ANALYSIS AND DESIGN OF RAILWAY PLATE GIRDER AND TRUSS BRIDGE

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ANALYSIS AND DESIGN OF RAILWAY PLATE GIRDER AND TRUSS BRIDGE

Major Project

Submitted in partial fulfillment of the requirements

For the degree of

Master of Technology in Civil Engineering
(Computer Aided Structural Analysis & Design)

By

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May 2008

CERTIFICATE

This is to certify that the Major Project - I entitled "**Analysis & Design of Railway plate girder and truss bridge**" submitted by **Ms. Sangita A. Patel (05MCL014),** towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) of Nirma University of Science and Technology, Ahmedabad is the record of work carried out by her under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

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ABSTRACT

Plate girder may be riveted, bolted, or welded. Plate girders resist transverse bending like beams and are provided where loads are heavy. For heavier loads, the section modulus required is not available in any standard rolled section. In such case riveting or welding plates fabricates a beam section and angle sections to forms a plate girder. Plate Girder Bridge is adopted for simply supported spans in the range of 20 to 50 m and for continuous span up to 250m.

Steel truss bridges are generally economical in the span range of 100 to 200m. Trussed bridges are economical since the members are subjected to direct forces and the open web construction facilitates the use of larger depths with a reduction in the self-weight.

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Abbreviation Notation and Nomenclature

D	=	Over all depth
d1	=	Center to center distance between the C.G of the flanges
L	=	Span of plate girder
Aw	=	Area of the web
Af’	=	Net area of tension flange
tw	=	Thickness of web
Af	=	Gross area one the one flange
M	=	Bending moment
Rv	=	Rivet value
p	=	Pitch of the rivets/welds
V	=	Maximum shear force at the section
c1	=	Actual distance between vertical stiffener
w	=	Total superimposed load on both the plate girder in KN
σbt	=	The bending tensile stress
σbc	=	Allowable bending compressive stress.
Ixx	=	Moments of inertia of girder about its neutral axis
Aw1	=	Area of web between flange angle (as per IS: 800-1984), i.e. equivalent web area on compression side.
ζ	=	Horizontal shear force /unit length
Y	=	Distance of remote fiber from the c.g of the section
dw	=	Depth of web
ac	=	Clear distance between the welds
a	=	Effective length of intermittent fillet weld
h	=	Outstand of stiffeners
λ	=	Slenderness ratio
d1eff	=	Effective depth
σp	=	Permissible bending stress
r	=	Radius of gyration perpendicular to the plane of web of the girder

R	=	Reaction at the support
n	=	Number of rivets in one row
n’	=	Number of rows on one side of the splice
ds	=	Depth of the moment plate
b	=	Clearance between flange angle and moment plate
ζ va	=	Average shear stress
σb2	=	Bending stress at the level of rivets connection the splice\
§	=	Clauses
P1	=	Wind load acting on the top chord
P2	=	Wind load acting on the Bottom chord
S	=	Spacing between two girder
R	=	Reaction
H	=	height of the truss

1. INTRODUCTION

1.1 GENERAL

Plate Girder Bridge is the most common type of the bridge generally used for railway crossing of streams and river. Plate Girder Bridge is adopted for simply supported spans in the range of 20 to 50 m and for continuous span up to 250m.

In the case of the railway bridge, the plate girder supports the sleepers over which the steel rails are fastened. Each rail is supported on a plate girder so that the wheel loads are transmitted directly to the plate girder without the effect of torsion. The plate girders are braced laterally at the level of the bottom flange to provide lateral stability. Cross bracing consisting of angles are provided at the end at interval of 3 to 4 m. The lateral bracing and the end cross frames resist the lateral load on the plate girder.

Steel truss bridges are generally economical in the span range of 100 to 200m.These types of bridges are generally preferred for long span railway bridges. Trussed bridges are economical since the members are subjected to direct forces and the open web construction facilitates the use of larger depths with a reduction in the self weight. The constructions of steel trussed bridges are faster due to lightness of members and fabrication of joint at site.

1.2 CLASSIFICATION OF PLATE GIRDER

Plate girders are flexural members. Their bending resistance can be increased by increasing distance between the flanges. This increases the shear resistance as the web area increases. Plate girder are primarily provided in bridges but are also very common in buildings where heavy concentrated loads (from upper storey columns, etc.) act on long span beams ,e.g. dinning hall floor beam of a restaurant requiring a clear space all through.

There are two forms of plate girder bridges;

1. Welded plate girder
2. Riveted plate girder

1.2.1 Welded plate girder

The welded plate girder may be used up to 100m span. Welded plate girder is economical as compared to the riveted plate girder because of the consideration of reduction in its self weight.

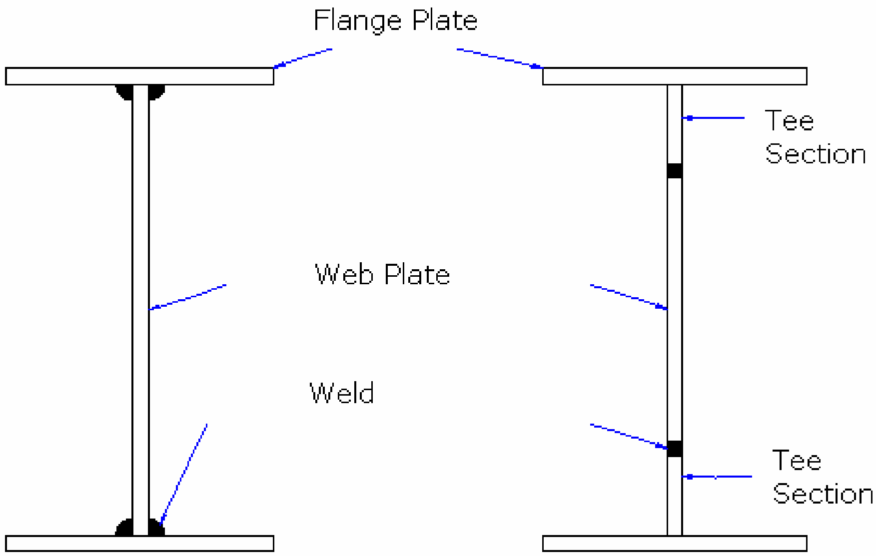


Figure 1.1 Welded Plate Girders

The figure1.1 (a) is most common. In welded plate girder, no flange angle is required and it is uneconomical to use a number of thin cover plates as flanges. One thick plate may be used as a flange and where it is desired to decrease the flange area, a thinner plate may be used. The thinner and thicker plates may then be butt welded. A plate girder can also be fabricated by welding two T – sections with a web plate, shown in figure1.1 (b).

1.2.2 Riveted plate girder

Riveted plate girder is economically used between 15 m to 30 m span. Figure 1.2 (a) shows the simplest type of riveted plate girder, in which each flange consists of a pair of an angles connected to solid web plate.

For large moments, the flange area can be increased by riveting additional plates (called cover plates) as shown in figure1.2 (b). In a simple beam, the maximum bending moment generally occurs near the mid span and it goes on decreasing towards the supports .The cover plates can be curtailed as the moment decreases.

To avoid a larger number of cover plates, some of the flange area may be provided by plates inserted between the web and the flange angles. Figure 1.2(c). Although this is not the most efficient use of material, but the arrangement is sometimes used to avoid excessively long rivet connecting the flange angles with the cover plates.

The box girder, shown in figure1.2 (d) is also quite commonly used in buildings. Due to the headroom restrictions in buildings, the larger depth of girder may not be permitted and to take the shear force, more than one plate may be needed as in a box girder. Moreover the box girder provides better lateral stability. The box girder can be made stronger by using more two webs and also by using more cover plates as in figure 1.2(e).

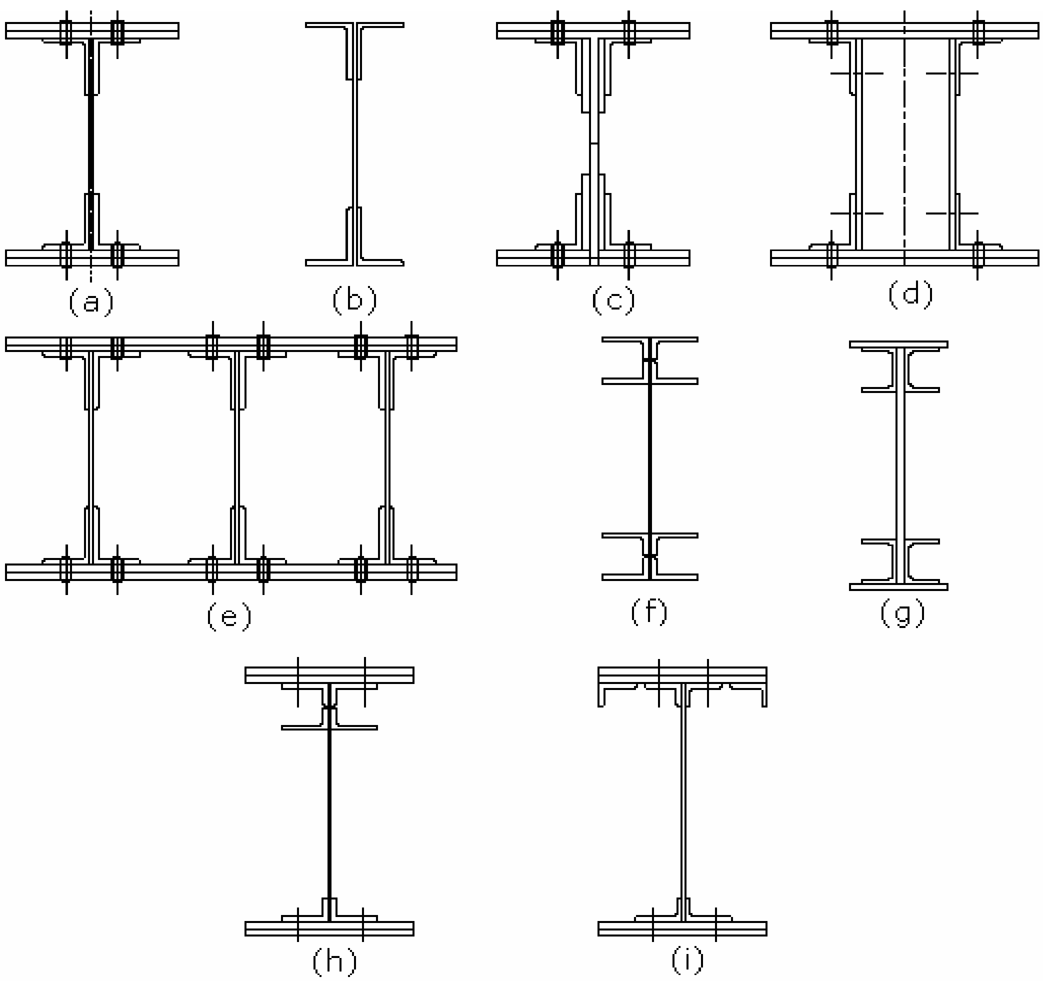


Figure 1.2 Riveted plate girders

1.3 ELEMENTS OF PLATE GIRDER

The various elements of a riveted plate girder are as listed below;

- Web plate
- Flange angle with or without flange cover plate.
- Bearing stiffeners at the end over the center line of bearing and at intermediate points under the point's loads.
- Vertical stiffeners fixed to the web plate
- Horizontal stiffeners fixed to the web plate, depth wise, one or more in number, to prevent buckling of web plate.
- Web splices- plates used to join the two web plates.
- Flange splices plates used to join the two flange plates.
- Angle splice – plate used to join the two flange angle.

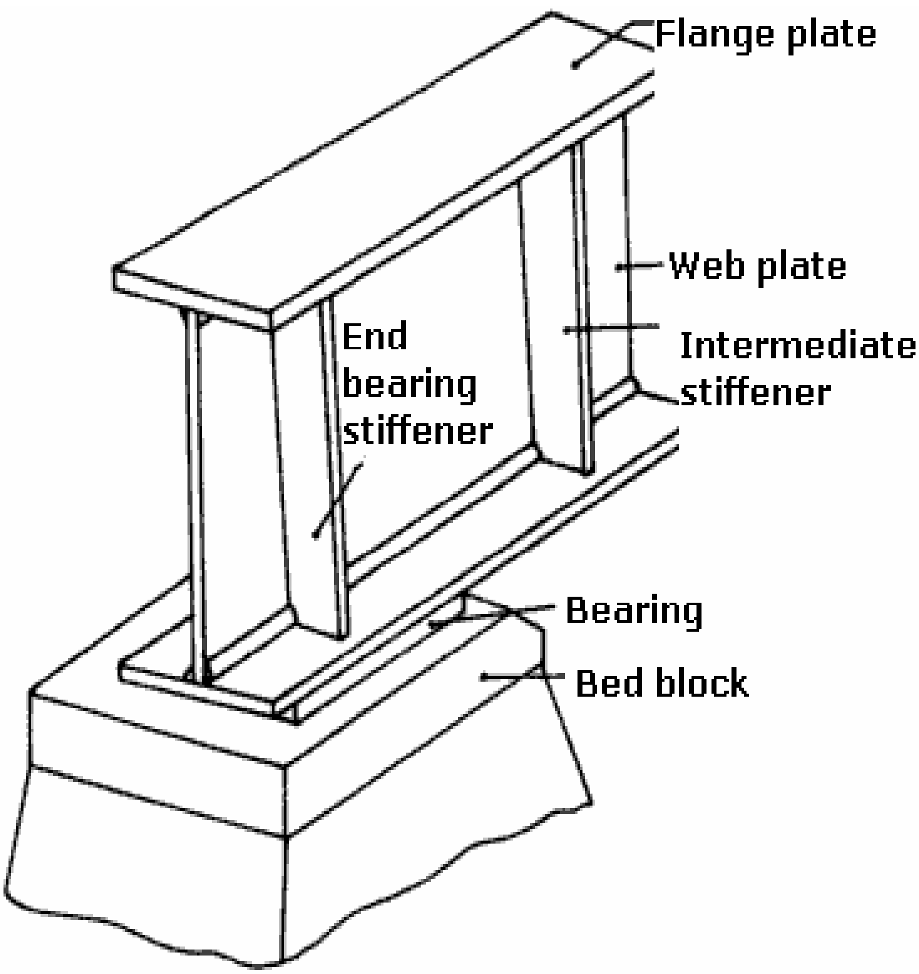


Figure 1.3 Elements of plate girder

1.4 TRUSSED GIRDER BRIDGE

Steel truss bridges are generally economical in the span range of 100 to 200m.These types of bridges are generally preferred for long span railway bridges. For spans greater than what can be spanned economically by the plate girders, are used truss bridges. It is difficult to draw a demarcating line in the lengths of the span above which the plate girders will not be economical. For the same weight a plate girder may be economical due to smaller cost of fabrication. Roughly a truss bride should be used for spans greater than about 30 m.

1.4.1 Components of truss Girder Bridge

A truss is a structure component of members connected together to form a rigid framework. Members are the load-carrying components of a structure. Members are arranged in interconnected triangles Because of the configuration, truss members carry load primarily in tension and compression.

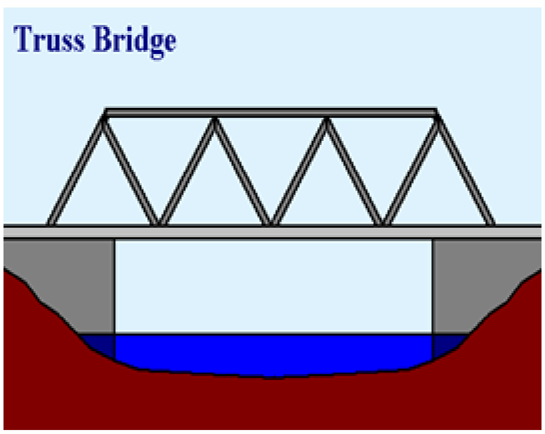


Figure 1.4 (a) Truss bridge

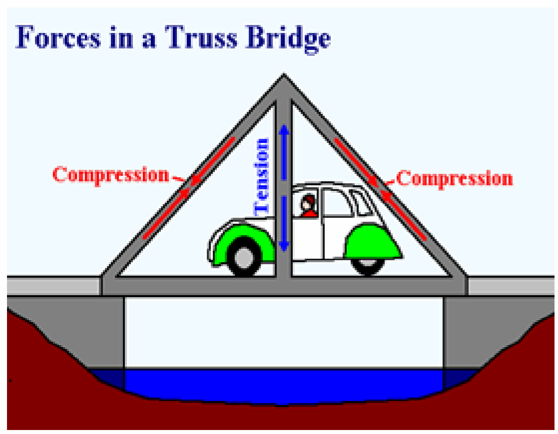


Figure 1.4 (b) Force in truss bridge

The truss bridge has two main load-carrying trusses. Each truss composed of a top chord, a bottom chord, and several verticals and diagonals. The two trusses are connected together by a series of transverse members: struts, lateral bracing, and floor beams.

All bridges are composed of the superstructure or span and the sub structure or foundation. Here consider superstructure of Railway Bridges.

The four major parts of the superstructure are;

- ✓ The Floor system
- ✓ The girder or the trusses
- ✓ The end shoe
- ✓ Bearing

An arrangement of the components of a truss girder bridge is shown in figure (1.5).

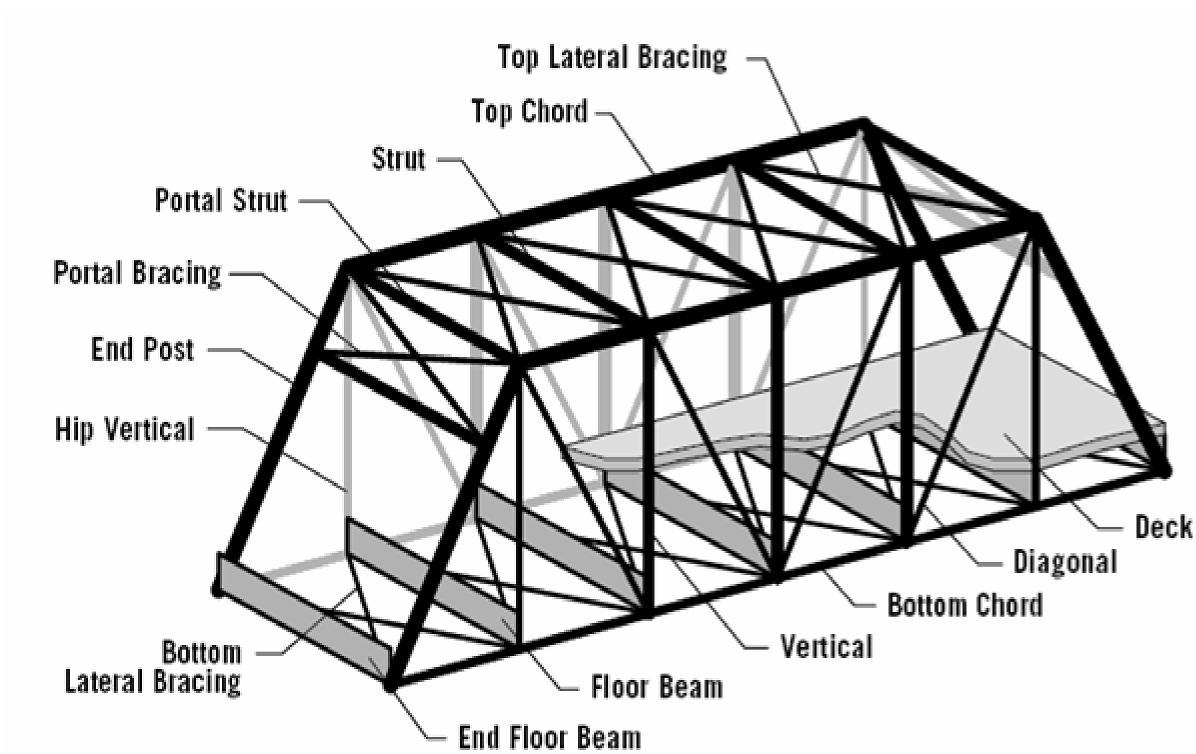


Figure 1.5 components of the bridge.

1.5 TYPES OF TRUSS BRIDGE

Many types of the Truss bridge are used in bridges. Some of the common types of the truss girder are shown in figure 1.6; the most common types is the parallel chord high Pratt truss (figure1.6 (a,b)) for through span of moderate length.

Through it is economical only up to 50 m span it is recommended even up to 70 m. The Warren trusses for through and deck bridges are shown in figure 1.6 (d,h).A high Warren truss is most economical for a deck Bridge and is seldom used for a through bridge. The curved Chord Pratt truss (figure1.6 (c)),also called Parker or camel back truss, is recommended for span more than 50 m.For span more than 90m a K – bar truss (figure1.6(f)) or a Pettit truss with Subdivided Panels (figure1.6) or with a broken end post (figure1.6) is used. The Baltimore truss, shown in figure1.6 (e), may be looked upon as intermediate between Parker and Pettit trusses, but being uneconomical is not used widely.

The economical height to span ratio of a bridge is about one sixth to one eighth. The optimum inclination of the diagonal is 45° to maintain the inclination for long bridges, the Pratt or the Warren trusses result shorten the span of the stringers is to subdivide these trusses. The examples are Pettit and Baltimore trusses .however, high secondary stresses in these trusses limit use and a K-truss is preferred.

There are different types of the truss listed are below;

- ✓ Pratt
- ✓ Double Intersection Pratt (Whipple)
- ✓ Baltimore
- ✓ Parker
- ✓ Pennsylvania
- ✓ Warren
- ✓ Double Intersection Warren
- ✓ Warren Quadrangular(Lattice)
- ✓ K Truss

Figure 1.6 (a) Pratt type truss

Figure 1.6 (b) Double Intersection Pratt truss

Figure 1.6 (c) Parker type truss

Figure 1.6 (d) Deck Warren type truss

Figure 1.6 (e) Baltimore types truss

Figure 1.6(f) K-type truss

Figure 1.6 (g) Pony types of the truss

Figure 1.6(h) through types of truss

Figure 1.6 different types truss bridge

1.6 OBJECTIVE OF THE STUDY

The objective of present work is to study the analysis and design of Railway bridges. Microsoft excel is used to analysis of IRS recommended bridge loadings for welded, riveted and truss bridge. I have checked my result through visual basic 6.0.

1.7 SCOPE OF THE WORK

The study was divided in two phases,

- **Phase I:**
    - Introduction of railway plate Girder Bridge and truss bridge.
    - Literature Review
    - Study of provision of IRS Loading standard
    - Example of Analysis and design of the railway plate Girder Bridge and truss bridge.
      (Manual and Excel Sheet)
- **Phase II:**
    - Design of Plate Girder and Railway Truss Bridge
    - Comparison of Cost with Same Span but Different D/tw Ratio
        - Riveted Plate girder
        - Welded Plate girder
        - Truss Girder
    - Programming in Visual Basic
        - Riveted Plate girder

1.8 ORGANIZATION OF REPORT

Chapter 1 Includes the introductory part of thesis, objective and Scope of the thesis.

Chapter 2 The literature review regarding information of various type of bridge, loading on bridge, economy criteria and brief from research paper is covered.

Chapter 3	It covers design methodology of Riveted Plate girder Bridge and Example of analysis and Design procedure along with sample calculation and detailed drawings.
Chapter 4	It covers design methodology of Welted Plate Girder Bridge and Example of analysis and Design procedure along with sample calculation and detailed drawings.
Chapter 6	Includes Programming of Riveted Plate girder Bridge in Visual Basic 6.
Chapter 7	Includes estimation of cost, in that quantity calculation and rate analysis is done for Riveted and Welded Plate Girder and Truss Bridged. Parametric study for obtaining effective depth to Web thickness (D/tw) ratio.

2. LITERATURE SURVEY

2.1 GENERAL

There are about literature 1, 20, 000 bridges of all types and spans on Indian Railways

Making an average of two bridges per route km. A rough break-up of the total is:

- Girder bridges 20%
- Arch bridges 19%
- Slab culverts 23%
- Pipe culverts 19%
- Other types 19 %

A plate girder is a built beam and is normally fabricated from plate sections and angle iron sections. These are used in building, factories and bridges for carrying heavy loads over span greater than 15 m. Plate girder are used extensively in every form of steel construction, because of their adaptability, with different form of flange of different size. Plate girder can be designed to serve a great Variety of purposes.

2.2 LITERATURE REVIEW

N.E.shanmugam [1] The paper is concerned with an experimental investigation on simply supported steel- concrete composite plate girder subjected to shear loading. Four composite and two bare steel plate girders were tested to failure in order study their ultimate strength behavior. It is observed from the tests that the ultimate shear capacity of composite plate girder increases compared to bare steel plate girders.

V.B.Sood [2] This paper concludes that the high performance steel is being used on high way and Railway Bridge successfully, all over the world because of it's inherent quality of better strength, resistance against fracture toughness, weld ability and a resistance against weathering /corrosion. The weight of the structure is reducing the cost of substructure and foundation and over all reduced life cycle costs. Its introduction on Indian Railway Bridge will be a very good decision for the up gradation of the present technology of design, fabrication and maintenance of steel bridge.

M. Ravindranath Reddy [3] The paper concludes result of the Instrumentation on Br.No.105 (30.5m span plate girder). Longitudinal forces at the bearing level are also assessed to be well within limits. Deflections in the girder & pier are also well within limits. No abnormality in pier has been noticed / recorded. The report also can be taken as basis for fitness certification of girder Bridges up to 30.5m (100ft span) for running of freight trains with 25 tons of axle load wagons.

Nobuaki [4] In this paper the specifications regarding the tensile fatigue endurance of studs is not clearly set forth, it is considered desirable that the studs be arranged avoiding the position just above a crossbeam connection. With regard to the bending deformation of the upper flange of a main girder, which leads to the separation of a floor slab from the main girder, it is considered necessary to give a certain extent of stiffness to the flange or provide reinforcement at adequate portions, especially in the case of a composite Girder.

Ahmad M. Itani1 [5] This paper discusses the behavior of steel plate girder bridges during recent earthquakes like Petrolia, Northridge, and Kobe. And also discusses the recent experimental and analytical investigations that were conducted on steel plate girder bridges and their components. Results of these investigations showed the importance of shear connectors in distributing and transferring the lateral forces to the end and Intermediate cross frames. Also, these investigations showed the potential of using end cross frames as ductile elements that can be used to dissipate the earthquake input energy.

CYNTHIA.J.ZAHN [6] In this paper plate girder design according to LRFD Method is described. The LRFD method is facilitated by the flow chart being introduced in the LRFD manual. These new design aids demonstrate graphically how the applicable specification and section interact.

Dr.T.K Bandyopadhyay [7] In this paper includes various code provisions of different countries have been dealt to high light the need for modifying IRC codes to match with international standard. Static shear connector capacity given in IRC: 22 are very low compared to BS & AASHTO provisions. Deflection stipulation is very stringent and in most cases this is guiding factor for sizing the bridge girder, which needs to be modified..

M.B.Vijay [8] This paper includes main purpose of the composite girder with restricted depth is to eliminate the task involved in raising the approaches. Thus, it proves to be economical as well as time saving project. As the level of track structure is almost same, the effective maintenance can be achieved.

Ravindra Kumar Goel, [9] The paper briefly describes the considerations involved in the design of web & flange connection for a railway plate girder bridge. Indian Railways is in the process of adopting welded connections for the design of Railway Bridges. The design of connection is to be done with respect of the fatigue strength for a specified number of cycles of maximum and minimum stress, to which the bridge component is subjected. The economy of the design depends not only upon the type of weld and welding methodology adopted but also the ease with which the fabrication can be done. An economical Welded connection has been designed by taking adequate considerations of the fabrication difficulties, welding technology available and the existing codal provisions.

J.C. Chapman [10] This paper describes part of an investigation which aims to provide a rational, usable and validated design method for transverse stiffeners. The proposed design method takes account of the axial compression in stiffeners caused by tension field action and external forces, the transverse forces on the stiffeners required to enforce effective division of the web plate into plate panels, and the tendency for the stiffened plate to buckle overall. This paper examines the effects causing axial forces in the stiffeners. Using the concept of the tension field theory, with modifications that take account of stiffener compressibility and panel deflection, a simple validated method for the estimation of the stiffener axial force has been developed, taking account of applied shear, compression and bending. Further papers will consider the effects causing stiffener bending.

John Ch. Ermopoulos [12] This paper presents an experimental and analytical study to access the condition of a historic railway steel truss bridge still in use. The task is pursued through static and dynamic field measurements, as well as laboratory tests. A validated analytical model is employed to evaluate the capacity of the bridge to carry seismic and wind loads specified by current design codes, as well as the heavier trainloads set by the owner. Strengthening and replacement measures are proposed for bridge upgrade. An estimation of the remaining fatigue life of the bridge in its present condition and after the suggested strengthening is also made.

3. ANALYSIS AND DESIGN OF RIVETED PLATE GIRDER

3.1. GENERAL

A plate girder is a beam built – up from the plate elements to achieve more efficient arrangement of material than it is possible with the rolled steel beams. Plate girders are economical where span are long enough to allow saving in cost by proportioning for the particular requirements. Plate girders may be riveted, bolted, or welded. Plate girders resist transverse bending like beams and are provided where loads are heavy. For heavier loads, the section modulus required is not available in any standard rolled section. In such case riveting or welding plates fabricates a beam section and angle sections to forms a plate girder. Plate girders are used extensively in every form of steel construction, because of their adaptability. With different forms of flanges of different size, plate girder can be designed to serve a great variety of purposes.

3.2. COMPONENTS OF THE PLATE GIRDER

The various element of riveted plate girder are shown in figure 3.1

i) Web plate, (depth of girder)
ii) Flange plates and their curtailments,
iii) Rivets connecting flange angles with web plate, and flange angles with Cover plates,
iv) Transverse or vertical stiffeners,
v) Longitudinal or horizontal stiffeners,
vi) Bearing stiffeners,
vii) Web splices,
viii) Flange splice,
ix) End Bearing.

In the following section the analysis and design for each element of a plate girder is dealt with separately and then over all design is developed.

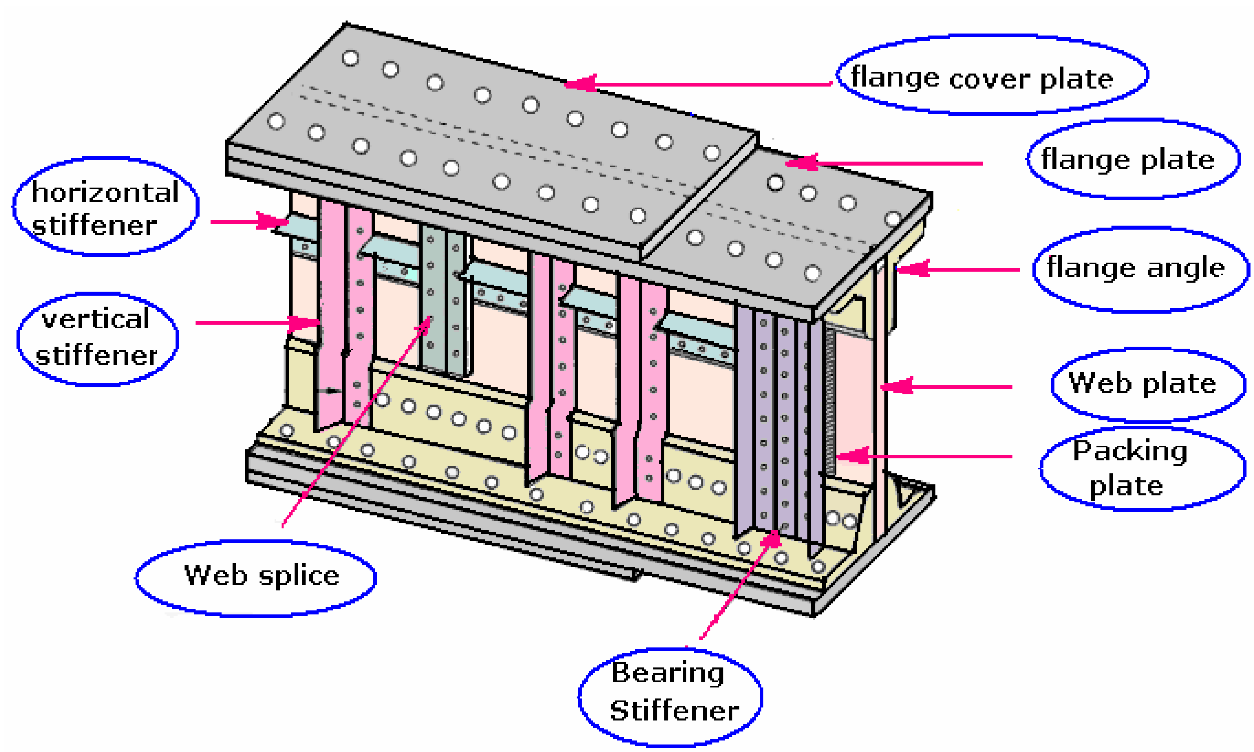


Figure 3.1 Elements of Riveted Plate girder

3.3. RAILWAY LOADING

The loads, which should be taken in to account in the design of a railway bridge, have been specified in bridge rules published by railway board. The following types of loads act on the superstructure of a bridge.

a) Dead Load
b) Live load
c) Impact load

3.3.1. Dead Load

The dead load consists of weight of rails, sleepers, and floor system and the supporting structure. In design one has to assume the dead weight of the structure. The best way estimating the dead loads is given by comparison with similar Structures. Many empirical formulas have been given by foreign authors, but those can not be applied directly for designing bridges according to Indian specification and standard loading.

The dead load acting on plate girder bridges also include the self weight of plate Girder of the bridges. The self weight of plate girder of the bridges are either assumed depending upon experience and by comparing with the existing the plate girder bridges on the similar spans or determined from Fuller's formula for 10 to 30 m span

$$w = [(a*L +b)/100] \text{ kN/m} \qquad \text{... (3.1)}$$

w = weight of both the plate girder in KN per meter and a, b constant.

