Design of Steel Bridge using STAAD.pro

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MAY-2010

Submitted in partial fulfillment of the Degree of Bachelor of Technology

to

DEPARTMENT OF CIVIL ENGINEERING

JAYPEE UNIVERSITY OF INFORMATION TECHNOLOGY

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CERTIFICATE

This is to certify that the work entitled, "Design of Steel Bridge using STAAD.pro" submitted by Udai Bhanu Keshari, Prashant Gupta and Arpit Bansal, in partial fulfillment for the award of degree of Bachelors of Technology in Civil Engineering of Jaypee University of Information Technology has been carried out under my supervision. This work has not been submitted partially or wholly to any other University or Institute for the award of this or any other degree or diploma.

Date: 27.5.2010

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Acknowledgement

We wish to express our deep regards and gratitude to **Mr. Anil Dhiman**, Sr. Lecturer, Department of Civil Engineering, Jaypee University of Information Technology, Waknaghat, Solan, for his immense help and valuable guidance, constant encouragement and inspiration throughout the progress of this project.

We are also thankful to **Dr. Yajulu Medury**, Vice Chancellor (Acting HOD, Civil Engineering Department), JUIT and to all other faculty members of civil engineering department for their support and time to time guidance.

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ABSTRACT

With the introduction of truss bridges made of steel and due to high tensile strength of steel, bridges of longer spans are possible to construct. They have greater efficiency than concrete structures as regard to absorbing vibrations, resisting seismic forces and blast loading. It is this ability which made steel to be used for railway bridges. In the current project a Railway through bridge to whose length is 36 m and width is 5.25 m has been considered for analysis and design. First, the bridge was analysed manually to design preliminary sections of various members. Thereafter, an extensive analysis has been carried out by STAAD.pro. Primary loads considered were - self weight, train's axel loads, wind and earthquake loads. Wind load was calculated as per IS-875: 1987 part-3 for the bridge site assumed in Chandigarh. IS-1893:2002 Part-1 was followed for seismic loads' calculations. Further, a number of load combinations have been considered as per IS 800: 2007. Axial forces have been found to be critical for dead loads combined with earthquake load acting perpendicular to the bridge main axis. Other critical load cases were train-loads plus deal loads. Wind loads caused larger shear forces in z-direction. The design was checked for safety with STAAD.pro and all the members were found to be safe.

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CHAPTER 1

INTRODUCTION

GENERAL

A bridge is a structure built to span a valley, road, railroad track, river, body of water, or any other physical obstacle, for the purpose of providing passage over the obstacle. Designs of bridges will vary depending on the function of the bridge and the nature of the terrain where the bridge is to be constructed.

1.1 ADVANTAGES OF STEEL BRIDGES

The main 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, structural steel is an efficient and economic material in bridges.

The following are some of the advantages of steel bridges that have contributed to their popularity in Europe and in many other developed countries.

- They could carry heavier loads over longer spans with minimum dead weight, leading to smaller foundations.
- Steel has the advantage where speed of construction is vital, as many elements can be prefabricated and erected at site.
- In urban environment with traffic congestion and limited working space, steel bridges can be constructed with minimum disruption to the community.

- Greater efficiency than concrete structures is invariably achieved in resisting seismic forces and blast loading.
- The life of steel bridges is longer than that of concrete bridges.
- Due to shallow construction depth, steel bridges offer slender appearance, which make them aesthetically attractive. The reduced depth also contributes to the reduced cost of embankments.

In India there are many engineers who feel that corrosion is a problem in steel bridges, but in reality it is not so. Corrosion in steel bridges can be effectively minimized by employing newly developed paints and special types of steel. These techniques are followed in Europe and other developed countries.

1.2 STEEL USED IN BRIDGES

Steel used for bridges may be grouped into the following three categories:

- Carbon Steel: This is the cheapest steel available for structural users where stiffness is more important than the strength. Indian steels have yield stress values up to 250 N/mm^2 and can be easily welded.
- High strength steels: They derive their higher strength and other required properties from the addition of alloying elements.
- Heat-treated carbon steels: These are steels with the highest strength. They derive their enhanced strength from some form of heat-treatment after rolling namely normalization or quenching and tempering.

The physical properties of structural steel such as strength, ductility, brittle fracture, weldability, weather resistance etc., are important factors for its use in bridge construction. These properties depend on the alloying elements, the amount of carbon, cooling rate of the steel and the mechanical deformation of the steel.

1.3 CLASSIFICATION OF STEEL BRIDGES

Steel bridges are classified according to:

- The type of traffic carried.

- The type of main structural system.
- The position of the carriage way relative to the main structural system.

1.3.1 CLASSIFICATION BASED ON TYPE OF TRAFFIC CARRIED

- Highway or road bridges.
- Railway or rail bridges.
- Road - cum - rail bridges.

1.3.2 CLASSIFICATION BASED ON MAIN STRUCTURAL SYSTEM

Many different types of structural systems are used in bridges depending upon the span, carriageway width and types of traffic. Classification, according to makeup of main load carrying system, is as follows:

Girder Bridge - Flexure or bending between vertical supports is the main structural action in this type. Girder bridges may be either solid web girders or truss girders or box girders. Plate girder bridges are adopted for simply supported spans less than 50 m and box girders for continuous spans upto 250 m. Cross sections of a typical plate girder and box girder bridges are shown in Fig. 1.1.Truss bridges are suitable for the span range of 30 m to 375 m. Cantilever bridges have been built with success with main spans of 300 m to 550 m. Girder bridges are discussed in detail later. They may be further, sub-divided into simple spans, continuous spans and suspended-and-cantilevered spans, as illustrated in Fig. 1.2 and 1.3.

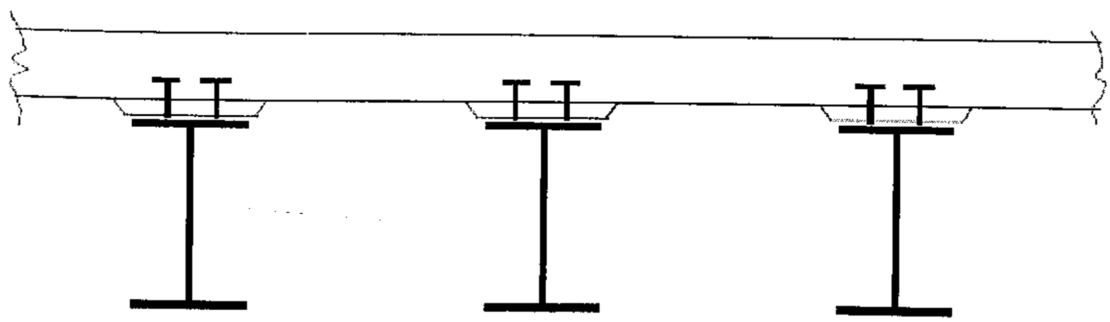


Fig 1.1: Plate Girder bridge section.

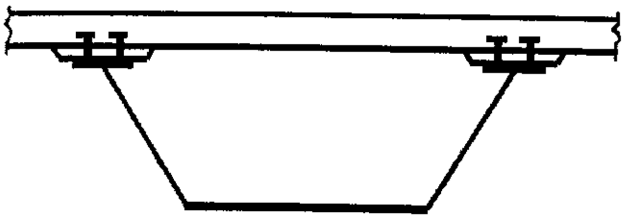


Fig 1.2: Box Girder bridge section.

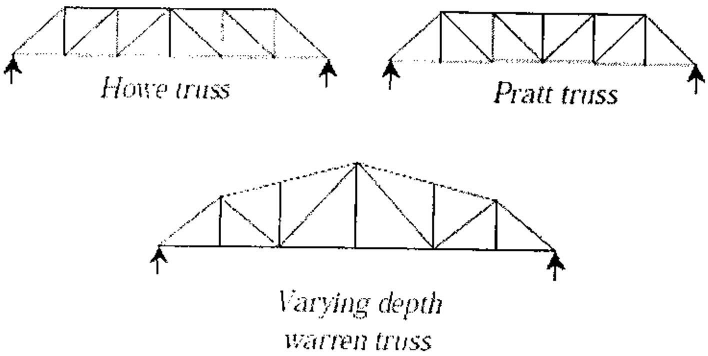


Fig 1.3: Some of the trusses used in steel bridges

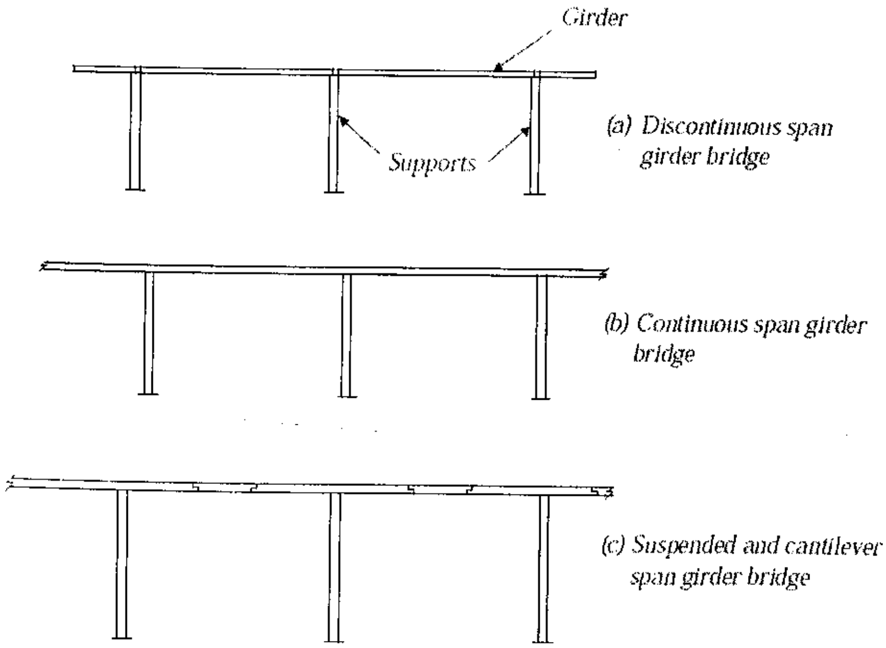


Fig 1.4: Typical Girder bridges

Rigid frame bridges - In this type, the longitudinal girders are made structurally continuous with the vertical or inclined supporting member by means of moment carrying joints as shown in Fig. 1.5. Flexure with some axial force is the main forces in the members in this type. Rigid frame bridges are suitable in the span range of 25 m to 200 m.

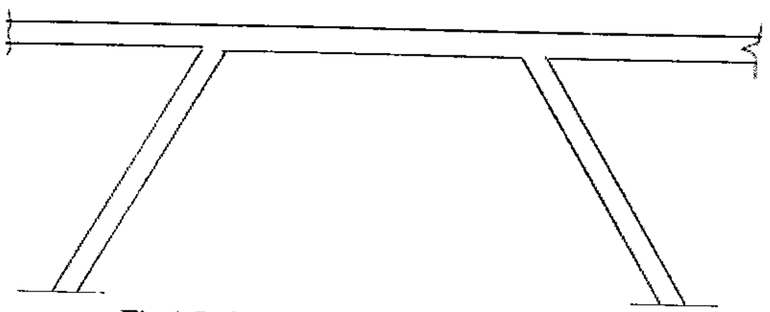


Fig 1.5: Typical Rigid Frame Bridge

Arch bridges - The loads are transferred to the foundations by arches acting as the main structural element. Axial compression in arch rib is the main force, combined with some bending. Arch bridges are competitive in span range of 200 m to 500 m. Examples of arch bridges are shown in Fig. 1.6

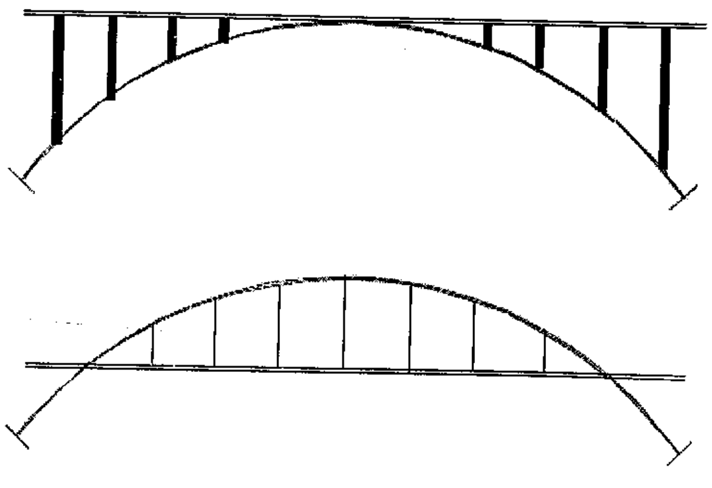


Fig 1.6: Typical Arch bridges

Cable stayed bridges - Cables in the vertical or near vertical planes support the main longitudinal girders. These cables are hung from one or more tall towers, and are usually anchored at the bottom to the girders. Cable stayed bridges are economical when the span is about 150 m to 700 m. Layout of cable stayed bridges are shown in Fig. 1.7 and 1.8

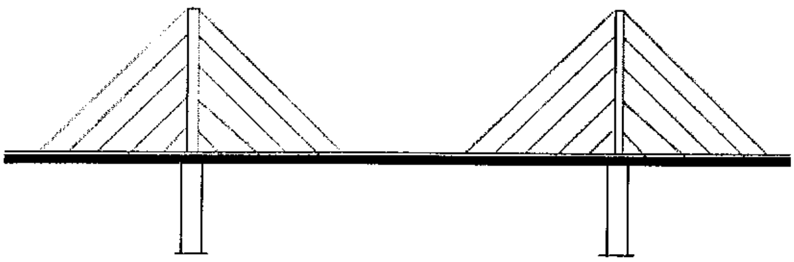


Fig 1.7: Layout of a Cable Stayed bridge type 1

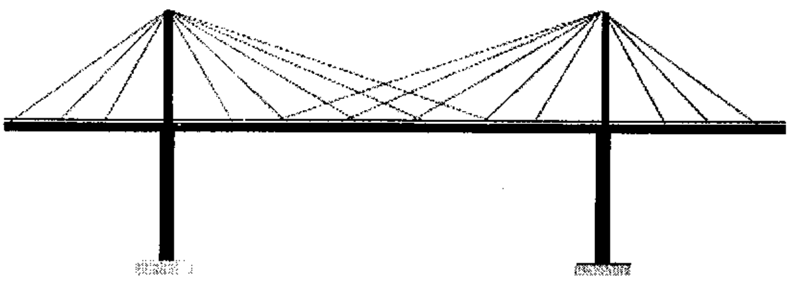


Fig 1.8: Layout of a Cable Stayed bridge type 2

Suspension bridges - The bridge deck is suspended from cables stretched over the gap to be bridged, anchored to the ground at two ends and passing over tall towers erected at or near the two edges of the gap. Currently, the suspension bridge is best solution for long span bridges. Figure 1.9 shows a typical suspension bridge. Fig shows normal span range of different bridge types.

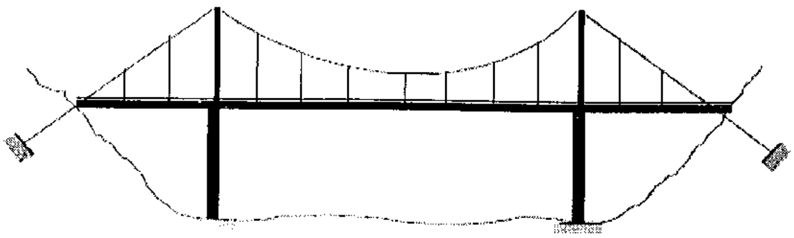


Fig 1.9: Suspension bridge

1.3.3 CLASSIFICATION BASED ON POSITION OF CARRIGEWAY

The bridges may be of the "deck type", "through type" or "semi-through type". These are described below with respect to truss bridges:

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. 1.11

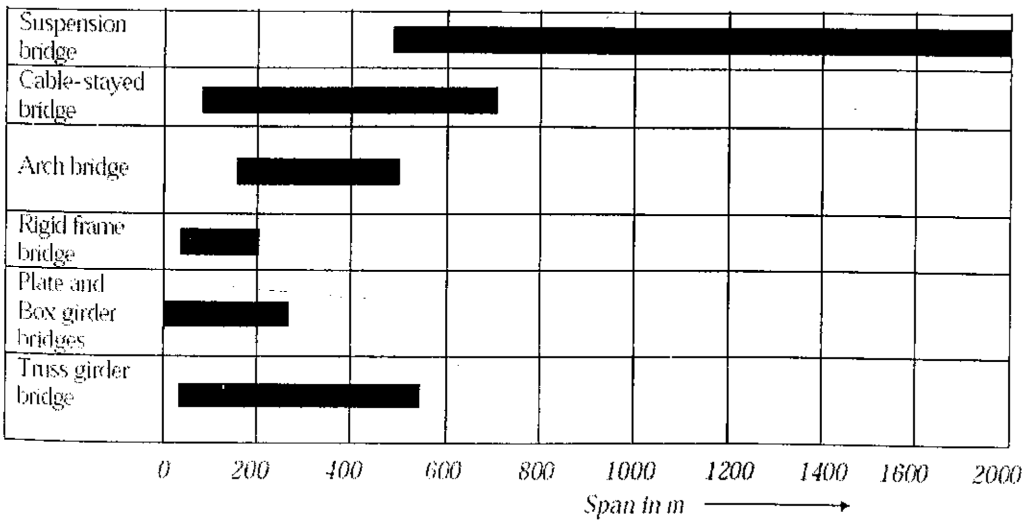


Fig 1.10: Normal span ranges of bridge system

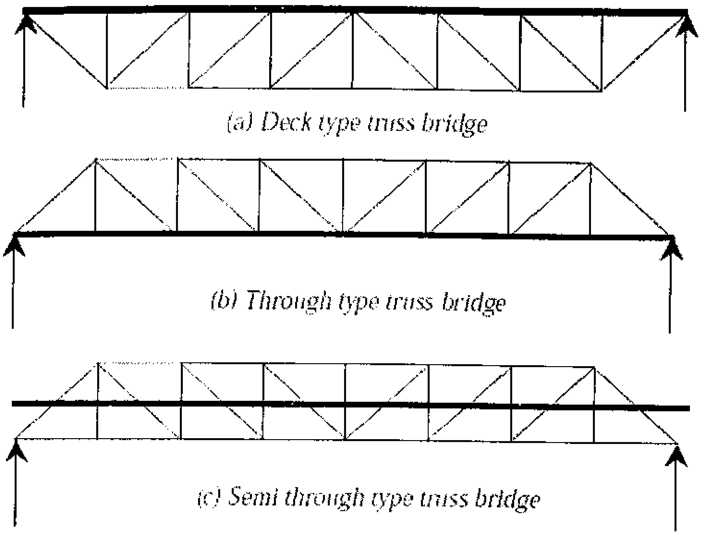


Fig 1.11: Typical Deck type, Through and Semi-Through type Truss bridges

Through Type Bridge - The carriageway rests at the bottom level of the main load carrying members. 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. The bracing of the top flange or lateral support of the top chord under compression is also required.

