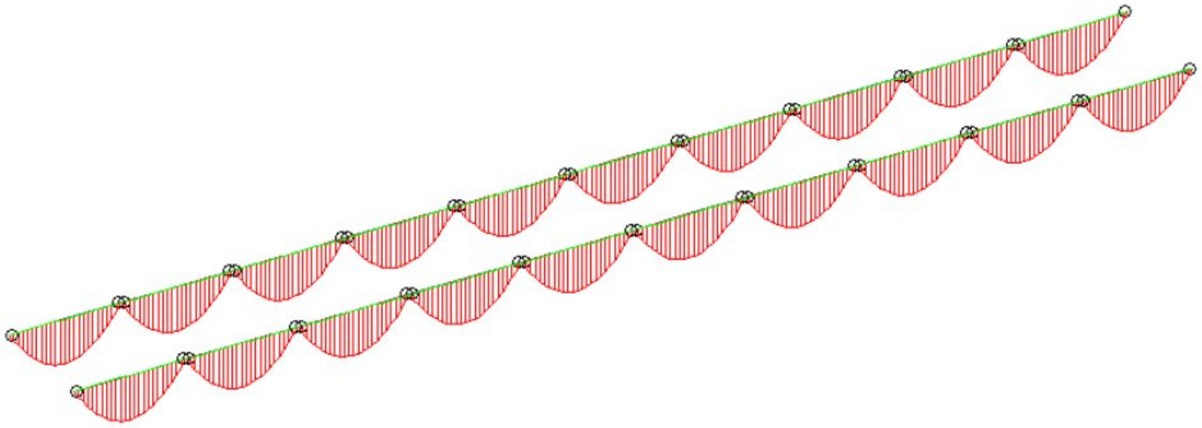


Fig.4.16 bending moment due to dead load

Bending moment variation is parabolic due to dead load

4.3.7 SHEAR FORCE IN CROSS GIRDER

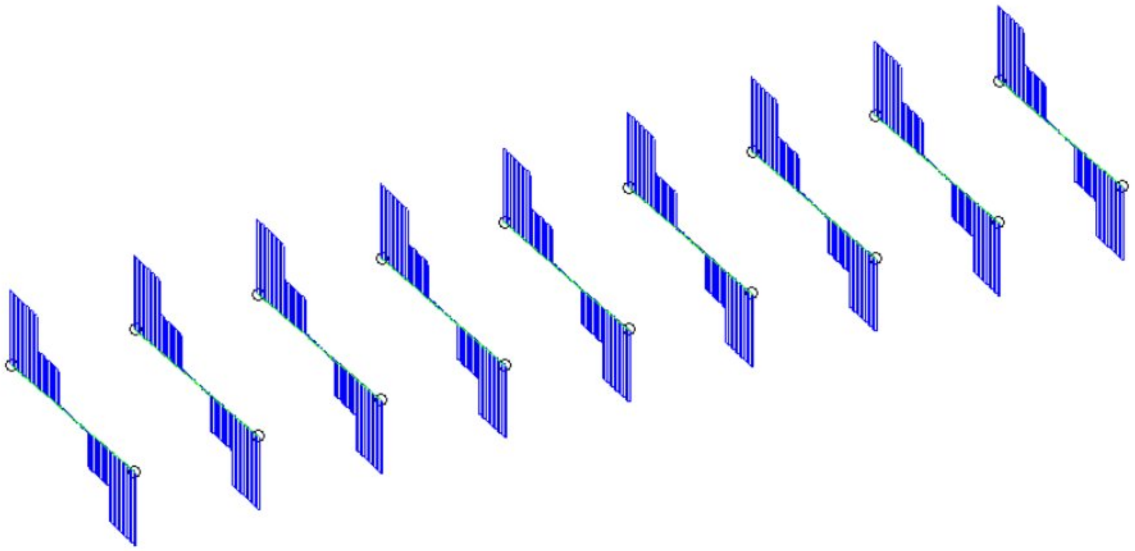


Fig.4.17 shear force due to dead load

Shear force variation is rectangular and decreasing at location of stringer beam in cross girder