Constant a and b depends upon types of the bridge, the dead load for which the bridge is designed and unit stress used. The term a*L represent principally the weight of those portion whose weight per linear length increase approximately in proportion to span. The term b represents mainly weight of floor .The weight of the floor per linear length depends of panel length and width of the bridge. It is approximately constant .the value of a and b constant are adopted as 20 and 100 respectively .so the Equation 3.1 can be written as

$$[(0.2\ L + 1] \quad \text{KN /m} \qquad \text{... (3.2)}$$

3.3.2. Live Load

Live loads due to the train loading have been specified in "Bridge Rules" for a various types of the track.

Standard Broad gauge (B.G) loading consists of a train with 224.6 kN axle loads and train of 75 kN per meter behind the engine. The Broad Gauge (B.G) loading of 1975 behind the engine. The Broad gauge (B.G) loading of 1975 consists of train loads of 75 kN per linear meter with a maximum axle load of 224.6 kN.

It is cumbersome to calculate the maximum force in all truss members due to the moving train with the concentrated wheel loads. For simplicity, Bridge Rules have given equivalent uniformly distributed loads for computing the maximum shear force and bending moment. The equivalent uniformly distributed loads for various types of loading have been given in Appendix D.

3.3.3. Impact Load

The dynamic effect of the moving load is taken care of by increasing the live load by a certain factor, called impact factor or dynamic augmentation. This factor should depend on many variable like type of loading, speed, types of the structure, material of the structure, loaded length etc. It is difficult to have a single expression involving all the variables. Design codes generally give difficult expression for impact factor for railway bridge, highway bridge, combined road rail bridges; foot bridge, steel bridge, concrete bridge or timber bridge, girder bridge, pipe culverts or arch bridge etc. The value of the impact factor for different span is given by Appendix D.

3.4. DESIGN OF WEB PLATE

The web of a plate girder is designed for shear force. The flanges share very little of the shear force and the shear stress are assumed to be uniformly distributed over the entire depth of the web.

Shear stress in web, $$\zeta va = \frac{V}{dw \times tw} \quad \text{... (3.3)}$$

The shear stress in the web should be less then the allowed shear stress as specified by the specification. The Depth of the Web is fixed from many other considerations, like the headroom restrictions, economy etc. The total Depth of the girder lies between one -tenth to one twelth of the span. The limitation is not very rigid and girders deeper then $1/8^{th}$ of span and shallower then $1/10^{th}$ of span can also be used. After fixing the depth. The thickness should be chosen to give the required area of cross section. The thickness of web plate should be such that maximum shear stress in the web does not exceed maximum allowable shear stress and average shear stress in the web does not exceed average allowable shear stress.

The thickness of web plate should also provide necessary bearing area for rivets connecting web plate and flange angles, so that rivets are provided at proper spacing. A minimum thickness of 6 mm is adopted to provide for corrosion. The thickness of web plate is fixed keeping in view the buckling of web in shear, and shear along with bending. If there is no restriction on the depth of the plate girder the depth may be assumed on the basis of minimum weight of the material according to the approximate expression

Economical depth $$d = 1.1\sqrt{\frac{M}{\sigma bc \times tw}} \quad \text{... (3.4)}$$

Where M is the design moment including all dead load and live load.

3.5. DESIGN OF FLANGE PLATE

In the riveted plate girder, the flange portion consists of flange plate, flanges angles and that part of the web, which is between the flange angles. The effective area tension flange is the gross area with deduction of holes.

In practice, the gross area of comparison flange and tension flange are kept equal. The tension flange of the plate girder is designed on the net area basis and the corresponding gross area is provided for compression flange. After selecting thickness and depth of web plate, flanges are designed by the following methods.

1. Flange area method.
2. Moments of inertia method.

3.5.1. Moment of inertia method

This is also known as exact method. A trial section of plate girder designed by flange area method is checked by moment of inertia method. The moment of resistance of trial section is computed. It is equal to or greater than the maximum bending moment; then the trial section designed is satisfactory.

The area of flanges should be so proportioned that the maximum bending stress in tension or compression flange is less than the allowed limit .The maximum bending stresses are commuted on the basis of gross moment of inertia and neutral axis assumed at the center of gravity of the gross area. The stresses so computed, however, are increased by multiplying with ratio of gross area of flange to net area of flange .On the tension side; the net area of the flange is equal to the gross area minus the deduction for holes, (including those filled rivets).

On compression flange the deduction for those holes will be made in full, which are not filled, by rivets or bolts.

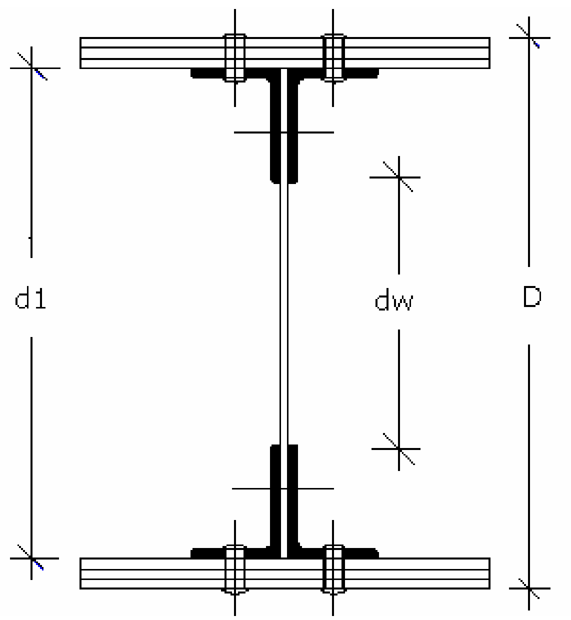


Figure 3.2 Cross section of a riveted plate girder

The account for rivet holes of the flange, the net area may be assumed to be3/4th of the gross area. If the rivet size is known at this stage the exact net area may be computed and used. Hence the net area of tension flanges.

$$Af = \frac{M}{\sigma bc \times dw} - \frac{Aw}{8} \qquad \text{... (3.5)}$$

3.6. CURTAILMENT OF PLATES

The girder section is required only for a short distance near the point of maximum bending moments. This moment in a simple plate girder under uniformly distributed loads occurs near the center of the span and reduces towards the supports.

The bending stresses thus generated are resisted by the flange area. It is therefore possible to use a smaller flange area towards the supports. The reduction in the section may be achieved by cutting of the cover plates one by one until only the web plate and flange angles remain near the supports. The curtailed length of the flange plates suits the variation in the bending moment. The length of curtailment of plate can be found conveniently bye the algebraic method or graphical method.

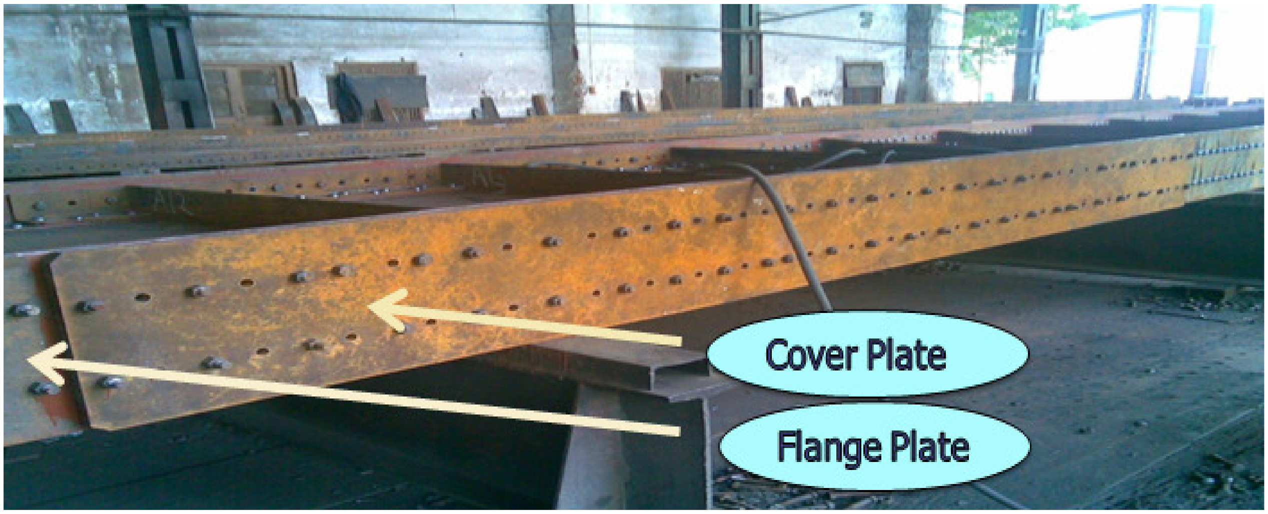


Figure 3.3 Curtailment of cover plates

When the plates girder is carrying uniformly distributed load, the bending moment diagram has parabolic curve. The parabolic bending moment is calculated by

$$y = \frac{4a}{L^2} X (L - X) \quad \text{... (3.6)}$$

The algebraic method of curtailment of flange may be used if the plate girder supports only uniformly distributed loads, where as the graphical method of curtailment of flange plate is applicable to any system of applied loads.

3.6.1. Algebraic method

Where, 2X1, 2X2 are the distances of the cut – off points from the center of the plate girder as the shown in figure3.3.

X1, X2, are calculated by following equation.

$$X1 = \frac{1}{2} \times L \times \sqrt{\frac{A_1}{Af_1}} \quad \text{... (3.7)}$$

$$X2 = \frac{1}{2} \times L \times \sqrt{\frac{A_1 + A_2}{Af_1}} \quad \text{... (3.8)}$$

$$X3 = \frac{1}{2} \times L \times \sqrt{\frac{A_1 + A_2 + A_3}{Af_1}} \quad \text{... (3.9)}$$

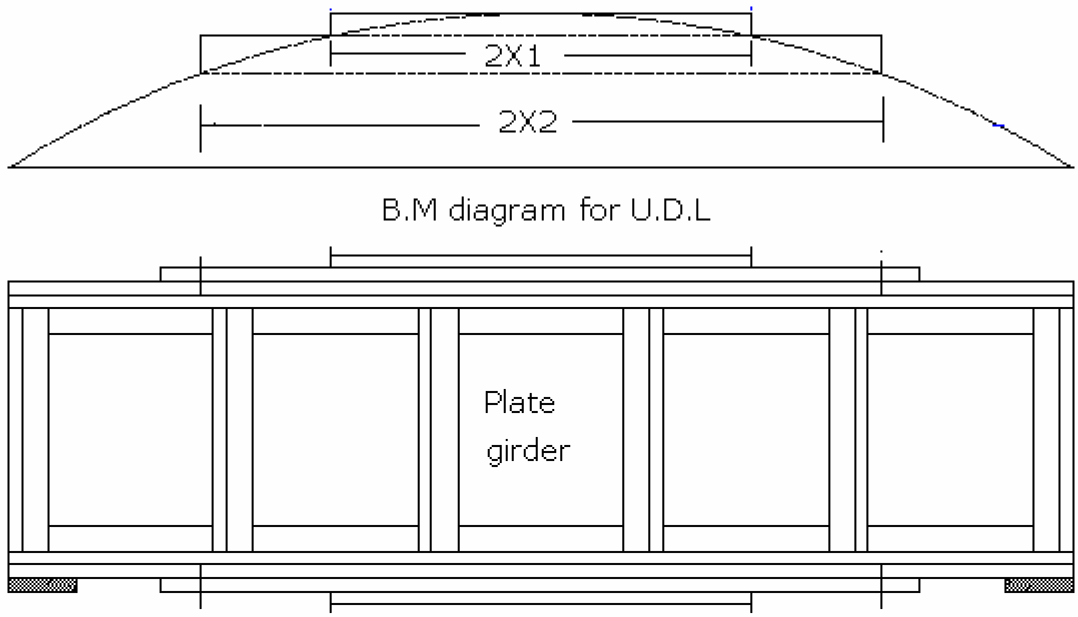


Figure 3.4 Moment diagram of flange plate curtailment

Let

A_1, A_2, A_3 = net area of 1st ,2nd ,3rd cover plates respectively, counted from outside

Where A_{f1} = Total area of flange including web equivalent.

The component are done for the curtailed of flange plate in tension flange. Net area of plate and flange area are used in calculation. The curtailment of plates in compression flange is done at the same position as that in tension flange, for the economic fabrication.

3.7. CONNECTIONS

3.7.1. Flange angle to web

These rivets pass through the angles and web of a plate girder and are designed to transmit the maximum horizontal shear force resulting form the bending moments and vertical shear associated due to any live load. It is assumed that the maximum web shear is equal to the average shear on the web.

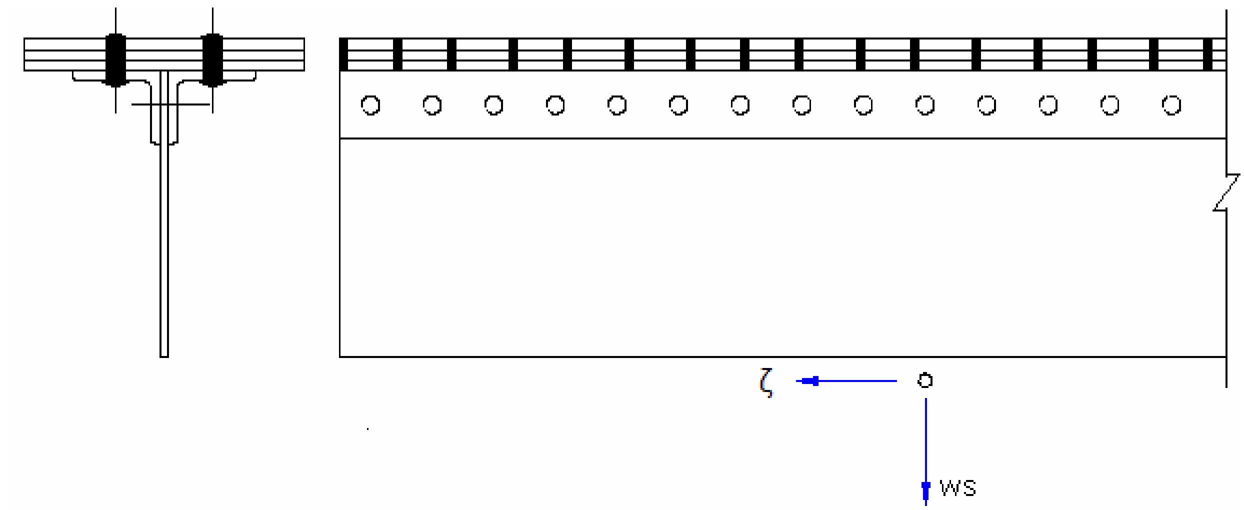


Figure 3.5 Flange angle to web connection

Therefore, the horizontal shear per unit length between the web and flange will be same as the horizontal shear per unit length in the web it self near the connection line.

The pitch of rivets connecting flange angle to web for loaded flange is given by

$$p = \frac{Rv}{\sqrt{\left[\frac{V}{dw} \times \frac{Af_1}{\left(Af_1 + \frac{1}{6}Aw\right)}\right]^2 + (W)^2}} \qquad \ldots (3.10)$$

The pitch of rivets connecting flange angle to web for unloaded flange (W=0) is given by

$$p = \frac{Rv}{\frac{V}{dw} \times \frac{Af_1}{\left(Af_1 + \frac{1}{8} \times Aw\right)}} \qquad \ldots (3.11)$$

3.7.2. Flange angle to web cover plate

The rivets pass through cover plates and flange angle, and are subjected to longitudinal shear only. The rivets are in single shear and bearing. The pitch of rivets connecting flange angle to flange plates for tension flange, if rivets are provided in one straight line, is given by

$$p = \frac{Rv \times dw}{V} \times \left(\frac{A'f_1 + \frac{1}{8}Aw}{A'_1 + A'_2 + \ldots\ldots\ldots A'_n} \right) \quad \ldots (3.12)$$

For the compression flange

$$p = \frac{Rv \times dw}{V} \times \left(\frac{Af_1 + \frac{1}{6}Aw}{A_1 + A_2 + \ldots\ldots\ldots A_n} \right) \quad \ldots (3.13)$$

3.8. STIFFENERS

The web of plate girder being relatively tall and thin is a poor compression member and hence, the possibility of vertical and diagonal buckling is always there. Either the web is stiffened vertically as well as horizontally or the compressive stress in the web is kept low enough to prevents buckling .The elements provided to stiffen the web against this are called intermediate stiffeners.

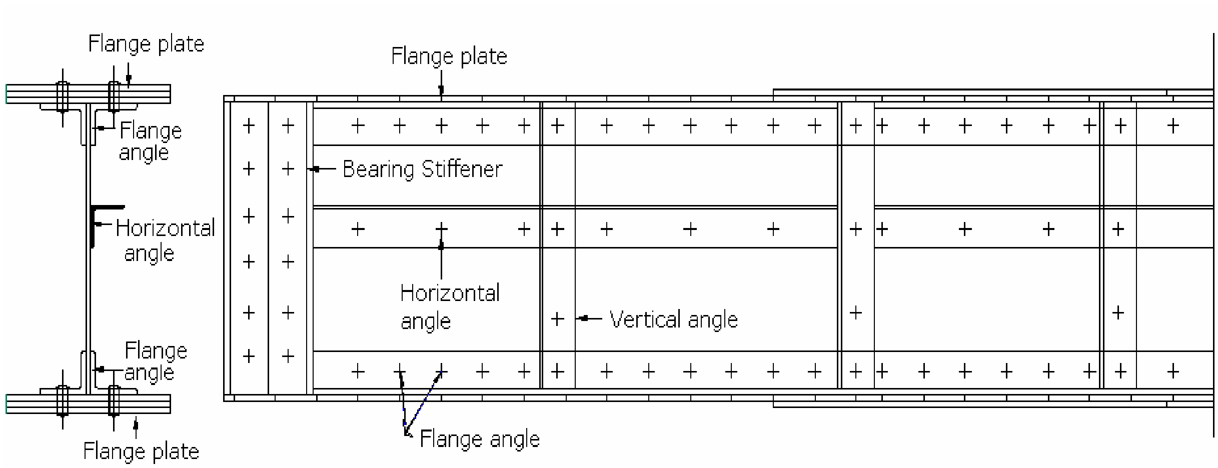


Figure 3.6 Arrangements of Stiffeners

3.8.1. Necessity of web stiffeners (I.S. provision)

IS: 800 -1984 recommended the provision of web stiffener is required as follows. Minimum thickness of the web plate shall be not less than the following:

a) For unstiffened webs :the greater of

$d_1 \frac{\sqrt{\tau va\,,cal}}{816}$ and $d_1 \frac{\sqrt{fy}}{1344}$ but not less than $\frac{d_1}{85}$

Where, d1= clear depth of web

ζva,cal = calculated average stress in the web due to shear force.

b) For vertically stiffened webs :the greater of

1/180 of the smallest clear panel dimension and $d_2 \frac{\sqrt{fy}}{3200}$ but not less than $\frac{d_2}{200}$

c) For webs stiffened both vertically and horizontally with a horizontal stiffener at a distance from the compression flange equal to 2/5 of the distance from the compression flange to the natural axis: the greater of 1/180 of the smallest clear panel dimension and $d_2 \frac{\sqrt{fy}}{4000}$ but not less than $\frac{d_2}{250}$

d) Where there is also a horizontal stiffener at the natural axis of the girder: the greater of 1/180 of the smallest clear panel dimension and $d_2 \frac{\sqrt{fy}}{6400}$ but not less than $\frac{d_2}{400}$

3.8.2 Intermediate stiffeners

These are also called stability stiffeners and are provided to check the diagonal buckling of the web. Depending upon the requirements as discussed 3.8.1 vertical and horizontal stiffeners are provided.

3.8.2.1 Vertical stiffeners

These are also called transverse stiffeners. Angle sections are provided for riveted plate girders and plate sections for welded plate girders. Angle section can be in pairs or may be one in number. When a single angle section is used as vertical stiffeners, the angles are placed alternately on the web plate (figure.3.6).

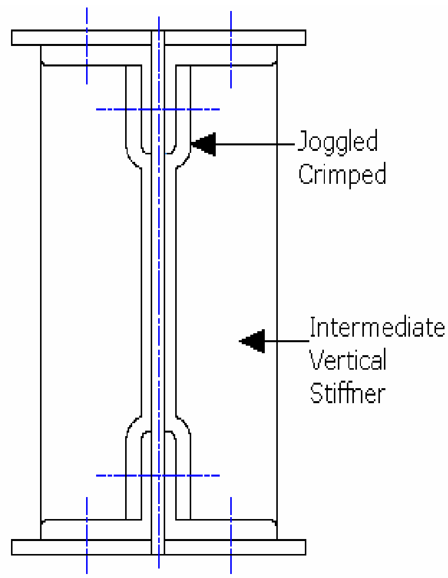


Figure 3.7 Intermediate (vertical) stiffeners

3.8.2.2 Design of Vertical stiffeners

1. The average shear stress in the web plate is computed.

$$\zeta va= \frac{V}{dw \times tw} \qquad ... (3.14)$$

2. Vertical stiffeners are provided at spacing not greater than 1.5 d and not less than 0.33d. Where d is the distance between the flange angles or where there are no flange angles, the clear ignoring fillets when the horizontal stiffeners are also provided d is the maximum clear depth of the web. Spacing can be reduced near the supports when the shear force is large compared to the center of the girder. The maximum clear dimension of the panels formed by intermediate stiffeners should note be greater than 270 t and the lesser dimension of the panels should note be greater than 180t where the t is the thickness of the web.
3. Calculate the ratio d/t and find the maximum spacing ‘c’ of the vertical stiffeners from table 6.6 of I.S: 800-1984.the spacing c, thus calculated should be less than 180 t.
4. The value of the allowable shear stress in the web is found from table 6.6 of I.S:800-1984, corresponding to the d/t ratio and maximum spacing c. this value should be greater than τva, in step (1).

5. The length of the outstanding leg of the stiffeners should no be greater than $\frac{256 \times t}{\sqrt{fy}}$ for angle sections and 12 t for flats, where t is the thickness of section.
6. The moment of inertia of the stiffeners is calculated. If the stiffeners are in a pair, the moment of inertia is found about the center line of the web. If the stiffener is a single section, the moment of inertia is found about the face of the web.
7. The moment of inertia that is provided should not be less than

$$1.5 \times \frac{d1^3 \times tw^3}{c^2} \quad \text{... (3.15)}$$

8. For a riveted plate girdler the diameter of the rivets is assumed and the rivets value in single shear (single angle) or in double shear (pairs of angles) and in bearing is calculated. The connections are designed to resist a shear of

$$\frac{125 \times tw^2}{h} \quad \text{... (3.16)}$$

The pitch of rivets can be calculated by dividing the rivet value by this value of shear force/m run. It is should not exceed the specified values of pitch of rivets as per I.S specifications.

3.8.3 Horizontal stiffeners

Horizontal stiffeners are also called longitudinal stiffeners. This increases the buckling resistance considerably as compared to transverse stiffeners when the web is subjected to bending.

The Horizontal stiffeners consisting of angle section for riveted plate girder and are provided in the compression zone of the web. Horizontal stiffeners are not continuous and are provided between vertical stiffeners.

The horizontal stiffeners are provided at distance of:

1. One fifth of the distance from the compression flange to the tension flange.
2. Another stiffener is provided at the natural axis.

3.8.3.1 Design of horizontal stiffeners

1. The moment of inertia of the first Horizontal stiffeners is calculated by

$$I = 4 \times c1 \times tw^3 \qquad \text{... (3.17)}$$

2. The moment of inertia of another stiffener, it provided, is calculated by

$$I = d2 \times tw^3 \qquad \text{... (3.18)}$$

   Where d2 =twice the clear distance from the compression flange angle to the neutral axis.

3. Horizontal stiffeners may be provided either as single angle sections or in pairs for a riveted plate girder. The outstand of the stiffener should not be greater than $\frac{256 \times t}{\sqrt{fy}}$ for angle section and 12t for flats.
4. Connections of Horizontal stiffeners to web are designed similar to that as for vertical stiffeners.

3.8.4 Bearing stiffeners

Bearing stiffeners are used to transfer concentrated loads on the girder and heavy reactions at the supports to the full depth of the web. A bearing stiffener at the supports is called an end bearing stiffeners. Bearing stiffeners should be placed:

(i) Tight with the web

(ii) Straight (it is not crimped or joggled),

(iii) In pairs of two or four angles, symmetrically placed on both side of the web.

As Bearing stiffeners are provided straight, the back of the connected leg of the stiffener angle will be at some distance (equal to the thickness of flange angle) from the face of web. To achieve tightness suitable packing plates (equal to the thickness of flange angle) are inserted between the web and bearing stiffener.

Additional rivets are calculated as per I.S specification, and are placed on packing.

3.8.4.1 Design of bearing stiffeners

Bearing stiffeners must have sufficient contact area between the flange and the stiffeners to transfer load in bearing.

The Bearing stiffeners are designed as follows:

1. The Bearing area required, A=reaction x σp

   Where σp =permissible bending stress (0.75 x fy) ... (3.19)
2. In the riveted connections angle sections are used. Only the Length of the outstanding leg is considered to provide Bearing Area and is counted outside the fillets of the flange angle. Therefore, the bearing length is calculated by subtracting the length of the root of fillet of flange angle from the length of the outstanding leg of stiffener angle.
3. The Bearing stiffeners are designed as columns with the length of the web 20times the thickness of the web on both sides.

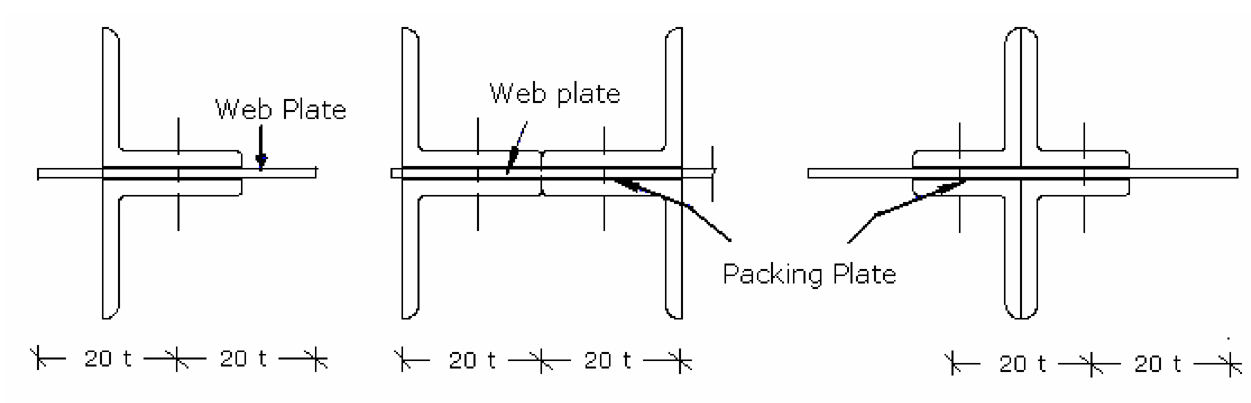


Figure 3.8 Bearing Stiffener

4. The slenderness ratio of the Bearing stiffeners is calculated by

$$\lambda = \frac{d1eff}{r} \quad \text{... (3.20)}$$

Where, d1 = The actual length of stiffeners provided (depth of the web).

r = radius of gyration perpendicular to the plane of web of the girder

$$r = \sqrt{\frac{I}{A}} \quad \text{... (3.21)}$$

Where, I is the moments of inertia And A is the cross Sectional area of the bearing stiffener.

d1eff = 0.7d1 (the effective depth is taken as 0.7 times the Depth of the web to allow for the fact that the stiffeners within the depth of web)

6. Allowable compressive stress corresponding to the calculated value of λ is Found from I.S: 800-1984.
7. The load carrying capacity of the stiffeners is calculated from the product of the allowable compressive stress and bearing area provided. This Value should be more than the concentrated load (or the reaction) to be resisted

3.9 WEB SPLICE

1. The plate girder web may be spliced one or more of the following reasons: The length of the plate girder may be large whereas the plate lengths available are limited.
2. A large size plate is difficult to handle, erect and place as they may get twisted. It is also difficult to transport such plate.

Web splice plate are required only when plate girder are riveted. In the case of welded plate girder the web plates may be butt-welded. Splice plates can be placed any where on the web, as and when required.

Some specification limits the use of splice at the mid span, probably because of large centerline moments, and recommended that the splices should be placed at $1/3^{rd}$ span, if required. But it is very clear that the moment at $1/3^{rd}$ span is also appreciable and is not substantially reduced.

Some of the specification recommended splice plate on both sides of the web, bellow vertical stiffener near $1/3^{rd}$ span, with the view that the stiffeners will provide additional strength to the splice.

3.9.1 Moment Splice

The number of splice plates required will be six. Four plates (two on each side of the web) are placed over the flange angles and two plates (one on each side of the web) are placed on the web. This arrangement is the best splice arrangement and is therefore, sometimes called rational splice. Splice plates are designed for the shear and moment to which the web section subjected, if not spliced. Rivets are designed for the vertical shear due to the bending moment at the section.

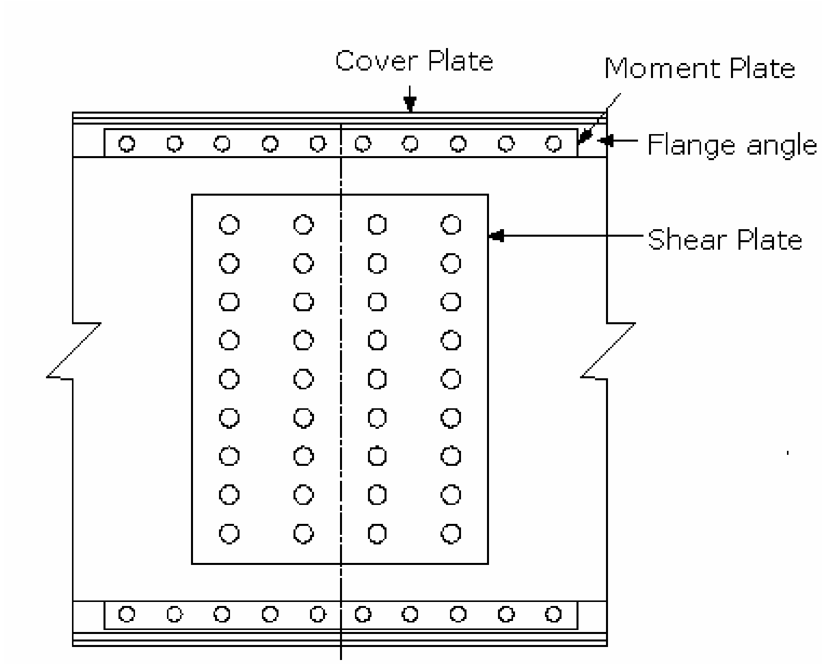


Figure 3.9 Moment Splice

3.9.1.1 Design of Moment Splice

1. The depth of the splice plate on the web will be equal to the depth of the web plate minus the depth of the vertical legs of flange angles. The thickness of the splice plate is kept equal to the half the web thickness and should not be less than 6 mm.
2. The number of vertical rows rivets (two or three) is assumed. the pitch of the rivets is calculated by

$$p = \frac{Rv}{\sqrt{(\sigma bt)^2 + (V/d1)^2}} \quad \text{... (3.22)}$$

3. The number of rows of rivets to be provided is decided and the width of the splice plate is computed.
4. The design of the splice plate over flange angle (for the web) consistence in calculating the length and thickness. Area of the web section below the flange angle to be spliced is computed. The area of splice plate each kept equal to area of the web below the flange angles.
5. The number of rivets required on one side of splice plate is found by

$$n = \frac{\sigma b2 \times Aw1}{Rv - \frac{V}{d_1}\left(\frac{Af}{Af + Aw1}\right) \times s} \qquad \text{... (3.23)}$$

6. The thickness of the splice plate is found by dividing the area calculated in step (4) with the length computed in step (5) and it should not be less than 6mm.

3.10 DESIGN STEPS OF PLATE GIRDER

1. Assume the self-weight the plate girder. Estimate the live load. Compute the Critical moment and shear force in the plate girder.
2. Assume the thickness of the web and calculate the depth of the plate girder.
3. Compute the flange area using equation select suitable flange angle and flange cover plate.
4. Compute the moments of the section and calculate the bending stress in compression. Compute gross area and net area of the section provided. Calculate bending stress in tension, which should be less than the permissible bending stress.
5. Curtail the flange cover plates.
6. Design the connection of flange angle to web and flange angle to cover plate in case of riveted plate girder. Design the connection between flange plate and web plate in case of a welded plate girder.
7. Design the bearing stiffeners and their connection.
8. Design the intermediate stiffeners if required, and their connection.
9. Design the web splice and its connection.
10. Design the flange splice and its connection.