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. The lateral restraint in the system is obtained usually by the U-frame action of the verticals and cross beam acting together.

1.4 LOADS ON BRIDGES

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

- Dead load

- Live load
- Impact load
- Longitudinal force
- Thermal force
- Wind load
- Seismic load
- Racking force
- Forces due to curvature.
- Forces on parapets
- Frictional resistance of expansion bearings

CHAPTER 2
MANUAL ANALYSIS AND DESIGN OF A RAILWAY TRUSS BRIDGE

2.1 RAILWAY TRUSS BRIDGE

A **truss bridge** is a bridge composed of connected elements (typically straight) which may be stressed from tension, compression, or sometimes both in response to dynamic loads. Truss bridges are one of the oldest types of modern bridges. The basic types of truss bridges shown in Fig. 2.1 have simple designs which could be easily analyzed by nineteenth and early twentieth century engineers. A truss bridge is economical to construct owing to its efficient use of materials.

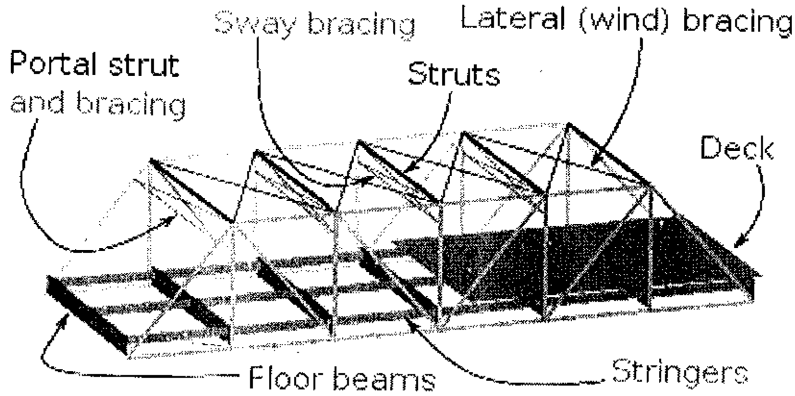


Fig 2.1: General Arrangement

2.2 GENERAL ARRANGEMENT

- For span greater than what can be spanned economically by the plate girder bridges, we use truss bridges.
- NOTE: It is difficult to draw a demarcating line in the lengths of span above which the plate girders will not be economical. For the same weight a plane girder may be economical due to smaller cost of fabrication.
- Roughly, a truss bridge should be used for spans greater than about 30m.
- There are 3 types of truss bridges
    - Through type
    - Deck type
    - Half- through bridge.
- The general arrangement of different members in through truss bridge is shown in Fig. 2.2

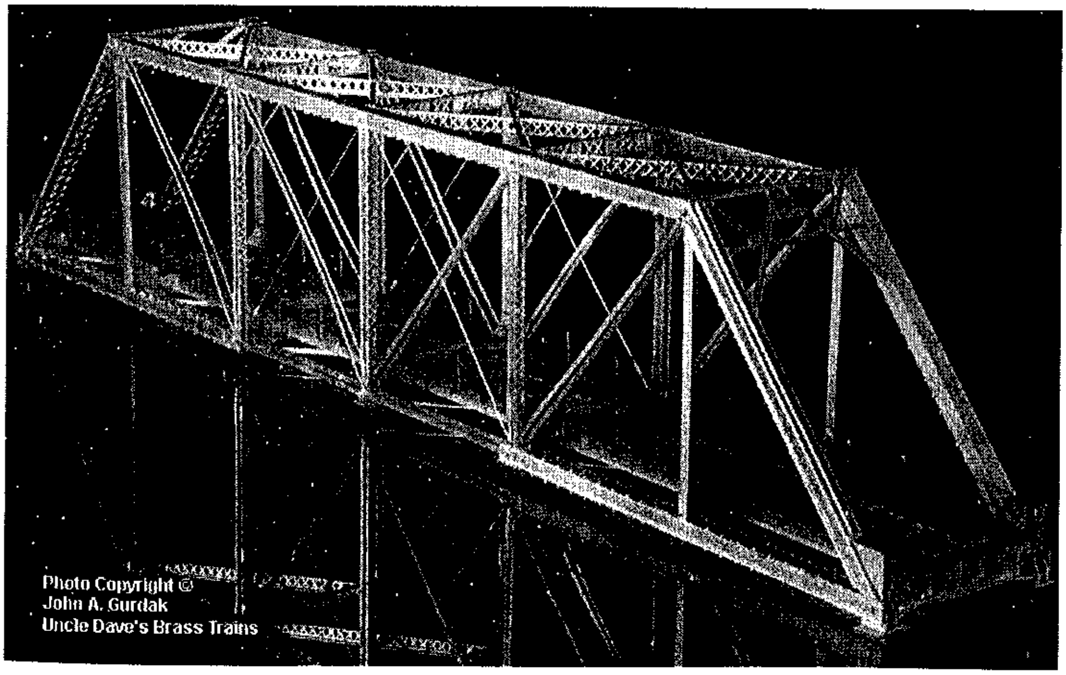


Fig. 2.2: General Arrangement For Truss Bridge

- Chord members form the perimeter of the truss figure. The diagonal and vertical members form the web system of the truss. The end members of the truss form a part of the web and there are called end posts. The point of intersection of web members (diagonals &vertical) with chord members is called panel points.
- The corresponding lower panel points of the two trusses are joined by girders, called cross beams or floor beams. These floor beams support stringers which run parallel to the length of the truss.
- In the railway bridges, the sleepers rest directly on the stringers.
- In addition to the vertical loads, a bridge is subjected to lateral forces due to wind, seismic and racking forces. To transfer these lateral forces to bearings, laterals are used at the bottom and top chords. Along with the bottom chord members, bottom laterals from a truss which can transfer lateral loads to bearings.
- The bracing provided in the plane of the end posts is called the portal bracing. Similar to portal bracing, sway bracing is used in the planes of corresponding verticals of the two trusses. The sway bracing keeps the rectangular shape of the bridge cross- section.
- The spacing b/w centers of the main girders should be sufficient to resist overturning with the specified wind pressures and loading conditions.

2.3 ECONOMIC PROPORTIONS

There are too many factors which affect the economy of bridge, so it is difficult to fix up mathematically the most economical configuration of the truss bridge. The permissible stress in the compression members, for example, is not constant and depends upon the slenderness ratio of the member. A truss configuration giving the minimum weight of the truss may not be the most economical because the cost of the floor system may be higher. The cost of fabrication will depend upon the number and types of joints and may not be the minimum when the weight of the truss is minimum.

2.4 THE PROBLEM

Design a through type single lane truss bridge for broad gauge main line loading for the following data:

The effective span of the bridge	:	36 m
Height	:	7 m
Overall width	:	5.25 m
Site	:	Chandigarh
Soil type	:	Medium Soil
Seismic zone	:	IV

The general arrangement adopted is shown in Fig. 2.3

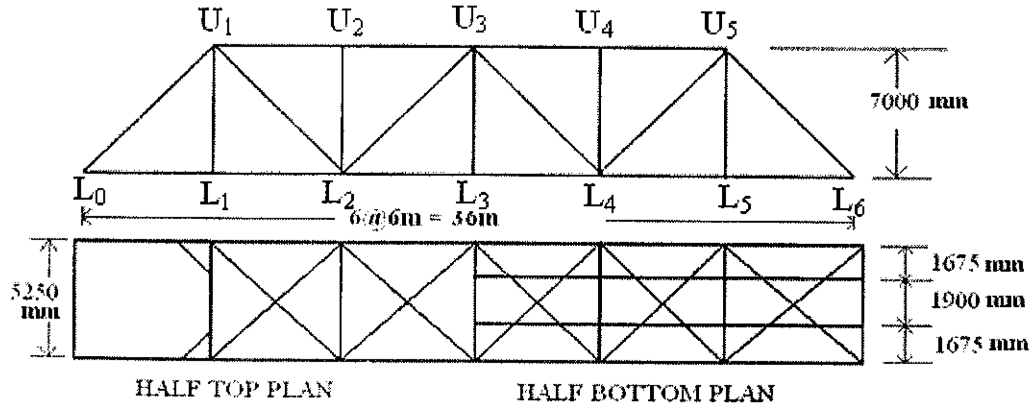


Fig. 2.3 General Configuration

Rivets dia	= 22mm
Holes dia	= 23.5mm
Vertical clearance of bridge	= 5.41m
Horizontal clearance of bridge	= 4.424m
Height of the truss	= 7.0m
Spacing of truss	= 5.25m

Stringers

Effective span = 6 m

Load	*For Bending*	*For Shear*
Live load	763.9kN	918.9kN
Impact load	624.1kN	750.7kN
Track load	15.7kN	15.7kN
Dead load	25.0kN	25.0kN
Total =	1428kN	1710.3kN

Maximum B.M. Per stringer = 535.8 kNm

Maximum S.F. in stringer = 427.6 kN

Assume the depth girder = 750 mm

Effective depth = 690 mm

For web d/t= 45<75mm

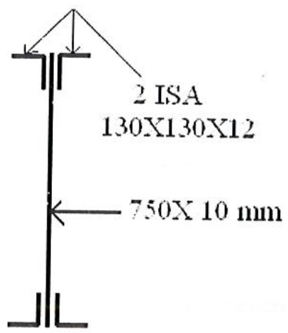


Fig 2.4 : Stringers' Section

F_b = 147 N/mm^2

Net area of flange required = $5282mm^2$

Flange element	*Gross area*	*Net Area*
2ISA 130 × 130 × 12	$59.64cm^2$	$54.00cm^2$
Equivalent web	$12.50cm^2$	$9.38cm^2$
Total	$72.14cm^2$	$63.38cm^2$

Moment of inertia = $1736\times10^6mm^4$

Stress on gross area = $115.7N/mm^2$

Stress on net area = $131.7N/mm^2 < 147$ (safe)

Shear in web = 57.0 < 100.0 (safe)

Strength of 22mm dia rivet:-

In double shear = 86.75kN

In bearing on 10mm plate = 54.52kN

Rivet Value = 54.52kN

Horizontal shear per mm = $VA\bar{y}/I$ = 497N/mm

Vertical shear per mm = 187N/mm

Pitch of rivets in upper flange = 102.5~100mm

Pitch of rivets in lower flange = 109.7~100mm

Stringer Bracings

Lateral load per meter due to:-

Racking force = 5.88 kN/m

Wind load on moving train = 5.25 kN/m

Wind load on exposed area of stringer above lower chord = 0.675 kN/m

Total = 11.805 kN/m

Total lateral load = 11.805 × 6 = 7083 kN.

Approx. length of diagonal = 2.76 m.

Force in end lateral = 34.30kN

Try ISA 75 ×75 × 8 mm, giving

A_0= 11.38 cm^2

R_{min}= 1.45cm, l/r = 133.5

Increased permissible stress when

wind load is considered = 61.3N/mm^2

Area required = 559.5mm^2 < 11.38cm^2 (safe)

Rivet value of 22mm rivet in single shear = 43.375kN

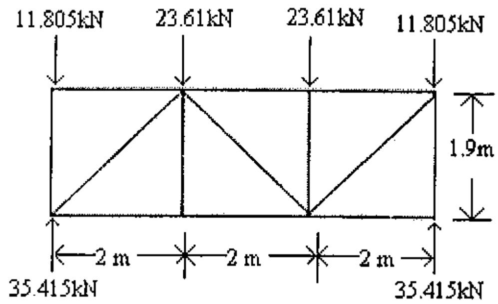


Fig. 2.5: Stringer Bracing

No. of rivets used = 2

Checking Stringer Design for Racking and Wind Effect

Racking force = 5.88kN/m

Lateral wind load on moving train = 5.25 kN/m

Longitudinal load = 36.77kN/m

Overturning effect of lateral load acting at 2.65m above top of stringer = 7.31kN/m

Additional load on the stringers due to the overturning effect of the wind force on the train is clear from the Fig. 2.6

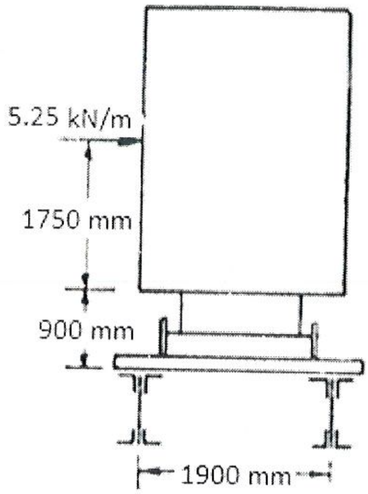


Fig. 2.6 : Additional loads on the stringer

Additional stresses in stringer flanges will be as follows:

(i) Direct stress in top flanges due to racking force = 1.93 N/mm^2

(ii) Direct stress in top flanges due to lateral wind force = 1.72 N/mm^2

(iii)Direct stress in each stringer due to longitudinal forces = 5.68N $/mm^2$ (compression)

(iv)Bending stress due to longitudinal force

(a) in top flange = 8.93 N/mm^2 (compression)

(b) in bottom flange = 10.17 N/mm^2 (tensile on net area)

(v) Bending stress due to overturning effect

(a) in top flange = 7.11 N/mm^2 (compression)

(b) in bottom flange = 8.09 N/mm^2(tensile on net area)

Table 2.1: Combination of stresses (Tensile '+' , Compressive '-')

Stress due to	Top flange (N/mm^2)	Bottom flange (N/mm^2)
Dead and Live Loads	-115.7	+ 131.7
Racking Force	-1.93	-
Lateral wind force	+1.72	-
Overturning effect	-7.11	+8.09
Longitudinal force	-5.68	-5.68
a) Direct Comp.	-8.93	+10.17
Total	-137.63	+144.28

Increased permissible stress= $1.167 \times 147 = 171.5$ N/mm^2 *(OK)*

Therefore, Design of stringer is satisfactory.

Weight of Stringers and its bracings per panel

Density of steel = 77 kN/m^3

	Weight (kN/m)
Main girder area= 194.28cm^2	17.95
3 Nos ISA 75×75×8, 2.25m	0.72
2 Nos ISA 75 × 75 × 8, 1.90m	0.33
Total	19.00 kN/m

Design of Floor Beams

Effective span = 5.25 m

Spacing of beams = 6.0 m

Loaded length for maximum B.M. and S.F. = 12.0 m

Loading	*For bending*	*For shear*
Live load reaction from stringer	307.0 kN	339.8 kN
Impact @ 0.594	182.4 kN	201.8kN
DL from stringer	9.5 kN	9.5 kN
DL from track @ 266 kg/m	7.8 kN	7.8 kN
Total	506.7 kN	558.9 kN

Assuming the self weight of girder to be 25 kN

Maximum B.M. = 865.1 kNm

Maximum S.F. = 571.4 kN

Using the section as shown Fig. 2.7

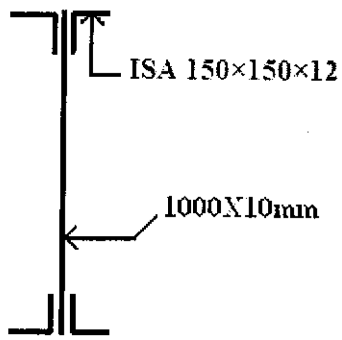


Fig. 2.7: Floor Beams

Flange element	*Gross area,cm²*	*Net area,cm²*
2ISA 150×150×12	69.18	63.54
Equivalent web	16.67	12.50

Total	85.85	76.04

$I = 3773 \times 10^6$ mm^4

Shear stress in web = 57.14 N/mm^2 < 100 N/mm^2 (OK)

Bending stress on net area = 129.4 N/mm^2

Unsupported depth of web/ Thickness of web = 75

Hence F_b = 147 N/mm^2 > 129.4 N/mm^2, therefore safe.