4.3.8 BENDING MOMENT VARIATION IN CROSS GIRDER

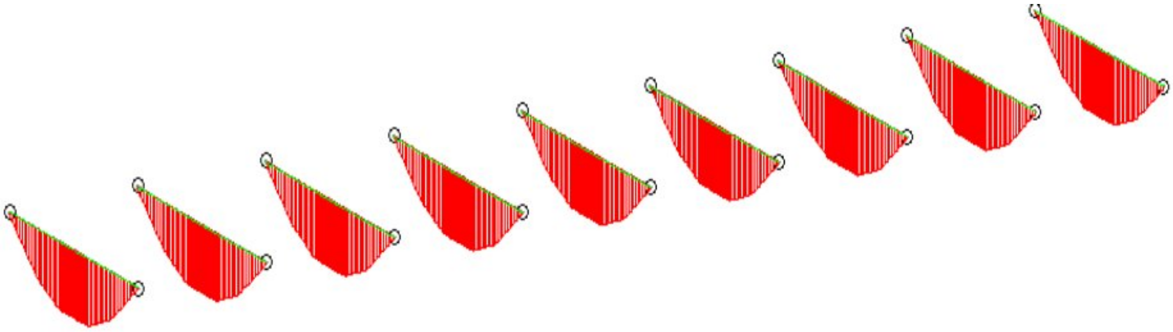


Fig.4.18 bending moment due to dead load

Bending moment variation is straight line having four kinks at the location of stringer beam.

4.3.8 DEFORMED SHAPE OF TRUSS

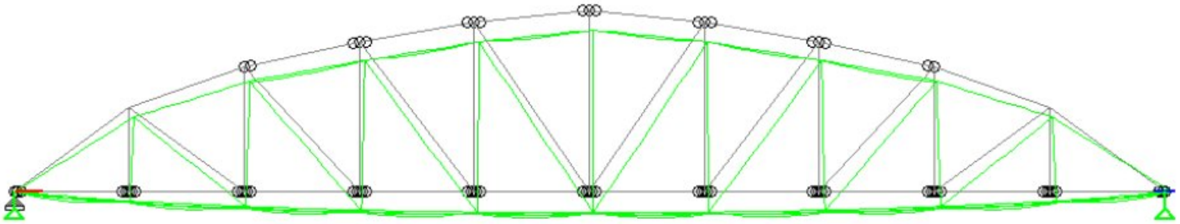


Fig.4.19 deformed shape due to dead load

4.3.9 AXIAL FORCE DUE TO LIVE LOAD

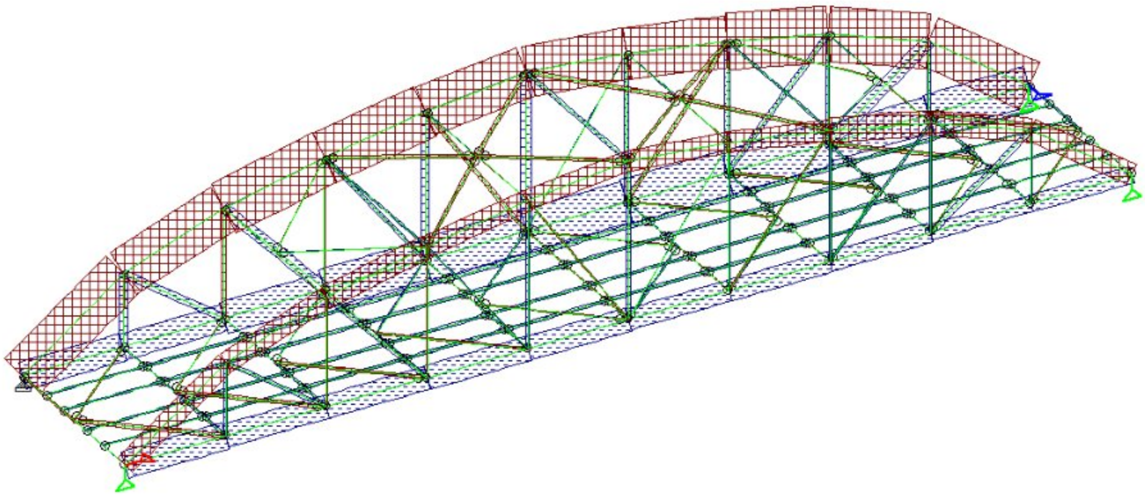


Fig.4.20 axial force due to live load

4.4 DEFLECTION

S.N.	PRATT TRUSS	Deflection(mm)	ARCH TRUSS	Deflection(mm)	
1	DL	47.0	DL	48	
2	Live Load	26.0	Live Load	28	
3	SIDL	18.0	SIDL	19	
	Total Deflection	91.0	**Total Deflection**	95	
Permissible Deflection = L/600 =			97.17	mm	**Check**