3.11 RESULT OF ANALYSIS AND DESIGN OF TYPICAL RIVETED PLATE GIRDER

INPUT DATA OF RIVETED PLATE GIRDER

Types of bridge		Deck type plate girder bridge			
Gauge		Broad gauge, single track, mainline			
Span of girder	=	30			m
Equivalent total live load for B.M per track	=	2727			kN
Equivalent total live load for S.F per track	=	2997			kN
Yield stress for steel (fy)	=	236			N/mm^2
Permissible bending stress (σbc)	=	141			N/mm^2
Average shear stress ζva	=	85			N/mm^2

OUTPUT RESULT

Bending Moment	=	8288.65			kN.m
Shear force	=	1197.78			kN.m
Web plate	=	2500 x 8			mm
Flange plate (Three Cover Plate)	=	600 x 12			mm
Diameter of Rivet (All rivets)	=	22			mm
Bending stress (σbc) in compression flange	=				
Plate 3	=	83.57	<	141	N/mm^2
Plate 2	=	72.96	<	141	N/mm^2
Plate 1	=	52.41	<	141	N/mm^2
Bending stress (σbt) in tension flange	=				
Plate 3	=	95.19	<	141	
Plate 2	=	84.43	<	141	N/mm^2
Plate 1	=	62.58	<	141	N/mm^2
Shear Stress	=	59.89	<	85	N/mm^2
Necessity of stiffener	=	Vertical as well as horizontal			
Vertical stiffener					

Angle section	=	ISA 90 x 90 x 8	
Moment of inertia	=	289504 < 1910784	mm^4
Pitch of rivets	=	190 c/c	mm
Horizontal stiffener (d1/5)			
Angle section	=	ISA 75 x 75 x 8	
Moment of inertia	=	2273280 < 2648384	mm^4
Pitch of rivets	=	190 c/c	mm
Horizontal stiffener (d1/2)			
Angle section	=	ISA 75 x 75 x 10	
Moment of inertia	=	1152000 < 1404962	mm^4
Pitch of rivets	=	190 c/c	mm
Bearing stiffener			
Angle section	=	ISA 130 x 130 x 15	
Load carrying Capacity of stiffener	=	1197.78 < 2492.06	kN
Pitch of rivets	=	190 c/c	mm
Splice			
Splice on Web plate	=	2300 x250 x 4	mm
Splice over flange angle	=	400 x 70 x 6	mm
Bracing			
Top Lateral Bracing			
Angle section	=	2 ISA 80 x 80 x 8	
Load carrying Capacity	=	251.42 < 576.31	kN
Bottom Lateral Bracing			
Angle section	=	2 ISA 50 x 50 x 6	
Load carrying Capacity	=	62.85 < 65.89	kN
Diagonal member			
Angle section	=	2 ISA 80 x 80 x 8	
End Strut			
Angle section	=	2 ISA 80 x 80 x 8	
End Cross Frame			
Angle section	=	ISA 80 x 80 x 12	

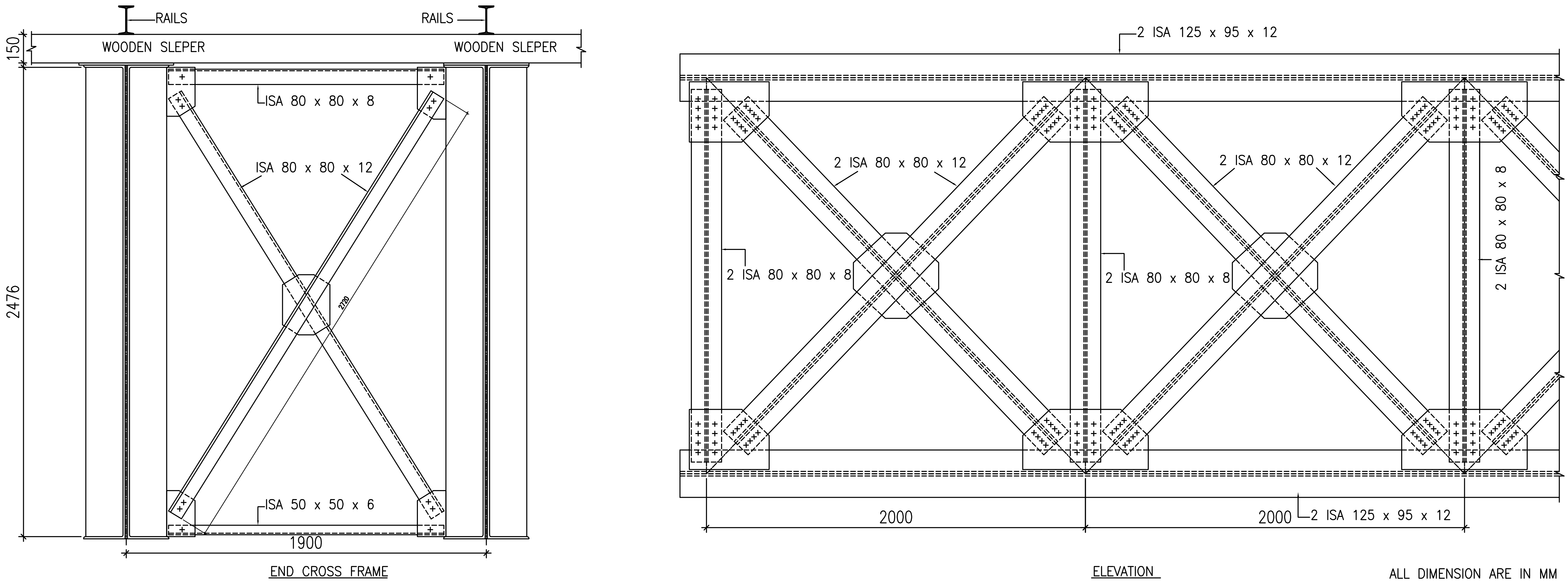


Figure 3.11 Details of Cross Frame in Riveted plate girder

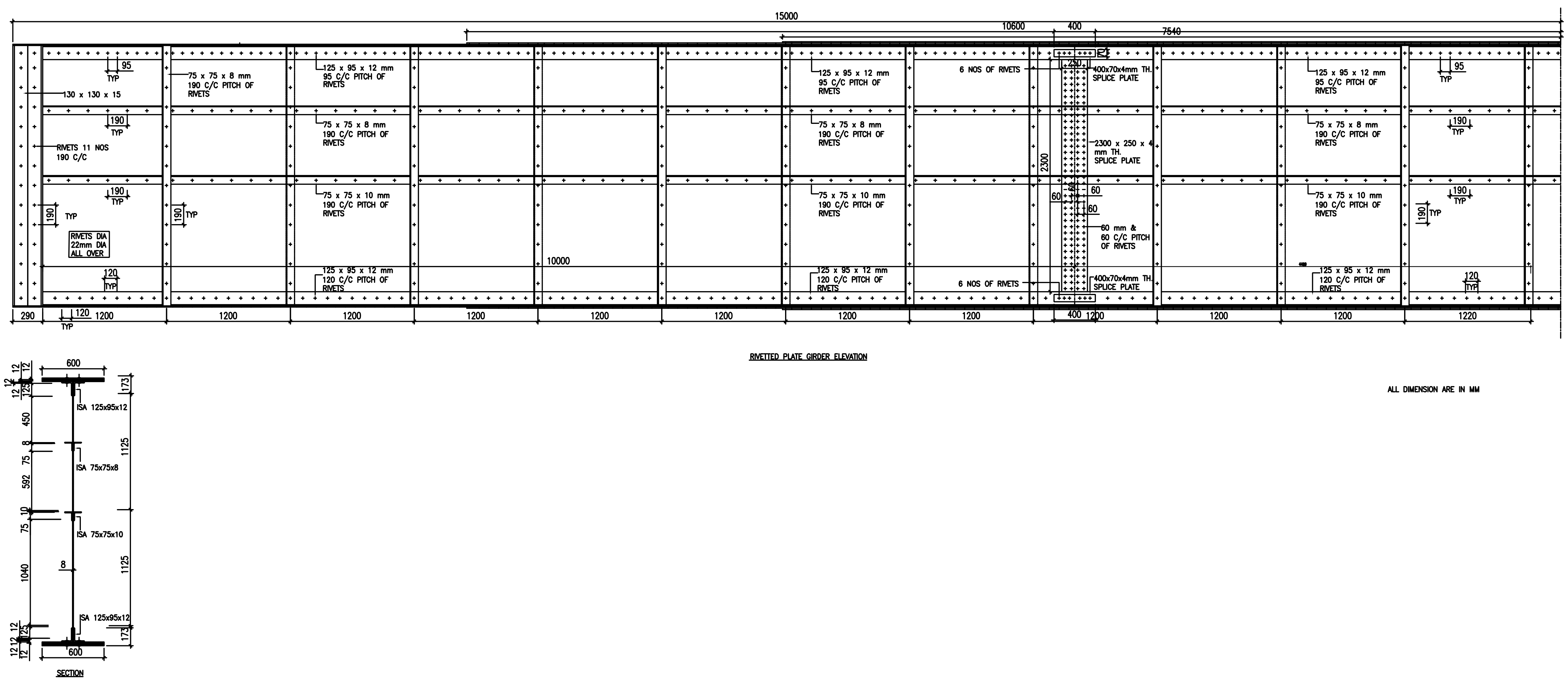


Figure 3.10 Details of drawing of Riveted plate girder

4. ANALYSES AND DESIGN OF WELDED PLATE GIRDER

4.1. GENERAL

A welded plate girder is a more efficient section than riveted plate girder because all the material is effective to resist the load. The typical welded sections are as shown in figure 4.1. The flange plate is welded directly to the web and no flange angles are used.

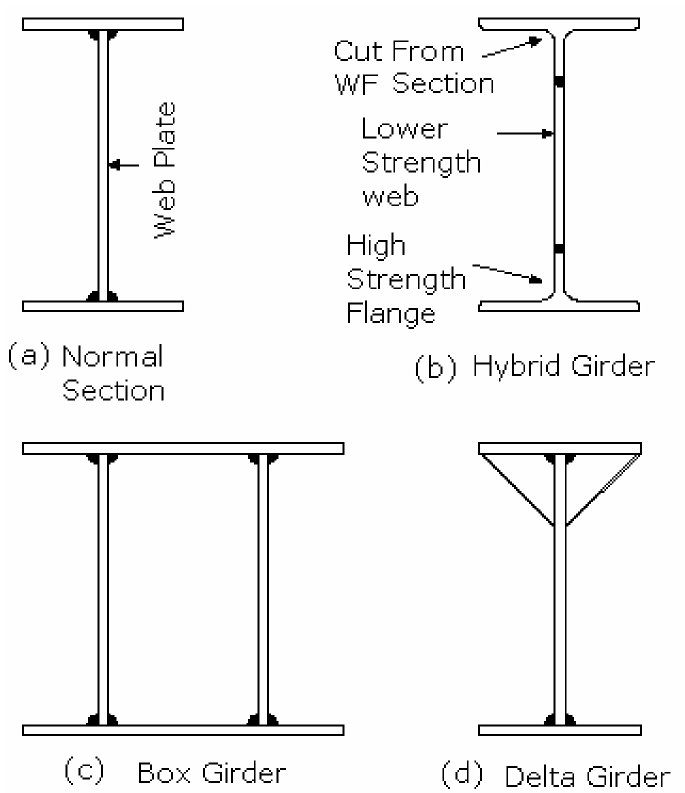


Figure 4.1 cross-section of welded Plate girder

The stacked flange plate not used for welded plate girder because of multiple welding involved, but it is possible to vary the cross section area by flanking the flange plate required for maximum bending moment with one or more successively thinner plate ,butt weld end to end. In welded plate girder, it is not necessary to make an allowance for rivets holes to determine the net section for flange. There is saving in the section of the tension flange. The welded plate girder is economical in material and cost. The design principals are the same as in the riveted plate girder.

4.2. DESIGN PRINCIPLES

Design involves

1. Cross-section design
2. Connection design between flange and web
3. Design of intermediate and bearing stiffeners
4. Stiffener connection with web

4.2.1. Design Steps

1. Compute the live load and dead load moment and shear force. Self weight of girder may be assumed as (0.2L+1) kN/m
2. The design moment and shear force are computed by applying impact factor to live load moment and shear.
3. The depth of a plate girder is usually taken as 1/8 to 1/10 of the span, and

$$\text{Economical depth} = 5 \times \sqrt[3]{\frac{M}{\sigma\ b}} \qquad \dots (4.1)$$

Where M is the design moment

= 141N/mm^2 to 150 N/mm^2 for clear depth to thickness ratio of web is greater or less than 35 respectively

Assuming the thickness of the web as't' (of less than 8 mm) the depth of web is obtained as

$$dw = \frac{V}{\tau v \times tw} \qquad \dots (4.2)$$

ζv = 85 N/mm^2 for mild steel with yield stress 236 N/mm^2 IRC – 24

4. The area of the flange is

$$Af = \frac{M}{\sigma b \times dw} - \frac{Aw}{6} \qquad \dots (4.3)$$

Where Aw is area of web Flange width = L/40 to L/45

5. The connection between flange and web is designed to resist a maximum horizontal shear force given by

$$\tau s = \frac{Va\ \bar{y}}{I} \qquad \dots (4.4)$$

The size of the weld is designed to resist this horizontal force.

6. According to IRC 24,the critical compressive stress for I section having equal M.I about yy axis is given by

$$Cs = \frac{2677300}{\left(\frac{L}{ry}\right)^2} \left\{ 1 + 0.05 \left(\frac{L \times te}{ry \times D} \right)^2 \right\}^{0.5} \quad \text{... (4.5)}$$

4.3. DESIGN OF FLANGE PLATES

The area of the flanges is determined by approximate flange area method and then it is checked by moment of inertia method. Allowance is not made for rivet holes to determine the area of tension flange. The gross area of flange is effective to resist the moment. The flange includes (1/6) th the web area in the tension flange. In order to avoid to buckling, the outstanding of flange plate of flanges plates, with unstiffened edges, that is their projection beyond the face of the web shall not exceed 16 t for the flanges in flexural compression and 20 t for the flanges in flexural tension, where t is the thickness of the plate.

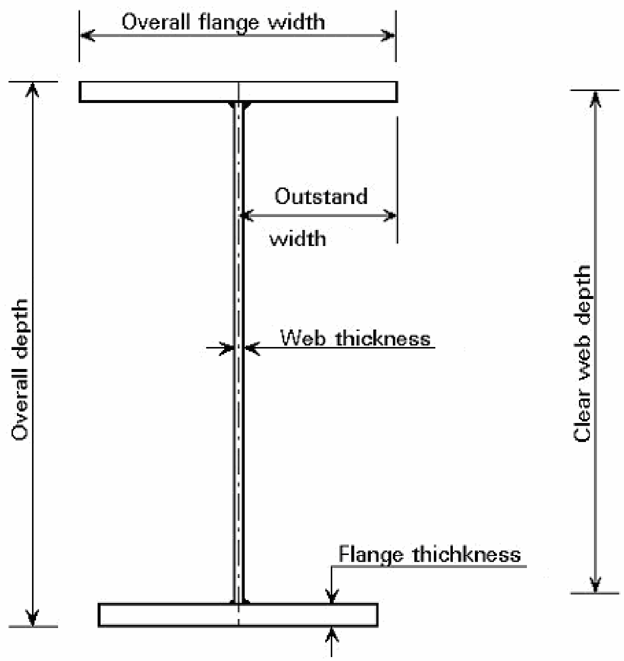


Figure 4.2 Plate girder Proportions

If the difference between the thickness of thicker plate and thinner plate to be joined together exceed 25 % of the thickness of the thinner plate or 3.2 mm, whichever is greater, the thicker plate towards the joint should to be tapered to a slope not greater than 1 in 5 as shown in figure 4.2(a). If this is not applicable the weld metal should be built up at the junction to a dimension at least 25 % of the thickness of the thinner plate as shown in figure (b) provided that the thicker plate is not more than 50% thicker than thinner plate.

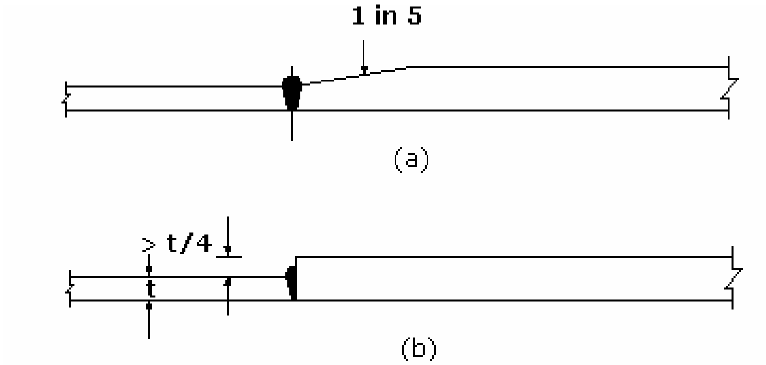


Figure 4.3 Reduction in flange plate thickness

4.4. STIFFENERS

4.4.1. Intermediate stiffeners & its weld connection

Intermittent or continuous fillet welds are provided to connect the flange plate with the web plate. If hand welding is employed, intermittent fillet welding is done and if automatic welding is employed continuous fillet welds may be provided. The welds are designed for horizontal shear**.**

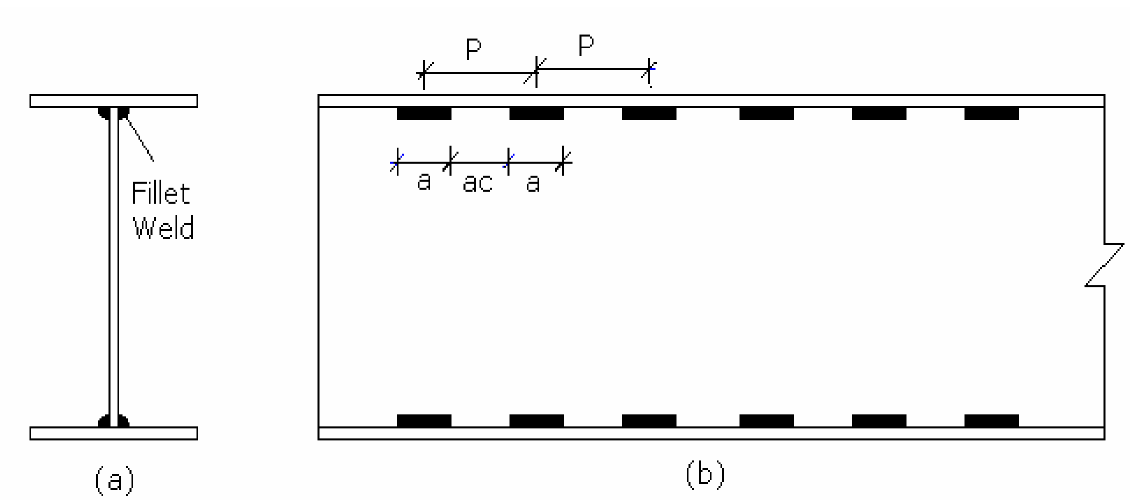


Figure 4.4 Intermittent weld

The clear distance between the intermittent fillet weld should not exceed 12 t or 200mm, whichever is least for a compression flange and 16 t or 200 mm, and whichever is least for tension flange, where t is the thickness of the thinner plate (web or flange).

4.4.2. Bearing stiffeners & its weld connection

The requirements of bearing stiffeners are the same as in a riveted plate girder. The welds joining the bearing stiffeners with the web should be designed to transmit the full reaction or load. The specification regarding welding stated above for intermediate stiffeners also apply to bearing stiffeners.

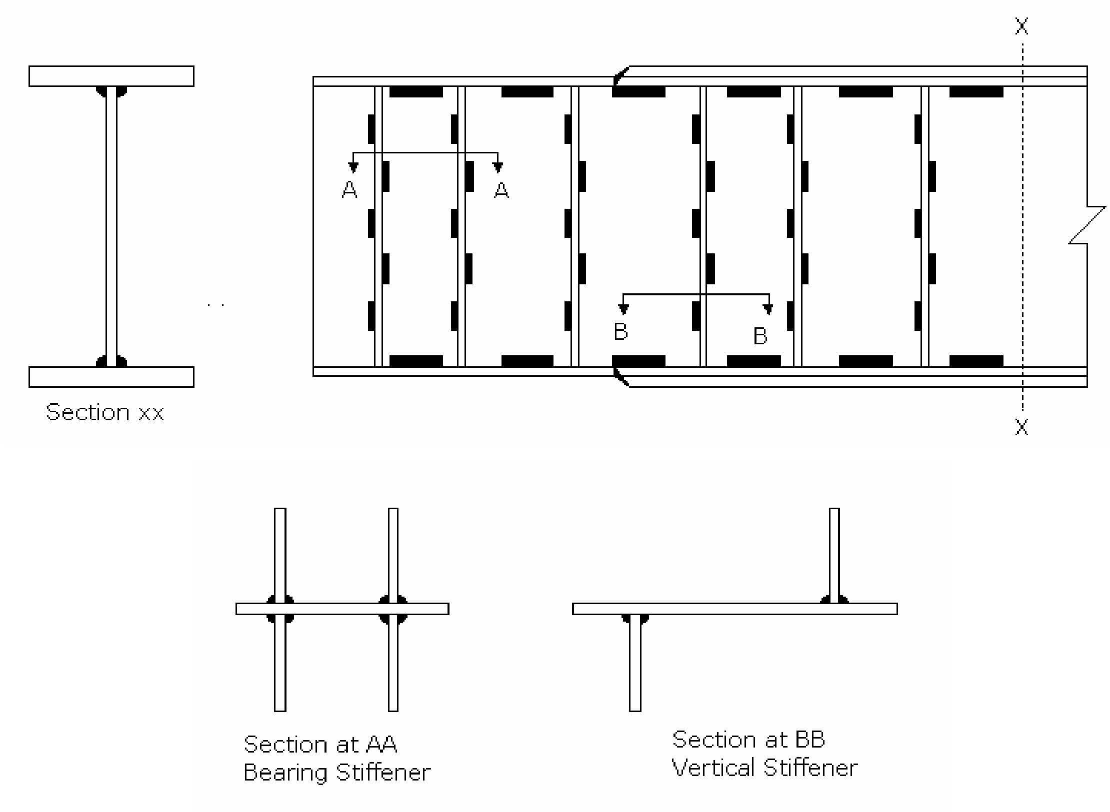


Figure 4.5 Weld connection of bearing stiffener

The Bearing stiffeners are designed columns with the length of the web is 20 times the thickness of the web on both sides.

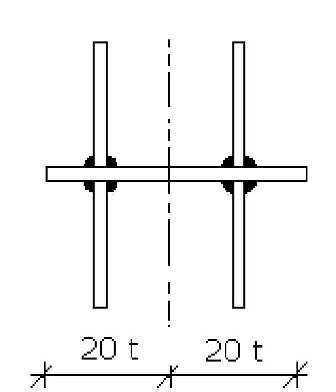


Figure 4.6 Length of bearing stiffener

4.5. STEPS OF STIFFENERS DESIGN

The steps of the stiffeners design please refer in chapter 3 (3.8)

4.6. RESULT OF ANALYSIS AND DESIGN OF TYPICAL WELDED PLATE GIRDER BRIDGE

INPUT DATA OF WELDED PLATE GIRDER

Types of bridge	=	Deck type plate girder bridge	
Gauge	=	Broad gauge, single track, mainline	
Span of girder	=	30	M
Equivalent total live load for B.M per track	=	2727	kN
Equivalent total live load for S.F per track	=	2997	kN
Yield stress for steel (fy)	=	236	N/mm^2
Permissible bending stress (σbc)	=	141	N/mm^2
Average shear stress ζva	=	85	N/mm^2

OUTPUT OF WELDED PLATE GIRDER

Bending Moment	=	8288.65			
Shear force	=	1197.78			kN.m
Web plate	=	2000	x	12	Mm
Flange plate	=	700	x	40	Mm
Reduction in flange plate	=	700	x	30	Mm
Reduction in flange plate	=	700	x	25	Mm
Bending stress (σbc & σbt)	=	130.08	<	158	N/mm^2
Shear Stress	=	59.89	<	87	N/mm^2
Connection between flange and web					
Size of Weld		5 mm Size 40 mm long weld 60 mm c/c			
Necessity of stiffener	=	Web stiffened vertical			
Vertical stiffener					
Moment of inertia	=	5184000 < 5760000			mm^4

Size of plate	=	120	x	10	mm
Size of Weld		5 mm			
Bearing stiffener					
Size of the plate	=	300	x	25	mm
Size of Weld		6 mm 200 c/c			
Area required	=	8555.56	<	21060	mm^2
Top Lateral Bracing					
Angle section	=	ISA 100 x 75 x 10			
Load carrying Capacity	=	148.5	<	204.6	kN
Bottom Lateral Bracing					
Angle section	=	ISA 100 x 75 x 10			
Diagonal member					
Angle section	=	ISA 80 x 80 x 10			
End Strut					
Angle section	=	ISA 100 x 75 x 10			
End Cross Frame					
Angle section	=	ISA 80 x 80 x 8			

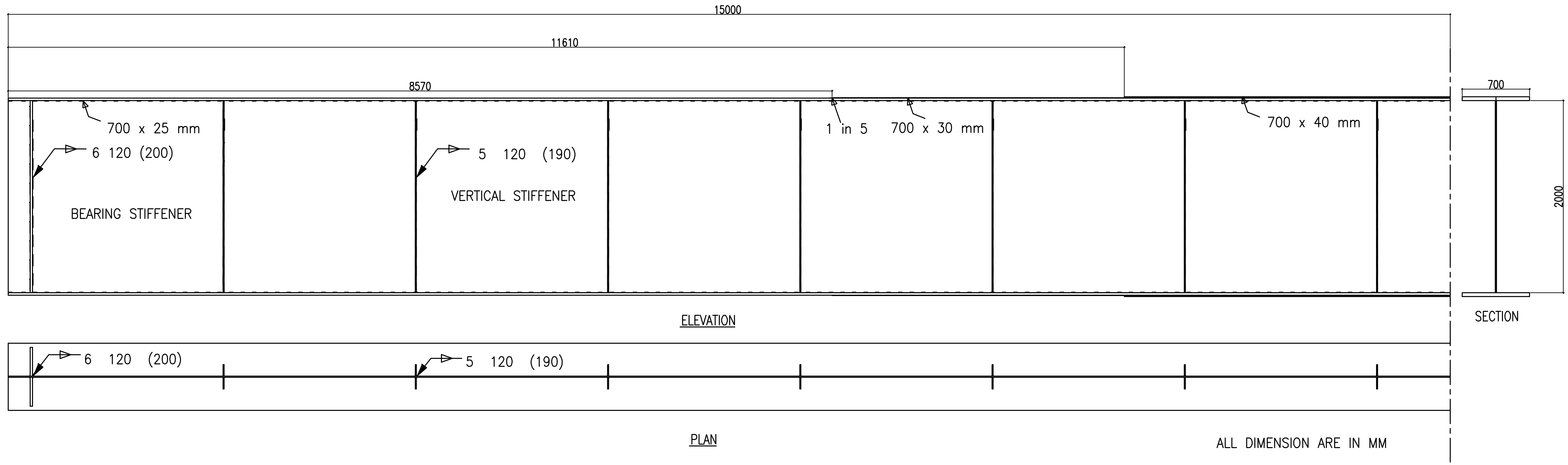


Figure 3.12 Details of drawing of Welded plate girder

5. ANALYSES AND DESIGN OF RAILWAY TRUSS GIRDER

5.1 GENERAL

The truss girders are used as the main load carrying members in the truss girder bridge. The members are subjected to direct tension and compression .The members of the truss girder are classified are chord members and the web members. The uppermost members constitute the top chord. The lowermost members constitute the bottom chord. The vertical member and the diagonal member are the web members.

Those mem bers which are absolutely necessary for the stability of the truss bridge girders are known as the main mem bers. In most of the truss girder bridges, the chord mem bers carry bending moment in the form of direct tension or compression and the vertical and /or diagonal mem ber carry the shear force in the form of direct tension or compression. The members of truss girder bridges are joined through the gusset plate by riveting, bolting or welding.

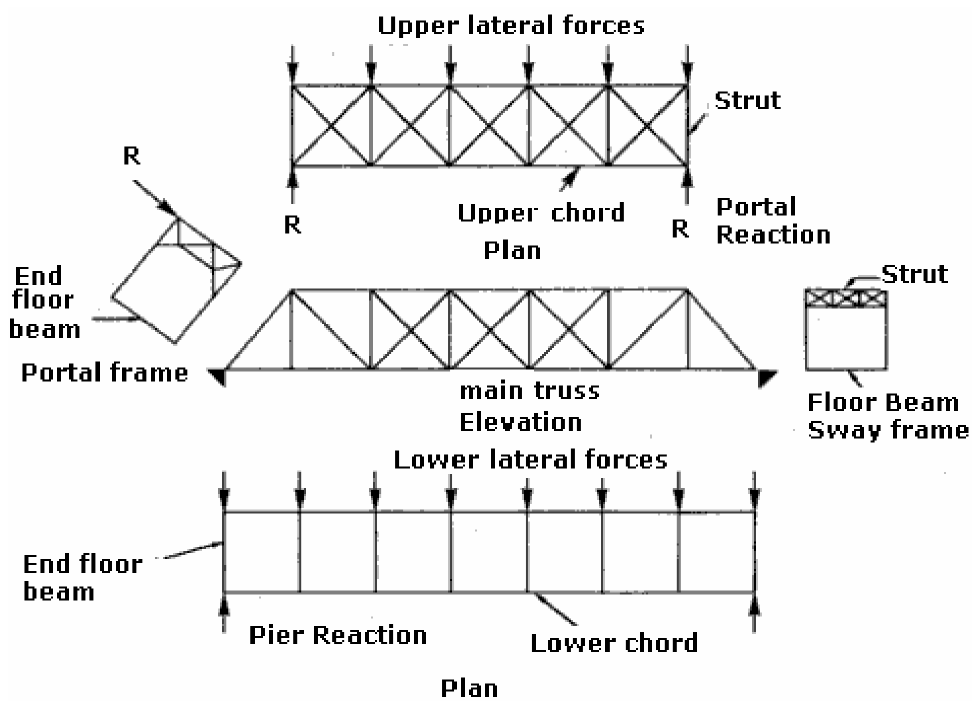


Figure 5.1 Through Truss bridge components

5.2 ECONOMIC PROPORTION OF TRUSS

The ratio of height of truss to the span of bridge producing the greatest economy of material is that which makes the weight of chord members equal to the weight of the members of truss. The economy height to span ratio of truss bridge is one - fifth to one – eighth. It varies with the types of truss bridges. The smaller ratios are used for long span bridges and the larger ratio is adopted for the relatively short span of the bridges. The depth /span ratio is also somewhat dependent upon the live load.

5.3 SELF WEIGHT OF TRUSS GIRDER

The dead load acting on the truss girder bridges also include the self weight of truss girder of the bridge.

The self weight of truss girders are determined by the using fuller's formula is

$$W = (a.l + b) \qquad \ldots (5.1)$$

Where, a and b constant

The dead load exercises a greater influence on the weight. The terms representing weight of truss girder increases faster than the span length. Constant a and b depend upon the types of the bridge.

For the truss girder bridges, the weight of trusses and bracings in kN per meter may be obtained from the formula given below up to 100 m span for the bridges.

$$W = \frac{15.L + 550}{100} \qquad \ldots (5.2)$$

5.4 ANALYSIS OF TRUSS USING INFLUENCE LINE

The influence line diagram for the top and bottom chords are drawn for the bending moment whereas the influence lines for the diagonal and vertical members are drawn for shear force. The influence line diagram will be depending upon the types of the truss and location of the member in the truss.

Bridge truss are subjected to moving loads and such the forces in the truss members can not be taken evaluated unless the assistance of the influence line is taken. Therefore, it is essential to draw the influence lines for forces in the members and the maximum value of force for each truss member is thus determined after placing the moving load for maximum effect.

5.5 TRUSS COMPONENTS

5.5.1 Chords and web member

Chords are top and bottom members that act like the flanges of a beam. They resist the tensile and compressive forces induced by bending. In a constant-depth truss, chords are essentially parallel. Web members consist of diagonals and also often of verticals. Where the chords are essentially parallel, diagonals provide the required shear capacity. Verticals carry shear, provide additional panel points for introduction of loads, and reduce the span of the chords under dead-load bending. When subjected to compression, verticals often are called posts, and when subjected to tension.

5.5.2 Joints

Joints are intersections of truss members. Joints along upper and lower chords often are referred to as panel points.

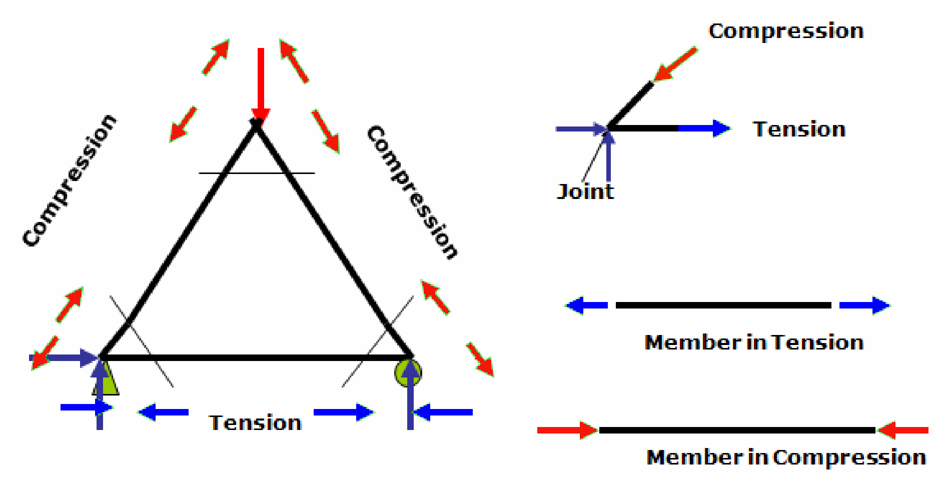
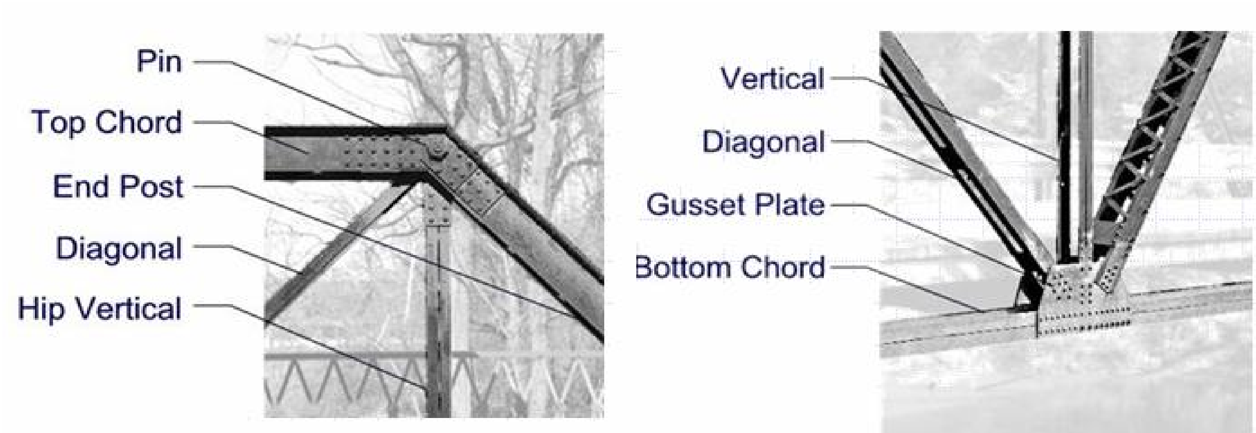


Figure 5.2 Joint of Truss components

To minimize bending stresses in truss members, live loads generally are transmitted through floor framing to the panel points of either chord in older, shorter-span trusses. Bending stresses in members due to their own weight was often ignored in the past. In modern trusses, bending due to the weight of the members should be considered.