Also no transverse stiffeners are required for $d/t < 75$.

Horizontal shear between angles and web per unit length = $VA\bar{y}/I$ = 424.9 N/mm

Rivet value of 22 mm dia. Rivets = 54.52 kN.

Pitch = 128.3 mm or 120 mm.

Connection between stringer and cross girder

Maximum reaction from stringer = 427.6 kN

Try two angle cleats ISA 150 × 115 × 10

Area = 51.04 cm^2

I = 2450 cm^2

r = 6.95cm

Allowable comp- stress = 135.8 N/mm^2

Actual stress = 83.8 N/mm^2 < 135.8 (OK)

Strength of 22 mm rivet in single shear = 43.375 kN

in double shear = 86.75 kN

bearing on 10 mm = 54.52 kN

Number of rivets connecting cleat angles with stringer web= 7.84

As 12 mm thick fillers will be used, total number of rivets

$$= 1.15 \times 7.84 = 9$$

Number of rivets connecting cleat angles with cross girder

when one panel length is loaded = 12

when two adjacent panels are loaded = 12

Therefore,Use 12 rivets.

Weight of cross-girder and panel-point dead load

Area of cross-section = $238.36\ cm^2$

Length = 5 m

Weight of steel = $77.0\ kN/m^3$

Weight = $(238.36/104) \times 5 \times 77.0 = 9.18$ kN.

Dead weight at each panel point of each truss

due to cross-girder = 4.59 kN

due to stringer & bracing = 9.50 kN

due to track = 7.80 kN

Dead Loads

Maximum tensile force in the tension chord due to live load and impact = 1241 kN

Net area of tension chord = $8928\ mm^2$

Weight of main trusses and bracing = 6.874 kN/m

Weight of trusses and bracing per truss = 3.436 kN/m

Weight of flooring system = 3.648 kN/m

Total weight per truss per m = 7.085 kN/m

Table 2.2: Member forces due to DL, LL and Impact

Member	Impact factor	Area of influence line	Load (kN)		Force in members (kN)			Total force
			Dead load	Live load	Dead load	Live load	Impact	
L_0L_1 L_1L_2	0.341	12.8	9.0	40.06	115.2	512.8	174	802.9
L_2L_3	0.341	23.1	9.0	40.06	207.9	925.4	315.6	1448.9
U_1U_2 U_2U_3	0.341	-20.5	9.0	40.06	-184.5	-821.2	-288.0	-1285.7
L_0U_1	0.341	-19.8	9.0	40.06	-178.2	-793.2	-270.5	-1241.9
U_1L_2	0.756	-.79	9.0	69.78	106.7	-55.1	-41.7	-9.9
	0.38	12.65		44.91		568.1	215.9	870.9
L_2U_3	.543	3.17	9.0	52.40	-35.6	166.1	90.2	220.7
	0.44	-7.12		47.96		-341.5	-150.3	-527.4

Panel L_0L_1

Loaded length = 36 m

Longitudinal load

Tractive effort = 735.5

Force per chord = 367.8 kN

Panel L_1L_2

Loaded length = 30 m

Longitudinal load

Tractive effort = 637.4

Force per chord = 318.7 kN

Panel L_2L_3

Loaded length = 24 m

Longitudinal load

Tractive effort = 588.4 kN

Force per chord = 294.2 kN

Wind Loads

Assume 350 mm deep chord, 300 mm wide diagonals, 250 mm wide verticals.

Wind pressure = 1.50 kN/m^2

Wind load on top chord = 36.6 kN

Wind load on bottom chord = 295.3 kN

Table 2.3 : Area of members for wind load

Details of area exposed	Top chord	Bottom chord
Projected area of stringer and track upto rail level = 1.25x36	-	45.00
Area above rail level and below moving load		
1. Verticals		0.75
2. Diagonals		0.95
3. End points		0.56
Top chord	8.40	-
Area above moving load and below top chord		
1. Verticals	2.06	-
2. Diagonals	2.61	-
3. End points	1.52	-
Gussets @ 20% of top chord area	1.68	-

Total	16.27	47.26
Exposed area of moving load		126.00

Horizontal bending of bottom laterals

Load on each node = 49.22 kN

Reaction = 147.66 kN

Forces in chord members :

L_0L_1 = 70.3 kN

L_1L_2 = 182.6 kN

L_2L_3 = 239.0 kN

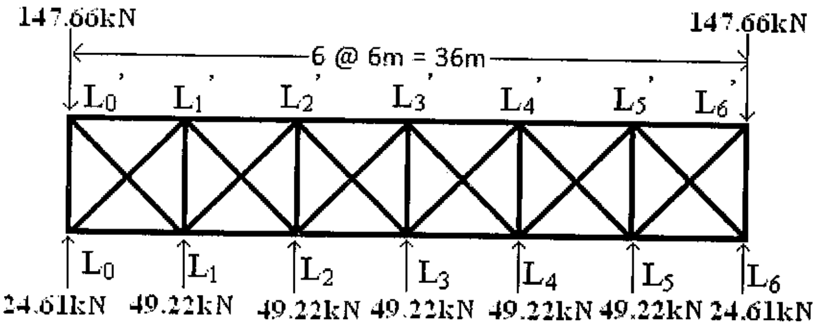


Fig. 2.8 : Bottom Lateral

Table 2.4 : Overturning effect

Detail	Force due to lateral wind (kN)	Lever arm above bottom bracing (m)	Moment (kN-m)
Stringer load track etc.	106.3	0.625	66.5
Moving load	189.0	3.60	680.4

Top chord	36.61	7.175	262.7
			1009.6

Additional vertical load per meter on leeward girder = 5.342 kN/m

Additional forces in all members of the truss = 59.35% of DL forces

Horizontal bending of top chord

Horizontal bending of top chord is similar to horizontal bending of lower lateral system. Forces in upper chord members due to horizontal bending of top laterals and due to overturning effect are opposite in nature. The force with larger magnitude is kept of the same sign as due to DL+ LL and the smaller force may be of opposite nature.

Total wind on top chord = 36.6 kN

Load on intermediate panel point = 36.6/ 4 9.15 kN

Force in U_1U_2= $\pm$ 1/5.25 (18.30-4.575) $\times$ 3 = $\pm$ 7.84 kN

Force in U_2 U_3 = $\pm$ 1/5.25(13.725 $\times$ 9-9.15 $\times$ 3) = $\pm$ 18.28 kN

Portal Effect

Although in design of the end portal system, a lateral shear equal to 3/2 % of the compression force in the two end posts is assumed in addition to the wind forces, only the wind force is accounted for in designing the bottom chord members.The positions of knee braces in has been fixed so as not to interfere with clearance diagram

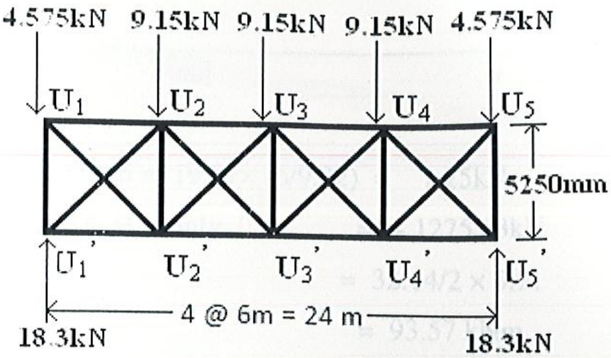


Fig. 2.9 :Top Lateral

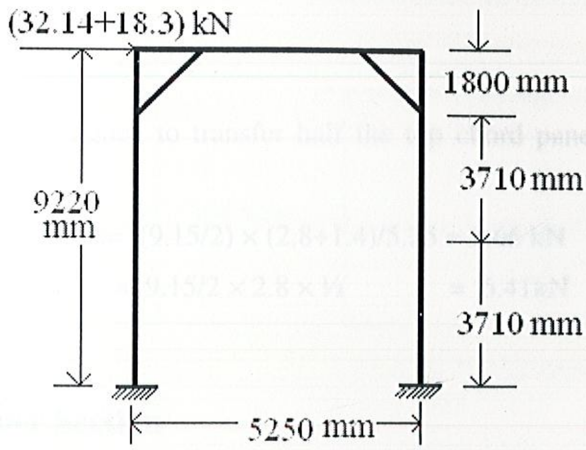


Fig.2.10 : Knee Bracing

Additional lateral load on portal due to 9/4% of force in two end top chords

= 2× 1285.7× (1.25/100) = 32.14 kN

Wind load reaction from top = 18.30 kN

Total = 50.44 kN

Additional axial load in end post due to

(a) Wind force = 18.3(3.71+1.80)/ 5.25 = 19.21 kN

(b) 9/2 % of end chord forces = 32.14(3.17+1.80)/5.25

	= 33.73kN
Total	= 52.94kN

Additional force in bottom chord = 19.21× (6/9.22) = 1.25kN

Total forces in L_0U_1 due to DL+LL only = - 1275.63kN

B.M. in L_0 U_1 without wind = 32.14/2 × 3.71

= 93.57 kNm

B.M. in L_0 U_1 with wind = 50.44/2× 3.71

= 93.57kNm.

Sway effect

The sway bracing is designed to transfer half the top chord panel point wind load to bottom bracing.

Additional force in verticals= (9.15/2) × (2.8+1.4)/5.25 = 3.66 kN

B.M. in verticals = 9.15/2 × 2.8 × ½ = 6.41kN

Design of Member Section

Forces in all the members are calculated due to DL+LL+Impact+Lateral and Longitudinal forces. 16.67% higher stresses are permissible when lateral and longitudinal loads are considered.

Members L_0L_1 and L_1L_2

The section of L_0L_1 is designed to consist of two web plates and four angles. The outstanding legs of angles are turned inside to facilitate the connection of floor beams with lower panel points. The floor beam is much deeper than the depth of the bottom chord and it is desirable that the outer surface of the bottom chord be smooth without projection.

Force without occasional load = + 802.9 kN

Force with occasional load = + 1385.1 kN

Area required without occasional load = $802.9\times1000/139 = 5776\text{mm}^2 = 57.76\text{cm}^2$.

Area required with occasional loads = $(1385.1\times1000)/(139\times1.167) = 8539\ \text{mm}^2$.

	Gross area, cm^2	Net area, cm^2
2 plates 300× 10m	60.0	2(30-3 ×2.35) 1.0 = 45.90
4 ISA 100 ×75 ×10	66.0	66-4 × 1 ×2.35 = 56.60
		102.50

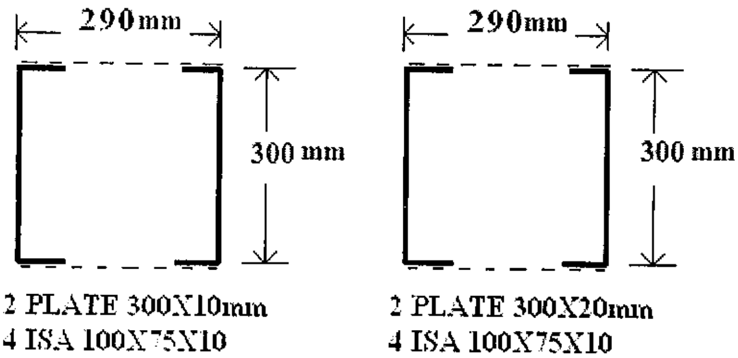


Fig. 2.11 : Bottom Chord

Member L_2L_3

Angles in all the bottom chords are of the same size, but the web plate for L_2L_3 is thickened to provide more area. Clearance between toes of the angles should be at least 100 mm to provide for fabrication and painting. Thus clear distance between the gusset plates is 290 mm. The distance is kept constant for all the main members of truss.

Force without occasional loads = 1448.9 kN

Force with occasional loads = 2118.0 kN

Area required without occasional loads

$$= (1448.9 \times 1000)/(139 \times 100) = 104.2 \text{ cm}^2$$

Area required with occasional loads

$$= (2118 \times 1000)/(139 \times 100 \times 1.167)$$

$$= 130.6 \text{ cm}^2$$

	Gross area, cm^2	Net area, cm^2
2 plates 300 ×20 mm	120.0	2(30-3 ×2.35)2.0
		= 91.80
4 ISA 100 × 75 × 10	66.0	66-4× 1.0 ×2.35
		= 56.60
		148.40

Member U_1U_2 or U_2U_3

The clear distance between gusset plates is 290 mm to match the lower chord. Assuming 16 mm thick gusset plates, the clear distance between the web plates of top chord will be 322 mm . Using the 10 mm thick web plates and top angles as ISA 75 × 75 × *10,* the minimum width of the cover plate is 500 mm and the distance between rivet lines is 385 mm.

The lower angles will be connected together by latticing. The latticing is designed for 2i% of the axial force in the member. Thus shear force i, shared by two planes and the latticing, connecting lower flanges, will be designed for a shear force equal to 9/4% of the axial load,

Force without occasional load = 1285.7 kN (comp.)

Force with occasional loads = 1550.6 kN (comp.)

This is 20.6% more than 1285.7 kN hence governs the design.

Assume allowable stress = 120 N/mm^2.

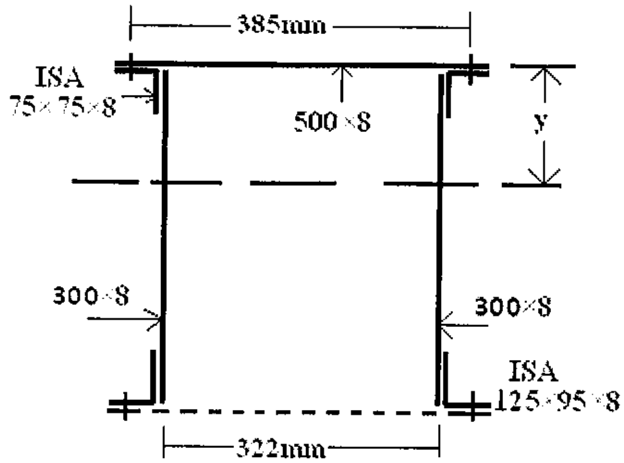


Fig. 2.12: Section of U_1U_2

Required area = (1550.6 × 1000)/(120 × 100) = 129.2 cm^2.

Try a section as shown in Fig. 2.12

b/t for cover plate = 385/8 = 48.1 < 50

Thus the whole width is effective.

Element	Gross area(cm^2)	Lever arm from top fibre(cm)	Moment of area (cm^3)
Cover plate 50 × 0.8	40.00	0.40	16.0
2 ISA 75×75×8	22.76	2.94	66.9
2 Web plates 30×0.8	48.0	15.80	760.0

2 ISA	33.96	28.49	966.0
	144.72		1808.9

Y= 1808.9/144.72 = 12.5 cm.

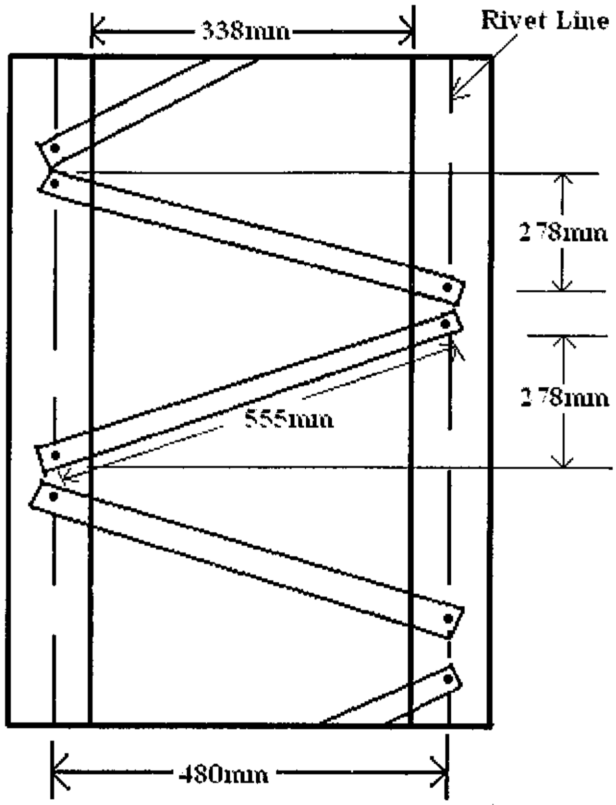


Fig. 2.13: Latticing

I_{xx} = 50×0.8 (12.5-0.4)^2 + 2× 0.8(30)^2/12+2×0.8 ×30 (15.8-12.5)^2 +2×59.0+

2 × 11.38 (12.5-2.94)^2 + 2 × 133.3+2 × 16.98 (28.-49-12.5)^2

= 21,096 cm^2

I_{yy} = 0.8(50)^3/8 + 2 × 30 ×0.8 (16.1+04)^2+2 × 59.0 +2 ×11.33

×(16.9+2.14)^2 +2 × 266.0+2 × 16.98 (16.9+3.80)^2

$= 49040\ cm^4$

$r_{min.} = (21096/144.72)^{1/2}$ $= 12.1$ cm.