It is observed that in Pratt truss, deflection is less as compared to Arch truss bridge

4.5 **WEIGHT CALCULATION FOR THE PRATT TRUSS**

Description	**Area of Member Provided** **m2**	**No. of members**	**Weight** **t/m**	**Length** **m**	**Wt of the member** **t**
BOTTOM CHORD					
L0-L1	0.035	4	0.276	5.830	6.444
L1-L2	0.035	4	0.276	5.830	6.444
L2-L3	0.035	4	0.276	5.830	6.444
L3-L4	0.038	4	0.301	5.830	7.030
L4-L5	0.046	4	0.364	5.830	8.494
TOP CHORD					
U1-U2	0.038	4	0.298	5.830	6.942
U2-U3	0.038	4	0.298	5.830	6.942
U3-U4	0.042	4	0.333	5.830	7.762
U4-U5	0.050	4	0.396	5.830	9.226
DIAGONALS					
L0-U1	0.043	4	0.339	10.890	14.772
U1-L2	0.030	4	0.232	10.890	10.122
L2-U3	0.024	4	0.186	10.890	8.097
U3-L4	0.016	4	0.124	10.890	5.389
L4-U5	0.016	4	0.124	10.890	5.389
VERTICALS					
U1-L1	0.017	4	0.130	9.200	4.770
U2-L2	0.027	4	0.210	9.200	7.719
U3-L3	0.027	4	0.210	9.200	7.719
U4-L4	0.017	4	0.130	9.200	4.770
U5-L5	0.016	4	0.129	9.200	4.763
TOP CROSS MEMBER					
U1-U1'	0.016	2	0.123	10.300	2.525
REST	0.022	7	0.171	10.300	12.316

BOTTOM BRACINGS						
L0-L1		0.010	4	0.082	7.779	2.562
L1-L2		0.010	4	0.082	7.779	2.562
L2-L3		0.010	4	0.082	7.779	2.562
L3-L4		0.010	4	0.082	7.779	2.562
L4-L5		0.010	4	0.082	7.779	2.562
TOP BRACINGS						
U1-U2		0.012	4	0.092	7.779	2.855
U2-U3		0.012	12	0.092	7.779	8.565
PORTAL BRACINGS						
L0-U1		0.002	4	0.018	2.200	0.156
Walkway						
(@200kg/m)			2	0.20	60.000	24.0
Stringer		0.014	40	0.110	5.83	25.629
Cross Girder		0.055	11	0.432	10.30	48.917
Total Steel Weight (C/C length)						**277.0**
Total Steel Weight (with 20% extra for connections)						**332.4**
	Effective Span =	58.300	m			**5.70 t/m**

4.6 WEIGHT CALCULATION FOR ARCH STEEL TRUSS

Description	Area of Member Provided	No. of members	Weight	Length	Wt of the member
	m2		t/m	m	t
BOTTOM CHORD					
L0-L1	0.047	4	0.371	5.830	8.641
L1-L2	0.043	4	0.339	5.830	7.908
L2-L3	0.043	4	0.339	5.830	7.908
L3-L4	0.048	4	0.377	5.830	8.787
L4-L5	0.052	4	0.408	5.830	9.519
TOP CHORD					
U1-U2	0.039	4	0.308	6.200	7.631
U2-U3	0.043	4	0.339	5.960	8.085
U3-U4	0.048	4	0.377	5.920	8.923
U4-U5	0.052	4	0.408	5.860	9.568
DIAGONALS					
L0-U1	0.047	4	0.371	7.210	10.686
U1-L2	0.014	4	0.108	7.210	3.115
L2-U3	0.014	4	0.108	8.620	3.724
U3-L4	0.012	4	0.097	9.580	3.706
L4-U5	0.012	4	0.097	10.390	4.019
VERTICALS					
U1-L1	0.013	4	0.101	4.250	1.710
U2-L2	0.013	4	0.102	6.350	2.579
U3-L3	0.013	4	0.102	7.600	3.087
U4-L4	0.017	4	0.130	8.600	4.459
U5-L5	0.016	4	0.129	9.200	4.763