5.5.3 Connections

Two types of connections are pinned connections and gusset connections. Single large metal pin that connects two or more members together in pin connection and in gusset plate connection members are joined by one or two heavy gusset plates, which are attached to individual members with rivets, bolts, or welds.



Pinned connections Gusset plate connections

Figure 5.3 Truss connections

5.6 DESIGN OF MEMBERS

Almost all members of a truss bridge are primarily tension or compression members.

5.6.1 Top chord (Compression)

In the truss bridge the top chord member are mostly in compression. The top chord design will be calculate by maximum force in the top chord member. The maximum force is calculated by

- ✓ Draw the I.L.D of top chord member.
- ✓ Find the force due to dead load and live load and impact load.
- ✓ Impact load (refer APPENDIX D)

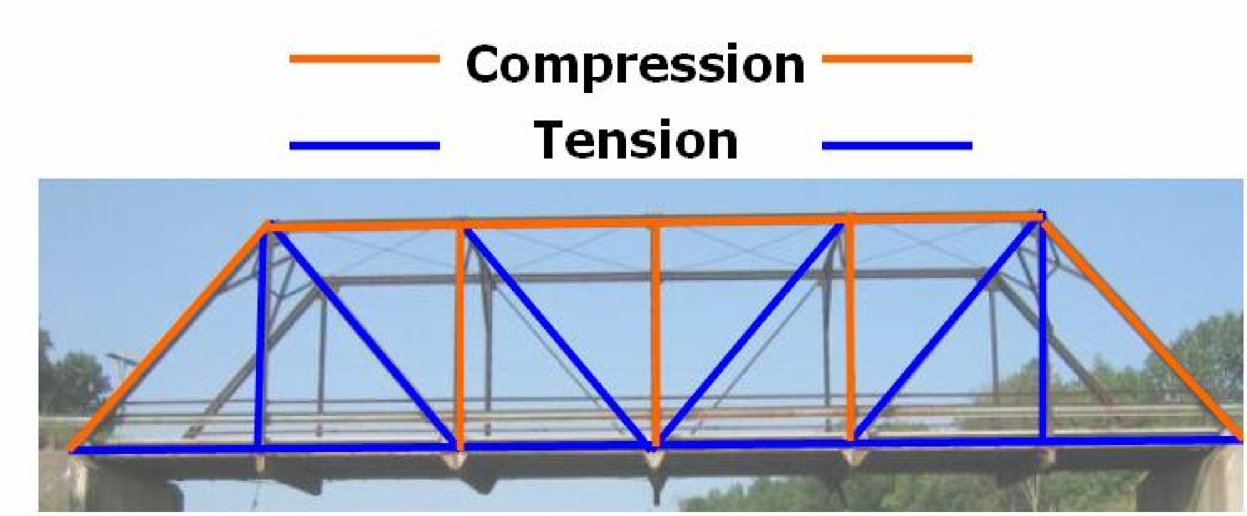


Figure 5.4 Chord member

5.6.2 Bottom chord (Tension)

In the truss bridge the bottom chord member are mostly in tension. The bottom chord design will be calculate by maximum force in the bottom chord member.

5.6.3 Diagonal member and Vertical member

The diagonal chord design will be calculate by maximum force in the diagonal chord member. The truss consists of top chord and bottom chord members connected by vertical and diagonals so that the diagonal members are in tension and the vertical members are in compression .So the diagonal member are design for tension member and check for compression member.

Similarly vertical member are deigned for compression member and check for compression member.

5.7 WIND LOAD AND WIND EFFECTS ON TRUSS GIRDER BRIDGE

Wind load, acting on the bridges, has to be transferred to the bearings by the truss members and the lateral bracings. For this purpose, lateral bracings, in the form of horizontal trusses, are provided both at the level of upper chords, as well as lower chords of the two trusses. Thus, the top lateral bracing joins the top chords of the two trusses while the bottom lateral bracing join the bottom chords of the two trusses. The end reaction of the top lateral bracing are transferred to the bearings through portal bracings provided at the ends, while the sway bracing provided in vertical planes at each panel joint, help in keeping the rectangular shape of the bridge .thus, the wind effects are resisted by the position of four elements

(i) Top lateral bracing
(ii) Bottom lateral bracing

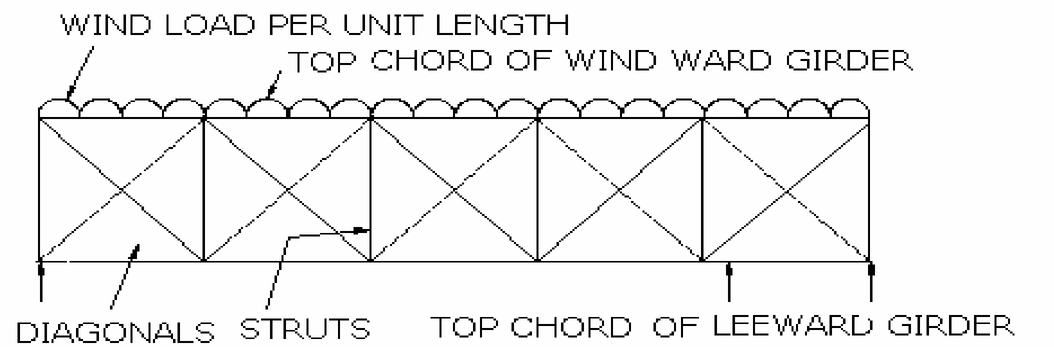


Figure 5.5 Wind Bracing

The entire wind load, acting on the exposed area of the truss is transferred to the top and bottom lateral bracing through the top and bottom panel point .it is assumed that the wind load acing on the truss members above the moving load acts on the top lateral truss while the remaining wind load (which is a major part) acts on the bottom lateral truss. This wind load transferred through panel points causes stresses in the top and bottom laterals and additional stresses in the top and bottom chords, as these form the chord members of the lateral trusses also. to start with, the sizes (i.e. width / depth) of chord and web members exposed to direction are assumed. The area of gussets is assumed to be equal to 20 % of top chord area.

Consider the following effects of wind load acting on truss girder bridges:

1) Overturning effects
2) Lateral effects on top chord and top lateral bracing
3) Lateral effects on bottom chord and bottom lateral bracing
4) Bending and direct stresses in the members transmitting the wind load form the top to the bottom chords

5.7.1 Overturning effect

The wind load P1 and P2 acting on the unloaded through type truss girder bridge as shown figure. The wind load acting over unloaded truss girder bridge is determined by multiplying the basic wind pressure and 1.75 times or twice the area of truss girder in elevation which on is one safer side. A part of this wind load, P1 is assumed to be acting on the top chord. This part of wind load P1 may also be determined by multiplying the basic pressure and twice the area of top chord, area of vertical, area of diagonals and end posts and twice the area of top panel point.

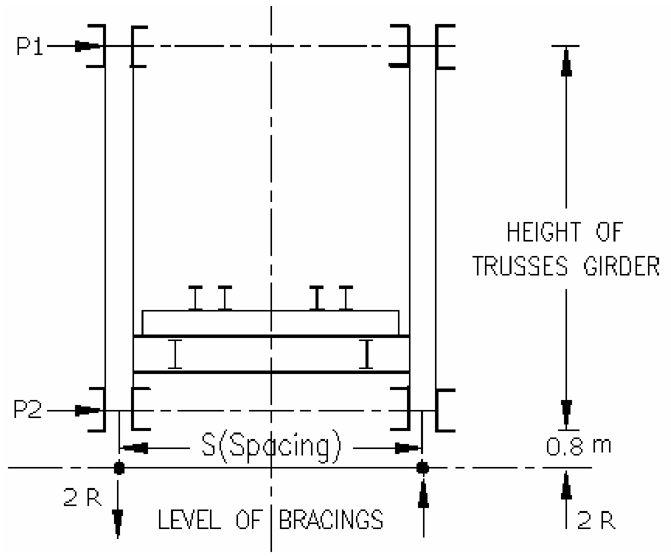


Figure 5.6 Overturning effect of wind

The remaining part of the wind load P2, is assumed to be acting on the bottom chord. This part p2 of wind load may be found by the product of basic wind pressure and twice the area of bottom chord, twice the area of bottom panel point, area of vertical, area of diagonals and area of end posts. These wind loads P1 and P2 are assumed to act at the center of gravities of top chord and bottom chord. The Overturning moment creates two equal and opposite reaction 1R as shown in figure. The value of reaction R may be found by talking the moment of wind load about one bearing

$$2Rxs = P1(h + 0.80) + P2x0.8$$

$$R = \frac{1}{2s}(P1(h + 0.80) + P2x0.8)$$

5.7.2 Top lateral bracing

In case off through type truss girder bridges, the top lateral (horizontal truss) bracing are provided in the horizontal plane between the top chords of bridges. The top lateral bracing are provides rigidity to the bridge structure .the top chords of through Type Bridge are the compression chords. The top lateral bracing connect both the top (compression) chords. This provides stability to these chords. The top chords of main truss girder also acts as chords of top horizontal lateral bracing as shown in figure 5.5.

The cross or double diagonal bracing is most commonly used. This type diagonal bracings gives better appearances. The diagonal are connected at the top points of their intersection. The struts are used to connect opposite points and to complete the horizontal truss. The top lateral (horizontal) truss bracing resists wind load, acting at the center of gravity of top chord. The maximum wind load on the top chord is in case of unloaded span and the same is taken into consideration.

5.7.3 Bottom lateral bracing

In case of the through type bridge, the bottom lateral (horizontal truss) bracing is provided in the horizontal plane between the bottom chords. The bottom chords of main truss girders also act as chords of bottom horizontal (lateral) bracing.

5.8 RESULT OF ANALYSIS AND DESIGN OF TYPICAL TRUSS GIRDER BRIDGE

INPUT DATA OF TRUSS RAILWAY BRIDGE

Types of bridge	=	Pratt truss girder railway bridge	
Gauge	=	Broad gauge, single track, mainline	
Span of girder	=	30	m
Number of panels	=	6	
Equivalent total live load for B.M per track		2727	kN
Equivalent total live load for S.F per track	=	2997	kN

Spacing between main girder	=	7	m
Panel length of bridge girder	=	5	m
Average shear stress ζva	=	85	m

OUTPUT RESULT

Top Chord Member U1U2		
Compression Force	1501.89	kN
Load Carrying Capacity	4674.87 > 1501.89	kN
Bottom Chord Member L2L3		
Tension Force	1335.01	kN
Load Carrying Capacity	1675.24 > 1335.01	
Vertical Chord Member U2L2		
Tension Force	211.13	kN
Compression Force	397.88	kN
Load Carrying Capacity (Compression)	717.57 > 397.88	kN
Load Carrying Capacity (Tension)	385.96 > 211.13	kN
Diagonal Member U2L3		
Tension Force	479.67	kN
Compression Force	387.16	kN
Joints	No. of Rivets	
Joint U1		
Member U1U2	24	
Member U1L1	12	
Member U1L2	12	
Member U1L0	16	
Joint U2		
Member U2U3	20	
Member U2L2	20	
Member U2L3	12	
Member U2U1	20	

Joint L0	
Member L0L1	16
Member L0U1	16
Joint L1	
Member L1L2	16
Member L1L0	16
Member L1U1	16
Joint L2	
Member L2U2	20
Member L2L3	20
Member L2L1	20
Member L2U1	12

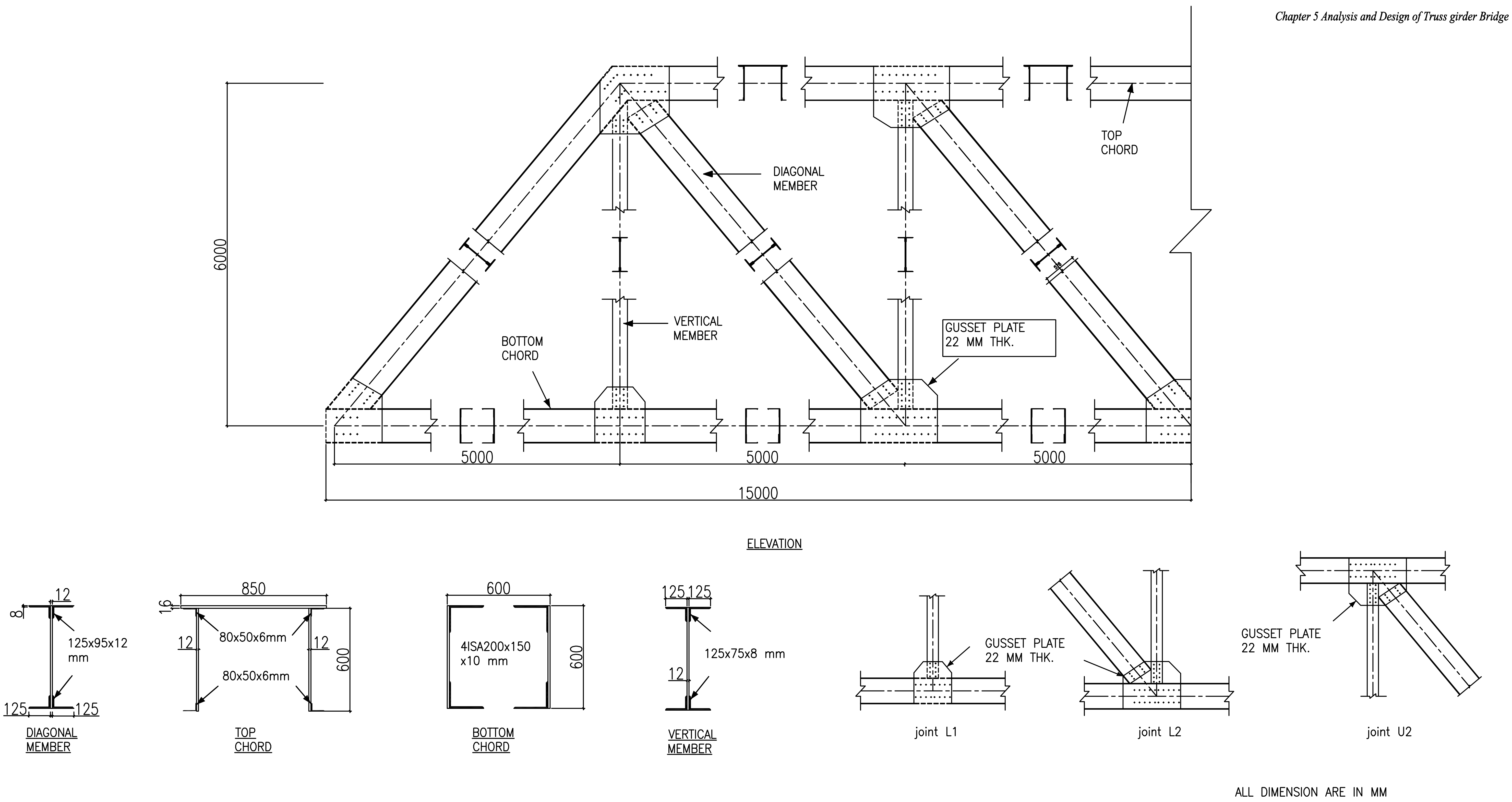


Figure 5.7 Analysis and Design of Truss girder Bridge

6. SOFTWARE FOR ANALYSIS AND DESIGN OF RIVETED PLATE GIRDER

6.1 GENERAL

Software for analysis and design of Riveted plate girder is prepared using Visual Basic as a tool. Software is capable of analyzing the Riveted plate girder. With input of span, web thickness, and selection of angle section. Complete output is available for flange plate, web plate, shear force, bending moment, intermediate vertical stiffener, bearing stiffener, and splice, and also pitch distance. Analysis of Riveted plate girder is carried out using as per I.R.S.

The process of using the software is explained in further section.

6.2 ANALYSIS OF RIVETED PLATE GIRDER

Input data for analysis of Riveted plate girder is as shown in fig.6.1. The software gives Maximum bending moment and Maximum shear force as shown in fig. 6.2.

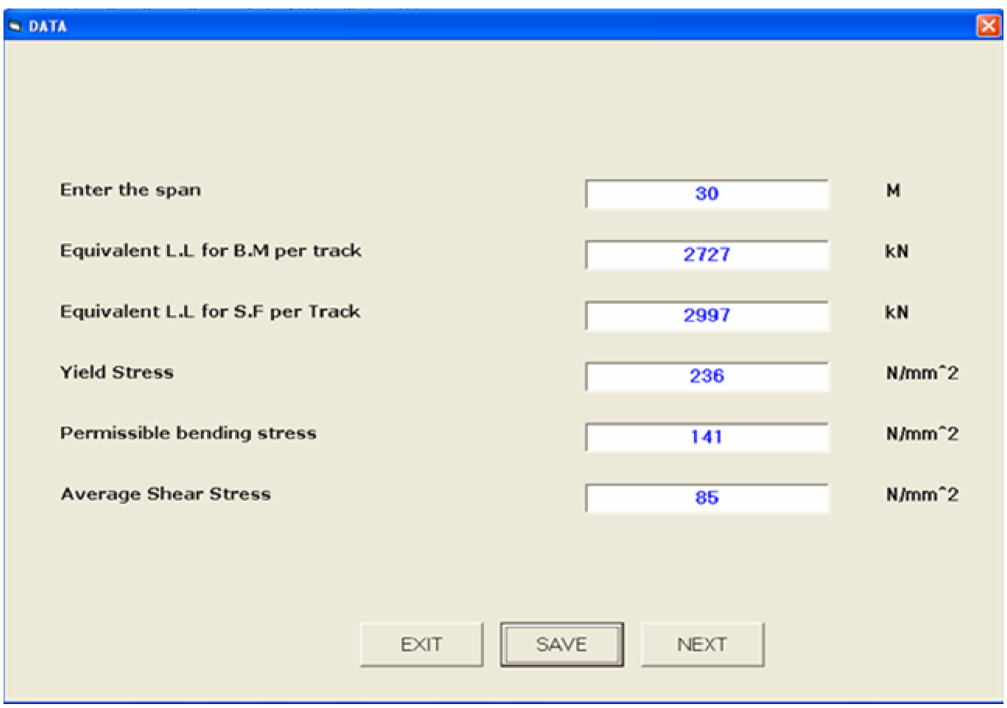


Fig. 6.1 Input for analysis of Riveted plate girder

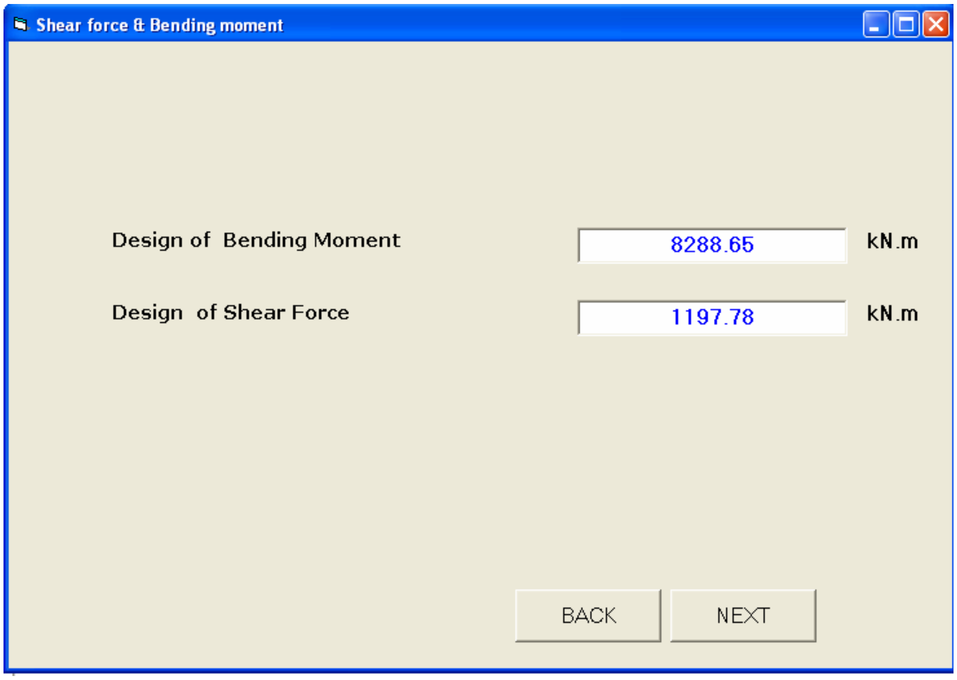


Fig. 6.2 bending moment and shear force of Riveted plate girder

Hence the software gives the economical web depth for Riveted plate girder as shown in fig. 6.3. The minimum flange plate thickness is obtained on selection of the web thickness and angle section as shown in fig. 6.4**.** This form also calls for rivet diameter.

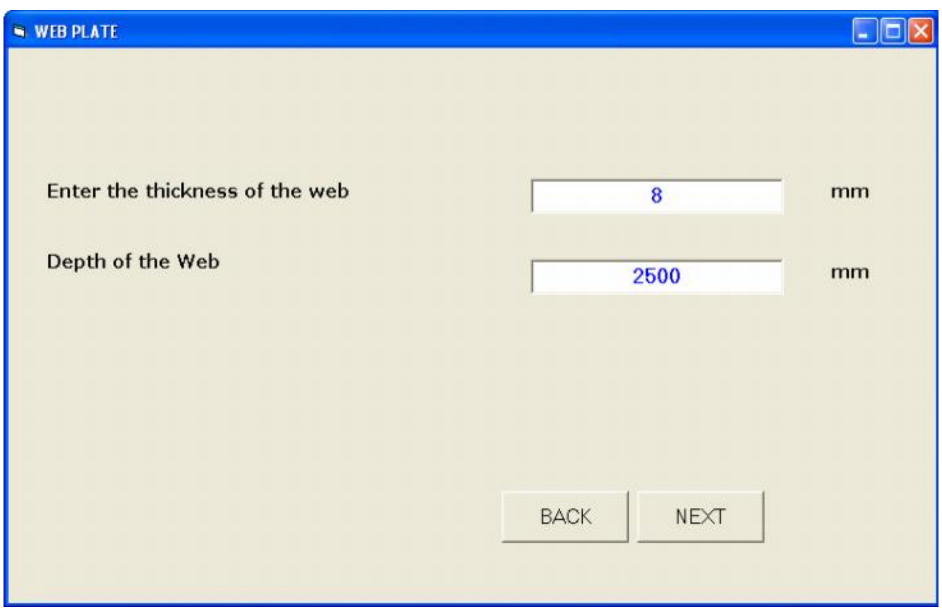


Fig. 6.3 Economical depth of the web

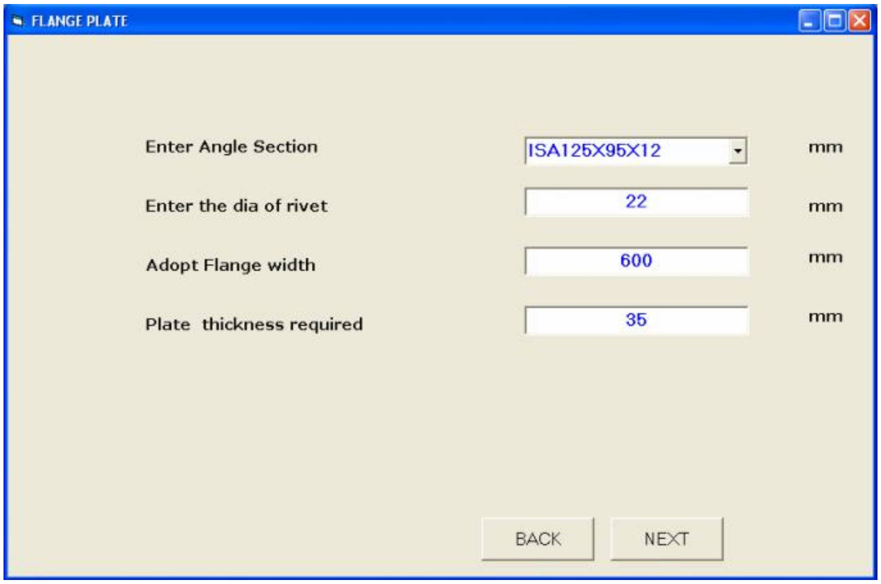


Fig. 6.4 Thickness of the flange plate

This application allows use of three plates in flange- top and bottom which can be curtailed at appropriate distances where the moment gets reduced. The form below requires input for both the top and bottom flange plates thickness. A reference figure is included for clarity.

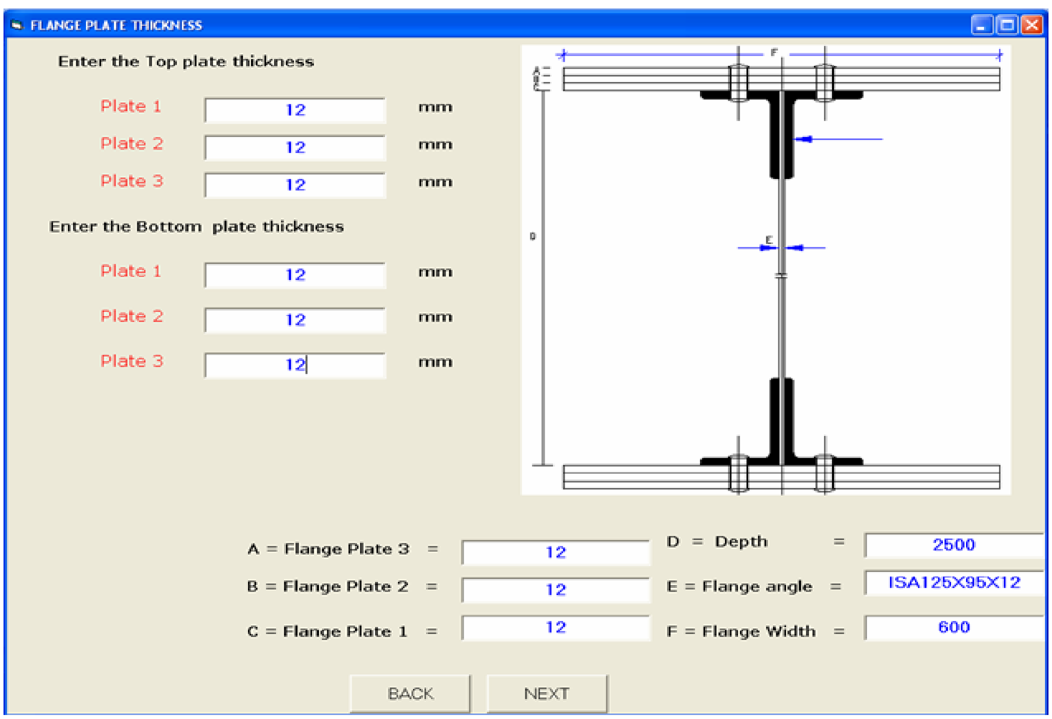


Fig. 6.5 propagation of the riveted plate girder

For the selected thickness of flange plates, check for Bending stress and Shear stress check is carried. Fig. 6.6 shows the check of bending stress of compression flange plate and tension flange plate. Fig. 6.7 provides the check of shear stress.

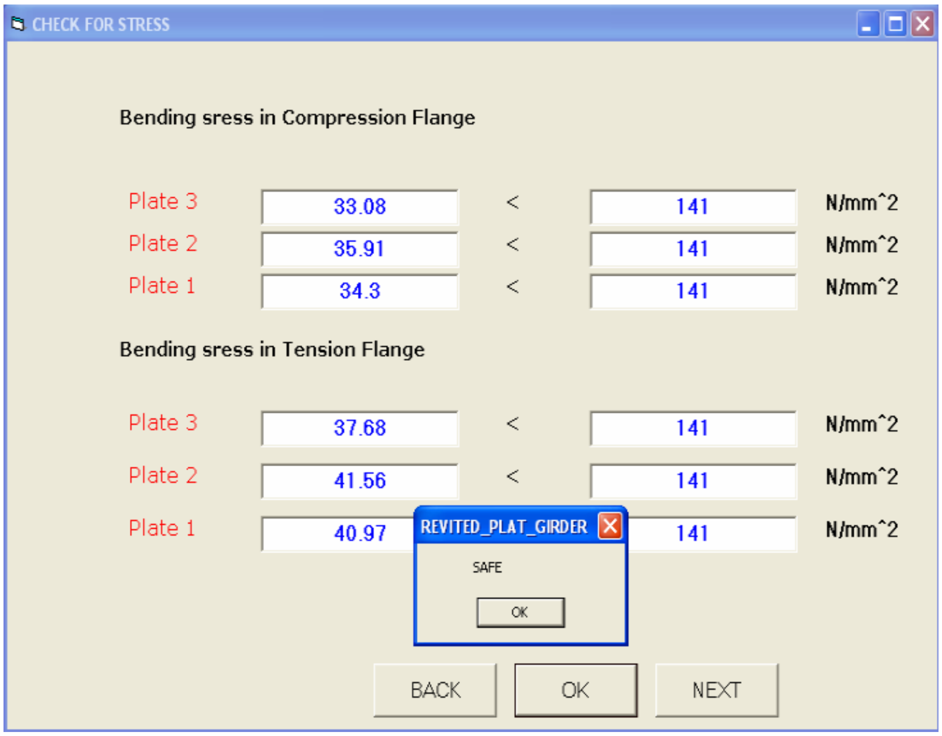


Fig. 6.6 Bending stress check

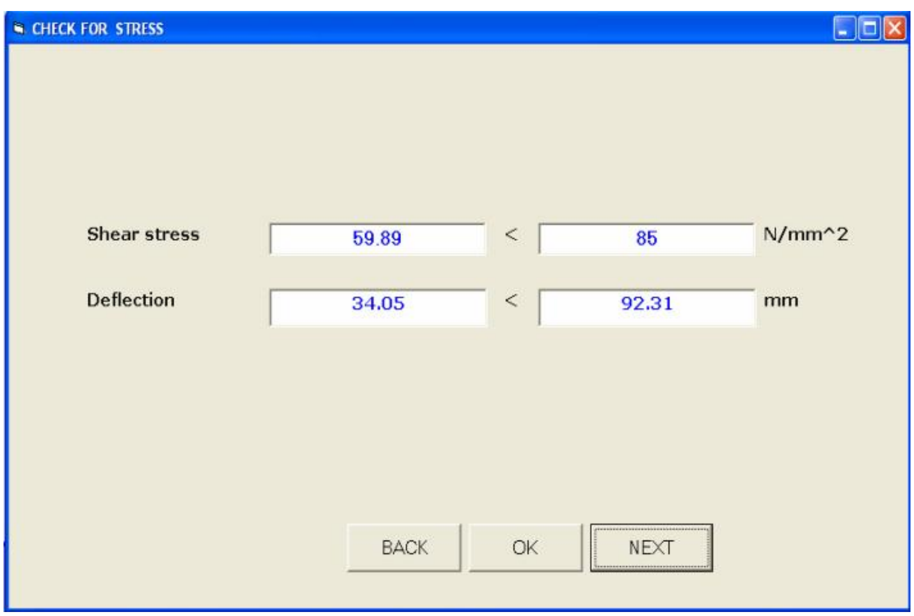


Fig. 6.7 Shear stress check

The next step of design is for the vertical stiffener. The selection of appropriate angle section can be done using the drop down menu. The pitch distance of the rivets for connecting angles to web plate is thus obtained.

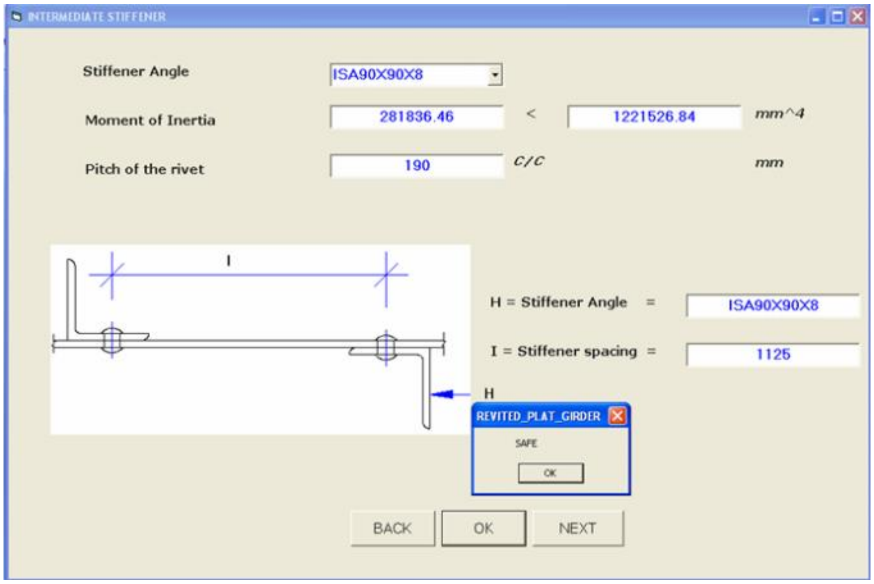


Fig. 6.8 Vertical Stiffener

Form shown in Figure 6.9 appears only if the need of horizontal stiffener arises. Selection of angle sections for use as horizontal stiffener can be done here.