Effective length 0.85×600 $= 510$ cm

$l/r = 510/12.1$ $= 42.2$

Permissible compressive stress $= 125.8$ N/mm2

Area required = $(1550.6\times1000)/(125.8\times100\times1.167) = 105.6cm^2$

$< 144.72\ cm^2$ *OK*

Shear force in latticing = $1/2(2.5/100\times 1550.6)$ $= 19.38$ kN

Force in lattice bar (at 60 to axis) = $19.38/\sin 60$ $= 22.38$ kN

Effective length $= 48/\sin60$ $= 55.5$ cm

Try ISA $75 \times 50 \times 8.0$ giving $A = 9.38\ cm^2$

$r_{min.} = 1.06$ cm

$l/r = 55.5/1.06 = 52.4$

Allowable stress (Table 16-3) = $120.2\ N/mm^2$ (for normal load)

Permissible load = $120.2\times100\times9.38/1000$ $= 112.7$ kN *OK*

Strength of 22 mm dia. rivet in single shear $= 43.38$ kN

Strength of 22 mm dia. rivet in bearing on 8 mm plate

$= 8 \times 23.5 \times 232 = 43616\ N = 43.62$ kN

For 22.38 kN force, use one rivet at each end.

Member U_1L_2

The outer distance between channels is kept 290 mm as shown in Fig. 16-49 so that these fit in the gusset plates.

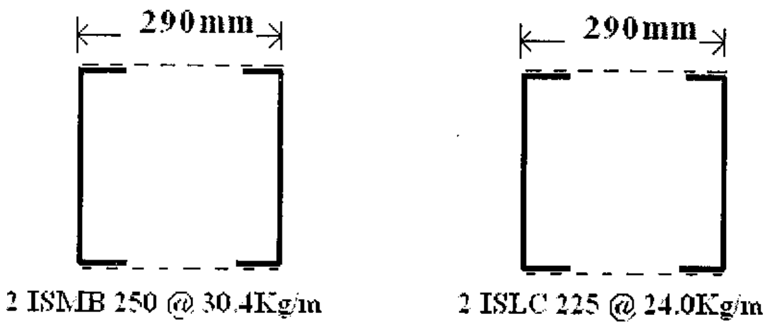


Fig. 2.14 : Member U_1L_2 and L_2U_3

Force without occasional load = 890.7 kN (tensile)

Force with occasional loads = 954.0 kN (tensile)

Area required = (890.7×1000)/(139 × 100) = 64.1 cm^2.

Use two ISMC 250 @ 0.298 *kN/m*

Net area = 2×38.67- 4 × 2.35 × 0.71 = 70.66 cm^2.

Member L_2U_3

The member L_2U_3 is subjected to reversal of stress. As this is primarily a compression member, no effect of fatigue is considered.

Force without wind = + 220.7 kN or - 527.4 kN

Force with wind = + 241.8 kN or - 548.5 kN

Let allowable compressive stress = 120 N/mm^2.

Area required = (527.4 × 1000)/(120 × 100) = 43.95cm^2 .

Try 2 ISLC. 225 @ 24,0 *kg/m* as shown in Fig. 2.14.

Gross area = 2 × 30.53 = 61 .06 cm^2.

r_{xx} = 9.14 cm

I_{yy} = 2 [209.5+30.53 $(14.5-246)^2$] = 9300 cm^4.

$r_{yy} = \quad (9300/61.06)^{1/2} \quad = 15.2$ cm

Length of member = 9.22 m

$l/r = 0.7 \times 922/9.14 = 71$

$F_c = 106.7$ N/mm^2

Area required = $(527.4\times1000)/(106.7\times100) = 49.4$ cm2 < 61.06 cm^2 *OK*

Lacing can be designed in the same manner as for U_1U_2 .

Member U_1L_1 or L_3U_3

Force without wind = + 595.6 kN (tensile)

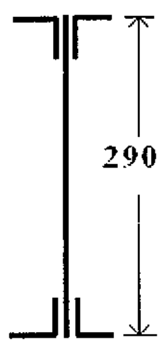
Force with wind = + 631.3 kN

moment = 6.41 kNm

Net area = $(595.6 \times 1000)/(139 \times 100) = 42.85$ cm^2.

Section is chosen as shown in Fig. 2.15.

Element	Net area, cm^2
Plate 290 × 8	(29.0-2×2.35)0.8 = 19.60
4 ISA 100 ×75 × 8	4 × 13.6-4×2.35×0.8 = 45.94
	Total = 65.54 OK



4 ISA 100×75×8

Fig. 2.15: Web Plates 90×8

Additional load and bending moment due to wind are very small and the section chosen will be safe for those also.

Member U_2L_3

Although the load on this member is much smaller as only the weight of top chord acts on it but the same section as for U_1L_1, and U_2L_3 will be adopted.

Member L_0 U_1

The member L_0U_1 is a compression member and the section similar to U_1U_2 could have been economical. The angles were turned inward to simplify the connection at Lo.

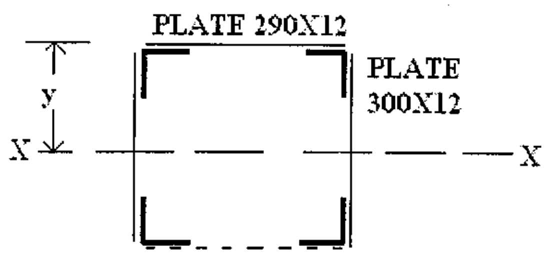
(I) Without wind, Direct load = 1275.6 kN (comp)

Bending moment = 59.62 kNm

(2) With wind, Direct load = 1400.6 kN

Bending moment = 93.57 kNm

Try the section shown in Fig. 2.16.



4 ISA 100×75×12

Fig. 2.16: Section of U_0L_1

Element	Gross area	Lever arm from	Moment

	(cm^2)	top fibre(cm)	(cm^3)
Cover plate	34.8	0.6	20.9
2 Web plates	72.0	16.2	1165.0
4 Angles	78.24	16.2	1127.0
	185.05		2455.9

$$\acute{y} = 2455.9/135.04 = 13.26 \text{ cm}$$

$$I_{xx} = 34.8\,(13.26-0.6)^2 + 2\times 1.2(34)^2/12 + 72.0\,(16.2-13.26)^2 + 4 \times 187.5$$
$$+ 2 \times 19.56(13.26-4.47)^2 + 2 \times 19.56(31.2-13.26-3.27)^2$$
$$= 23{,}750 \text{ cm}^4$$

$$I_{yy} = 1.2\,(29)^3/12 + 2 \times 30 \times 1.2\,(14.5-0.6)^2 + 4 \times 89.5$$
$$+19.56\,(14.5-1.2-2.03)^2 = 26{,}500 \text{ cm}^4$$

$r_x = (23750/185.04)^{1/2} = 11.3$ cm , $r_y = (26500/185.04)^{1/2} = 11.96$ cm

$1/r_x = 0.7 \times 922/11.3$ cm, $1/r_y = 0.85 \times 922/11.96 = 65.6$

$F_a = 111.2$ N/mm^2, $F_b = 147$ N/mm^2

$f_a = (1275.6\times1000)/(185.04\times 100) = 689.4$ N/ mm^2

$f_b = (59.62\times10^6)/(26500\times 10^4) \times 14.5\times 10 = 32.62$N/mm^2

$f_a/F_a + f_b/F_b = 68.94/11.2 + 32.62/147 = 0.620 + 0.222 = 0.842 < 1$ OK

If wind is also considered,

$f_a = (1400.6\times 1000)/(185.04 \times 100) = 75.69$ N/mm

$f_b = (93.57\times10^6)/(26500\times 10^4) \times 145 = 51.20$ N/mm^2.

$f_a/F_a + f_b/F_b$ = 75.69/111.2 = 51.20/147 = 0.681 + 0.348 = 1.029

<1.167 OK

Joint L_2

The bottom chord is not being spliced at L_2 . The larger section of L_2 L_3 is taken about I m to the left of L_2 and a splice has been provided there. Due to the continuous bottom chord at L_2, the rivets connecting the gusset plates with the bottom chord carry the difference of forces in L_1 L_2 and L_2 L_3.

The difference (L_2 L_3 – L_1 L_2) is not maximum when span is loaded.

The influence line for (L_2 L_3 – L_1 L_2) has been drawn in Fig. 2.17.

The investigation of gusset plates has been omitted here.

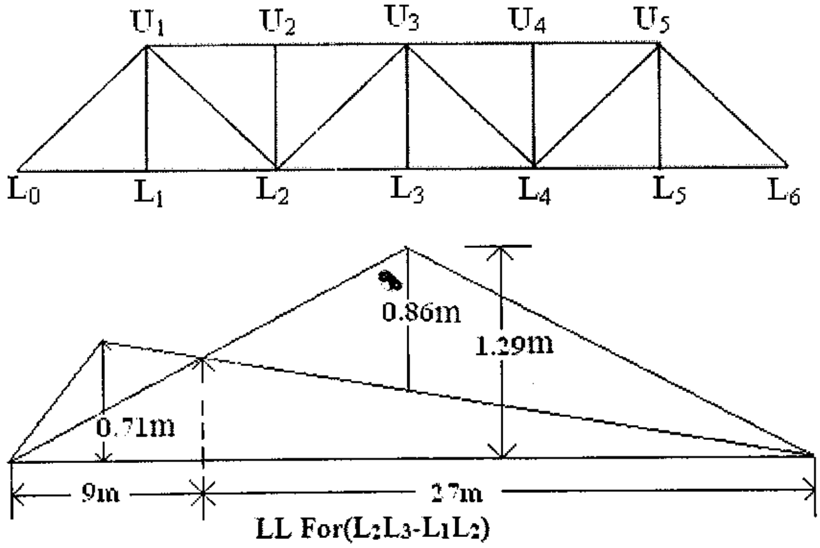


Fig. 2.17: Max. force difference in L_2L_3 and L_1L_2

For maximum force difference in L_2L_3 and L_1L_2, only 27 m length of the bridge should be loaded.

Total live load = 2253 kN

Impact factor = 0.3925

Load per truss = 2253/(2 × 27) = 41.72 kN/m

L_2L_3- L_1L_2due to LL + impact

= (1/2 ×27×O.86) × 41.72 × (1+0.3925) = 674.5kN

L_2L_3- L_1L_2 due to dead load

= 207.9-115.2 = 92.7 kN

L_2L_3 – L_1L_2 due to wind effects

= 263.5-151.2 = 112.3 kN

Total L_2L_3 – L_1L_2 without occasional loads

= 674.5+92.7 = 767.2 kN

Total L_2L_3 – L_1L_2 with occasional loads

= 767.2+112.3 = 879.5 < 1.167×767.2 kN

Design load in U_1L_2 = 890.7kN

Design load in L_2U_3 = 527.4 kN or +220.2 kN

Rivet value of 22 mm dia. rivets in single shear

= 43370 N, or 43.37 kN

Number of rivets in L_2U_1 = 890.7/43.37 = 20.5 or 22 rivets.

Number of rivets in L_2U_3 = (527.4 +220.2)/43.37 = 17.2. or 20

Number of rivets in bottom chord = 761.2/43.37 = 17.7 or 20

Floor beam reaction = 571.4 kN

Number of rivets connecting floor beam cleat angle with bottom chord

= 571.4/43.37 = 13.2 or 14

Rivet value in bearing on 10 mm plate

= 10 × 23.5 × 232 = 54250 N = 54.25 kN

Number of rivets connecting floor beam cleats with web

$$= 571.4/54.25 \quad = \quad 10.5 \text{ or } 11$$

Total actual number of rivets used is shown in the drawing of the girders.

Joint U_3

The top chord will be spliced at this joint with the help of top cover plate, splice plate, bottom angle splice plates and the gusset plates. The rivets are distributed in the splice material in the same proportions as the area they splice.

The cover plate and outstanding legs of the top angles will be spliced with cover plate splice. The outstanding legs of the bottom angles will be spliced with separate plates. The remainder of the section will be spliced with gusset plates and additional vertical splice plates.

The actual stress in top chord = $1285.7 \times 1000/14470$

$$= 88.9 \text{ N/mm}^2.$$

Number of rivets on member L_2U_3 or U_3L_4 as on lower chord = 20

Number of rivets on member U_3L_3 = 595.6/43.37 = 13.7 or 14

Joint L_0

Force in L_0L_1 without occasional loads = 802.9 kN

Force in L_0L_1 with occasional loads = 1321.9 kN

Force in L_0U_1 without occasional loads = 1275.6 k.N

No. of rivets in L_0L_1 = $1321.9/(1.167 \times 43.37)$ = 26.1 or 27

No. of rivets in L_0U_1 = 1275.6/43.37 = 29.4 or 30.

Joint U_1

No. of rivets in U_1L_0 = 1275.6/43.37 = 29.4 or 30.

No. of rivets in U_1L_1 = 595.6/43.37 = 13.7 or 14

No. of rivets in U_1U_2 = 1285.7/43.37 = 29.6 or 30.

Portal Bracing

Lateral load = 50.44 kN

Vertical load due to overturning moment = 52.94 kN

Bending moment at D or E (Fig. 2.18)

$= 25.22 \times 5.51 - 52.94 \times 1.25 = 72.79$ kNm

Force in EF (By equating moments at G in EG to zero)

= + 51.98 kNm

Force on CD = -102.4kN

Force in DE = -25.22 kN

Perpendicular CM = $1.80 \cos\theta$

= 1.027

Forces in knee brace BD = 135.3 kN

Length of knee brace = 2.19 m

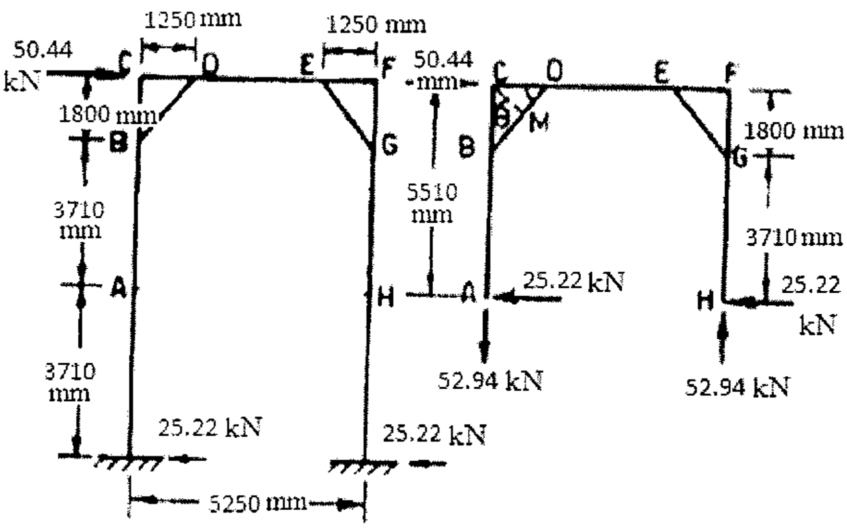


Fig. 2.18 : Portal Bracing

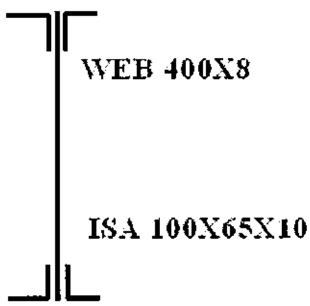


Fig. 2.19 : Section of CF

Choose section for CF as in Fig. 2.19

I_{xx} = 25.370 cm^4.

$I_{yy} = 4 \times 153.2 + 4 \times 15.51\,(3.37+0.4)^2 = 1493\ cm^4$.

$A = 4 \times 15.51 + 40 \times 0.8 = 94.04\ cm^2$.

r_{yy} = 3.98 cm

l/r = 525/3.98 = 132.

F_a = 53.6 N/mm

f_a = 10.89 N/mm^2.

f_b = 57.38 N/mm^2.

C_s = 187.5 N/mm^2.

F_b (Table 16-4c) = 73.5 N/mm^2.

$f_a/F_a + f_b/F_b$ = 0.203 + 0.781 = 0.984 <1.167 OK

Design of knee braces BD and EG :

Maximum force in knee brace = ± 135.3 kN

Area provided = 2 × 11.38 = 22.76 cm^2.

r_x = 2.28 cm

Effective length = 0.7×2.19 = 1.53 m

l/r = 153/2.28 = 67.2

F_a = 109.9 N/mm^2

Actual compressive stress = 59.45 N/mm^2. OK

Sway Bracing

The procedure of design will be same as for portal bracing.

Use 4 Nos. ISA 90 ×60 × 8 mm as shown in Fig. 2.20 for horizontal member and single angle 75 × 75 × 8 mm for knee braces.