TOP CROSS MEMBER					
U1-U1'	0.014	0	0.110	10.300	0.000
REST	0.015	7	0.122	10.300	8.766
BOTTOM BRACINGS					
L0-L1	0.010	4	0.082	7.779	2.562
L1-L2	0.010	4	0.082	7.779	2.562
L2-L3	0.010	4	0.082	7.779	2.562
L3-L4	0.010	4	0.082	7.779	2.562
L4-L5	0.010	4	0.082	7.779	2.562
TOP BRACINGS					
U1-U2	0.012	0	0.092	7.779	0.000
U2-U3	0.012	12	0.092	7.779	8.565
Walkway					
(@200kg/m)		2	0.20	60.000	24.0
Stringer	0.014	40	0.110	5.83	25.629
Cross Girder	0.055	11	0.432	10.30	48.917
Total Steel Weight (C/C length)					**247.5**
Total Steel Weight (with 20% extra for connections)					**297.0**
	Effective Span =	58.300 m			**5.09 t/m**

4.7 WEIGHT OF MEMBERS OF PRATT TRUSS

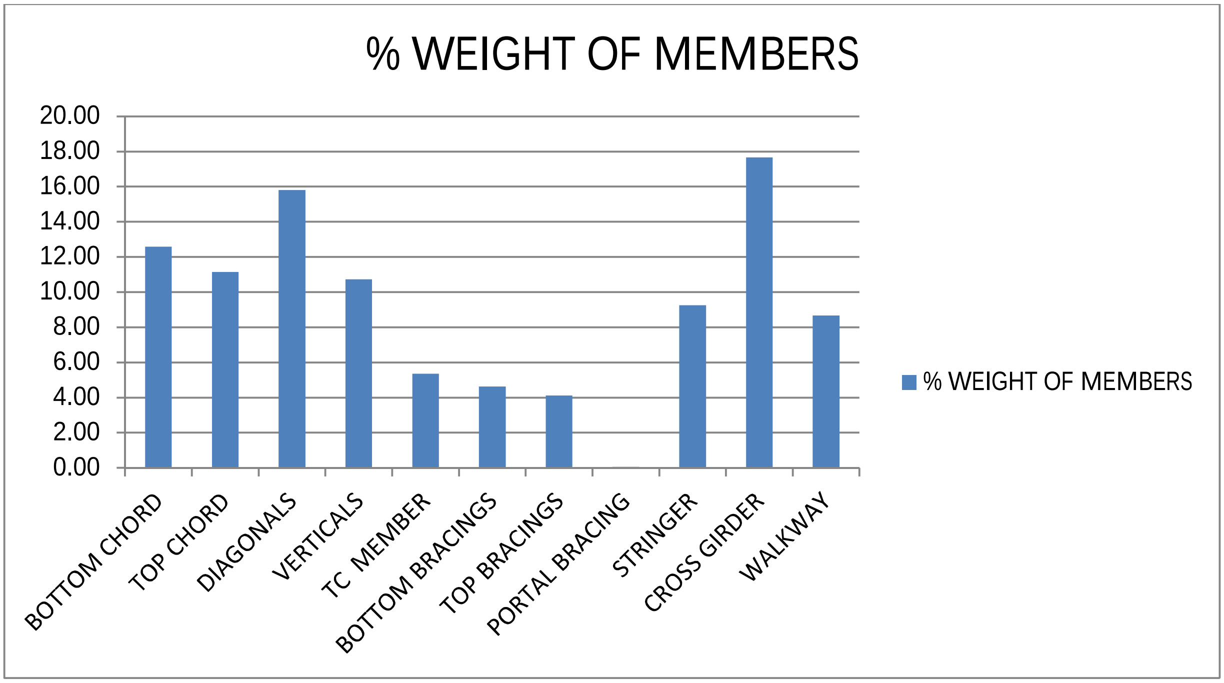


Fig.4.21 %Weight of members

4.8 WEIGHT OF MEMBERS OF ARCH TRUSS

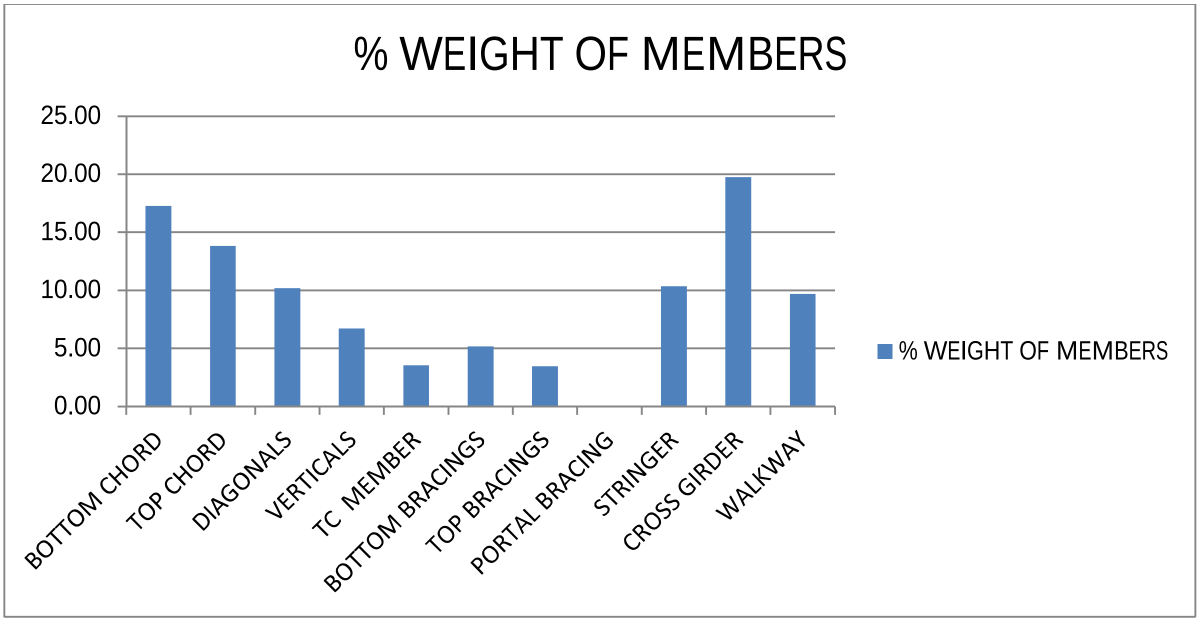


Fig.4.22 %Weight of members

4.9 COMPARION OF WEIGHT OF MEMBERS IN PRATT AND ARCH TRUSS BRIDGE

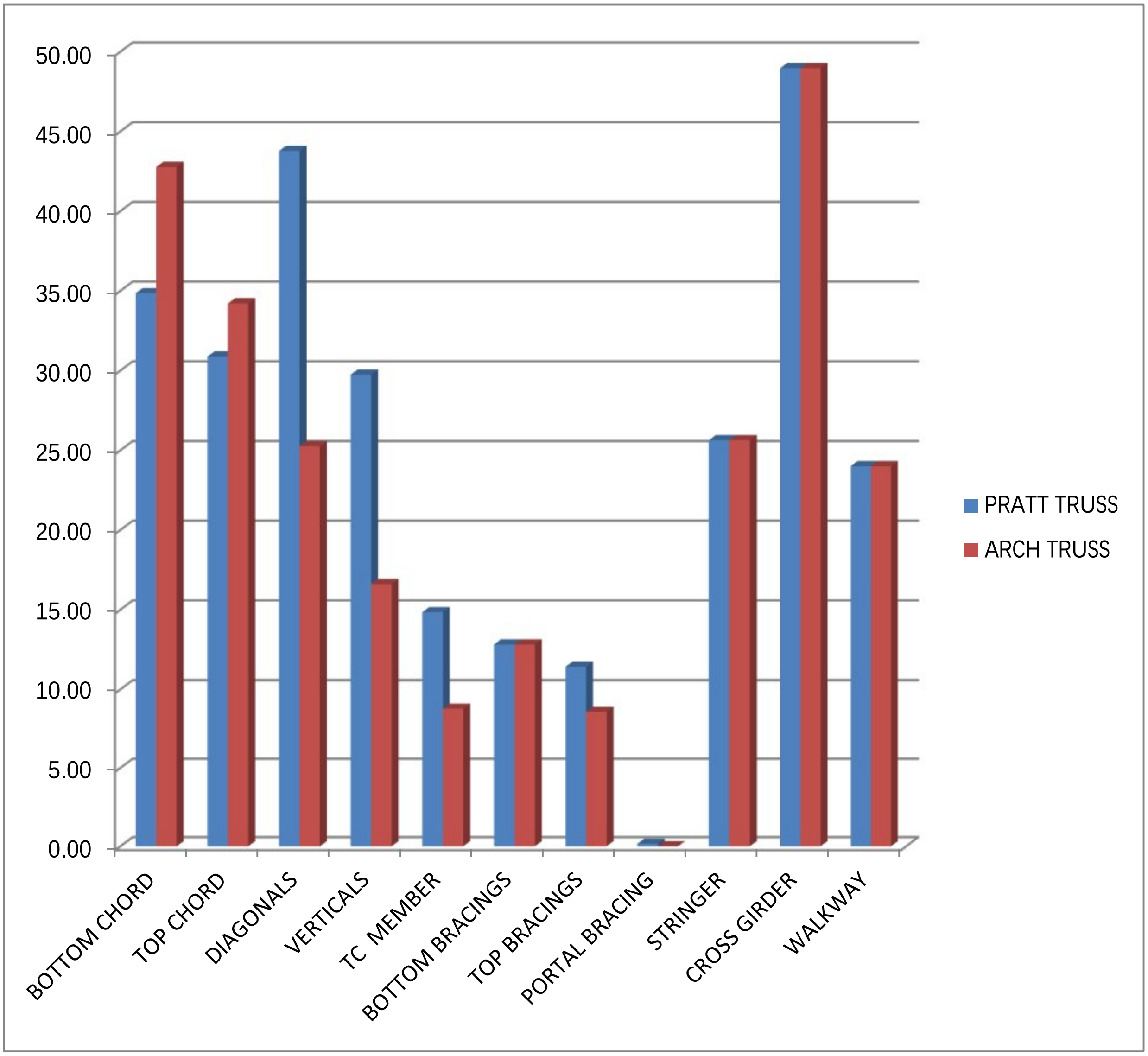


Fig.4.23 comparison between weight of members

CHAPTER 5
SUMMARY AND CONCLUSIONS

5.1 SUMMARY

Structural steel has been the natural solution for long span bridges since 1890, when the Firth of Forth cantilever bridge, the world's major steel bridge at that time was completed.

Advantages of structural steel over other construction materials are its strength and ductility. It has a higher strength to cost ratio in tension and a slightly lower strength to cost ratio in compression when compared with concrete. The stiffness to weight ratio of steel is much higher than that of concrete. Thus it has become