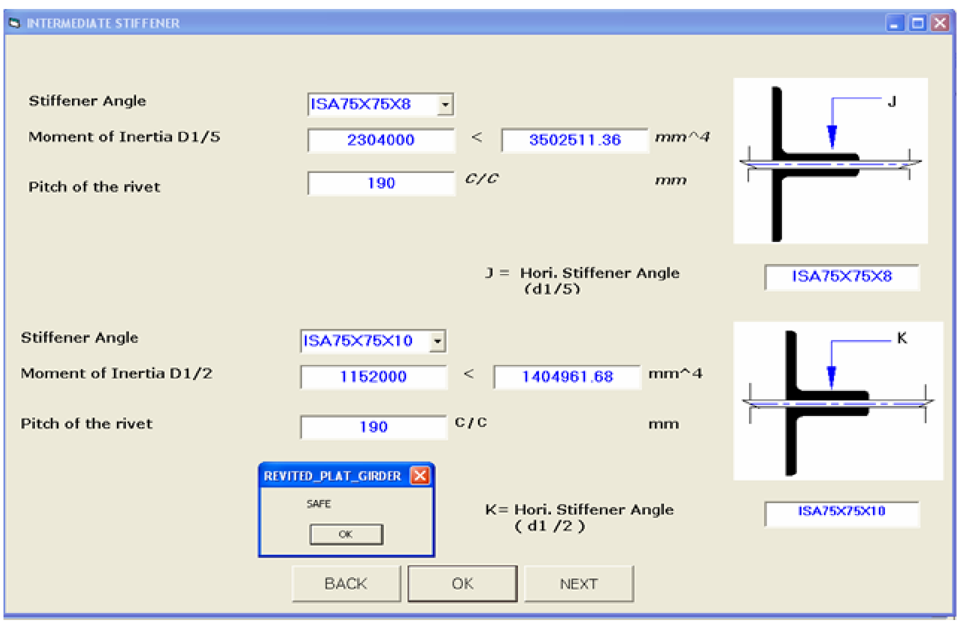


Fig. 6.9 Horizontal Stiffener

The design of bearing stiffener also needs selection angle sections. If the design for selected section is safe, the message box displays SAFE.

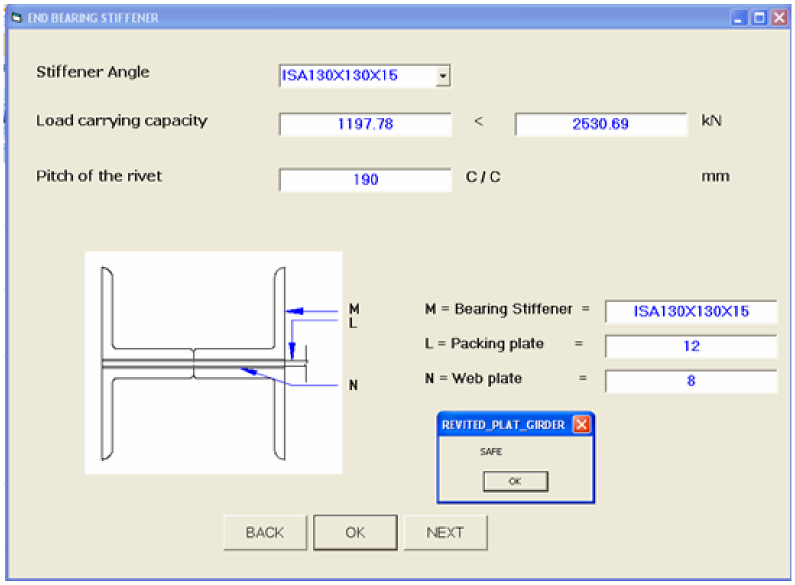


Fig. 6.10 Bearing Stiffener

Finally the design of web as well as flange splice plate is done, for required size of plates and rivets spacing.

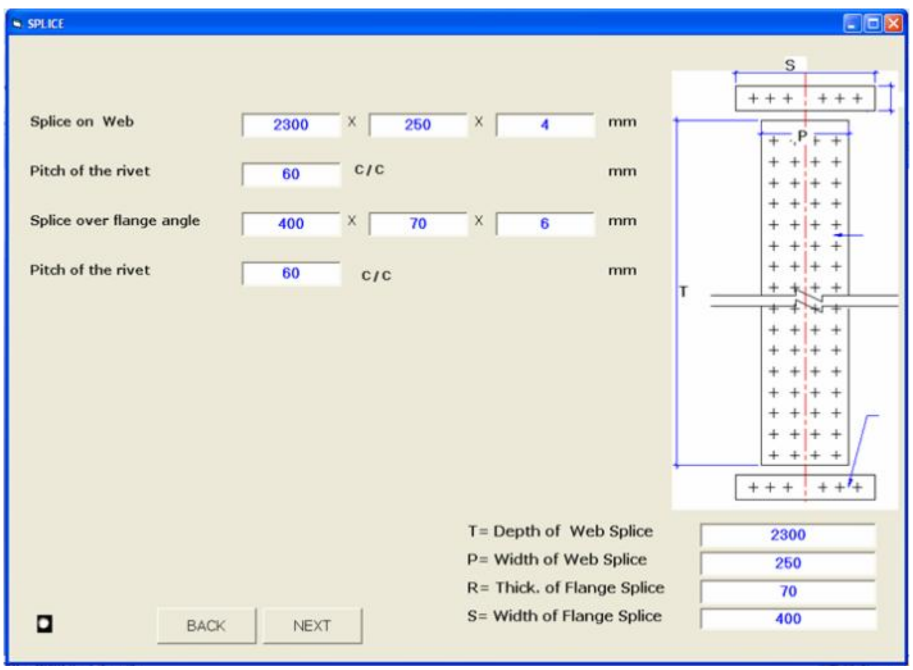


Fig. 6.11 Splice of Web and flange

7. PARAMETRIC STUDY

7.1. GENERAL

Parametric study for obtaining effective depth to Web thickness (D/tw) ratio is carried out with the use of the software prepared. Quantity calculation for total steel required for different depth of webs is carried out, which provides an idea of the economical section. A study of Bending stress, shear stress, deflection and Quantity are done and further compared on behalf of Depth (D) to Web Thickness (tw), which is varying for 30 meters span in riveted and welded plate girder Railway Bridge.

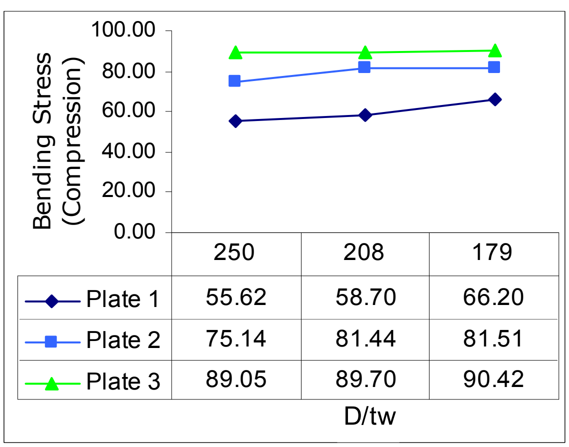
Result of study for different case parameters indicates the variation, which can been identified in below graphs, where span is a fixed length of 30 meters in every case.

Here three different cases are taken into consideration in riveted as well as welded plate Girder Bridge:

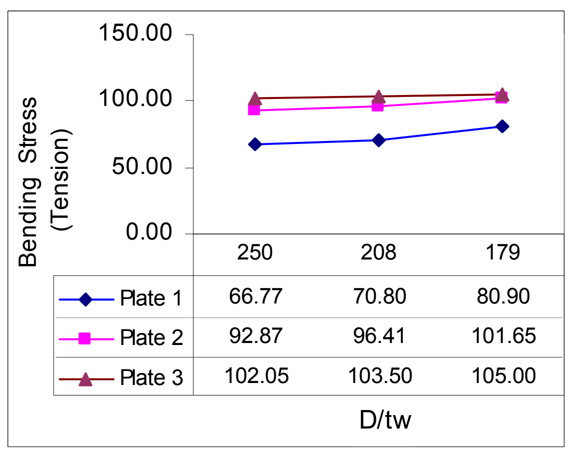
1. Depth (D) is Fixed and Web Thickness (tw) varying
2. Variation in Depth (D) and Web Thickness (tw) Fixed
3. Variation in both Depth (D) and Web Thickness (tw)

7.2. Riveted plate girder - 30 m span

7.2.1. Variation of bending stress in riveted plate girder

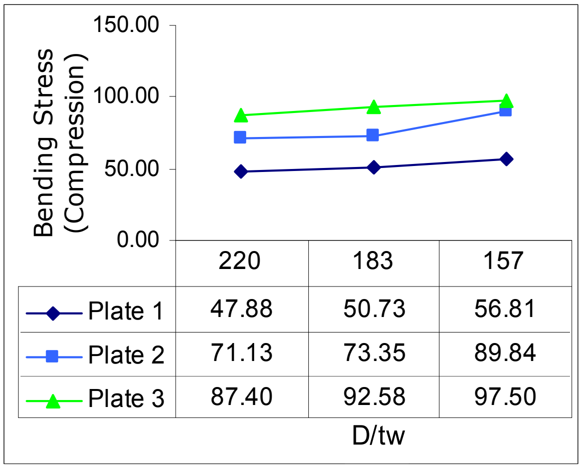


	250	208	179
Plate 1	55.62	58.70	66.20
Plate 2	75.14	81.44	81.51
Plate 3	89.05	89.70	90.42

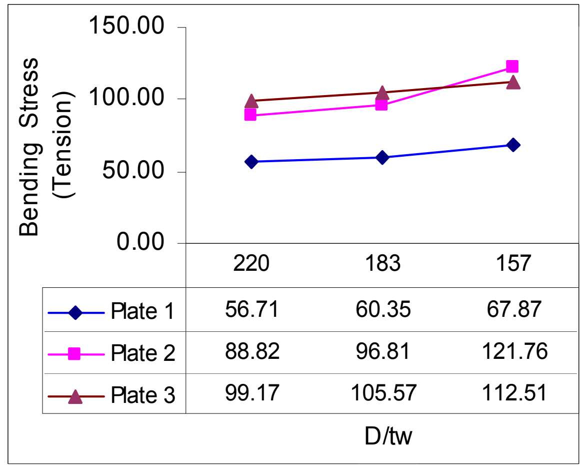


	250	208	179
Plate 1	66.77	70.80	80.90
Plate 2	92.87	96.41	101.65
Plate 3	102.05	103.50	105.00

Figure 7.1 Effect of bending stress due to depth -2500 mm in Riveted plate girder

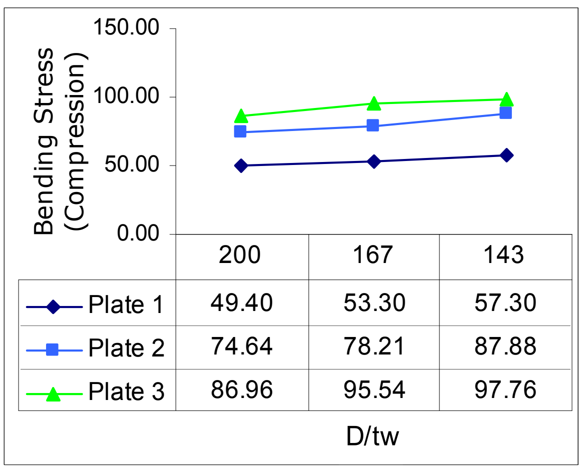


D/tw	220	183	157
Plate 1	47.88	50.73	56.81
Plate 2	71.13	73.35	89.84
Plate 3	87.40	92.58	97.50

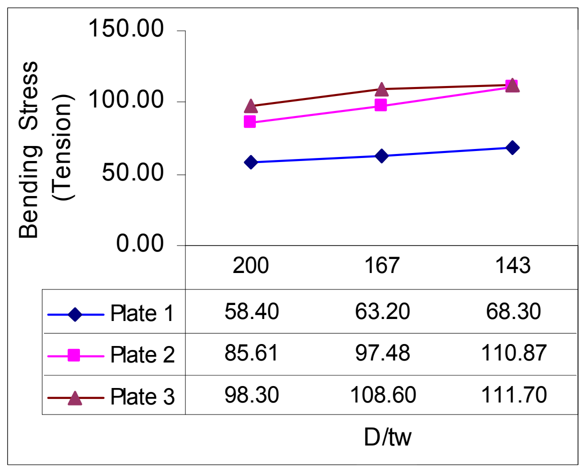


D/tw	220	183	157
Plate 1	56.71	60.35	67.87
Plate 2	88.82	96.81	121.76
Plate 3	99.17	105.57	112.51

Figure 7.2 Effect of bending stress due to depth - 2200 mm in Riveted plate girder

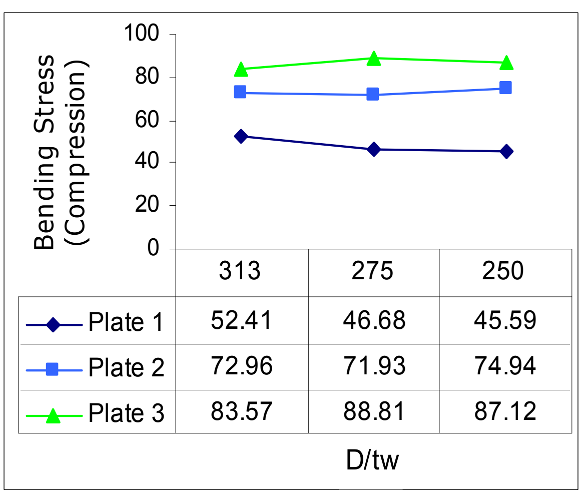


D/tw	200	167	143
Plate 1	49.40	53.30	57.30
Plate 2	74.64	78.21	87.88
Plate 3	86.96	95.54	97.76

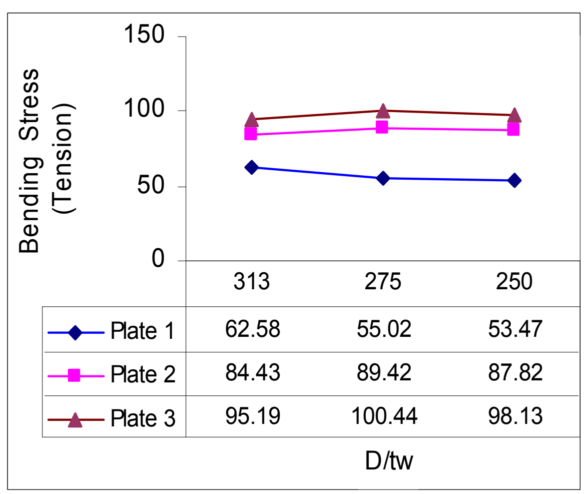


D/tw	200	167	143
Plate 1	58.40	63.20	68.30
Plate 2	85.61	97.48	110.87
Plate 3	98.30	108.60	111.70

Figure 7.3 Effect of bending stress due to depth - 2000 mm in Riveted plate girder

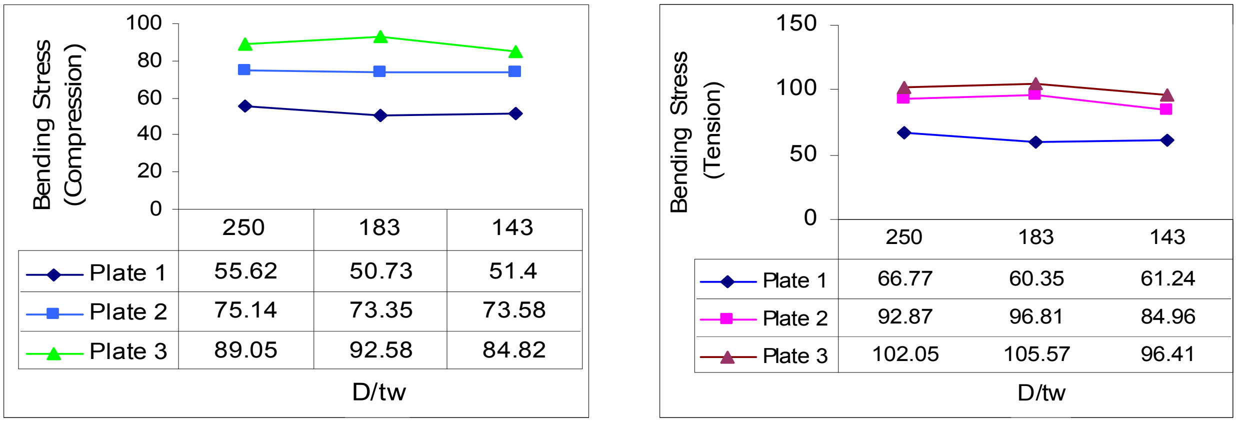


D/tw	313	275	250
Plate 1	52.41	46.68	45.59
Plate 2	72.96	71.93	74.94
Plate 3	83.57	88.81	87.12



D/tw	313	275	250
Plate 1	62.58	55.02	53.47
Plate 2	84.43	89.42	87.82
Plate 3	95.19	100.44	98.13

Figure 7.4 Effect of bending stress due to depth Variation and Web thickness 8 mm in Riveted plate girder

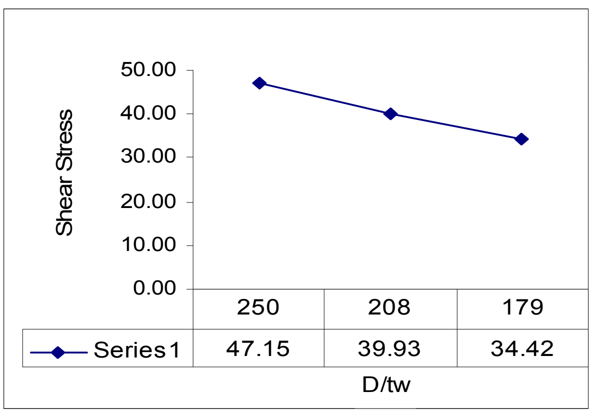


	250	183	143
Plate 1	55.62	50.73	51.4
Plate 2	75.14	73.35	73.58
Plate 3	89.05	92.58	84.82

	250	183	143
Plate 1	66.77	60.35	61.24
Plate 2	92.87	96.81	84.96
Plate 3	102.05	105.57	96.41

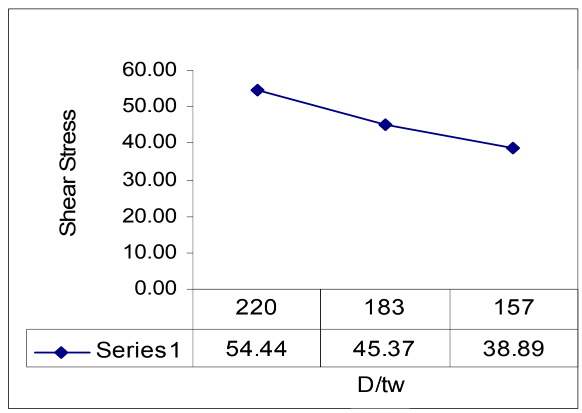
Figure 7.5 Effect of bending stress due to Variation in both depth and Web thickness in Riveted plate girder

7.2.2. Variation of Shear stress in riveted plate girder



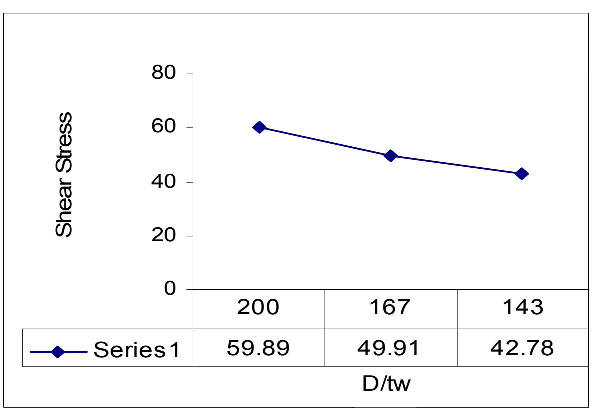
	250	208	179
Series1	47.15	39.93	34.42

Figure 7.6 Effect of shear stress due to depth - 2500 mm in Riveted plate girder



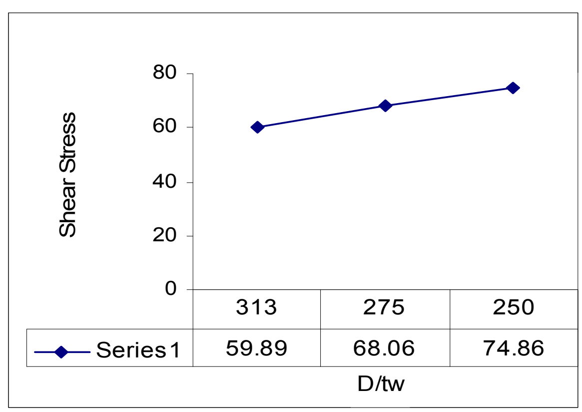
	220	183	157
Series1	54.44	45.37	38.89

Figure 7.7 Effect of shear stress due to depth – 2200 mm in Riveted plate girder



	200	167	143
Series1	59.89	49.91	42.78

Figure 7.8 Effect of shear stress due to depth - 2000 mm in Riveted plate girder



	313	275	250
Series1	59.89	68.06	74.86

Figure 7.9 Effect of shear stress due to Variation in depth and Web thickness 8 mm in Riveted Plate girder

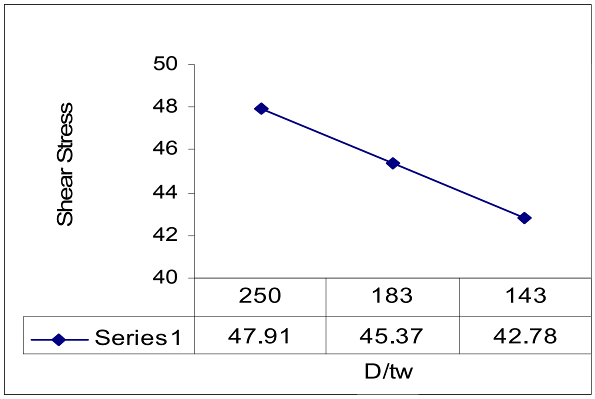


Figure 7.10 Effect of shear stress due to Variation in both Depth and Web thickness in Riveted plate girder

7.2.3. Variation on Deflection in riveted plate girder

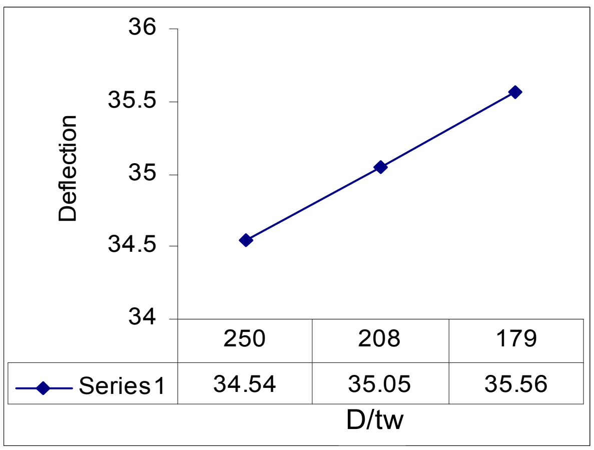


Figure 7.11 Effect of deflection due to depth - 2500 mm in Riveted plate girder

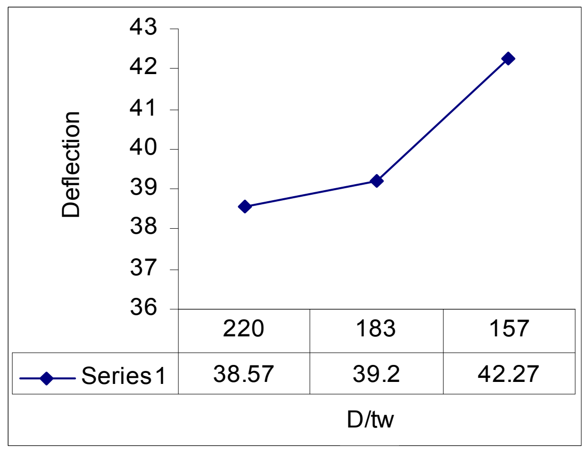


Figure 7.12 Effect of deflection due to depth - 2200 mm in Riveted plate girder

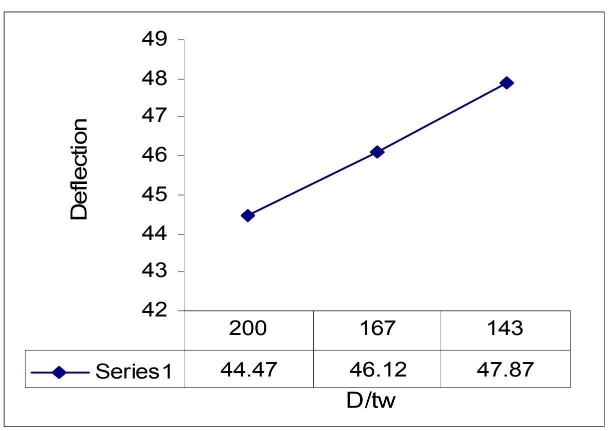


Figure 7.13 Effect of deflection due to depth - 2000 mm in Riveted plate girder

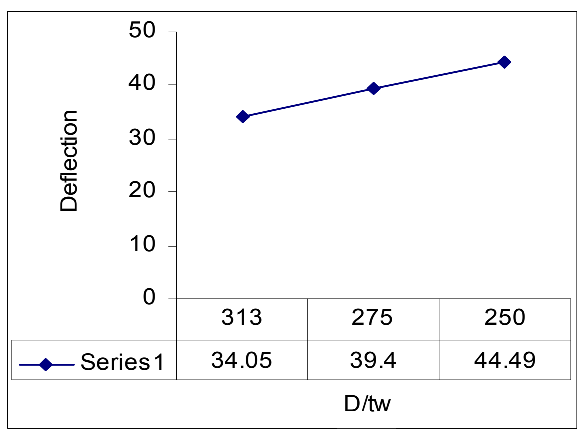


Figure 7.14 Effect of deflection due Variation in depth Web thickness mm in Riveted plate Girder

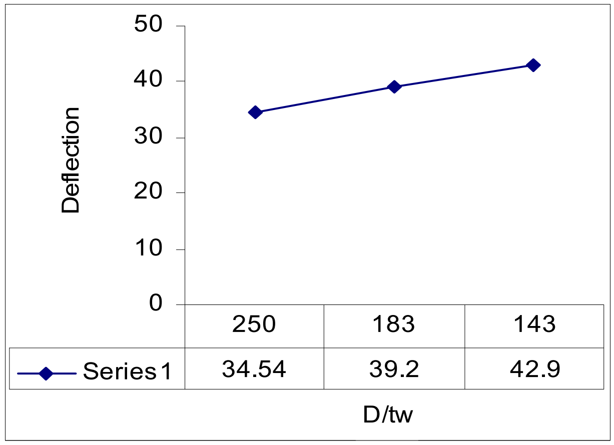


Figure 7.15 Effect of deflection due to Variation in both Depth and Web thickness in Riveted plate girder

7.2.4. Variation of Quantity in riveted plate girder

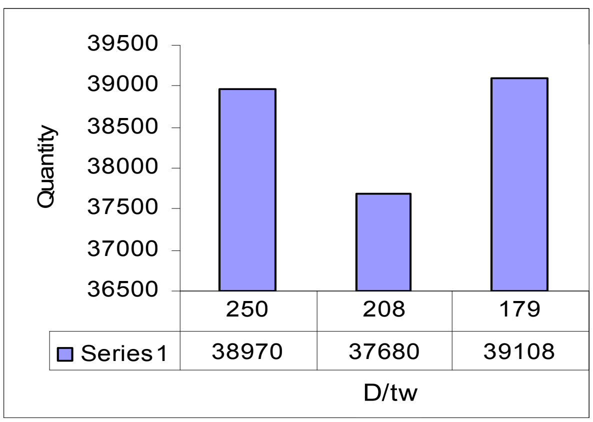


Figure 7.16 Effect of Quantity due to depth - 2500 mm in Riveted plate girder

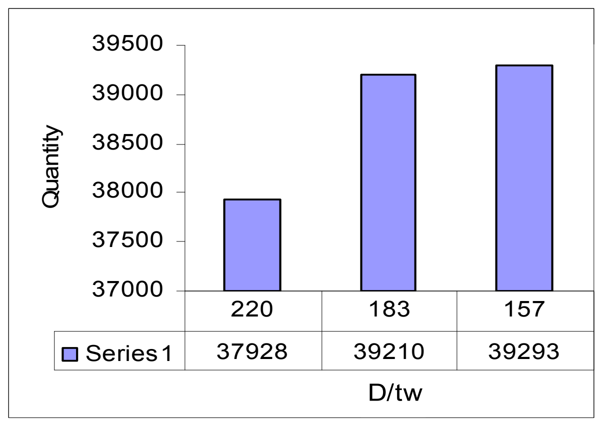


Figure 7.17 Effect of Quantity due to depth – 2200 mm in Riveted plate girder

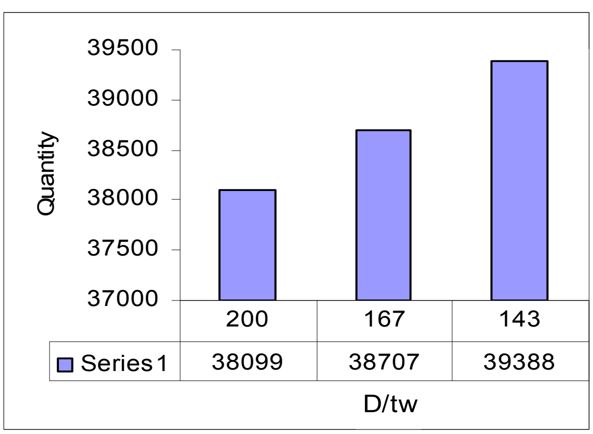


Figure 7.18 Effect of Quantity due to depth - 2000 mm in Riveted plate girder.

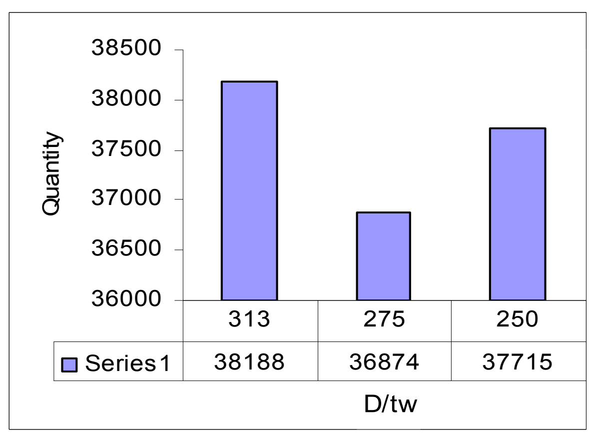


Figure 7.19 Effect of Quantity due to variation in depth and web thickness in Riveted plate girder.