4 ISA 90X60X8

Fig.2.20 : Knee Bracing

Top Lateral Bracing

Forces on top lateral bracing due to wind are shown in Fig. 2.21

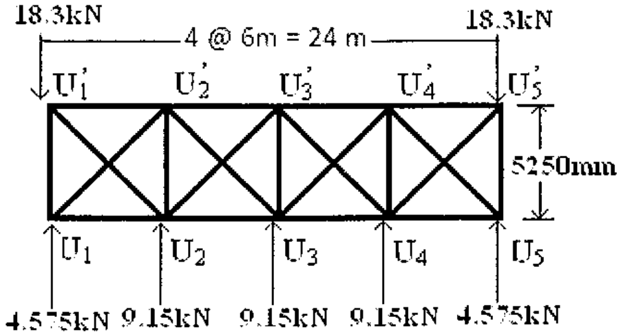


Fig. 2.21: Top Lateral Bracing

1.25% of top chord force = $2 \times 1285.7 \times 3.5/100$ = 64.29 kN

Shear due to wind = 18.30 - 4.575 = 13.725 kN

Total shear = 64.29 + 13.73 = 78.02 kN

Length of U_1U_2' = 7.98 m

Force in U_1U_2 = 59.29 kN

Use the section shown if. Fig. 2.22

$r_{\cdot}$ = 3.07 cm

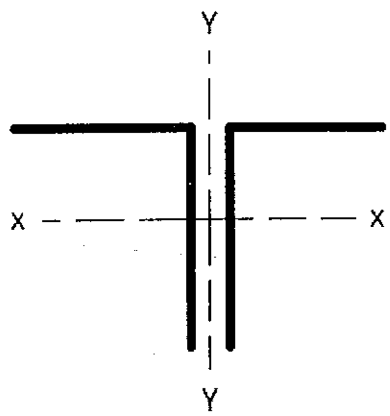


Fig.2.22 : Section of Top Lateral Bracing

Using 10 mm gusset plate,

$I_y = 2 \times 145.1 + 2\times 15.39 \times (2.76+0.5)^2 = 613cm^4.$

$F_a = 55N/mm^2.$

$f_a = 19.26\ N/mm^2.$ OK

Bottom Lateral Bracing

(a) Forces due to wind (Fig. 2.23)

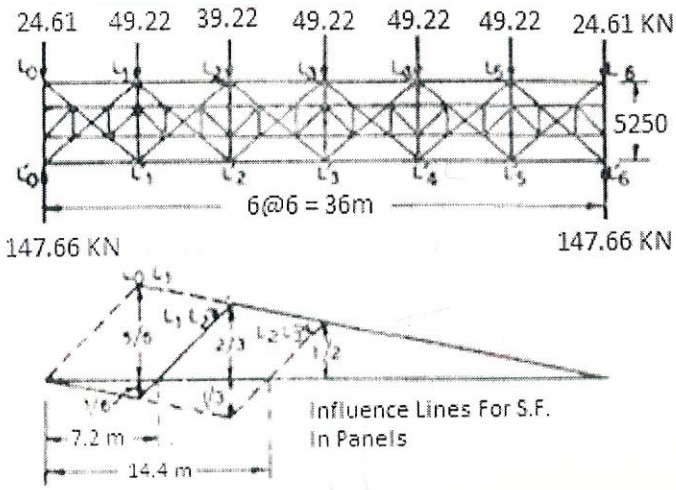


Fig. 2.23: Bottom Lateral Bracing

Shear force in panel L_0L_1 = 147.66-24.61 = 123.05 kN

Shear force in panel L_1L_2 = 123.05-49.22 = 73.83 kN

Shear force in panel L_2L_3 = 73.83-49.22 = 24.61 kN

Forces due to racking force of 5.88 kN/m (moving load).

Panel L_0L_1

Loaded length = 36 m

Shear force = 88.2 kN

Panel L_1L_2

Panel length = 28.8 m

Shear force = 56.45 kN

Panel L_2L_3

Loaded length = 21.6 m

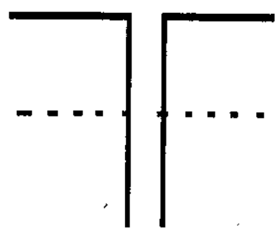
Shear force = 31.75 kN

Total shear force in panel L_0L_1 = 123.05+88.2 = 211.25 kN

Total shear force in panel L_1L_2 = 73.83+56.45 = 130.28 kN

Total shear force in panel L_2L_3 = 24.61+31.75 = 56.36 kN

Length of bracing member = 7.98 m



2 ISA 180X180X8

Fig. 2.24: Section of Floor Beams

Member Forces due to longitudinal forces.

In each panel, the longitudinal force is transferred from

stringers to trusses through bottom lateral braces.

Longitudinal force in one panel will not affect the bottom lateral bracing in other panels.

Loaded length = 6 m

Longitudinal force = 220.6 kN

Force in bracing member = 73.4 kN.

Max. force in L_0L_1' or $L_0'L_1$ = 160.6+73.4 = 234.0 kN

Max. force in L_1L_2' or $L_1'L_2$ = 99.01 +73.4 = 172.41 kN

Max. foree in L_2L_3' or $L_2'L_3$ = 42.83 × 73.4 = 116.23 kN

Effective length[12] = 7.98/5.25 × 1.875 = 2.84 m

Try 2 ISA 110 × 110 × 8 giving A = 2 × 17.02 cm^2

r = 3.38 cm.

l/r = 284/3.38 = 84

F_a = 95 N/mm^2.

f_a = (234.0 × 1000)/(2 × 17.02 × 100) = 68.7 < 1.16 ×95 N/mm^2 OK

CHAPTER 3

ANALYSIS AND DESIGN OF THE TRUSS BRIDGE USING STAAD.pro

GENERAL

In the present chapter stepwise procedure followed to analyse the truss bridge using STAAD.pro is explained. After analysis, code check was performed to verify whether these members follow IS-code requirements or not.

3.1 Geometrical Input

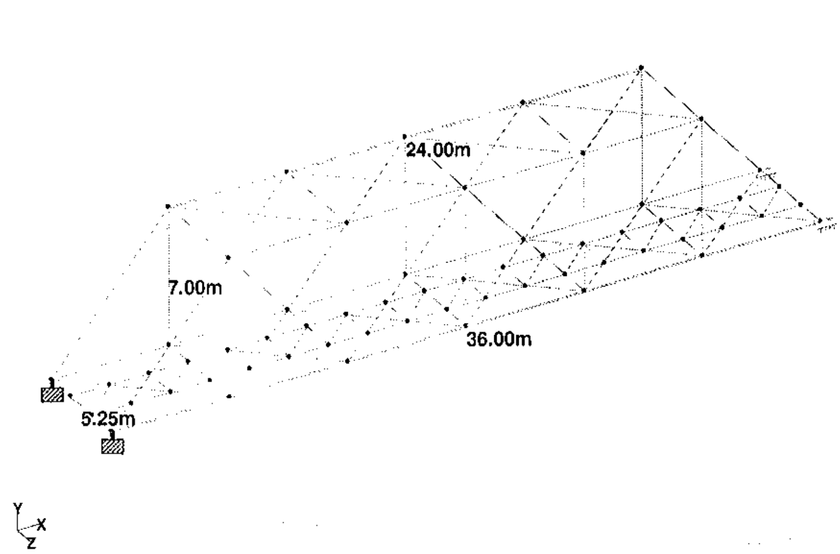


Fig 3.1: Overall Dimensions of the Truss Bridge.

3.2 Steel Sections were drawn in SECTION WIZARD and then the same transferred to STAAD.pro as a User-table.

3.3 Member grouping was carried out as

- **3.3.1** Top Bracing
- **3.3.2** Top Chord
- **3.3.3** Bottom Bracing
- **3.3.4** Bottom Stringer
- **3.3.5** Stringer Bracing
- **3.3.6** Floor Beams
- **3.3.7** Top Laterals