the obvious choice for long span bridges as steel is more efficient and economic. The design and analysis of steel truss bridge is never been an easy task, a lot of parameter have to kept in mind. One of the important and challenging task is to reduce the weight of dead load of superstructure on the substructure. In order to address this matter, the aim of the present project is to carry out the analysis and design of steel truss bridge. To achieve this, a 60m span of metro viaduct is selected in Delhi which is in ZONE 4 and two truss model are modeled and analyzed using STAAD.Pro.V8i software. Steel truss is designed manually and weight of steel truss bridge is calculated. On the basis of weight of steel truss, it is decided that which steel truss is better.

5.2 CONCLUSION

The objectives of the project is to understand the behavior of structural action of steel truss Bridge, perform the analysis and design of Pratt truss and Arch truss bridges for metro Viaduct, find out the efficient and economical option best suited for obligatory situations in metro viaduct.

In the study, 2 models are analyzed and designed and on the basis of their results following conclusions are drawn from the study:

1) In both Pratt and Arch truss, top chord members are in compression and bottom chord are in tension. Axial forces are increasing in members which are in center in both top chord as well as bottom chord in Pratt truss. But in case of Arch truss, axial forces are increasing in top chord members which are near support.

2) In Pratt truss, diagonal members are in tension and vertical members are in compression. Diagonal members and verticals which are near support carry more axial force than center members. But in case of Arch truss bridge, both diagonal and vertical members are in tension. Diagonal members which are near support carry more axial force than center members. But in case of vertical members near center carry more axial force.

3) Axial forces in bottom lateral bracings are compressive in nature and increasing towards support in both truss.

4) Axial forces in top lateral bracings are compressive in nature and increasing towards center in both truss.

5) Bending moment and shear force variation in stringer is parabolic and triangular.

6) Bending moment variation in cross girder is straight line having kinks at the location of stringer and shear force variation is rectangular and decreasing at location of stringer beam in cross girder.

7) Deflection of Pratt truss due to Dead load, SIDL, and live load is 91mm whereas in case of Arch truss bridge it is 95mm.

8) Weight of bottom chord and top chord members of Arch truss bridge is more than that of Pratt truss bridge but in case of diagonals, verticals and top chord members, Pratt truss members have more weight.

9) In Pratt truss, weight of diagonals is more as compared to other members.

10) In case of Arch truss, weight of bottom chord is more as compared to other members.

11) Total weight of arch truss bridge is 297tonne, whereas in case of Pratt truss it is 332.4tonne.

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