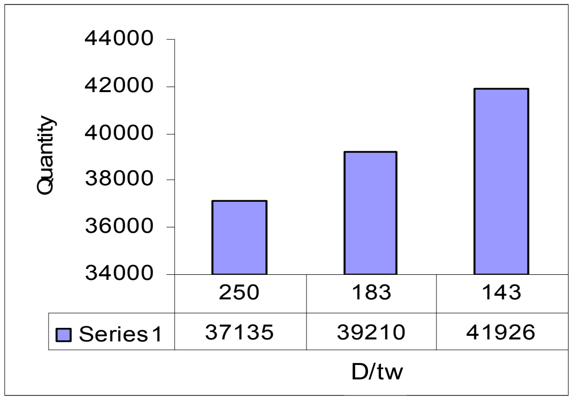


Figure 7.20 Effect of Quantity due to Variation in both depth and Web thickness in Riveted plate girder

7.3. Welded plate girder - 30 m span

7.3.1. Variation of bending stress in Welded plate girder

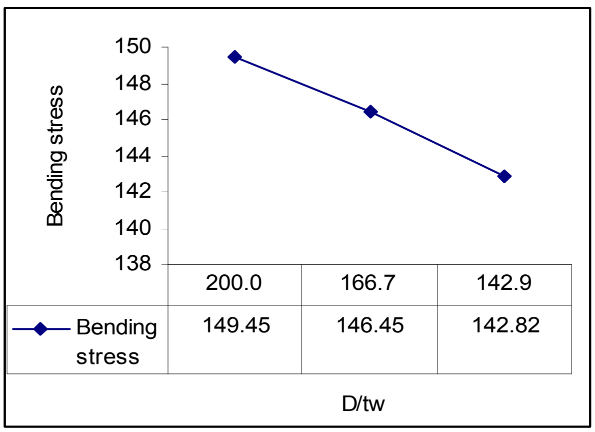


Figure 7.21 Effect of bending stress due to depth - 2000 mm in Welded plate girder

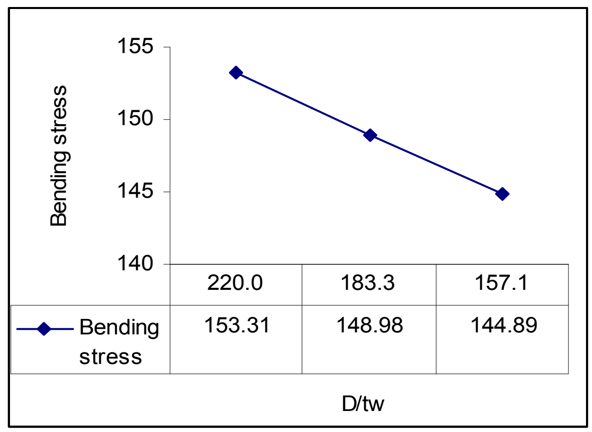


Figure 7.22 Effect of bending stress due to depth -2200 mm in Welded plate girder

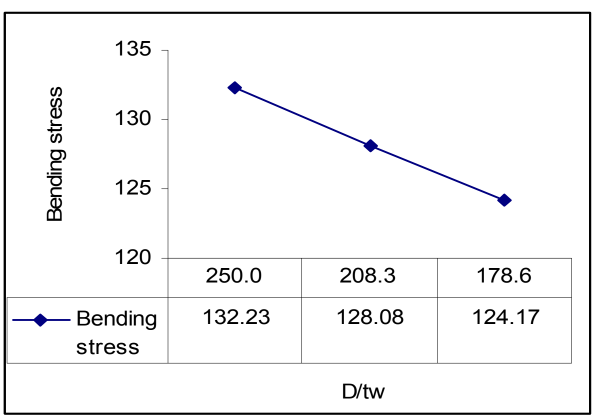


Figure 7.23 Effect of bending stress due to depth - 2500 mm in Welded plate girder

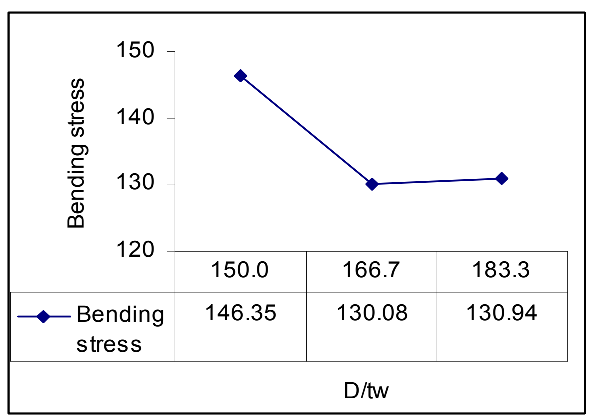


Figure 7.24 Effect of bending stress due to depth variation and Web thickness8 mm in Welded plate girder

7.3.2. Variation of shear stress in Welded plate girder

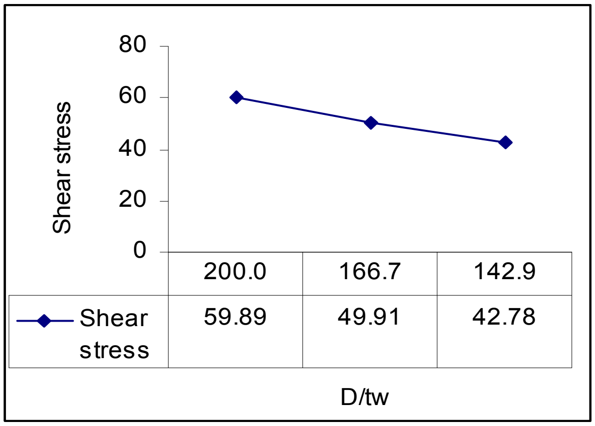


Figure 7.25 Effect of shear stress due to depth - 2000 mm in Welded plate girder

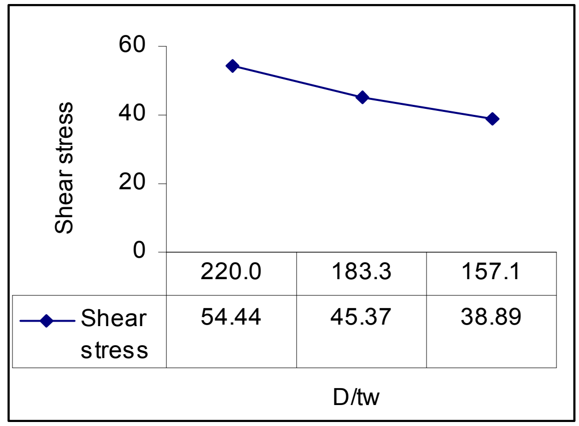


Figure 7.26 Effect of shear stress due to depth -2200 mm in Welded plate girder

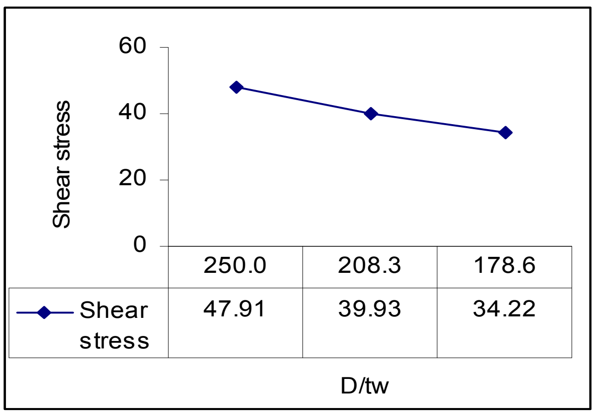


Figure 7.27 Effect of shear stress due to depth - 2500 mm in Welded plate girder

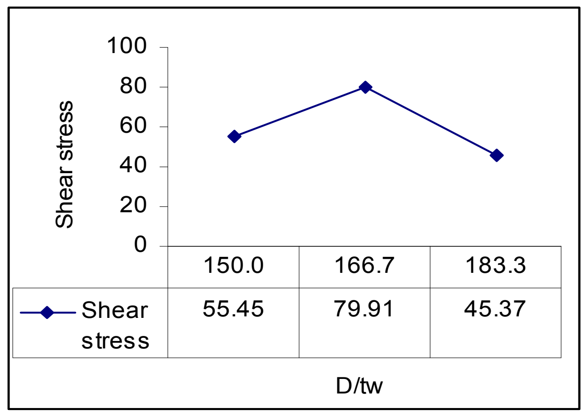


Figure 7.28 Effect of shear stress due to in depth variation and Web thickness8 mm Welded plate girder

7.3.3. Variation on deflection in Welded plate girder

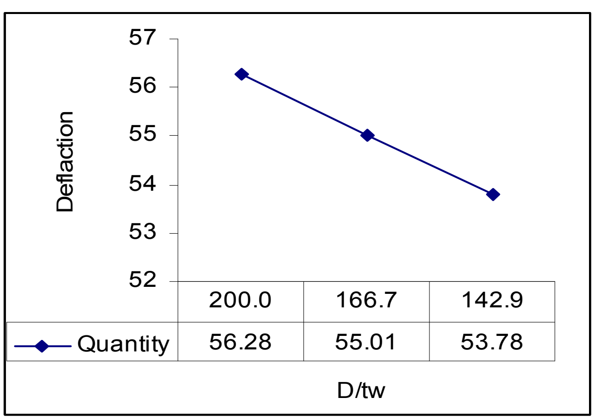


Figure 7.29 Effect of deflection due to depth - 2000 mm in Welded plate girder

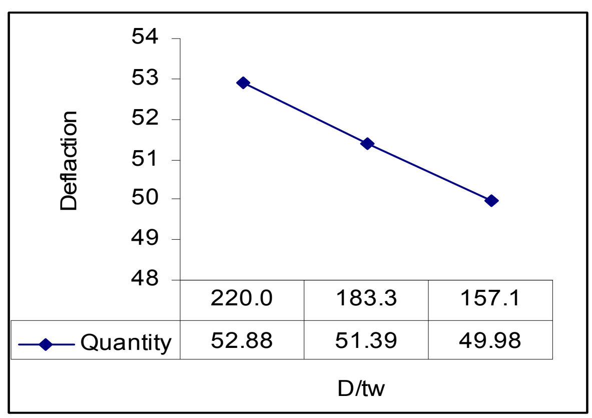


Figure 7.30 Effect of deflection due to depth -2200 mm in Welded plate girder

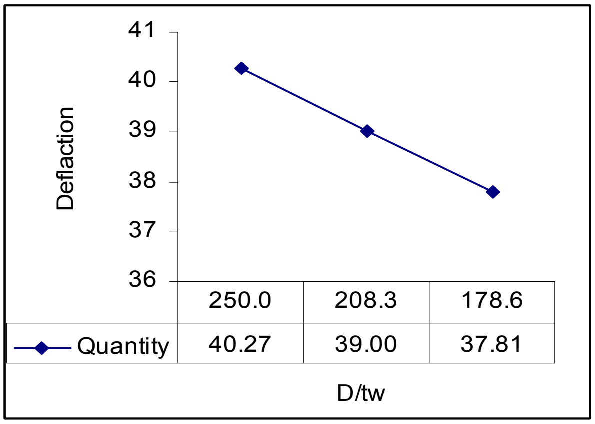


Figure 7.31 Effect of deflection due to depth - 2500 mm in Welded plate girder

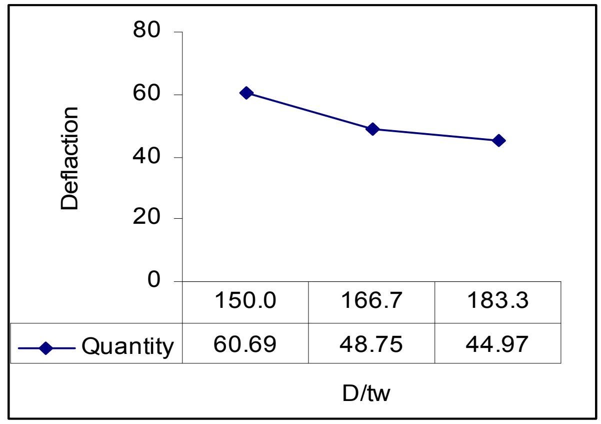


Figure 7.32 Effect of deflection due to depth variation and Web thickness8 mm in Welded plate girder

7.3.4. Variation of Quantity in Welded plate girder

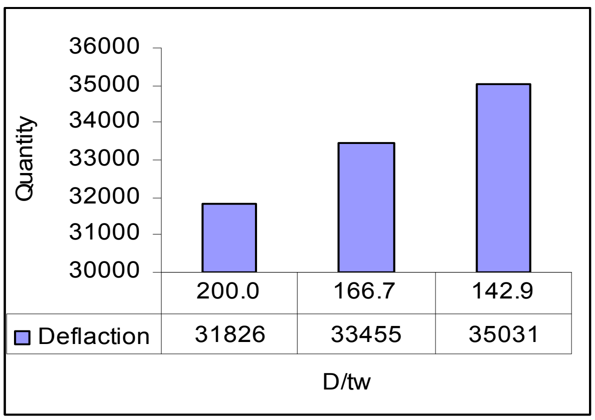


Figure 7.33 Effect of Quantity due to depth - 2000 mm in Welded plate girder

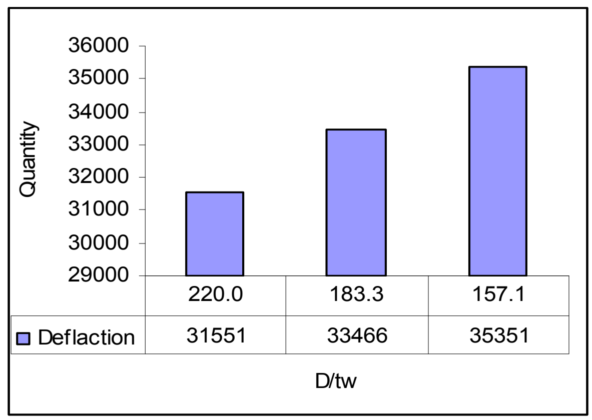


Figure 7.34 Effect of Quantity due to depth -2200 mm in Welded plate girder

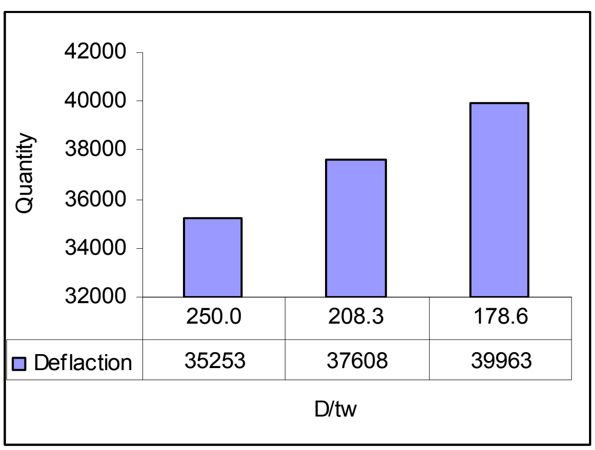


Figure 7.35 Effect of Quantity due to depth - 2500 mm in Welded plate girder

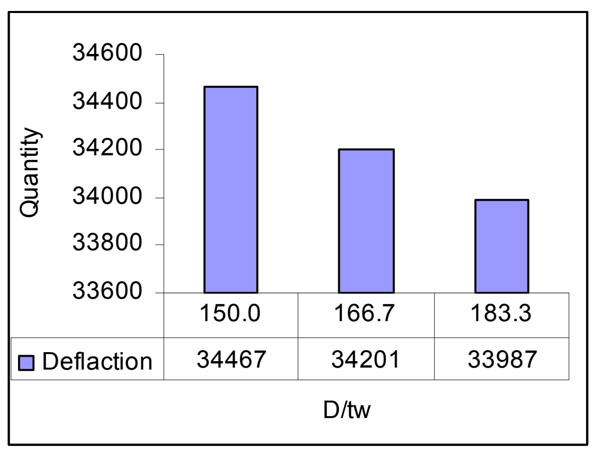


Figure 7.36 Effect of Quantity due to depth variation and Web thickness8 mm in Welded Plate girder

7.4. Truss girder Bridge – 30 m span

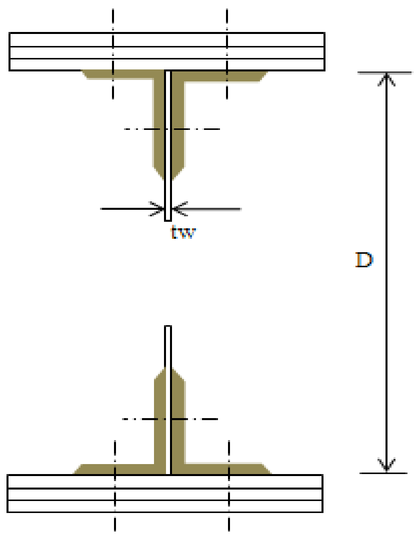
7.4.1. Variation in Quantity in Truss girder bridge

Table 7.1 Effect of Quantity in truss Girder Bridge

Height	panel	Quantity
5	5	52354
6	5	62991
5	2.5	49567
6	2.5	52558

Table 7 .2 Parametric study of Riveted plate girder

D/tw	Web plate		Flange plate	Total Thick.	Flange plate 1 Thickness		Flange plate 2 Thickness		Flange plate 3 Thickness		Angle (web & flange)	Bending stress < 141 N/mm^2						Shear stress < 85	Ixx of Plate 1	Ixx of Plate 2	Ixx of Plate 3	Y1 Plate 1	Y2 Plate 2	Y3 Plate3
												Plate 1		Plate 2		Plate 3								
	Depth	Thick.			Top	Bottom	Top	Bottom	Top	Bottom		Comp.	Tension	Comp.	Tension	Comp.	Tension							
	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm		N/mm^2		N/mm^2		N/mm^2		N/mm^2	mm^4	mm^4	mm^4	mm	mm	mm
Depth change and web thickness same																								
313	2500	8	600	36	12	12	12	12	12	12	ISA125X95X12	52.41	62.58	72.96	84.43	83.57	95.19	59.89	4814256	7129540	9488854	1256.0	1268.0	1280.0
275	2200	8	600	42	15	15	15	15	12	12	ISA125X95X12	46.68	55.02	71.93	89.42	88.81	100.44	68.06	4073803	6341853	8200185	1107.5	1122.5	1135.0
250	2000	8	600	46	16	16	15	15	15	15	ISA125X95X12	45.59	53.47	74.94	87.82	87.12	98.13	74.86	3435428	5321056	7262358	1008.0	1023.5	1037.5
Depth change and web thickness change																								
250	2500	10	600	34	12	12	12	12	10	10	ISA125X95X12	55.62	66.77	75.14	92.87	89.05	102.05	47.91	5074673	7389957	9352976	1256.0	1268.0	1280.0
183	2200	12	600	40	15	15	15	15	10	10	ISA125X95X12	50.73	60.35	73.35	96.81	92.58	105.57	45.37	4428742	6696787	8242667	1108.0	1122.5	1135.0
143	2000	14	600	45	15	15	15	15	15	15	ISA125X95X12	51.40	61.24	73.58	84.96	84.82	96.41	42.78	3711679	5593624	7531179	1008.0	1022.5	1037.5
Depth same and web thickness change																								
250	2500	10	600	34	12	12	12	12	10	10	ISA125X95X12	55.62	66.77	75.14	92.87	89.05	102.05	47.15	5074673	7389957	9352976	1256.0	1268.0	1280.0
208		12	600	32	12	12	10	10	10	10	ISA125X95X12	58.70	70.80	81.44	96.41	89.70	103.50	39.93	5335090	7650374	9613393	1256.0	1268.0	1280.0
179		14	600	30	10	10	10	10	10	10	ISA125X95X12	66.20	80.90	81.51	101.65	90.421	105.00	34.42	5213877	7134157	9084917	1256.0	1268.0	1280.0
Depth same and web thickness change																								
220	2200	10	600	42	15	15	15	15	12	12	ISA125X95X12	47.88	56.71	71.13	88.82	87.40	99.17	54.44	4251275	6519320	8377652	1108.0	1122.5	1136.0
183		12	600	40	15	15	15	15	10	10	ISA125X95X12	50.73	60.35	73.35	96.81	92.58	105.57	45.37	4428742	6696787	8377652	1108.0	1125.5	1136.0
157		14	600	35	15	15	10	10	10	10	ISA125X95X12	56.81	67.87	89.84	121.76	97.50	112.51	38..89	4606208	6111498	7643788	1108.0	1125.5	1136.0
Depth same and web thickness change																								
200	2000	10	600	45	15	15	15	15	15	15	ISA125X95X12	49.40	58.40	74.64	85.61	86.96	98.30	59.89	3445012	5326957	7264523	1008.0	1022.5	1038.0
167		12	600	42	15	15	15	15	12	12	ISA125X95X12	53.30	63.20	78.21	97.48	95.54	108.60	49.91	3578346	5460291	7005854	1008.0	1022.5	1038.0
143		14	600	39	15	15	12	12	12	12	ISA125X95X12	57.30	68.30	87.88	110.87	97.76	111.70	42.78	3711679	5212811	6749437	1008.0	1022.5	1038.0



D = Depth of the Web
tw = Thickness of the Web

Figure 7.37 Riveted plate girder

D/tw	Vertical Stiffener	Shear stress < 85 N/mm^2	Pitch of Rivet mm	Horizontal Stiffener d1/5	Pitch mm	d1/2	Pitch mm	Bearing Stiffener	No. of Rivets	Splice on web Length mm	Width mm	Thick mm	Splice on flange angle Length mm	Width mm	Thick mm	Quantity kg	Deflaction < 92.31 mm
Depth change and web thickness same																	
313	ISA90X90X8	59.89	190	ISA75X75X8	190	ISA75X75X10	190	ISA130X130X15	22	2300	250	4	400	70	6	38188	34.05
275	ISA75X75X8	68.06	190	ISA75X75X8	190	Not Required	-	ISA130X130X15	22	2300	250	4	400	70	6	36874	39.4
250	ISA75X75X8	74.86	190	ISA75X75X8	190	Not Required	-	ISA130X130X15	22	2300	250	4	400	70	6	37715	44.49
Depth change and web thickness change																	
250	ISA75X75X8	47.91	190	ISA75X75X8	190	Not Required	-	ISA130X130X15	18	2300	250	5	400	70	6	37135	34.54
183.3	ISA75X75X8	45.37	190	Not Required	-	Not Required	-	ISA130X130X15	16	2000	250	6	400	70	6	39210	39.2
143	ISA75X75X8	42.78	190	Not Required	-	Not Required	-	ISA130X130X15	14	1800	250	7	400	70	6	41926	42.9
Depth same and web thickness change																	
250	ISA75X75X8	47.15	190	ISA90X90X8	190	ISA75X75X10	190	ISA130X130X15	18	2300	250	5	400	70	6	38970	34.54
208.3	ISA75X75X8	39.93	190	Not Required	-	Not Required	-	ISA130X130X15	16	2300	250	6	400	70	6	37680	35.05
178.6	ISA75X75X8	34.22	190	Not Required	-	Not Required	-	ISA130X130X15	14	2300	250	7	400	70	6	39108	35.56
Depth same and web thickness change																	
220	ISA75X75X8	54.44	190	Not Required	-	Not Required	-	ISA130X130X15	18	2000	250	5	400	70	6	37928	38.57
183	ISA75X75X8	45.37	190	Not Required	-	Not Required	-	ISA130X130X15	16	2000	250	6	400	70	6	39210	39.2
157	ISA75X75X8	39.89	190	Not Required	-	Not Required	-	ISA130X130X15	14	2000	250	7	400	70	6	39293	42.27
Depth same and web thickness change																	
200	ISA75X75X8	59.89	190	Not Required	-	Not Required	-	ISA130X130X15	18	1800	250	5	400	70	6	38099	44.47
167	ISA75X75X8	49.91	190	Not Required	-	Not Required	-	ISA130X130X15	16	1800	250	6	400	70	6	38707	46.12
143	ISA75X75X8	42.78	190	Not Required	-	Not Required	-	ISA130X130X15	14	1800	250	7	400	70	6	39388	47.87

Table 7.3 Parametric study of Welded plate girder

D/tw	Web plate Depth mm	Web plate Thick. mm	Flange plate mm	Total Thickness mm	Reduction in Thick. Plate 1 mm	Reduction in Thick. Plate 2 mm	Bending stress < 158 N/mm^2	Shear stress < 87 N/mm^2	Quantity	Deflaction < 92.31 mm
Depth change and web thickness are same										
150.0	1800	12	700	40	30	25	146.35	55.45	34467	60.69
166.7	2000	12	700	40	30	25	130.08	79.91	34201	48.75
183.3	2200	12	700	35	30	25	130.94	45.37	33987	44.97
Depth same and web thickness are change										
200.0	2000	10	700	35	30	25	149.45	59.89	31826	56.28
166.7		12	700	35	30	25	146.45	49.91	33455	55.01
142.9		14	700	35	30	25	142.82	42.78	35031	53.78
Depth same and web thickness are change										
220.0	2200	10	700	30	30	25	153.31	54.44	31551	52.88
183.3		12	700	30	30	25	148.98	45.37	33466	51.39
157.1		14	700	30	30	25	144.89	38.89	35351	49.98
Depth same and web thickness are change										
250.0	2500	10	700	30	30	25	132.23	47.91	35253	40.27
208.3		12	700	30	30	25	128.08	39.93	37608	39.00
178.6		14	700	30	30	25	124.17	34.22	39963	37.81

D = Depth
tw = Thickness of the web

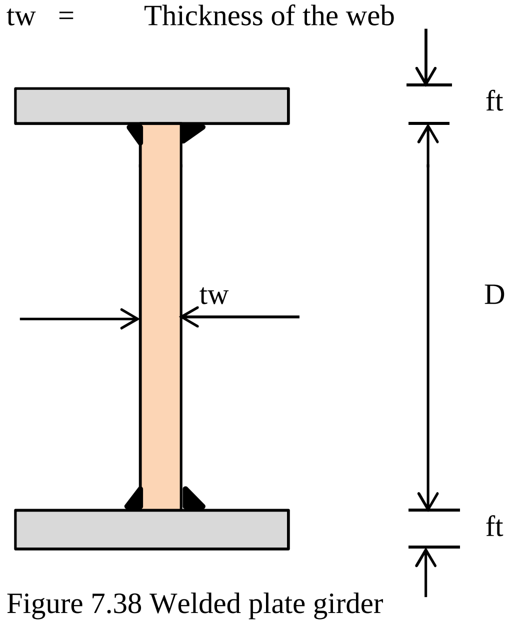


Figure 7.38 Welded plate girder

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8. SUMMARY AND CONCLUSION

8.1 SUMMARY

Bridges are an integral part of the transport routes. Different types of bridges are in use depending upon its specific purpose and their advantages. Steel bridges provide an economical solution for larger span. This work deals with steel railway bridges – plate girder riveted and welded as well as riveted truss bridge.

Excel is used for meticulous calculations of plate girder riveted and welded bridge. The software in Visual Basic is prepared for Plate girder riveted bridge. For truss bridge only riveted connections are considered and excel is used for calculation.

Two different systems i.e. Plate Girder Bridge- welded and riveted are studied for 30 m span with different depths of girder. Parametric study for Riveted Truss Bridge 30 m span is carried out for different height of truss and change in panel length.

Quantity estimate for all bridges is carried out to conclude for the economical section.

8.2 CONCLUSION

The variation in depth of plate girder section violating the empirical formulae of economical depth for section results into safe section in terms of bending and shear stresses also safe in deflection.

Welded plate girder is economical as compared to riveted plate girder.

In Truss girder Railway Bridge decreasing in both the height of the truss and panel length is more economical compare to more height of truss and panel length of the girder.

8.3 FUTURE SCOPE OF WORK

- ✓ Programming in other software languages
- ✓ Design and parametric study of Welded truss bridge
- ✓ Larger span than 30 m can be studied for all types of bridges
- ✓ Comparison of Different types of trusses

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APPENDIX A   ANALYSIS AND DESIGN OF RIVETED PLATE GIRDER

Types of bridge	:	Deck types plate girder bridge		
Gauge	:	Broad gauge, single track, main line		
Total Span of the Girder (L)	=	30		m
	=	30000		mm
Equivalent total L.L for B.M per track(W)	=	2727		kN
Equivalent total L.L for S.F per track(W)	=	2997		kN
Yield stress of steel (fy)	=	236		N/mm^2
Permissible bending stress (σbc)	=	141		N/mm^2
Average shear stress (ζva)	=	85		N/mm^2

1. DEAD LOAD

Dead load of track (open floor)

Weight of stock rail per track per meter	=	2 x 0.6	kN /m
	=	1.2	kN /m
Weight of guard rail per track per meter	=	2 x 0.4	kN /m
	=	0.8	kN /m
Weight of fastenings (Assume)	=	0.2	kN /m
Weight of sleepers per track per meter	=	2.11	kN /m
Self wt of girder = (0.2 L+1)	=	7	kN /m
Total dead Loads per track (w)	=	11.31	kN /m

2. LIVE LOAD

Equivalent total L.L for B.M per track(W)	=	2727	kN
Total live load per girder (W/2)	=	1363.5	kN
(Track on two girder)			
Equivalent total L.L for S.F per track (v)	=	2997	kN
Total live load per girder (v/2)	=	1498.5	kN

3. IMPACT FACTOR

Coefficient of dynamic augment (CDA)

$$0.15 + \frac{8}{6+L} \prec 1.0 \quad = \quad 0.37 \quad \textbf{Ok}$$

4. BENDING MOMENT

B.M due to dead load		=	1272.30	kN.m
B.M due to live load	$\frac{W*L}{8}$	=	5113.13	kN.m

B.M due to impact on live load	=	1903.22	kN.m
Total Design B.M(M)	=	8288.65	kN.m

5. SHEAR FORCE

S.F due to dead load $\frac{w * L}{2}$	=	169.64	kN
S.F due to live load per girder	=	749.25	kN
S.F due to impact on live load	=	278.89	kN
Total Design S.F (V)	**=**	1197.78	kN

6. TRIAL SECTION (WEB SECTION)

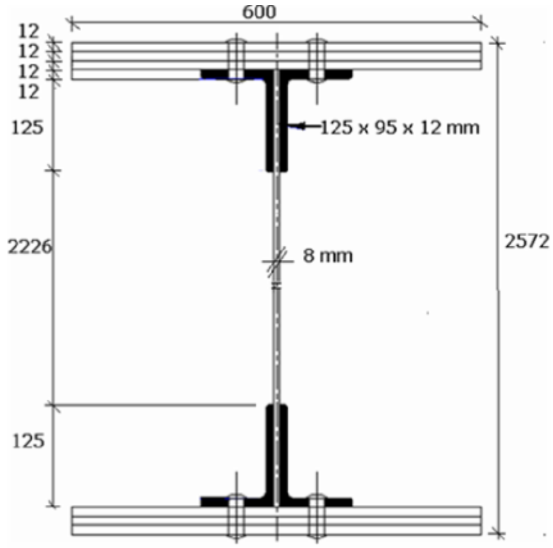
Thickness of the web (tw)	=	8	mm
Depth of girder 1/8 to 1/10 of span	=	3750 to 3000	
Economical Depth $d = 1.1\sqrt{\frac{M}{\sigma bc * tw}}$	=	2434.64	mm
Depth due to shear force $\frac{V}{\tau va * tw}$	=	1761.44	mm
Adopt Depth of Web (dw)	=	2500	mm
Try Web plate (M)	=	2500 x 8	mm

7. FLANGE PLATE

Approximate flange area required

$Af = \frac{M}{\sigma bc * dw} - \frac{Aw}{8}$	=	21013.9	mm^2
Adopt Flange width (fw)	=	600	mm
Angle section		ISA 125 x 95 12	mm
Gross area of two flange angles	=	4996	
Using gross Diameter of the rivets	=	23.5	mm
Area of 4 rivets hole	=	4x23.5x12	mm^2
	=	1128	mm^2
So, net area of 2 flange angles	=	4996 - 1128	mm^2
	=	3868.00	mm^2
Net area provided by the cover plates	=	17145.90	mm^2
Net width of cover plate	=	600 - 2 x 23.5	mm^2
	=	553	mm^2
Thickness of cover plate required			
(Net area/ width)	=	16201.74/553	mm
	=	31.01	mm
Say	=	35	mm
Provide three cover plates of Thickness (tf) of each cover plate			
	=	12	mm
So, the area provided	=	21600	mm^2

	=	21600 > 21013.90	mm^2
			Ok



Dimension of the Cross section

Adopt cover plate of size	=	600 x 12	mm
Number of cover plate	=	3	
Provide 3 flange plates at top & bottom	=	600 x 12	mm
Top plate (thickness)			
plate 1 tf1	=	12	mm
plate 2 tf2	=	12	mm
plate 3 tf3	=	12	mm
Bottom plate (thickness)			
plate 1' tf1'	=	12	mm
plate 2' tf2'	=	12	mm
plate 3' tf3'	=	12	mm
width of the flange	=	600	mm
Over all depth (D)	=	2572	mm
Angle section	=	ISA 125 x 95 12	

8. FLANGE AREA

Description	Gross Area	Deduction of hole	Net flange area (Afn)		
	(Afg) mm^2	mm^2	mm^2		
Flange angles					
2ISA125 x 95x12	2 x 2498	4 x 23.5 x 12	=	3868	mm^2
Cover plate					
plate 1	A1 = 7200	564	=	6636	mm^2
plate 2	A2 = 7200	564	=	6636	mm^2
plate 3	A3 = 7200	564	=	6636	mm^2

Equivalent Web area							
	(Aw/6)					(Aw/8)	
	3333.33				=	2500	mm^2
Total Afg =	29929.33		Total(Afn)		=	26276	mm^2

9. GROSS MOMENT OF INERTIA

Moment of inertia of gross section is computed below :

Io	+	Ay^2		Ixx	
Web					
$(2500^3 \times 8)/12$		Nill	=	1041666.67	mm^4
Angle					
4 x 1904000	+	$4 \times 2498 \times (1250-24.7)^2$	=	1500920.60	mm^4
Cover plate					
Plate 1 Ixx_1					
$2 \times 600 \times 12^3/12$	+	$2 \times 600 \times 12 \times 1256^2$	=	4814256.39	mm^4
Plate 2 Ixx_2					
$2 \times 600 \times 12^3/12$	+	$2 \times 600 \times 12 \times 1268^2$	=	7129540.23	mm^4
Plate3 Ixx_3					
$2 \times 600 \times 12^3/12$	+	$2 \times 600 \times 12 \times 1280^2$	=	9488853.51	mm^4

10. CURTAILMENT OF PLATE

Curtailment of flange plate, in tension flange

Length X1 $\quad X1=\frac{1}{2}*L*\sqrt{\frac{A1}{Afn}}$ = 7.54 m

Length X2 $\quad X2=\frac{1}{2}*L*\sqrt{\frac{A1+A2}{Afn}}$ = 10.66 m

Length X3 $\quad X3=\frac{1}{2}*L*\sqrt{\frac{A1+A2+A3}{Afn}}$ = 13.06 m

Distance to section for theoretical cut off point

Distance to cross section 1,From A	=	7.46	m
Distance to cross section 1,From A	=	4.34	m
Distance to cross section 1,From A	=	1.94	m

11. Bending moment at section for theoretical cut off

M1 = B.m at section 1-1

$$\frac{4*a}{L^2}*X*(L-X) = 6195.35 \text{ kN-m}$$

M2 = B.m at section 2-2

$$\frac{4*a}{L^2}*X*(L-X) = 4102.05 \text{ kN-m}$$

M3 = B.m at section 3-3

$$\frac{4*a}{L^2}*X*(L-X) = 2008.76 \text{ kN-m}$$

12. CHECK FOR STRESS

Bending stress in compression flange at center of the plate.

σbc_1 For plate 3	y3	=	1280	mm
	$\frac{M1 * y3}{Ixx_3}$	=	83.57	N/mm^2
		=	83.57 < 141	N/mm^2 **Ok**
σbc_2 For plate 2	y2	=	1268	mm
	$\frac{M2 * y2}{Ixx_2}$	=	72.96	N/mm^2
		=	72 96 < 141	N/mm^2 **Ok**
σbc_3 For plate 1	y1	=	1256	mm
	$\frac{M3 * y1}{Ixx_1}$	=	52.41	N/mm^2
		=	52.41 < 141	N/mm^2 **Ok**

Bending stress in tension flange center of the cover plate.