Respective sections were assigned

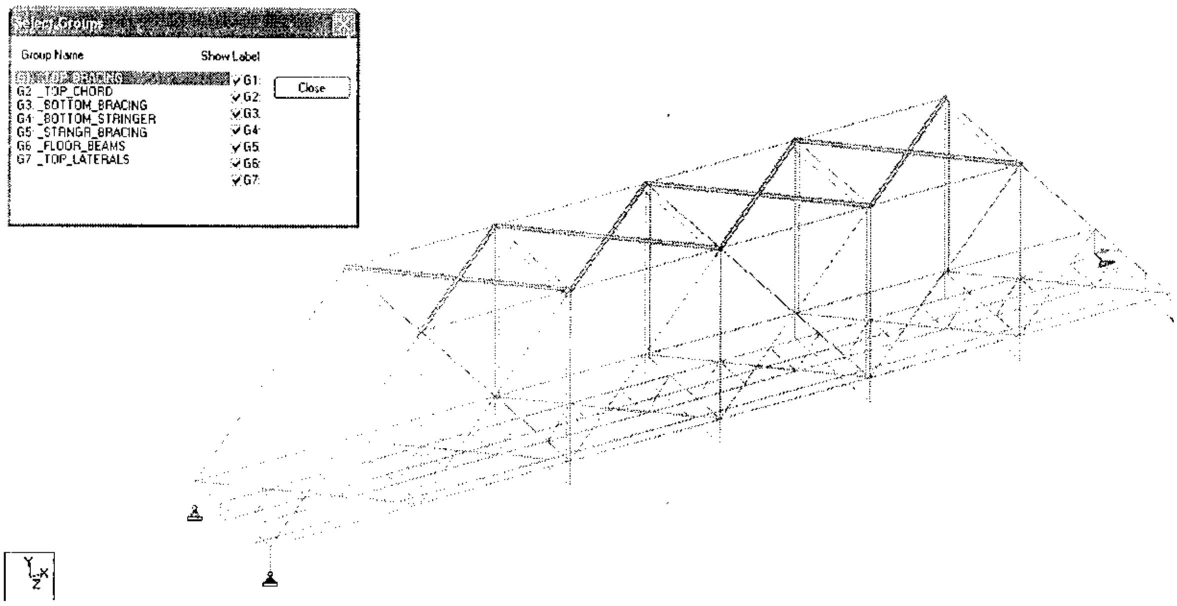


Fig 3.2: Top Bracing

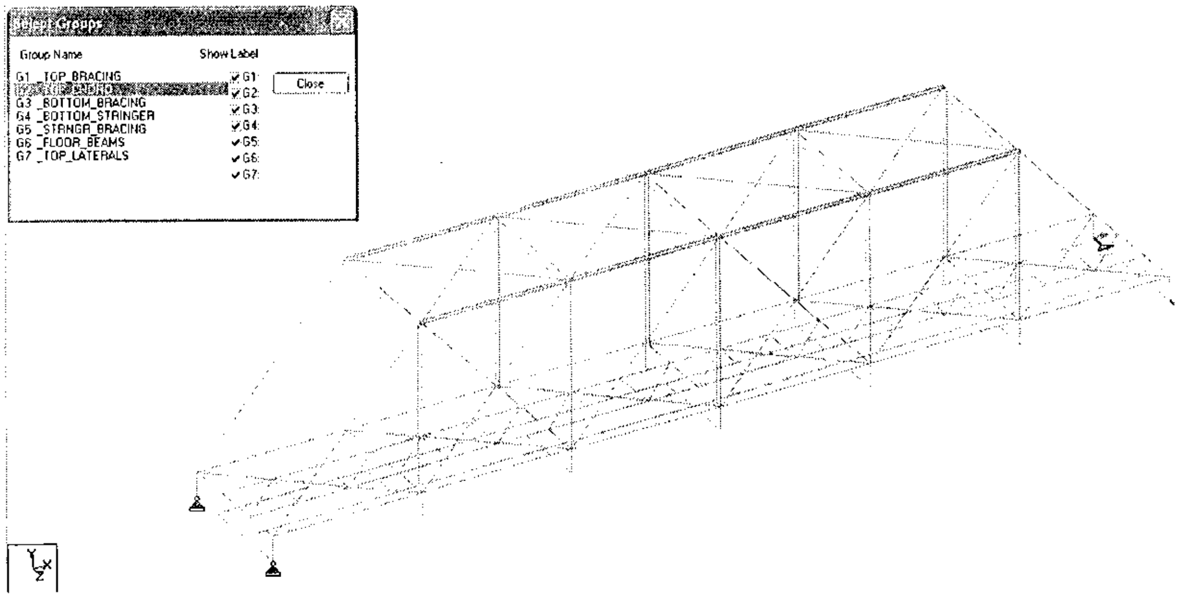


Fig 3.3: Top Chord

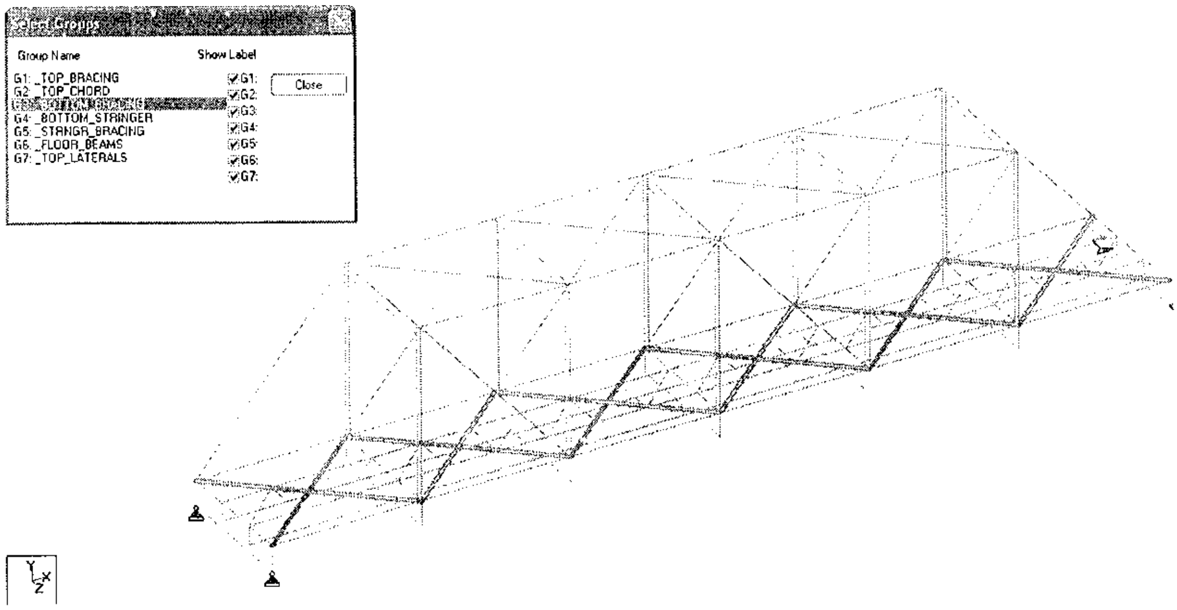


Fig 3.4: Bottom Bracing

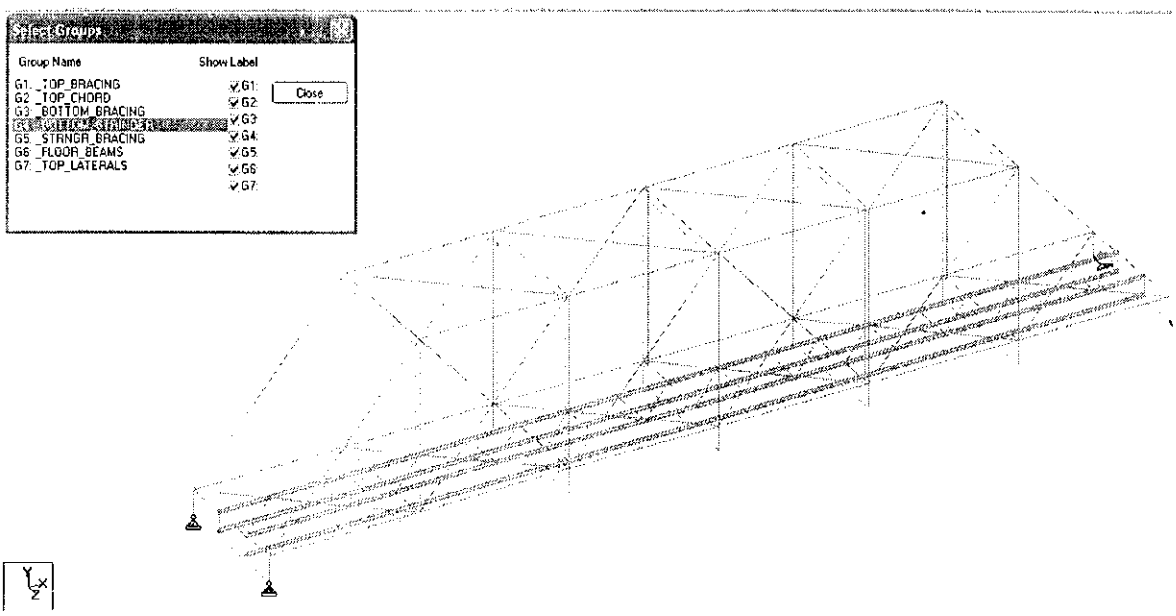


Fig 3.5: Stringer

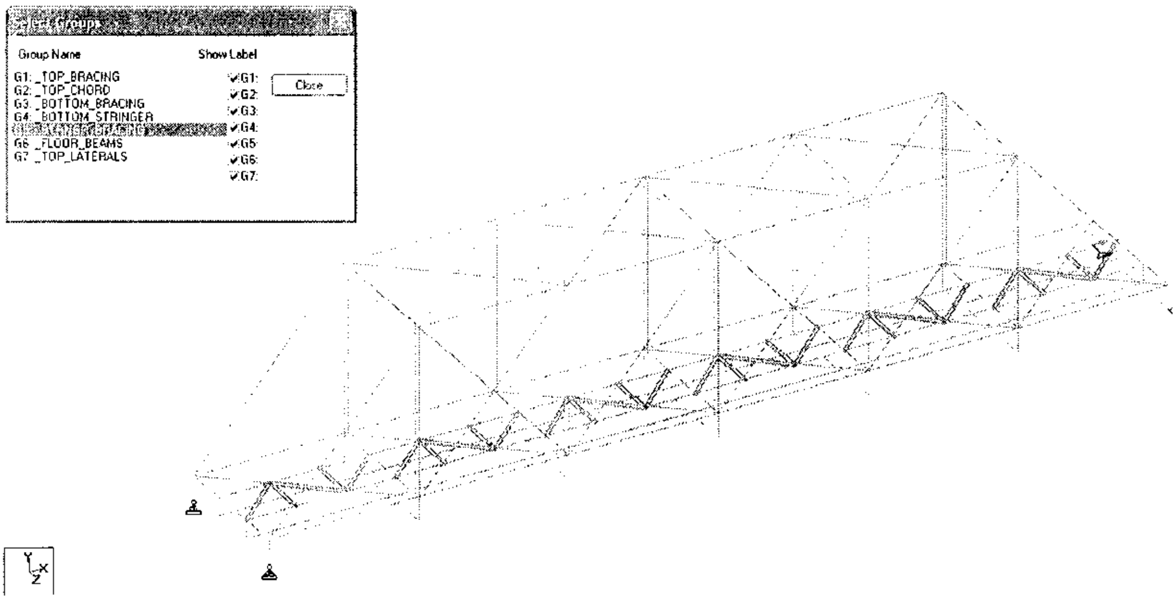


Fig 3.6: Stringer Bracing

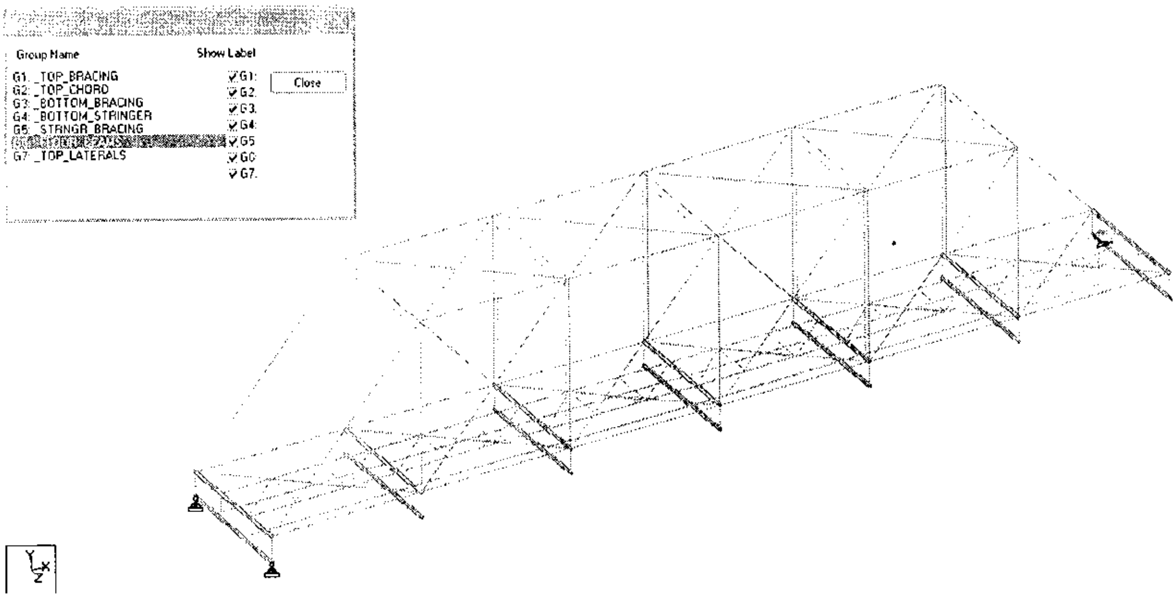


Fig 3.7: Floor Beams

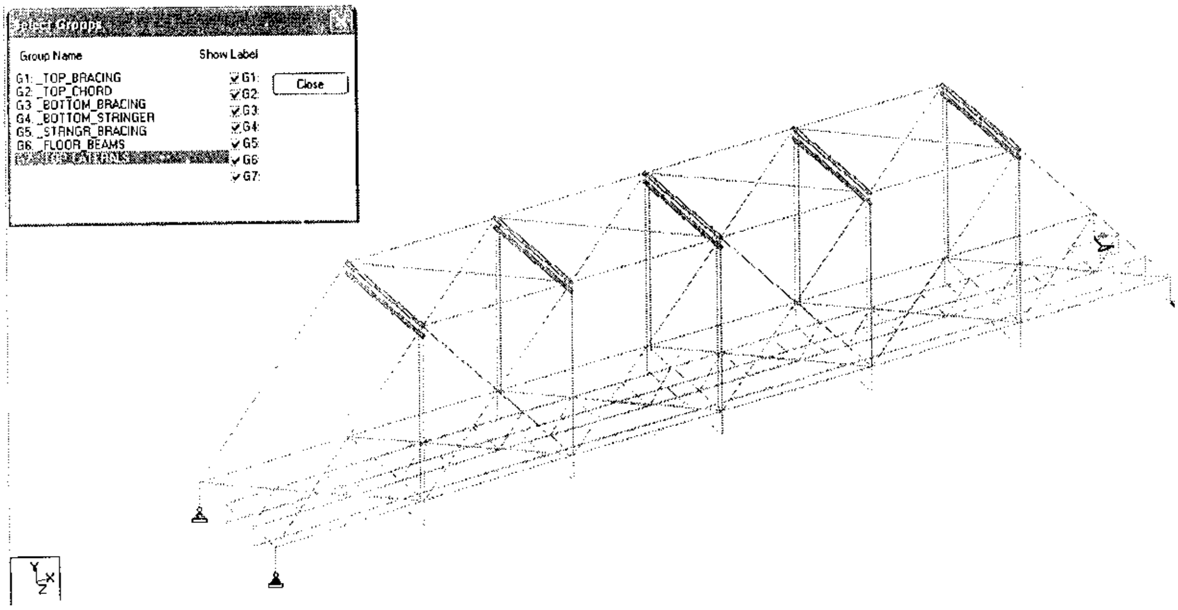
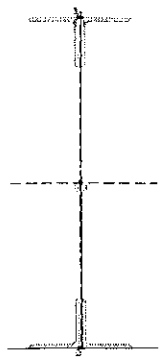
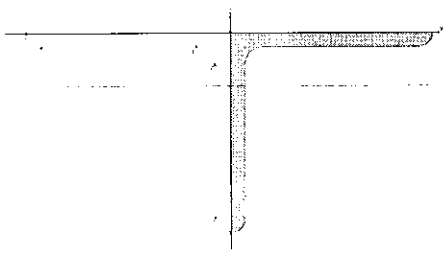
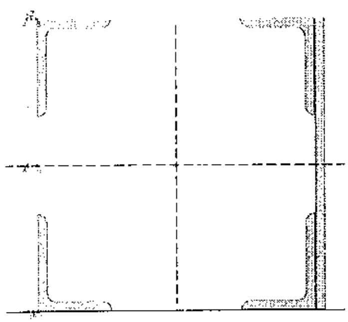
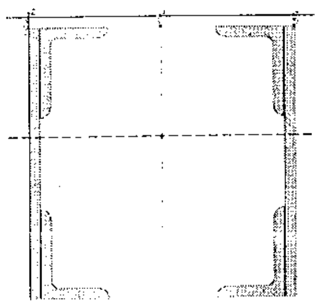
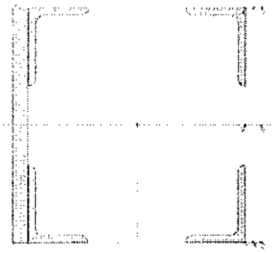
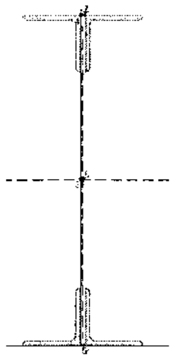
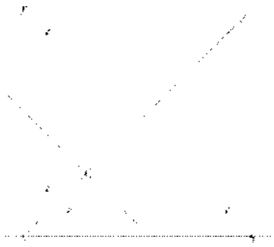
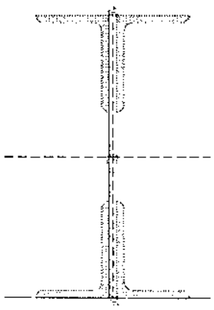
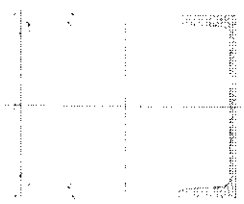


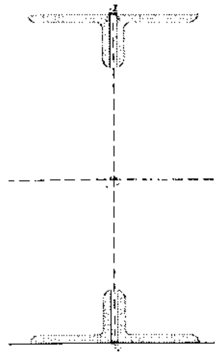
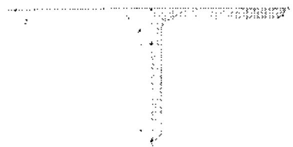
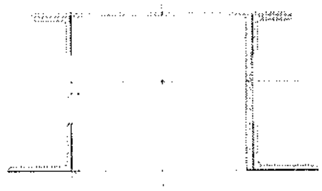
Fig 3.8: Top Laterals

3.4 Properties were assigned to the different sections drawn in section wizard.

Table 3.1: Sections drawn in section wizard

Member	Cross section	Remark
Floor beam		1 × PLATE 1000 × 10 4 × ISA 150 × 150 × 12
L_0 L_1'		2 × ISA 110 × 110 × 8
L_0 L_1 & L_1 L_2		2 × PLATE 300 × 10 4 × ISA 100 × 75 × 10
L_0U_1		1 × PLATE 290×12 2 × PLATE 300×12 4 × ISA 100×75×12

$L_2\ L_3$		2 × PLATE 300×20 4 × ISA 100×75×10
Stringer		1× PLATE 750×10 4 × ISA 130×130×12
Stringer bracing		1 × ISA 75×75×8
U_1L_1 & U_2L_2		1 × PLATE 290 × 8 4 × ISA 100 × 75 × 8
U_1L_2		2 × ISMC 250

U_1U_1'		1 × PLATE 400 × 8 2 × ISA 100 × 65 × 10
U_1U_2'		2 × ISA 100 × 100 × 8
U_1U_2 & U_2U_3		1 × PLATE 500 × 8 2 × PLATE 300 × 14 2 × ISA 75 × 75 × 8 2 × ISA 125 × 95 × 8

3.5 Loads were assigned to the sections

Flowing loads were assigned:

3.5.1 Dead Load (DL): Self Weight taken in –Y direction.

3.5.2 Live Load (LL): Weight of engine and bogie were assigned on structure

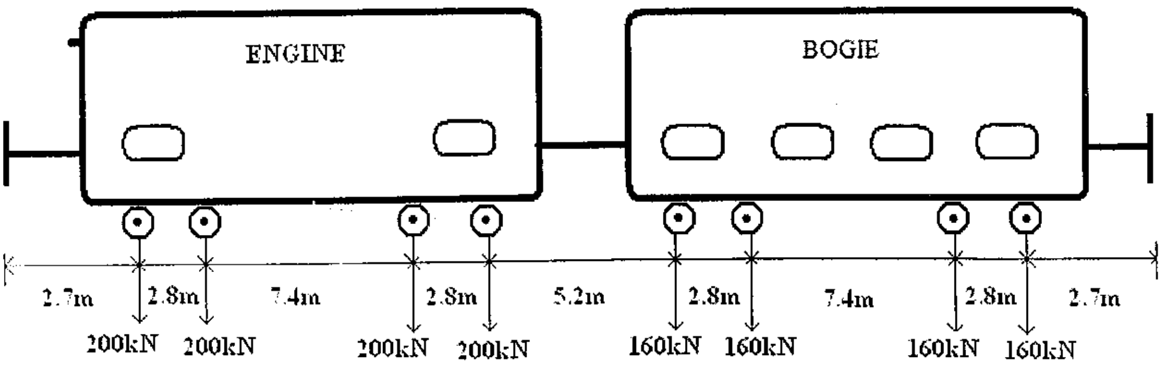


Fig 3.9: Engine and Bogie load on structure.

3.5.3 Wind Load(WL): Wind load was assigned in X and Z direction of the structure.

IS 875 (Part-2)-1987 was followed to calculate the wind load.

$T_x = T_z = 0.085\times h^{0.75} = 0.365$ sec.

$V_{b,Chandigarh} = 47$m/sec.

$P_z = 0.6 \times(V_z)^2$

$V_z = k_1k_2k_3V_b$

$k_1 = 0.9$

$k_2 = 0.98$

$k_3 = 1$

So, $V_z = 41.454$

$P_z = 41.454$

It will be constant for height of the bridge.

3.5.4 Seismic Load: Earthquake load is applied on the structure in X and Z direction.

Zone factor was takes as 0.24 for Zone IV.

Importance factor was takes as 1.

Response reduction factor is 5.

Rock and soil type was taken as 2.

Type of structure is type 2.

Damping ratio is 5%.

Period in X direction is 0.37 s.

Period in Z direction is 0.37 s.

The depth of foundation was assumed as 10 m.

The code used was IS 1893:2002.

3.6 Following load combinations (as per IS-800:2007) were considered for analysis:

- 1.5DL+ 1.5TL + 1.05AL
- 1.2DL +1.2TL +1.05AL+0.6WL in X- direction.
- 1.2DL +1.2TL +1.05AL+0.6WL in Z- direction.

- 1.2DL +1.2TL +1.05AL+0.6EL in X- direction.
- 1.2DL +1.2TL +1.05AL+0.6EL in Z- direction.
- 1.5DL + 1.5WL in X- direction.
- 1.5DL + 1.5WL in Z- direction.
- 1.5DL + 1.5EL in X- direction.
- 1.5DL + 1.5EL in Z- direction.

Abbreviations used:

DL – Dead Load

TL – Leading Load

AL – Accompanying Load

EL – Seismic Load

WL – Wind Load

3.7 After defining the appropriate loads, the final analysis was carried out using STAAD.pro.

CHAPTER 4

STAAD.pro RESULTS AND DRAWINGS

4.1 GENERAL

The results obtained by STAAD.pro analysis are presented in this chapter. All the members have been found safe and passed code-check command. Figure 4.1 shows the truss bridges with its joints numbered.

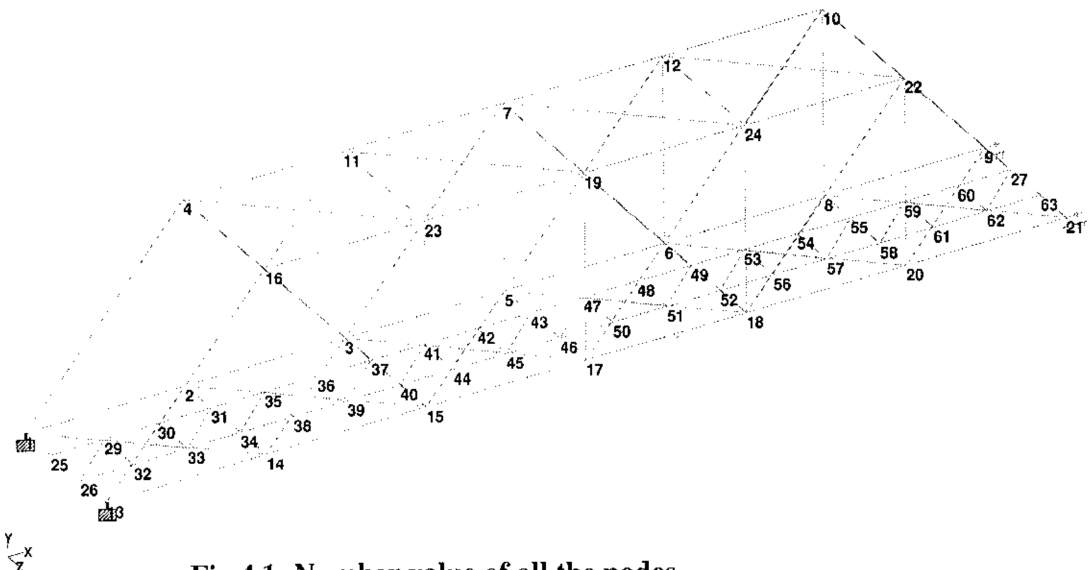


Fig 4.1: Number value of all the nodes

4.2 JOINTS OUTLINE

NOTE: The diagrams of the joints depicted are a rough outline.