σbt_1 For plate 3				
	$\sigma bc_1 * \frac{\text{gross area of the flange}}{\text{Net area of tension flange}}$	=	95.19	
		=	95 .19 < 141	N/mm^2 **Ok**
σbt_2 For plate 2				
	$\sigma bc_2 * \frac{\text{gross area of the flange}}{\text{Net area of tension flange}}$	=	84.43	N/mm^2
		=	84.43 < 141	N/mm^2 **Ok**
σbt_3 For plate 3				
	$\sigma bc_3 * \frac{\text{gross area of the flange}}{\text{Net area of tension flange}}$	=	62.58	N/mm^2
		=	62. 58 < 141	N/mm^2 **Ok**

Shear stress

shear stress ζv.cal	$\frac{V}{dw * tw}$	=	59.89	N/mm^2
		=	59. 89 < 85	N/mm^2

Ok

13. FORCE IN THE FLANGE PLATE

Tensile force in each cover plate at Respective section

Cover plate 3	F1	95.19 x 6636	=	631.69	kN
Cover plate 2	F2	84.43 x 6636	=	560.29	kN
Cover plate 1	F3	62.58 x 6636	=	415.31	kN

14. DESIGN OF RIVETED CONNECTIONS FOR FLANGE PLATE

Diameter of the power driven rivets (d) = 22 mm

Gross diameter (d+1.5) = 23.5 mm

Strength of rivets in Single shear

$$\tau_{vf} * \frac{\pi}{4} * d^2 = 43.35 \text{ kN}$$

Strength of rivets in bearing

$$\sigma_{bf} * d * tw = 56.4 \text{ kN}$$

Rivet value Rv (Minimum value) = 43.35 kN

Number of rivets at section 1-1 (plate 3)

$$\frac{F1}{Rv} = 14.57$$

say = 16

Number of rivets at section 2-2 (Plate 2)

$$\frac{F2}{Rv} = 12.92$$

say = 16

Number of rivets at section 3-3 (Plate 1)

$$\frac{F3}{Rv} = 9.58$$

say = 16

15. ACTUAL LENGTH OF CURTAILED

Cover plate 3	=	16.20	m
Cover plate 2	=	22.45	m
Cover plate 1	=	27.24	m

16. CONNECTION BETWEEN FLANGE ANGLE AND WEB

Connection of flange angle to web in compression flange

Strength of rivets in double shear

$$2 * \tau_{vf} * \frac{\pi}{4} * d^2 = 86.70 \text{ kN}$$

Strength of rivets in bearing

$2 * \sigma_{bf} * d * tw$ = 56.4 kN

Rivet value Rv = 56.40 kN

pitch of rivet

(L.L + I.L) pe girder, per mm Run (w) = 0.19 KN/mm

Dead load of track ,per girder = 0.5 * 4.31 KN/m

= $2.15*10^{-3}$ KN/mm

One flange plate is available (Af) = 12196.00 mm^2

$$p = \frac{Rv}{\sqrt{\left[\frac{V}{de} \cdot \frac{Af}{Af + \frac{1}{6} Aw}\right]^2 + w^2}} = 134.21 \text{ mm}$$

So , provide pitch of the rivet = 95 mm

Connection of flange plate to flange angle in compression flange

Strength of rivets in single shear = 43.35 kN

Strength of rivets in bearing = 56.4 kN

Rivet value Rv = 43.35 kN

pitch of the rivet

One Flange plate is available (Af) = 7200 mm^2

= 12196.00

Area of plate (Ap) = 195.16 mm

Rivets are provided in two rows, = 2 x 195.16

= 390.32 mm

so, provide pitch of rivets = 95 mm

Connection of flange angle to web in tension flange

Strength of rivets in double shear = 86.70 kN

Strength of rivets in bearing = 56.4 kN

Rivet value Rv = 56.4 kN

At support, flange plate is not available. Af' = 3868 mm^2

$$p = \frac{Rv}{\frac{V}{de} * \frac{Af'}{\left(Af' + \frac{1}{8} * Aw\right)}} = 193.80 \text{ Mm}$$

so, provide pitch of rivets = 120 mm

Connection of flange to flange angle in tension flange

Strength of rivets in single shear = 43.35 kN

Strength of rivets in bearing = 56.4 kN

Rivet value Rv = 43.35 kN

Pitch of the rivets connection flange angle to web

Only one flange plate is available. Af' = 10504 mm^2

$$p = \frac{Rv * de}{V} * \left(\frac{Af' + \frac{1}{8} Aw}{Ap'} \right)$$ Ap'	=	6636	mm^2
	=	177.31	mm
so, provide pitch of rivets	=	120	mm

17. CHECK FOR NECESSITY OF STIFFENER

Web thickness (tw)	=	8	mm
Web Depth (D)	=	2500	mm
Clear depth between flange angle on the plate girder (d1)			
	=	2250	mm
Shear stress (ζva.cal)	=	59.89	N/mm^2

[A] For Unstiffened Web

tw min	$d1 \frac{\sqrt{\tau_{va.cal}}}{816}$	=	21.34	mm
tw min	$d1 \frac{\sqrt{fy}}{1344}$	=	25.72	mm
tw min	$\frac{d1}{85}$	=	26.47	mm

Here, thickness tw is less than the min. tw so, the stiffener are required.

[B] For Web Stiffened Vertical

tw min	$d2 \frac{\sqrt{fy}}{3200}$	=	10.80	mm
tw min	$\frac{d2}{200}$	=	11.25	mm

Here, thickness tw is less than the min. tw so, the stiffener are required.

[c] For Web Stiffened Vertical and horizontal

tw min	$d2 \frac{\sqrt{fy}}{4000}$	=	8.64	mm
tw min	$\frac{d2}{250}$	=	9.00	mm

Thickness tw is greater the min. tw so, the vertical and horizontal stiff. are required.

[D] For Web Stiffened as above

tw min	$d2 \frac{\sqrt{fy}}{6400}$	=	5.40	mm
	$\frac{d2}{400}$			

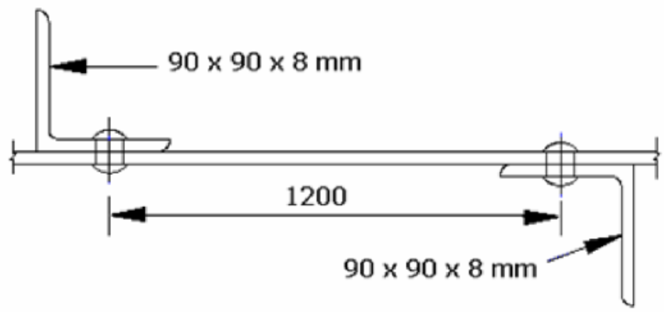
tw min	=	5.63	mm

18. DESIGN OF VERTICAL STIFFENER

The smaller clear Panel dimension for the actual thickness of web			
(180 *tw)	=	1440	mm
The greater clear Panel dimension for the actual thickness of web			
(270 *tw)	=	2160	mm
Minimum spacing of Stiffener Cmin	=	0.33 *d'	
	=	675	N/mm^2
Maximum spacing of Stiffener Cmax	=	1.5* (d')	
	=	1875	mm
Ratio d'/tw	=	140.63	mm
From IRS the permissible average shear Stress			
	=	87	N/mm^2
	=	87 > 59.89	N/mm^2
			Ok
Try of angle section		ISA 90 X 90 X 8	
Clear distance between vertical stiffenerc1	=	(1200 - 90)	
	=	1110	mm
Ixx about face of the web plate provided	=	2E+06	mm^4
Maximum outstand of vertical Stiffener			
$\frac{256 * tw}{\sqrt{fy}}$	=	133.31	mm
Minimum required thickness of Web tw1	=	5.51	mm
Moments of inertia of vertical stiffener (I)			
$1.5 * \frac{d1^3 * tw^3}{c1^2}$	=	289503.77 < 2E+06	mm^4
			Ok

19. CONNECTION OF VERTICAL STIFFENER TO WEB PLATE

Shear force $\frac{125 * tw^2}{h}$	=	88.89	kN/m
Strength of rivets in single shear	=	43.35	kN
Strength of rivets in bearing	=	56.4	kN
Rivet value R	=	43.35	kN
pitch of the rivets	=	256	Mm
So, adopt pitch	=	190 c/c	mm



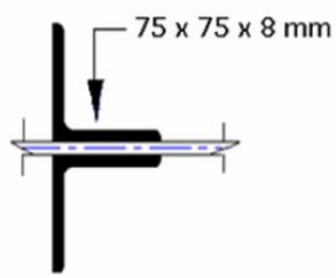
Vertical stiffener

20. DESIGN OF HORIZONTAL STIFFENER

First stiffeners at distance d1/5 depth

Moments of Inertia required , I

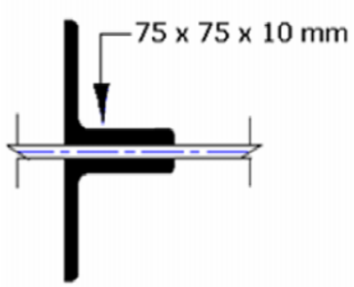
$I = 4 * C1 * tw^3$	=	2E+06	mm^4
Try the section on both side of the web	=	ISA 75 x 75 x 8	
I of the pair of stiffeners about the center of the web			
	=	2648384.16>2E+06	mm^4
Adopt Pitch	=	190 mm c/c	
			Ok



Second stiffeners at distance d1/2 depth

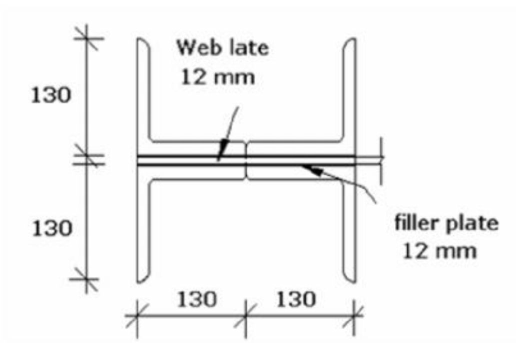
Moments of Inertia required , I

$I = d2 * tw^3$	=	1152000	mm^4
Try single angle		ISA 75 x 75 10	
I on the face of the web	=	1404962	
		1404962 > 1E+06	mm^4
			Ok



21. DESIGN OF BEARING STIFFENER

At end reaction (V)	=	1197.78	kN
Allowable Bearing stress ($\sigma_{p)}$	=	189	N/mm^2
Bering Area Required = (V/σp)	=	6337.45	mm^2
For flange angle 2ISA 125 x 95 x12 mm r1	=	9.0	
Try angle section	=	4-ISA 130 x 130x 15	
Bearing area provided by outstanding legs	=	7200	mm^2
	=	7200 > 6337.45	mm^2
			Ok
Using filler plate thickness	=	12	mm
Width of the stiffener	=	300	mm
Actual depth of bearing stiffener d1	=	2476	mm
Effective depth of column = (0.7*d1)	=	1733.2	mm
Cro.secti. area of the column section (A)	=	18324	mm^2
moment of inertia (I)	=	7E+07	mm^4
r min = $(I/A)^{1/2}$	=	62.77	mm
slenderness ratio = d1eff/rmin	=	27.61	
Allowable stress σac	=	136	N/mm^2
Load carrying capacity of stiffener	=	2492.06	mm^2
	=	2492.06 > 1197.78	mm^2
			Ok



22. CONNECTION OF BEARING STIFFENER TO WEB PLATE

Strength of rivets in double shear	=	86.70	kN
Strength of rivets in bearing	=	56.4	kN
Rivet value R	=	56.40	kN
Number of rivets	=	21.24	
provide rivets	=	22	
Thickness of packing plate	=	12	mm

23. WEB SPLICE

web spliced at 1/3 of span	=	10.00	m

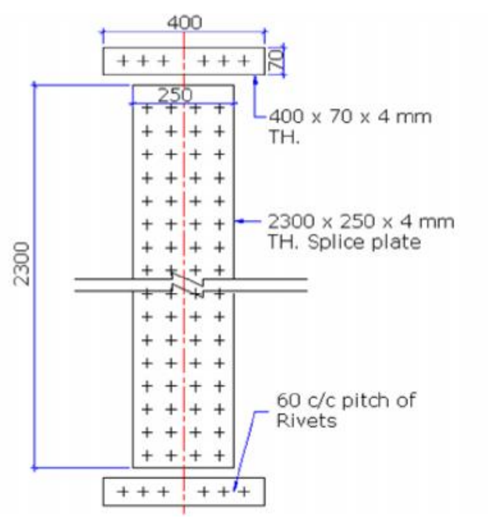
Maximum shear force	=	598.89	kN
Maximum Bending moment			
$\frac{4 * a}{L^2} * X * (L - X)$	=	7367.69	kN.m

24. SPLICE PLATE ON WEB

depth of the splice plate	=	2298	mm
say	=	2300	mm
Thickness of the splice plate	=	4	mm
σb_2	=		
$P = \frac{Rv}{\sqrt{(\sigma b_2 * tw)^2 + \left(\frac{V}{d1^2}\right)^2}}$	=	95.71	mm
provide rivets in two row at pitch	=	191.43	mm
provide distance of adjacent rows of rivets	=	60	mm
Edge distance	=	33	mm
Width of the splice plate	=	252	mm
	=	250	mm
Splice plate	=	2300 x 250 x4	mm

25. SPLICE OVER FLANGE ANGLE

Area of web plate flange angles Aw_2	=	95 x 8	mm^2
	=	760	mm^2
Provide wide of the splice plate both side	=	70	mm
Thickness of the splice plate	=	10.86	mm
Say	=	10	mm
Thickness of the each splice plate	=	10.86/2	mm
	=	5.43	mm
Say	=	6	mm
σb2	=	41.14	



Splice on web and flange

Assume pitch of the rivets	=	60	mm
Number of rivets			

$$n = \frac{\sigma b2 \quad * \ Aw1}{Rv \ - \ \frac{V}{d1} * \frac{Af}{(Af \ + \ Aw\,1)} * \ p} = 0.56$$

say	=	3	
Length of the splice plate	=	372	mm
say	=	400	mm
provide splice plate	=	400 x 70 x 6	mm

26. Design of top lateral bracing

Consider the bridge to be loaded

lateral load will act	=	2.97	mm
say	=	3	mm
(i) Wind load on both girders @1.5kN/m	=	Pw*(1+k)*h1	
	=	5.63	KN/m
(ii) Wind load on train @1.5 kN/m	=	1.5 x 3.5	
	=	5.25	KN/m
(iii) Racking force @ 600 kg/m			
600 x 9.81 x 10 -3	=	5.89	KN/m
Total Horizontal force	=	16.76	KN/m
Lateral load each intermediate panel point	=	33.52	KN
Lateral load on end panel point	=	16.76	KN
End reaction	=	251.42	KN
tanθ = 0.95 , θ	=	42.14 ;	
sin θ	=	0.67	
Compressive force in end strut	=	251.42	KN
shear in end panel	=	234.65	KN
Tensile force in end diagonal	=	234.65 cosec θ	
	=	350.23	KN

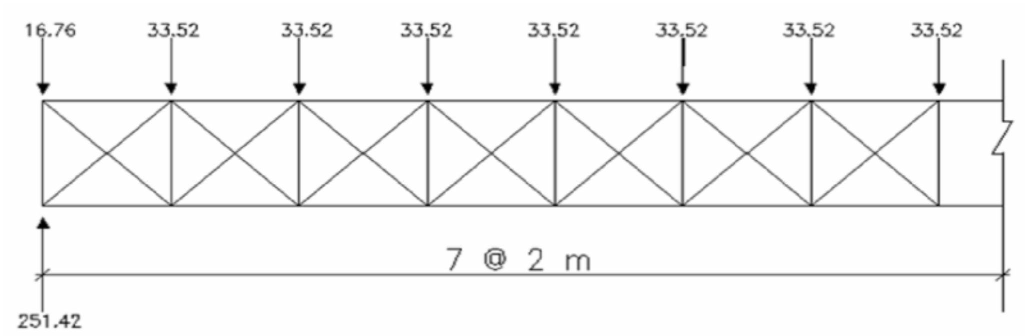
27. Design of end strut

P	=	251.42	kN

Let us provided two angles connected to the same side of gusset plate.

Hence , effective length (L)	=	1.90	m
Let us Assume (λ)	=	83.70	
So , 6ac	=	101	N/mm^2
Required area	=	2489.26	mm^2
provided 2ISA		ISA 80 x 80 x 8	mm
Actual (λ) = L/r	=	83.70	
Corresponding to which 6ac	=	101	N/mm^2
Load capacity	=	493.28	kN
The rivets will be in single shear			
No. of 20 mm dia rivets	=	5.80	

Hence , provide 3 rivets on each angle for corresponding these to 10 mm thick gusset plate at the end



28. Design of diagonal member

P	=	350.23	kN
Permissible tensile stress	=	141	N/mm^2
Required area Anet	=	2483.90	mm^2
provided 2ISA	=	80 x 80 x 12	

Using 20 mm dia rivets, deduction for rivets holes for each angle

	=	172	
Total net area provided	=	3218	mm^2
			Ok
No. of 20 mm dia rivets	=	8.08	

Hence provide 4 rivets on each angle

29. Design of bottom lateral bracing

Hence , force in the strut	=	62.85	kN
provided 2ISA	=	50 x 50 x 6	
Actual (λ) = L/r	=	131.03	
Corresponding to which 6ac	=	63	N/mm^2
Load carrying capacity	=	65.89	kN
tensile force in diagonal member	=	87.56	kN
Required area (Anet)	=	620.97	mm^2
Using 20 mm dia rivets,			
deduction for rivets holes	=	21.5 x 6	
	=	129	mm
Total net area provided	=	2[568 -129]	
	=	878	mm^2
			Ok

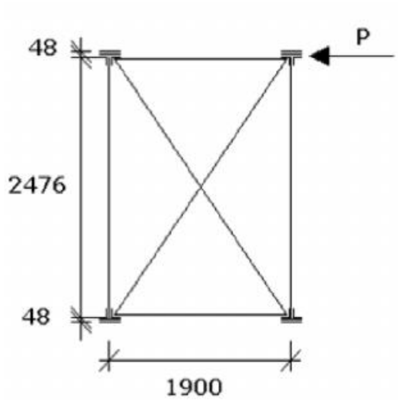
Using one rivet, on each angle, for connection this member.

30. Design of end cross frames

P	=	251.42	kN
Length of diagonal member	=	2.72	mm
Cosθ = 2.476/2.72	=	0.69	
Tensile force in diagonal member	=	366.39	kN
Allowable stress in axial tension	=	141	N/mm
Required Anet	=	2598.53	mm^2

provided 2ISA	=	80 x 80 x 12	
Using 20 mm dia rivets, deduction for rivets holes		for each angle	
24.2*12	=	290.4	mm
Total net area provided	=	2981.2	mm^2
			Ok
No. of 20 mm dia rivets	=	5.80	

Hence provide 2 rivets for each angle.



End cross frame

APPENDIX B ANALYSIS AND DESIGN OF WELDED PLATE GIRDER BRIDGE

1. DATA

Types of bridge	:	Deck types welded plate Girder	
Gauge	:	Broad gauge, single track,	
Span of the Girder (L)	=	30	m
	=	30000	mm
Equivalent total L.L for B.M per track	=	2727	KN
Equivalent total L.L for S.F per track	=	2997	KN
Permissible bending stress (σb)	=	141	N/mm^2
Average shear stress (ζ)	=	85	N/mm
Yield stress fy	=	236	N/mm

2. DEAD LOADS

Dead load of track (open floor)

Weight of stock rail per track per meter	=	2 x 0.6	kN /m
	=	1.2	kN /m
Weight of guard rail per track per meter	=	2 x 0.4	kN /m
	=	0.8	kN /m
Weight of fastenings (Assume)	=	0.2	kN /m
Weight of sleepers per track per meter	=	2.11	kN /m
Self wt of girder = (0.2 L+1)	=	7	kN /m
Total dead Loads per track (w)	=	11.31	

3. LIVE LOADS

Equivalent total Live Load for B.M per track (W)			
	=	2727	KN
Total live load per girder (W/2)	=	1363.5	KN
(Track on two girder)			
Equivalent total Live Load for S.F per track (W)			
	=	2997	KN
Total live load per girder (W/2)	=	1498.5	KN

4. IMPACT FACTOR

Coefficient of dynamic augment (CDA)	=	0.37	Ok

5. BENDING MOMENTS

B.M due to dead load	=	1272.30	KN.m
B.M due to live load	=	5113.13	KN.m
B.M due to impact on live load	=	1903.22	KN.m

Total Design B.M (M)	=	8288.65	KN.m

6. SHEAR FORCE

S.F due to dead load	=	169.64	KN
S.F due to live load	=	278.89	KN
Total Design S.F (V)	=	1197.78	KN

7. PROPIORTIONING OF TRIAL SECTION OF THE WEB PLATE

Thickness of the web (tw)	=	12	mm
Depth of girder 1/8 to 1/10 of span	=	3750 to 3000	mm
Economical Depth d	=	1944.13	mm
Depth due to shear d	=	1174.29	mm
Adopt Depth of Web (dw)	=	2000	mm
Try a Web plate	=	2000 X 12	mm

8. FLANGE PLATE

Approximate flange area required	=	25392.37	mm^2
Flange width (fw) =L/40 to L/45	=	750 to 666.67	mm
Adopt Flange width (fw)	=	700	mm
Thickness of plate required	=	36.27 mm	
Adopt Flange Thickness (ft)	=	40	mm^2
Provided Area of the flange	=	28000 mm2	
	=	28000 > 25392.37	mm
Adopt flange plate of size	=	700 X 40	mm
Top plate	=	40	mm
Bottom plate	=	40	mm

9. CHECK FOR MAXIMUM STRESS

The moment of inertia of the section

Ixx of section	=	66269866667	mm^4
Iyy of section	=	2286954667	mm^4
Area of the section (A)	=	80000	mm^2
Radius of gyration about y-y axis (ry)	=	169.08	mm

According to IRC: 24, the critical compressive stress for I section having equal moment of inertial about Y-Y axis is given by

Effective length of the compre. Flange (L)	=	6000	mm
Over all depth (D)	=	2080	mm
Radius of gyration about y-y axis (ry)	=	169.08	mm
Effective thick. Of the compre. Flange (te)	=	40	mm
Critical stress Cs is given by	=	2127.23	N/mm

The allowable Working stress for different value of Cs prescribed in IRC: 24 .permissible bending stress corresponding to a value of Cs = 2127.23 N/mm^2

obtained as 158 N/mm^2 for σy	=	236	N/mm^2
Actual maximum bending stress			
σbc = σbt	=	30.08 < 158	N/mm^2 Ok
Ratio of (dw/tw)	=	166.67	
Stiffener spacing (c)	=	1248	mm
Using stiffener spacing (c)	=	1250	mm
Average shear stress	=	49.91 < 87	N/mm^2

10. REDUCTION OF FLANGE PLATE THICKNESS

Reduce the thickness of flange plate 1	=	30	mm
Maximum outstand	=	344	mm
Permissible outstand	=	360	mm
	=	360 > 344	Ok
Moment of inertia of plate girder Ixx1			
	=	51272600000	Nmm
	=	51272.6	kNm
Moments of Resistance MR			
	=	7865117282	Nmm
	=	7865.11	kNm
Using, parabolic equation y	=	kx (L-x)	
Where y= maximum B.M	=	8288.65	kN.m
l = span	=	30	m
k= a constant			
x=Distance of the section from support	=	15	
y=kx (L-x)	=	8288.65=k x 15 (30-15)	
so, the constant k	=	36.84	
X @ y	=	7865.12	kN.m
y	=	k x (L-x)	
7865.12	=	36.84*x (30-x)	
	=	x2 -30x+213.49=0	
So, the distance from the support (x)	=	11.61	m
Provide, Flange Plate	=	700 x 30	mm

Therefore, the plate thickness can be reduced to 30 mm from 40mm for a length of 11.61 m from the supports. Provide 700 x 30 mm plates up to 11.61 m from the supports.

Reduce the thickness of flange plate 2	=	25	mm
Maximum outstand (fw-tw)/2	=	344	mm
Permissible outstand (16 * ft)	=	400	mm
	=	400 > 344	mm Ok
Moment of inertia of plate girder Ixx2	=	43882291667	mm^4

Moments of Resistance MR

	=	6764294715	N/mm
Using, parabolic equation			
y	=	kx (L-x)	
k= a constant	=	36.84	
x=Distance of the section from support	=	30/2	m
x @ y	=	6764.29	kN.m
y	=	k x (L-x)	
6764.29	=	36.84*x(30-x)	
	=	x2 -30x+183.61=0	
so, the distance from the support (x)	=	8.57	m
Provide, Flange Plate	=	700 x 25	mm

Therefore, the plate thickness can be reduced to 25 mm from 30mm for a length of 8.57 m from the supports. Provide 700 x 30 mm plates up to 8.57 m from the supports.

Distance (x)	Flange plate	
From supports up to 8.57 m	700 x 25	mm
From 8.57 m to 11.61 m	700 x 30	mm
From 11.61 m to mid span	700 x 40	mm

11. Connection between Flange and web

Maximum shear force at the junction of the web and flange			
So, Shear per unit length	=	483.64	N/mm
Size of the weld			
Welding is done on both sides of the web,			
Strength of weld of size's' is	=		
483.64	=	2x (0.7x s x 1x 108)	
s	=	3.20	mm
Minimum size of the weld	=	5	mm

So, the minimum size of fillet weld for a 25mm thick is 5 mm. Let us provide an intermittent fillet weld. The effective weld length is 4 *s (4x5=20 mm) OR a 40 mm whichever is more .Therefore, provide 40 mm long intermittent fillet weld.

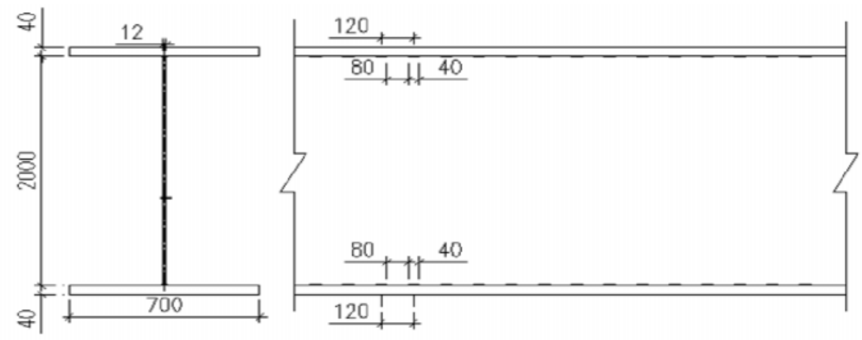
$$\text{pitch of the weld} = \frac{\text{Strength of weld on both faces of web}}{\text{horizontal shear/mm length}}$$

	=	2x40x0.7x5x102/483.64	
	=	59.05	
So, provide pitch of the rivets	=	120	mm

The clear spacing between the weld should not be more than 12 t OR 200 mm whichever is less.

Maximum clear spacing	=	144	mm
Clear spacing	=	80	mm

Provide 40 mm fillet welds at a pitch of 60 mm .The flange plate 40 mm, 30 mm and 25 mm thick are connected together by butt weld.



Connection between flange plate and web

11. Connection between Flange Plates:

Reduction of plate thickness		=	30	to	25	mm
Strength of weld (P)	tx l x ζvf	=	1890			kN
Throat thickness	(5/8)*t	=	15.63			mm
Provide taper 1 in 5 mm						
Reduction of plate thickness		=	40	to	30	mm
Strength of weld (P)	tx l x ζvf	=	2268			kN
Throat thickness	(5/8)*t	=	18.75			mm
Provide taper 1 in 5 mm						

12. CHECK FOR NECESSITY OF STIFFENER

ζva cal. = $\frac{V}{dw * tw}$ = 49.91 N/mm^2

[A] For Unstiffened Web

tw min = $d1 \frac{\sqrt{\tau_{va.cal}}}{816}$ = 17.31 mm

tw min = $d1 \frac{\sqrt{fy}}{1344}$ = 22.86 mm

tw min = $\frac{d1}{85}$ = 23.53 mm

Therefore, intermediate web stiffener will be required.

[B] For Web Stiffened Vertical

tw min = $d2 \frac{\sqrt{fy}}{3200}$ = 9.60 mm

tw min = $\frac{d2}{200}$ = 10 mm

Therefore, horizontal web stiffener will be not required.

13. DESIGN OF VERTICAL STIFFENER

The smaller clear Panel dimension for the actual thickness of web

= 180x12 = 2160 mm

The greater clear Panel dimension for the actual thickness of web

= 270x12 = 3240 mm

Number of stiffener = 13.89

say = 20

Actual smaller clear panel dimension = 1500 mm

Spacing of Stiffener

Maximum Spacing of Stiffener = 1.5 *dw

= 3000 mm

Minimum Spacing of Stiffener = 0.33 * dw

= 660 mm

Adopt spacing (c1) = 2000 mm

Moments of inertia of vertical stiffener (I)

$$1.5 * \frac{dw^3 * tw^3}{C_1^2} = 5184000.00$$

Using the thickness of the plate = 10 mm

Outstand stiffener = 120 mm

Moments of inertia (I)

$$\frac{10 \times 120^3}{3} = 5760000 > 518400 mm^4$$

Ok

14. CONNECTION OF VERTICAL STIFFENER

Shear force

$$\frac{125 * tw^2}{h} = 150.00 \quad KN/m$$

Size of the weld = 1.98

So, provide size of the weld = 5 mm

Effective length of weld = 10 * tw

= 120 mm

Strength of welds /mm = 0.7 x 5 x 1 x 108

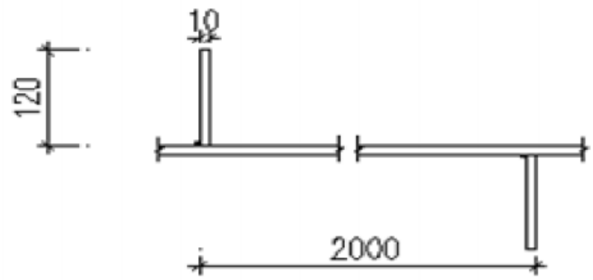
= 378 mm

$$\text{C/C spacing of the welds} = \frac{2 \times 378 \times 120}{150} = 604.8 \quad mm$$

Spacing =32 tw = 32 x 12 = 384 mm or 300 mm, whichever is less

So, = 604.8 > 384 mm

Therefore provided 5 mm size 120 mm long fillet welds at a center to center spacing of 190mm on both side of the web.



Connection between vertical stiffener and web

15. DESIGN OF BEARING STIFFENER

At end reaction	=	1197.78	kN
Allowable Bearing stress	=	189	N/mm^2
Bering Area Required	=	6337.45	mm^2

The end Bering stiffener is designed as a column

If outstand stiffener	=	300	mm
H/t is not grater than 12mm			
So, the thickness of the plate	=	25	mm
Provide plate size	=	300 x 25	mm

Let us try two plates 300 mm wide as a stiffener.

So, the total area provided	=	300 x 25 x2	
	=	15000 > 6337.45	mm^2

The Length of web plate which acts along with stiffener plate in bearing the reaction

= 20 *tw = 240 mm

Moment of inertia (I)	=	477578160	mm^4
Total area of the stiffener (A)	=	21060	mm^2
r min = (I /A)(1/2)	=	(477578160/21060)1/2	
	=	150.59	mm
Effective depth of stiffener (d1eff)	=	1400	mm
Slenderness ratio = (d1eff/r)	=	9.30	
Permissible stress σac	=	140	N/mm^2
Area required	=	8555.56	mm^2
	=	8555.56 < 21060	mm^2
			Ok

16. CONNECTION OF BEARING STIFFENER TO WEB PLATE

Length available for welding using alternate intermittent welds

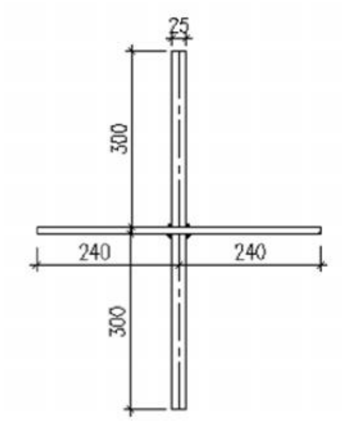
	=	3920	mm
Required strength of weld	=	305.56	N/mm

Required strength of weld	=	4.04	mm

So, provide 6 mm size intermittent fillet welds

Strength of welds /mm	=	0.7 x 6 x 1 x 108	
	=	453.6	N/mm
C/C spacing of the welds	=	178.14	mm
	<	300	mm
			Ok

Hence provided 200 mm c/c spacing



17. Lateral Bracing

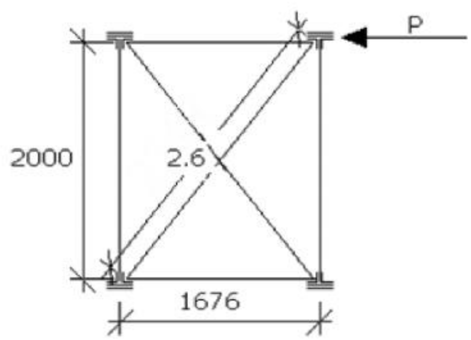
Wind Load		=	1.5	kN/m^2
Depth of the girder		=	2.08	m
Coefficient for wind load on leeward girder		=	0.25	
Wind Load on windward girder		=	(1.5 x 2.08 x 30)	
		=	93.6	kN
Wind Load on leeward girder		=	(0.25 x 93.6)	
		=	23.4	kN
Total wind load		=	(93.6 + 23.4)	
		=	117	kN
Lateral load due to racking forces		=	6	kN/m
Total racking force		=	180	kN
Total lateral load		=	297	kN
Maximum tension in the diagonal		=	(297/2)cosecθ	
		=	299.32	kN
Area required		=	[(299.32 x 103)/ (141)]	
		=	2122.810541	mm
Use ISA 80 x 80 x 10 mm thickness				
	a	=	1505	mm^2
Using 20 mm dia rivets	21.5 x 10	=	215	mm
Area provided	2 (1505 -215)	=	2580	mm^2
				Ok

Maximum compressive force in U1L1		=	148.5	kN
Length of member		=	1.676	m
Effective length	= 0.65 L	=	1.09	m
Use ISA 100 x 75 x 10 mm thickness				
	Area (A)	=	1650	mm^2
	rxx	=	21.6	mm
λ =	L/rxx	=	50.44	
Corresponding Permissible stress 6ac		=	124	N/mm
Safe load on member		=	1650 x 124/1000	
		=	204.6 > 148.5	N/mm^2
				Ok

18. Design of cross Frame

Lateral load to be resisted by one frame		=	148.5	kN
Tension in diagonal		=	148.50 sec θ	
		=	230.37	kN
Area required		=	[(190.20 *1000)]/ (141)]	
		=	1633.83	mm^2
Use 2 ISA 80 x80 x 8 mm thick				
	a	=	1221	mm^2
Using 20 mm dia rivets		=	172	mm
Area provided		=	2 (1221 -172)	
		=	2098	mm^2
				Ok

The cross frames are provided at 6 m interval.