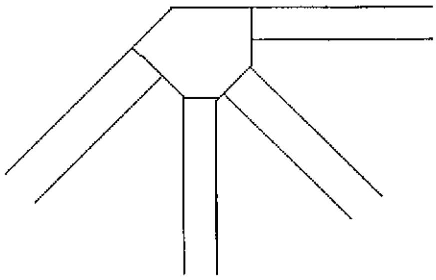


Fig. 4.2: Joints 4, 16

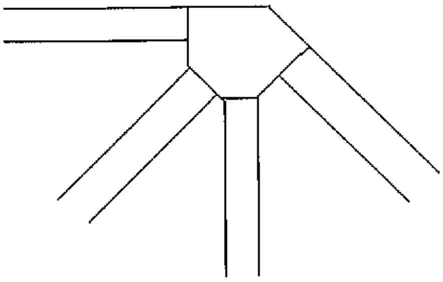


Fig. 4.3: Joints 10, 22

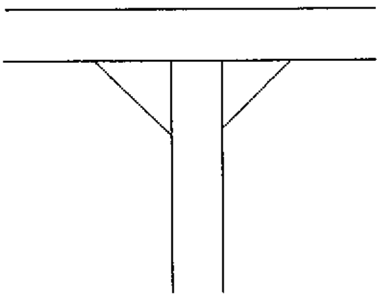


Fig. 4.4: Joint 11, 23, 12, 24

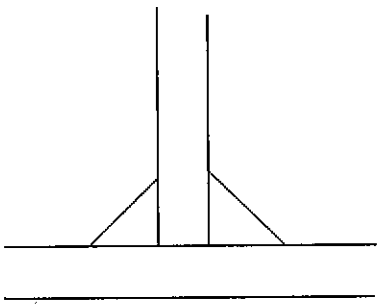


Fig. 4.5: Joint 2, 5, 8, 14, 17, 20

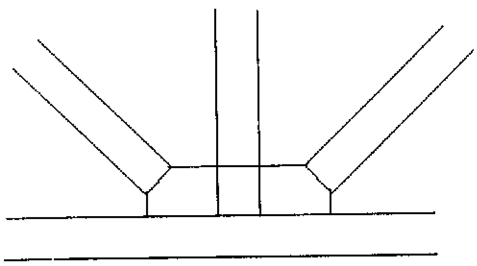


Fig. 4.6: Joint 3,6,15,18

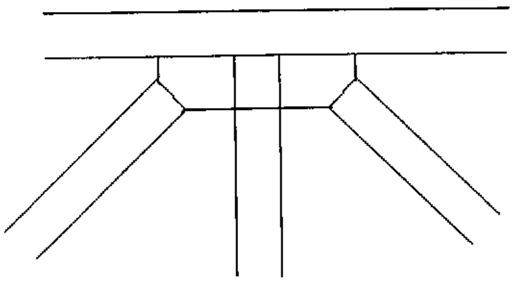


Fig. 4.7: Joint 7,19

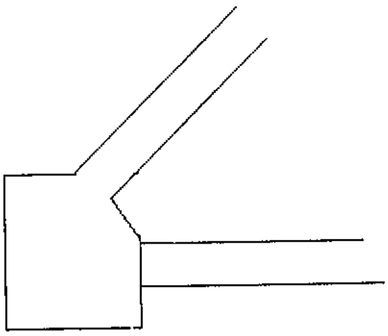


Fig. 4.8: Joint 1, 13

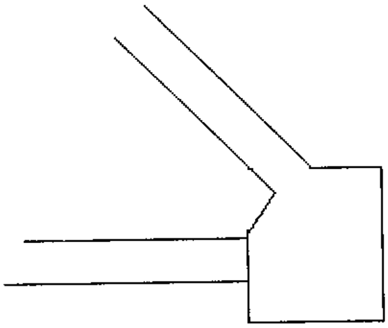


Fig. 4.9: Joint 9, 21

4.3 BEAM STRESS DIAGRAM IN STAAD.pro

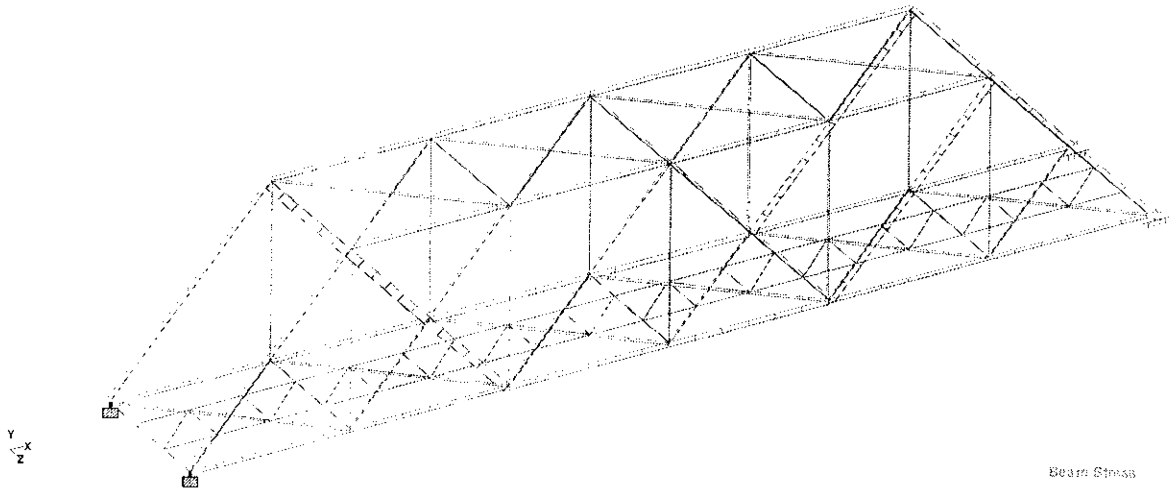


Fig. 4.10: Beam Stress Diagram

4.4 DEFLECTED SHAPE

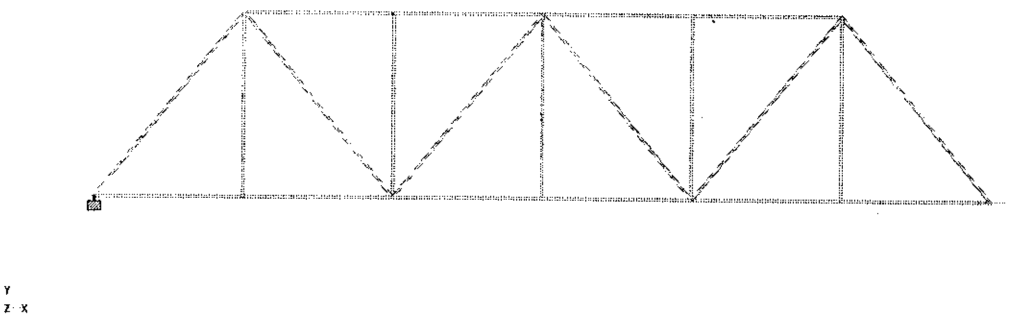


Fig. 4.11: Deflected shape due to DL + TL + AL (Side View)

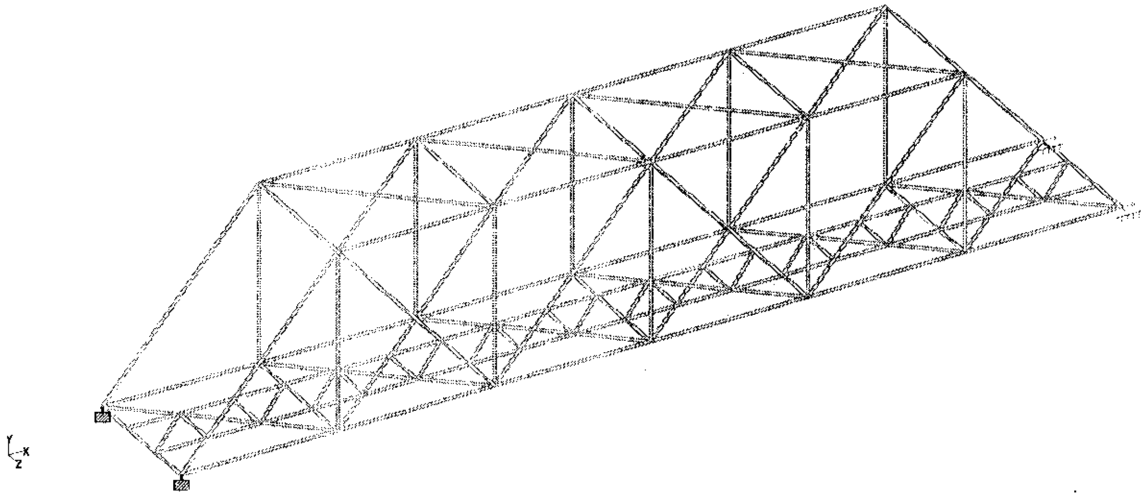


Fig. 4.11: Deflected shape due to DL + TL + AL (3D-View)

4.5 AXIAL FORCE DIAGRAM

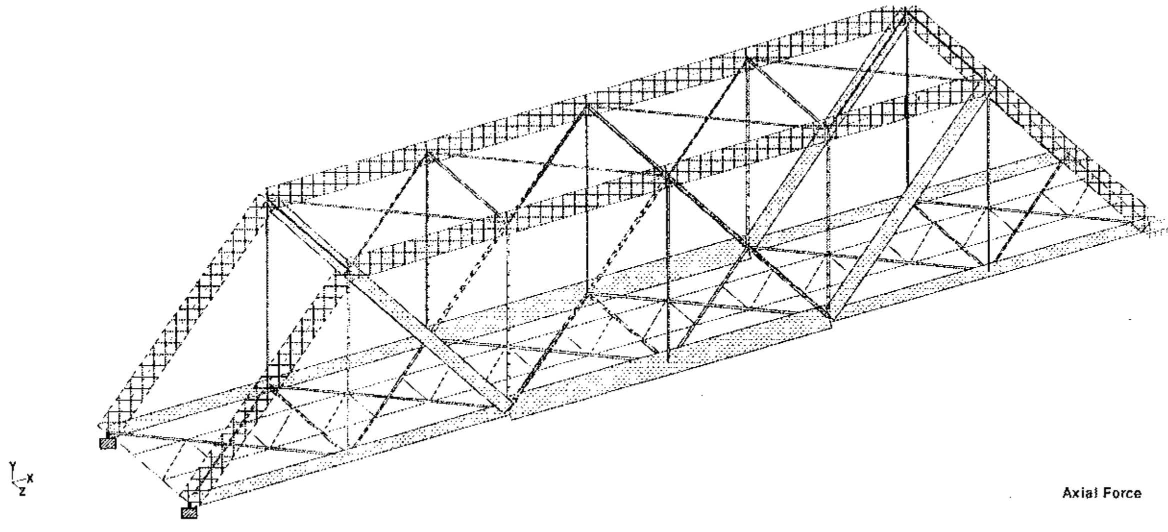


Fig. 4.13: Axial Force On Members

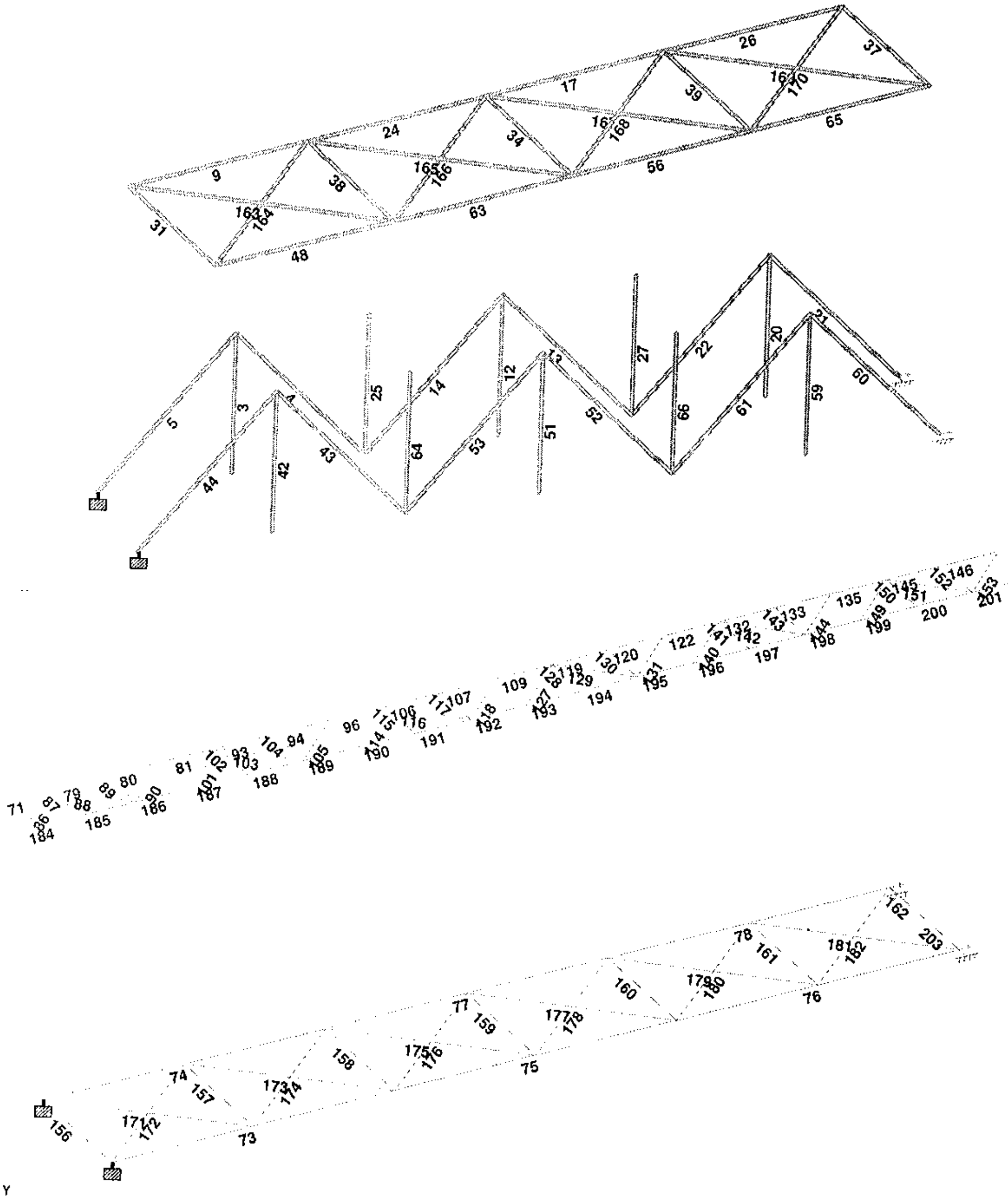


Fig. 4.14: Member Numbers

4.6 RESULT SUMMARY

Table 4.1: Summary for beam end forces

	Beam	L/C	Node	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
Max Fx	9	16 1.5DL+1.5EL IN Z	4	219.013	6.709	0	0	0	0
Min Fx	75	16 1.5DL+1.5EL IN Z	15	-270.96	12.86	0	0	0	0
Max Fy	75	8 1.5DL+1.5TL+1.05AL	15	-186.733	12.86	0	0	0	0
Min Fy	71	8 1.5DL+1.5TL+1.05AL	29	0	-803.24	0	0	0	0
Max Fz	156	13 1.5DL+1.5WL IN X	1	0	7.21	4.056	0	0	0
Min Fz	156	13 1.5DL+1.5WL IN X	13	0	-7.21	-4.056	0	0	0
Max Mx	3	8 1.5DL+1.5TL+1.05AL	2	-10.337	0	0	0	0	0
Min Mx	3	8 1.5DL+1.5TL+1.05AL	2	-10.337	0	0	0	0	0
Max My	3	8 1.5DL+1.5TL+1.05AL	2	-10.337	0	0	0	0	0
Min My	3	8 1.5DL+1.5TL+1.05AL	2	-10.337	0	0	0	0	0
Max Mz	3	8 1.5DL+1.5TL+1.05AL	2	-10.337	0	0	0	0	0
Min Mz	3	8 1.5DL+1.5TL+1.05AL	2	-10.337	0	0	0	0	0

Table 4.2: Summary for reactions and support

			Horizontal	Vertical	Horizontal	Moment		
	Node	L/C	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
Max Fx	1	16 1.5DL+1.5EL IN Z	168.454	187.072	-9.302	0	0	0
Min Fx	13	16 1.5DL+1.5EL IN Z	-168.454	121.562	9.302	0	0	0
Max Fy	1	16 1.5DL+1.5EL IN Z	168.454	187.072	-9.302	0	0	0
Min Fy	9	12 1.2DL+1.2TL+1.05AL +0.6EL IN Z	0	108.511	-7.442	0	0	0
Max Fz	13	13 1.5DL+1.5WL IN X	-80.852	148.47	17.781	0	0	0
Min Fz	1	13 1.5DL+1.5WL IN X	-82.32	148.47	-17.781	0	0	0
Max Mx	1	8 1.5DL+1.5TL+1.05AL	0	154.317	-9.302	0	0	0
Min Mx	1	8 1.5DL+1.5TL+1.05AL	0	154.317	-9.302	0	0	0
Max My	1	8 1.5DL+1.5TL+1.05AL	0	154.317	-9.302	0	0	0
Min My	1	8 1.5DL+1.5TL+1.05AL	0	154.317	-9.302	0	0	0
Max Mz	1	8 1.5DL+1.5TL+1.05AL	0	154.317	-9.302	0	0	0
Min Mz	1	8 1.5DL+1.5TL+1.05AL	0	154.317	-9.302	0	0	0

4.7 CODE CHECK RESULTS

STAAD.pro outputs for a few representative members are shown below.

4.7.1 Top chord:

Geometry | Property | Loading | Shear Bending | Deflection | Design Property | Steel Design

Beam no. = 9. Section: U1U2U2U3

Length = 6

DESIGN STRESSES (NEW, MMS)

YLD	248.21	FA	180.58
FCZ	0	FTZ	0
FCY	0	FTY	0
FT	32.73	FV	0

Critical load (KN ,METE)

Load	16
Locatio	0
FX	219.013275 C
MY	0
MZ	0

Code	Result	Ratio	Critical	KLR
IS-800	PASS	0.06249178	COMPRESSION	51.25082

4.7.2 Bottom chord

Geometry | Property | Loading | Shear Bending | Deflection | Design Property | Steel Design

Beam no. = 73. Section: L0L1L1L2

Length = 12

DESIGN STRESSES (NEW, MMS)

YLD	248.21	FA	225.65
FCZ	0	FTZ	0
FCY	0	FTY	0
FT	32.73	FV	0

Critical load (KN ,METE)

Load	16
Locatio	0
FX	238.607117 T
MY	0
MZ	0

Code	Result	Ratio	Critical	KLR
IS-800	PASS	0.08392359	TENSION	111.862

4.7.3 Top Laterals

Geometry | Property | Loading | Shear Bending | Deflection | Design Property | Steel Design

Beam no. = 31. Section: U1U1I

Length = 5.25

DESIGN STRESSES (NEW, MMS)

YLD	248.21	FA	225.65
FCZ	0	FTZ	0
FCY	0	FTY	0
FT	32.73	FV	0

Critical load (KN ,METE)

Load	8
Locatio	0
FX	8.937469 T
MY	0
MZ	0

Code	Result	Ratio	Critical	KLR
IS-800	PASS	0.004211852	TENSION	136.0666

4.7.4 Floor beams

Geometry | Property | Loading | Shear Bending | Deflection | Design Property | Steel Design

Beam no. = 156. Section: FLOORBEAM

Length = 5.25

DESIGN STRESSES (NEW, MMS)

YLD	248.21	FA	0
FCZ	0	FTZ	0
FCY	0	FTY	0
FT	32.73	FV	0

Critical load (KN ,METE)

Load	1
Locatio	0
FX	0.000000
MY	0
MZ	0

Code	Result	Ratio	Critical	KLR
IS-800	PASS	0	MINM. THK.	107.359

4.7.5 Stringer

Geometry | Property | Loading | Shear Bending | Deflection | Design Property | Steel Design

Beam no. = 79. Section: STRINGER

Length = 2

DESIGN STRESSES (NEW, MMS)

YLD	248.21	FA	225.65
FCZ	0	FTZ	0
FCY	0	FTY	0
FT	32.73	FV	0

Critical load (KN ,METE)

Load	14
Locatio	0
FX	116.295685 T
MY	0
MZ	0

Code	Result	Ratio	Critical	KLR
IS-800	PASS	0.02652813	TENSION	45.22226

4.7.6 Bottom laterals

Geometry | Property | Loading | Shear Bending | Deflection | Design Property | Steel Design

Beam no. = 171. Section: L0L1I

Length = 7.97261

DESIGN STRESSES (NEW, MMS)

YLD	248.21	FA	225.65
FCZ	0	FTZ	0
FCY	0	FTY	0
FT	32.73	FV	0

Critical load (KN ,METE)

Load	13
Locatio	0
FX	27.511362 T
MY	0
MZ	0

Code	Result	Ratio	Critical	KLR
IS-800	PASS	0.03581737	TENSION	232.5762

CHAPTER 5
CONCLUSIONS

Conclusions

The railway truss bridge of 36 m length, 7 m height, and an effective width of 5.25 m, was manually analyzed and designed, then checked by STAAD Pro 2006. Maximum axial force in X direction was found to be 218.013 kN for critical load case number 16 (1.5 DL + 1.5 EL in Z) in beam 9 at node 4. Minimum axial force in X direction was found to be -270.96 kN for critical load case number 16 (1.5 DL + 1.5 EL in Z) in beam 75 at node 15. Maximum shear force in Y direction was found to be 12.86 kN for critical load case number 8 (1.5 DL + 1.5 TL + 1.05 AL) in beam 75 at node 15. Minimum shear force in Y direction was found to be -803.24 kN for critical load case number 8 (1.5 DL + 1.5 TL + 1.05 AL) in beam 71 at node 29. Maximum force in Z was found to be 4.056 kN for critical load case 13 (1.5 DL + 1.5 WL in X) in beam 156 at node 1. Minimum force in Z was found to be -4.056 kN for critical load case 13 (1.5 DL + 1.5 WL in X) in beam 156 at node 13. Maximum bending was found for the critical load case 8 (1.5 DL + 1.5 TL + 1.05 AL) in beam 3 at node 2.

All the members were rendered safe after IS 800: 2007 code check.

REFERENCES

- Subramanian, N., "*Design of Steel Structures*", Oxford University Press, New Delhi, 2008, 1st Ed.
- Duggal, S.K, "*Design of Steel Structures*", McGraw-Hill, 2000, 2nd Ed.
- IS 1893: 2002 – "Criteria for earthquake resistant design of structures".
- IS 875 Part 3: 1987 – "Code of practice for design loads (other than earthquake) for building and structures".
- IS 800: 2007 – "General construction in steel – code of practice".
- www.google.com
- www.bentley.com
- www.dmc.utep.edu/steel/staad_eg47.org
- www.wikipedia.com

APPENDIX-A: STAAD.pro Source code

```
STAAD TRUSS
START JOB INFORMATION
ENGINEER DATE 09-May-10
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
JOINT COORDINATES
1 0 0 0; 2 6 0 0; 3 12 0 0; 4 6 7 0; 5 18 0 0; 6 24 0 0; 7 18 7 0; 8 30 0 0;
9 36 0 0; 10 30 7 0; 11 12 7 0; 12 24 7 0; 13 0 0 5.25; 14 6 0 5.25;
15 12 0 5.25; 16 6 7 5.25; 17 18 0 5.25; 18 24 0 5.25; 19 18 7 5.25;
20 30 0 5.25; 21 36 0 5.25; 22 30 7 5.25; 23 12 7 5.25; 24 24 7 5.25;
25 0 0 1.675; 26 0 0 3.575; 27 36 0 1.675; 29 2 0 1.675; 30 4 0 1.675;
31 6 0 1.675; 32 2 0 3.575; 33 4 0 3.575; 34 6 0 3.575; 35 8 0 1.675;
36 10 0 1.675; 37 12 0 1.675; 38 8 0 3.575; 39 10 0 3.575; 40 12 0 3.575;
41 14 0 1.675; 42 16 0 1.675; 43 18 0 1.675; 44 14 0 3.575; 45 16 0 3.575;
46 18 0 3.575; 47 20 0 1.675; 48 22 0 1.675; 49 24 0 1.675; 50 20 0 3.575;
51 22 0 3.575; 52 24 0 3.575; 53 26 0 1.675; 54 28 0 1.675; 55 30 0 1.675;
56 26 0 3.575; 57 28 0 3.575; 58 30 0 3.575; 59 32 0 1.675; 60 34 0 1.675;
61 32 0 3.575; 62 34 0 3.575; 63 36 0 3.575;
MEMBER INCIDENCES
3 2 4; 4 4 3; 5 4 1; 9 4 11; 12 5 7; 13 7 6; 14 7 3; 17 7 12; 20 8 10; 21 10 9;
22 10 6; 24 11 7; 25 3 11; 26 12 10; 27 6 12; 31 4 16; 34 7 19; 37 10 22;
38 11 23; 39 12 24; 42 14 16; 43 16 15; 44 16 13; 48 16 23; 51 17 19; 52 19 18;
53 19 15; 56 19 24; 59 20 22; 60 22 21; 61 22 18; 63 23 19; 64 15 23; 65 24 22;
66 18 24; 71 25 29; 73 13 15; 74 1 3; 75 15 18; 76 18 21; 77 3 6; 78 6 9;
79 29 30; 80 30 31; 81 31 35; 86 26 29; 87 29 32; 88 29 33; 89 30 33; 90 33 31;
93 35 36; 94 36 37; 96 37 41; 101 34 35; 102 35 38; 103 35 39; 104 36 39;
105 39 37; 106 41 42; 107 42 43; 109 43 47; 114 40 41; 115 41 44; 116 41 45;
117 42 45; 118 45 43; 119 47 48; 120 48 49; 122 49 53; 127 46 47; 128 47 50;
129 47 51; 130 48 51; 131 51 49; 132 53 54; 133 54 55; 135 55 59; 140 52 53;
141 53 56; 142 53 57; 143 54 57; 144 57 55; 145 59 60; 146 60 27; 149 58 59;
150 59 61; 151 59 62; 152 60 62; 153 62 27; 156 1 13; 157 2 14; 158 3 15;
159 5 17; 160 6 18; 161 8 20; 162 9 63; 163 4 23; 164 11 16; 165 11 19;
166 7 23; 167 7 24; 168 12 19; 169 12 22; 170 10 24; 171 1 14; 172 2 13;
173 2 15; 174 3 14; 175 3 17; 176 5 15; 177 5 18; 178 6 17; 179 6 20; 180 8 18;
181 8 21; 182 9 20; 184 26 32; 185 32 33; 186 33 34; 187 34 38; 188 38 39;
189 39 40; 190 40 44; 191 44 45; 192 45 46; 193 46 50; 194 50 51; 195 51 52;
196 52 56; 197 56 57; 198 57 58; 199 58 61; 200 61 62; 201 62 63; 202 21 63;
DEFINE PMEMBER
9 24 17 26 PMEMBER 1
START GROUP DEFINITION
MEMBER
_STRINGER 71 79 TO 81 93 94 96 106 107 109 119 120 122 132 133 135 145 146 -
184 TO 201
_STNGR_BRACING 86 TO 90 101 TO 105 114 TO 118 127 TO 131 140 TO 144 149 TO 153
_FLOOR_BEAM 156 TO 162
```

```
_TOP_LATERAL 163 TO 170
_BOTTOM_LATRALS 171 TO 182
_L0_L6 73 TO 78
_U1_U5 9 17 24 26 48 56 63 65
_U1U1' 31 34 37 TO 39
_L0L0' 156 TO 162
JOINT
END GROUP DEFINITION
START USER TABLE
TABLE 1 C:\SPRO2006\STAAD\PLUGINS\BRIDGE\BRIDGE.UPT
END
DEFINE MATERIAL START
ISOTROPIC STEEL
E 2.05e+008
POISSON 0.3
DENSITY 76.8195
ALPHA 1.2e-005
DAMP 0.03
END DEFINE MATERIAL
MEMBER PROPERTY AMERICAN
86 TO 90 101 TO 105 114 TO 118 127 TO 131 140 TO 144 149 TO 152 -
153 UPTABLE 1 SBRACING
71 79 TO 81 93 94 96 106 107 109 119 120 122 132 133 135 145 146 184 TO 200 -
201 UPTABLE 1 STRINGER
156 TO 162 202 UPTABLE 1 FLOORBEAM
73 74 76 78 UPTABLE 1 L0L1L1L2
75 77 UPTABLE 1 L2L3
9 17 24 26 48 56 63 65 UPTABLE 1 U1U2U2U3
4 22 43 61 UPTABLE 1 U1L2
13 14 52 53 UPTABLE 1 L2U3
3 12 20 25 27 42 51 59 64 66 UPTABLE 1 U1L1U2L3
5 21 44 60 UPTABLE 1 L0U1
31 34 37 TO 39 UPTABLE 1 U1U1I
163 TO 170 UPTABLE 1 U1U2I
171 TO 182 UPTABLE 1 L0L1I
CONSTANTS
MATERIAL STEEL ALL
SUPPORTS
1 13 FIXED
9 21 FIXED BUT FX
DEFINE 1893 LOAD
ZONE 0.24 RF 5 I 1 SS 2 ST 2 DM 5 PX 5 PZ 0.37 DT 10
SELFWEIGHT
MEMBER LOAD
71 184 CON GY -100 4.8
71 184 CON GY -100 7.6
71 184 CON GY -100 15
71 184 CON GY -100 17.8
71 184 CON GY -80 23
71 184 CON GY -80 25.8
```

```
71 184 CON GY -80 33.2
71 184 CON GY -80 36
DEFINE WIND LOAD
TYPE 1
INT 1.03 1.03 HEIG 0 10
EXP 1 JOINT 1 TO 27 29 TO 62
DEFINE MOVING LOAD
TYPE 1 LOAD 200 200 200 200 160 160 160 160
DIST 2.8 7.4 2.8 5.162 2.8 7.4 2.8 WID 1.9
TYPE 2 LOAD 160 160 160 160 160 160 160 160
DIST 2.8 7.4 2.8 5.162 2.8 7.4 2.8 WID 1.9
LOAD 1 LOADTYPE Seismic  TITLE EL IN X
1893 LOAD X 1
LOAD 2 LOADTYPE Seismic  TITLE EL IN Z
1893 LOAD Z 1
LOAD 3 LOADTYPE Wind  TITLE WL IN X
WIND LOAD X 1 TYPE 1 XR 0 36 YR 0 8 ZR 0 5.25 OPEN
LOAD 4 LOADTYPE Wind  TITLE WL IN Z
WIND LOAD Z 1 TYPE 1 XR 0 36 YR 0 8 ZR 0 5.25 OPEN
LOAD 5 LOADTYPE Dead  TITLE DL
SELFWEIGHT Y -1
LOAD 6 LOADTYPE Live  TITLE TL
MEMBER LOAD
71 184 CON GY -100 4.8
LOAD 7 LOADTYPE Live  TITLE AL
MEMBER LOAD
71 184 CON GY -100 7.6
71 184 CON GY -100 15
71 184 CON GY -100 17.8
71 184 CON GY -80 23
71 184 CON GY -80 25.8
71 184 CON GY -80 33.2
71 184 CON GY -80 36
LOAD COMB 8 1.5DL+1.5TL+1.05AL
5 1.5 6 1.5 7 1.05
LOAD COMB 9 1.2DL+1.2TL+1.05AL+0.6WL IN X
5 1.2 6 1.2 7 1.05 3 0.6
LOAD COMB 10 1.2DL+1.2TL+1.05AL+0.6WL IN Z
4 0.6 7 1.05 6 1.2 5 1.2
LOAD COMB 11 1.2DL+1.2TL+1.05AL+0.6EL IN X
5 1.2 6 1.2 7 1.05 1 0.6
LOAD COMB 12 1.2DL+1.2TL+1.05AL+0.6EL IN Z
2 0.6 7 1.05 6 1.2 5 1.2
LOAD COMB 13 1.5DL+1.5WL IN X
5 1.5 3 1.5
LOAD COMB 14 1.5DL+1.5WL IN Z
5 1.5 4 1.5
LOAD COMB 15 1.5DL+1.5EL IN X
5 1.5 1 1.5
LOAD COMB 16 1.5DL+1.5EL IN Z
```

```
5 1.5 2 1.5
PERFORM ANALYSIS PRINT LOAD DATA
PRINT MEMBER FORCES ALL
PARAMETER 1
CODE IS800 LSD
CHECK CODE ALL
PARAMETER 2
CODE IS800 LSD
STEEL MEMBER TAKE OFF LIST 3 TO 5 9 12 TO 14 17 20 TO 22 24 TO 27 31 34 37 -
38 TO 39 42 TO 44 48 51 TO 53 56 59 TO 61 63 TO 66 71 73 TO 81 86 TO 90 93 -
94 96 101 TO 107 109 114 TO 120 122 127 TO 133 135 140 TO 146 149 TO 153 -
156 TO 182 184 TO 202
PARAMETER 3
CODE IS800 LSD
STEEL TAKE OFF LIST 3 TO 5 9 12 TO 14 17 20 TO 22 24 TO 27 31 34 37 TO 39 -
42 TO 44 48 51 TO 53 56 59 TO 61 63 TO 66 71 73 TO 81 86 TO 90 93 94 96 101 -
102 TO 107 109 114 TO 120 122 127 TO 133 135 140 TO 146 149 TO 153 -
156 TO 182 184 TO 202
FINISH
```