APPENDIX C ANALYSIS AND DESIGN OF TRUSS GIRDER RAILWAY BRIDGE

DATA

Types of bridge	:	Pratt type through truss Girder Bridge	
Gauge	:	Broad gauge, single track, main line	
Span of the Girder (L)	=	30	m
	=	30000	mm
Number of panels	=	6	
Spacing between main girder	=	7	m
Panel length of bridge girder	=	5	m
Equivalent total L.L for B.M per track	=	2727	KN
Equivalent total L.L for S.F per track	=	2997	KN

1. DEAD LOADS

Dead load of track (open floor)			
Weight of stock rail per track per meter (Assume)	=	0.6	kN
	=	2 x 0.6	kN /m
	=	1.2	kN /m
Weight of guard rail per track per meter (Assume)	=	0.4	kN
	=	2 x 0.4	kN /m
	=	0.8	kN /m
Weight of fastenings (Assume)	=	0.2	kN /m
Unit weight of timber (Assume)	=	7.5	mm
Size of the sleeper (Assume)	=	250 x 150 x 3	mm
C/C spacing of sleepers	=	400	
Weight of sleepers per track per meter	=	2.92	kN /m
Weight of stringer per track per meter (Assume)	=	3	kN /m
Weight of stringer per track per meter (Assume)	=	5	kN /m
Self wt of girder	=	10	kN
Total dead Loads per track (w)	=	23.12	kN /m
Total dead Loads per girder per meter (w)	=	11.56	kN /m

2. INFLUENCE LINE DIAGRAMS FOR FORCE IN THE MEMBER

Height of the truss girder		=	1/8 to 1/5 of the span	
		=	3.75 to 6	m
So, Provide Height of the truss girder		=	6.00	m

Top chord members

Member : U1U2 this is a compression member
Opposite joint : L2

Height of the I.L.D = $\dfrac{a(L-a)}{Lh}$

With Respect to the opposite joint L2

= 1.11 m

Member : U2U3 this is a compression member
Opposite joint : L3

Height of the I.L.D = $\dfrac{a(L-a)}{Lh}$

With Respect to the opposite joint L3

= 1.25 m

Bottom chord members

Member : L0L1 this is a tension member
Opposite joint : U1

Height of the I.L.D = $\dfrac{a(L-a)}{Lh}$

With Respect to the opposite joint U1

= 0.69 m

Member : L1L2 this is a tension member
Opposite joint : U1

Height of the I.L.D = $\dfrac{a(L-a)}{Lh}$

= 0.69 m

Member : L2L3 this is a tension member
Opposite joint : U2

Height of the I.L.D = $\dfrac{a(L-a)}{Lh}$

With Respect to the opposite joint U2

= 1.11 m

Vertical members

Member : U1L1

When the unit load is exactly at L2 the force in the member U1L1 will be equal to 1

Member : U2L2

y1	=	0.33	m
y2	=	0.50	m

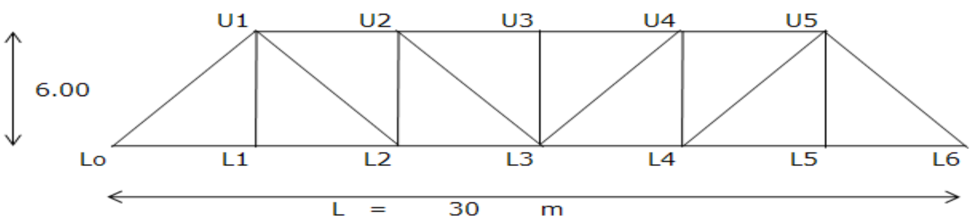
Diagonal members

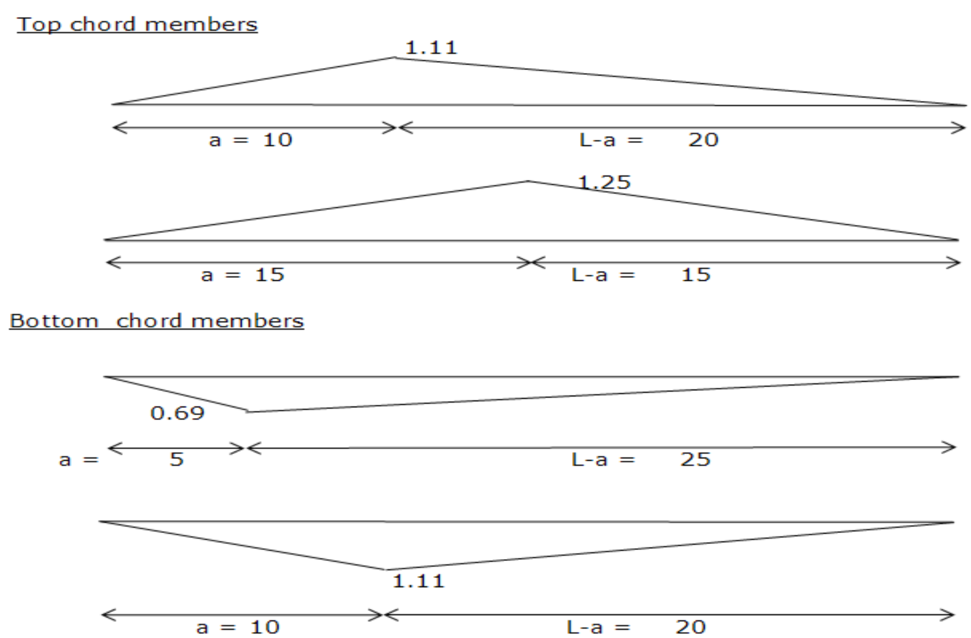
Member : U1L2

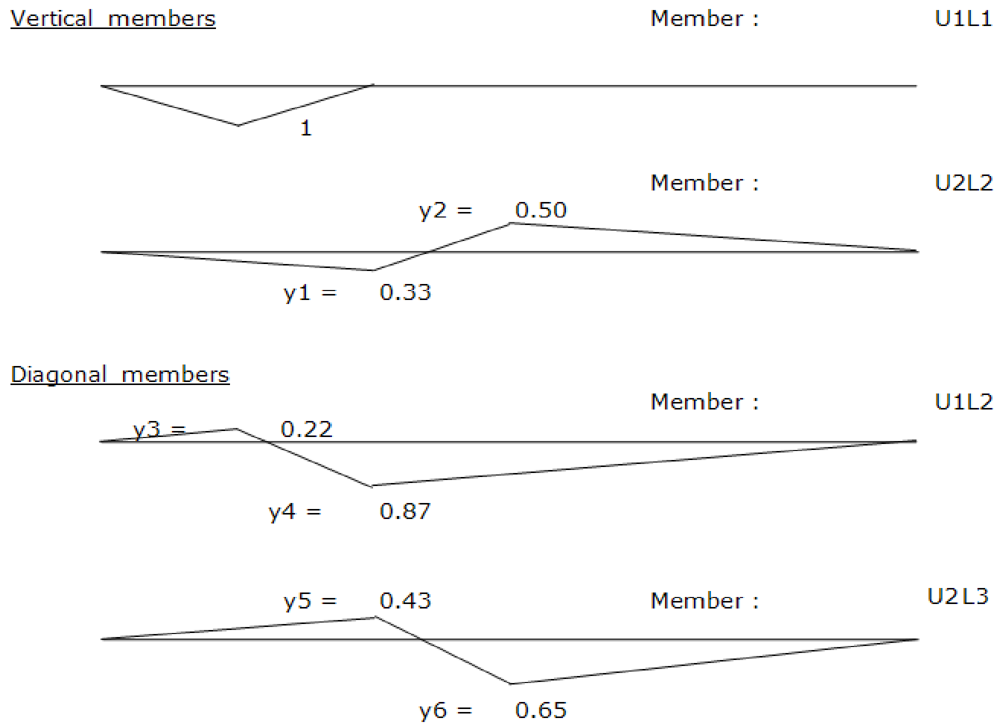
y3	=	0.22	m
y4	=	0.87	m

Member : U2L3

y5	=	0.43	m
y6	=	0.65	m







I.L.D diagram for different member

3. Force in the member due to dead load

Dead load		=	11.56	kN/m
Force in the member	U1U2		i.e., compression	
		=	11.56 x 30 x 1.11 x 1/2	
		=	192.64	kN
Force in the member	U2U3		i.e., compression	
		=	11.56 x 30 x 1.25 x 1/2	
		=	216.72	kN
Force in the member	L0L1		i.e., tension	
		=	11.56 x 30 x 0.69 x 1/2	
		=	120.40	kN
Force in the member	L1L2		i.e., tension	
		=	11.56 x 30 x 0.69 x 1/2	
		=	120.40	kN
Force in the member	L2L3		i.e., tension	
		=	11.56 x 30 x 1.11 x 1/2	
		=	192.64	kN
Force in the member	U1L1		i.e., compression	
		=	11.56[30 x 1 x 1/2]	
		=	173.38	kN
Force in the member	U2L2		i.e., compression	
		=	11.56[18x0.50x1/2-12 x0.33x1/2]	
		=	28.90	kN
Force in the member	U1L2		i.e., compression	

= 11.56[24 x 0.87 x 1/2- 6 x 0.22 x1/2]
= 112.84 kN

Force in the member U2L3 i.e., compression
= 37.61 kN

4. Force in the member due to live load and impact load

Member : U1U2
Loaded length L = 30 m

Impact factor $0.15+\dfrac{8}{6+L}\prec 1.0$ = 0.37 Ok

Live load +Impact load per girder = 1.372 x (1/2 x 2997)
= 2056.28 kN

Force in the member due to Live load +Impact load
i.e., compression
= 1142.38 kN

Member : U2U3 i.e., compression
= 1285.17 kN

Member : L1L2 i.e., tension
= 713.98 kN

Member : L2L3 i.e., tension
= 1142.38 kN

Member : U1L1 i.e., compression
= 1028.14 kN

Member : U2L2 (For tension)
Loaded length = 12 m
Impact factor
$0.15+\dfrac{8}{6+L}\prec 1.0$ = 0.59 Ok

Live load +Impact load per girder
= 1.594 x (1/2 x 1589)
= 1266.79 kN

Force in the member due to Live load +Impact load i.e., tension
= 211.13 kN

Loaded length (For compression)
= 18 m

Impact factor

$0.15+\dfrac{8}{6+L}\prec 1.0$ = 0.48 Ok

Live load +Impact load per girder = 1.594 x (1/2 x1990) kN
= 1475.92 kN

Force in the member due to Live load +Impact load i.e., compression
= 368.98 kN

Member : U1L2
The force in the member i.e., tension

	=	Force in U1L1 x cosec θ	
	=	1028.14 x1.3	kN
	=	1338.34	kN

Member : U2L3

When the force in U2L2 is 368.98 KN (compression).The corresponding force in member

The force in the member U2L3		i.e., tension	
	=	Force in U2L2 x cosec θ	kN
	=	368.98 x1.3	
	=	480.30	kN

When the force in U2L2 is 211.13 kN (tension) the corresponding force in member

The force in the member U2L3		i.e., compression	
	=	Force in U2L2 x cosec θ	
	=	211.13 x1.3	kN
	=	274.83	kN

Design forces in the members are as follows:

Members	Force in the member				Design forces	
	D.L		L.L+I.L		D.L+L.L+I.L	
	Compre. kN	Tension kN	Compre. kN	Tension kN	Compre. kN	Tension kN
U1U2	192.64	-	1142.38	-	1335.01	-
U2U3	216.72	-	1285.17	-	1501.89	-
L0L1	-	120.40	-	713.98	-	834.38
L1L2	-	120.40	-	713.98	-	834.38
L2L3	-	192.64	-	1142.38	-	1335.01
U2L2	28.90	-	368.98	211.13	397.88	211.13
U1L2	37.61	-	-	1338.34	37.61	1338.34
U2L3	112.84	-	274.83	480.30	387.67	480.30

5. Design of the top chord member

Force in the member	=	1501.89	kN
Depth of the truss girder	=	6.00	m
	=	6000	mm
Depth of top chord member h	=	1/10 x 6000	
	=	600	mm
Assuming Gusset plate thick.	=	22	mm
Width of top chord member (b)	=	1/10 x 6000 +2 x22	
	=	644.00	mm
Assume the thick of the web	=	12	mm
Overall width of the section	=	828.00	mm
So, Overall width of the section	=	850.00	mm
Using angle	=	ISA 80 x 50 x 6	

Width of top cover plate of the section between center to center of rivet line

B		=	768.00	
Ratio	= b/t	=	48.00 < 50	
Which gives	t	=	768/50	
		=	15.36	

Provide the thickness of the top cover plate

		=	16	mm

Therefore, the whole width of top cover plate is effective in compression

Top cover plate	=	850 x16	
	=	13600	mm^2
Web	=	2x 600 x 12	
	=	14400	mm^2

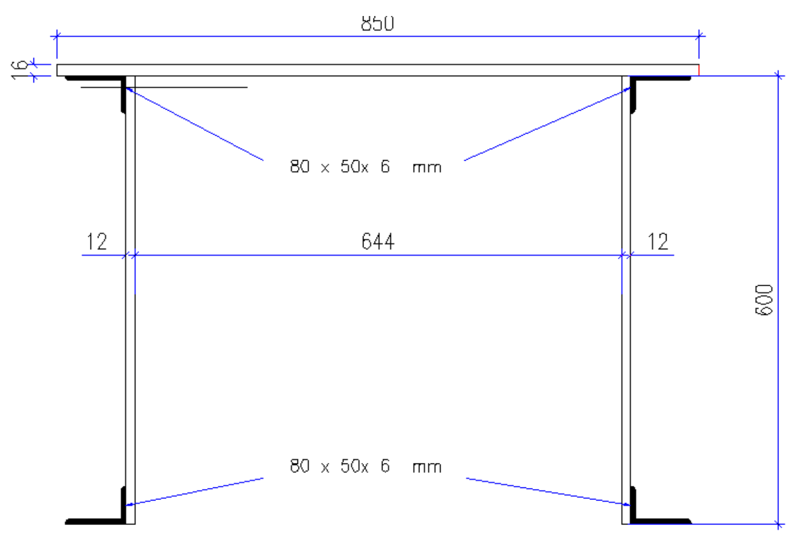
Using 4 ISA 80 mm X 50 MM X 6 MM

	=	2984	mm^2

The area of the section provided as

Total area	=	30984	mm^2

Try the section as shown in figure



Approximate radius of gyration

rx	= 0.39 * h	=	234	mm
ry	= 0.55*b	=	354.2	mm
r min		=	234.00	

Length of member from center to center of intersection is 5 mm

Effective length of the member	=	0.85 x 5000	mm
	=	4250	mm
Maximum slenderness ratio (λ)	=	4250/234	
	=	18.16	

From IRS allowable stress in axial compression, for the steel having value of yield stress as 236 N/mm^2

σac	=	140.00	N/mm^2
Area required	=	1501.89 X 103/140	
	=	10727.79	mm^2

More area is provided in order to adjust increase of force due to wind effect.
The c.g of section from top is at a distance (Y)

$$\overline{Y} = \left[\frac{850x16x8 + 2x746x(11.6+16) + 2x746x(600- \ 11.6+16) + (2x600 \times 12 \times 316)}{850 \times 16 + 2x746 + 2x746 + 2 \times 600 \times 12)}\right]$$

	=	180.81	mm
Ixx	=		

$$(2 \times \frac{1}{12} \times 1.2 \times 60^3 + 2 \times \ 60 \times \ 1.2 \times \ (30 \ - \ 18.08)^{\ 2} + 85 \ \times \ 1.6 \times \ (18.08 \ - 0.8)^{\ 2}$$

$$+ 2 \times 7.46 \ \times \ (18.08 \ -2.76)^{\ 2} + (2 \times 7.46 \ \ \times (45.32 \ -1.16)^{\ 2} + 4 \times 14.4) \ \times 10^4$$

	=	297014885.60	mm^4
Iyy	=		

$$(\frac{1}{12} x1.6 \times 85^3 + 2 \times 60 \times 1.2x(32.1+0.6)^2 + 4 \times 48)x10^4$$

	=	236996.29	mm^4
rmin	=	(297014885.6/30984)1/2	
	=	97.91	mm
Maximum slenderness ratio	=	4250/210.22	
	=	43.41	mm

From IRS allowable stress in axial compression, for the steel having value of yield stress as 236 N/mm^2

σac	=	130	N/mm^2
Force carrying capacity of the member			
	=	136 X 30984/1000	
	=	4027.92	kN
	=	4027.92 > 1501.89	kN
			Ok

6. Design of the vertical member

Compressive Force	=	397.88	kN
Tensile Force	=	211.13	kN

The over all depth of vertical member is kept equal to the internal width of top chord members less twice the thickness of gusset plate

Depth of the member	=	600	mm
Length between c/c of intersection	=	6000	mm
Effective length of member	=	0.7x 6000	
	=	4200	mm

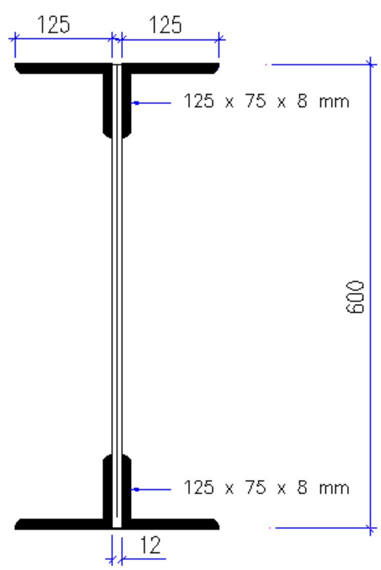
Assuming allowable stress in axial compression, for the steel having value of yield stress as 236 N/mm^2

σac	=	115	N/mm^2
Area required	=	397.88x1000/115	
	=	3459.78	
Provide 4 ISA 125mm x 75 mm x 8 mm			
Area provided	=	4 x 1538	
	=	6152	mm^2

Ixx	=	496092180.50	mm^4
Iyy	=	23700450	mm^4
rmin	=	$(23122162/6152)^{1/2}$	
	=	62.07	mm
Slenderness ratio (λ)	=	4200/61.31	mm
	=	67.67	

From IRS , for the steel having value of yield stress as 236 N/mm^2

σac	=	118.0	N/mm^2
Force carrying capacity of the member			
	=	118 X 6152/1000	
	=	725.93	kN
	=	725.94 > 397.88	kN
			Ok



To check for the tension member

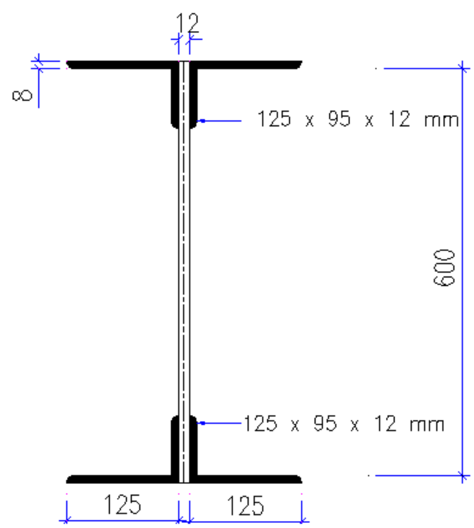
Using dia of the rivets	=	22	mm
A1	=	121 x 8 -23.5 x 8	
	=	780.00	kN
A2	=	71 x 8	
	=	568.00	kN
K	=	0.80	
Allowable stress in axial tension,			
σat	=	141.00	N/mm^2
Anet Required	=	P/σat	
	=	211.13X103/141	
	=	1497.38	mm^2
Anet Provided	=	2(A1+k x A2)	
	=	2474.11	
	=	2474.11 > 1497.38	mm^2
			Ok

Load carrying capacity	=	2474.11 x141		
	=	348.8499037		kN
	=	348.85	> 211.13	kN
				Ok

7. Design of the diagonal member

Tensile force	=	480.30	kN
Compressive force	=	387.67	
Provide 4 ISA 125mm x 95 mm x 12 mm			
Area provided	=	9992	mm^2
A1	=	1146	mm^2
A2	=	1068	mm^2
	=	0.76	

Figure



Allowable stress in axial tension

σat	=	141.00	N/mm^2
Anet Required	=	P/σat	
	=	480.30X103/141	
	=	3406.41	mm^2
Anet Provided	=	2(A1+kx A2)	
	=	3921.73	mm^2
	=	3921.73 > 3406.41	kN
			Ok
Load carrying capacity	=	552.96	kN
	=	552.96 > 480.30	kN
			Ok

To check for the compression member
Assuming allowable stress in axial compression, for the steel having value of yield stress as 260 N/mm^2

σac	=	115	N/mm^2

Area required	=	387.67x1000/115	
	=	3371.06	
Provide 4 ISA 125mm x 95 mm x 12 mm			
Area provided	=	9992	mm^2
Ixx	=	764910579.30	mm^4
Iyy	=	36080965.12	mm^4
rmin	=	$(36080965.12/9992)^{1/2}$	mm
	=	60.09	
Slenderness ratio (λ)	=	69.89	mm
From IRS, for the steel having value of yield stress as 236 N/mm^2			
σac	=	118	N/mm^2
Force carrying capacity of the member			
	=	1179.05	kN
	=	1179.05 > 387.67	kN
			Ok

8. Design of the bottom member

Forces in the member	=	1335.01	kN
4 ISA 200mm x150mm x 18 mm			
Area provided	=	23904	mm^2
A1	=	191 x 18 - 23.5 x 18	mm^2
	=	3015	
A2	=	141 x18	
	=	2538	mm^2
$k = \frac{3A1}{3A1 + A2}$	=	0.78	

Allowable stress in axial tension			
σat	=	156.00	N/mm^2
Net area required	=	P/σat	
	=	1335.01x1000/156	
	=	8557.78	mm^2

The over all dimension of the section is kept same as that for top chord. 2 web plates 600 x 12 mm

	=	2 x 600 x 12	
	=	14400	mm^2
Anet Provided	=	2(A1+k*A2)	
	=	10738.74	mm^2
	=	10738.74 > 8557.78	mm^2
			Ok
Load carrying capacity	=	1675.24	kN
	=	1675.24 > 1335.01	kN
			Ok

9. Design of the joint U2

Use 22 mm diameter power driven rivets
Strength of rivets in single shear

$$\frac{\pi}{4} x \left(\frac{23.5^2 x100}{1000} \right)$$

$$= \quad 43.35 \text{ kN}$$

Strength of rivets in bearing

$$\left(\frac{23.5x300x10}{1000}\right) = 70.5 \text{ kN}$$

Rivet value	=	43.35	kN
Force in U1U2	=	1335.01	kN
Force in U2U3	=	1501.89	kN

The top chord member is a continuous member. The rivets are provided for the difference of forces.

Number of rivets required $= \dfrac{1501.89 - 1335.01}{43.35}$

$= 3.85$

Provide 16 rivets

Force in U2L2 $= 397.88$ kN

Number of rivets required $= \left(\dfrac{397.88}{43.35}\right)$

$= 9.18$

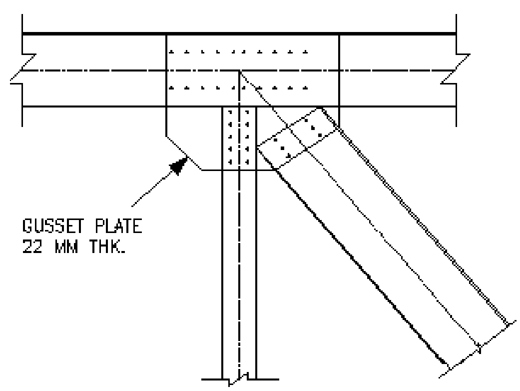
Provide 16 rivets

Force in U2L3 $= 480.30$ kN

Number of rivets required $= \left(\dfrac{479.67}{43.35}\right)$

$= 11.08$

Provide 16 rivets



10. Design of the joint L2

Use 22 mm diameter power driven rivets

Strength of rivets in single shear $= \dfrac{\pi}{4} x \left(\dfrac{23.5^2 x100}{1000}\right)$

$= 43.35$ kN

$$\left(\frac{23.5x300x10}{1000}\right)$$

Strength of rivets in bearing

	=	70.5	kN
Rivet value	=	43.35	
Force in L2L3	=	1335.01	kN
Force in L2L1	=	834.38	kN

The Bottom chord member is a continuous member. The rivets are provided for the difference of forces.

Number of rivets required = $\frac{211.13}{43.35}$

= 11.55

Provide 16 rivets

Force in L2U2 = 211.13 kN

Number of rivets required = $\frac{1335.01 - 834.38}{43.35}$

= 4.87

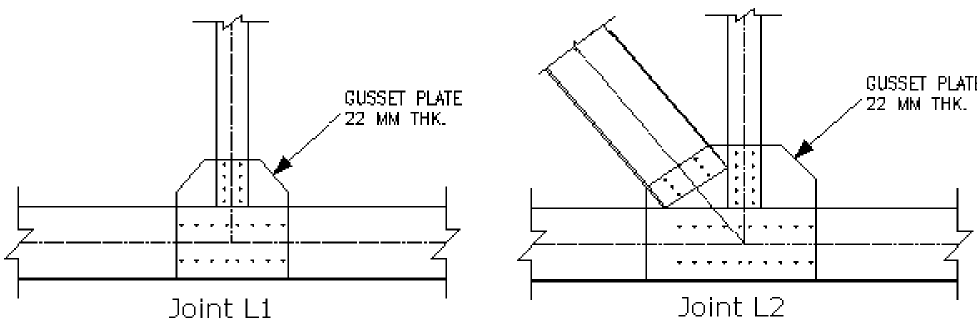
Provide 8 rivets

Force in L2U1 = 1338.34 kN

Number of rivets required = $\frac{1336.58}{43.35}$

= 30.87

Provide 16 rivets



11. Design of the joint L0

Strength of rivets in single shear	=	43.35	kN
Strength of rivets in bearing	=	70.5	kN
Rivet value	=	43.35	
Force in L0L1	=	834.38	kN
Number of rivets required	=	834.38/43.35	
	=	19.25	
		Provide 16 rivets	

12. Additional Forces in members of the truss girder due to wind load

Case I Bridge is unloaded			
Wind pressure	=	2.4	kn/m^2
Assume			
Depth of top & Bottom chord members	=	600	mm
Width of top & Bottom chord members	=	644	mm
Depth of Vertical members	=	600	mm
Width of Vertical members	=	260	mm
Depth of Diagonal members	=	600	mm
Width of Diagonal members	=	644	mm
End posts depth	=	600	mm
End posts width	=	644	mm

Gusset plate area @ 0.5 m2 for each gusset 9 x 0.5 m at top and 11 x 0.5 m at bottom.

	=	4.5	mm^2
	=	5.5	mm^2

Wind load on windward girder is as follows:

1. Wind load on top chord	=	20 x 0.60 x 2.4	
	=	15.6	kN
2. Wind load on Bottom chord	=	30 x 0.60 x 2.4	
	=	43.2	kN
3. Wind load on Vertical	=	5 x 6 x 2.4 x 0.26	
	=	18.72	kN
4. Wind load on Diagonal	=	4 x 7.81x 2.4 x 0.26	
	=	19.49	kN
5. Wind load on end posts	=	2 x7.81x 2.4 x 0.26	
	=	9.75	kN
6. Wind load on top gussets	=	4.5 x 2.4	
	=	10.8	kN
7. Wind load on Bottom gussets	=	5.5 x 2.4	
	=	13.2	kN
Spacing between main girders	=	7	m

Wind load is assumed 75 percent of wind load on windward girder for leeward girder.

Wind load on leeward girder is as follows:

1. Wind load on top chord	=	0.75 x 15.6	
	=	11.70	kN
2. Wind load on Bottom chord	=	0.75 x 43.2	
	=	32.40	kN
3. Wind load on Vertical	=	0.75 x 18.72	
	=	14.04	kN
4. Wind load on Diagonal	=	0.75 x 19.49	
	=	14.62	kN
5. Wind load on end posts	=	0.75 x 9.75	
	=	7.31	kN

6. Wind load on top gussets	=	0.75 x 10.8	
	=	8.10	kN
7. Wind load on Bottom gussets	=	0.75 x 13.2	
	=	9.90	kN

Wind load acting on top chord

P1 = Wind load on top chord + 1/2 wind load on vertical + 1/2 wind load on diagonal and end posts +wind load on top gussets.

= [15.6+11.70) + 1/2 (18.72 +14.04) + 1/2 (19.49 +14.62) + 1/2 (9.75+7.31) + (10.8+8.10)]

= 88.17 kN

P2 = Wind load on Bottom chord + 1/2 wind load on vertical + 1/2 wind load on diagonal and end posts +wind load on Bottom gussets.

= [43.2+32.40) + 1/2 (18.72 +14.04) + 1/2 (19.49 +14.62) + 1/2 (9.75+7.31) + (13.2+9.9)]

= 140.67 kN

Overturning effect due to wind, when bridge is unloaded

Take the moment about level of Bearings

2R x 7	=	(P1 x 6.8 + P2x 0.8)	
	=	(88.17x 6.8 + 140.67 x 0.8)	
	=	712.06	kN
2R	=	101.72	kN

Due to overturning effect, a thrust of 101.72 kN acts downward on leeward girder. Increase of stress in central top chord member U2U3 of the leeward girder

Comp. $\frac{1}{2}\left(\frac{1}{6}x\frac{15x15}{30}\right)(30)\left(\frac{101.72}{30}\right)$ = 63.58 kN

Increase of stress in central Bottom chord L2L3 of the leeward girder

Tension $\frac{1}{2}\left(\frac{1}{6}x\frac{10x20}{30}\right)(30)\left(\frac{101.72}{30}\right)$ = 56.51 kN

Lateral effect of top chord bracing when the bridge is unloaded

The top chord members of the leeward girder are subjected to tension due to Lateral effect of the top lateral bracing. Therefore, the force in these Members decrease. Decrease in force in central top chord member, U2U3 Due to top lateral bracing.

= (88.17 x20/8) x (1/7)

= 31.49 kN

Lateral effect of Bottom chord bracing when the bridge is unloaded

The top chord members of the leeward girder are subjected to tension. Therefore, the force in these members increase. Increase in force in central bottom chord member due to bottom lateral bracing.

Tension = (140.67 x30/8) x (1/7)

= 75.36 kN

Case II Bridge is loaded

Wind pressure	=	1.5	kn/m^2
1. Wind load on top chord	=	55.10	kN
2. Wind load on Bottom chord	=	87.92	kN
3. Wind load on moving train	=	30 x 3.5 x 1.5	
	=	157.5	kN

Overturning effect due to wind, when bridge is loaded

Take the moment about one of Bearings

2R x 7 = 55.10x6.8+87.92x0.8+157.5(0.75+0.60+0.80+0.80)

= 909.67 kN

2R = 129.95 kN

Due to overturning effect, a thrust of 129.95 kN acts downward on leeward girder. Increase of stress in central top chord member U2U3 of the leeward girder

$$\frac{1}{2}\left(\frac{1}{6}x\,\frac{15x15}{30}\right)(30)\left(\frac{129.95}{30}\right) = 81.22 \text{ kN}$$

Increase of stress in central Bottom chord member L2L3 of the leeward girder

$$\frac{1}{2}\left(\frac{1}{6}x\,\frac{10x20}{30}\right)(30)\left(\frac{129.95}{30}\right) = 72.20 \text{ kN}$$

Lateral effect of top chord bracing when the bridge is loaded

The top chord members of the leeward girder are subjected to tension due to lateral effect of the top lateral bracing. Therefore, the force in these members decrease. Decrease in force in central top chord member, U2U3 due to top lateral bracing.

	=	(88.17 x20/8) x (1/7)	
	=	31.49	kN

Decrease in force in central top chord member due to top lateral bracing. When the bridge is loaded

	=	(31.49x55.10)/88.17	
	=	19.67	kN
Wind load on top lateral bracing, when the bridge is unloaded			
	=	88.17	kN
Wind load on top lateral bracing, when the bridge is loaded			
	=	55.10	kN

Lateral effect of Bottom chord bracing when the bridge is loaded

Wind pressure acting on bottom lateral bracing when bridge is unloaded			
	=	40.67	kN
Increase in force in central bottom chord member due to bottom lateral bracing.	=	31.49	kN
Wind pressure acting on bottom lateral bracing when bridge is loaded P2 + P3	=	188.99	kN
Increase in force in central bottom chord member due to bottom lateral bracing.	=	93.60	kN

Top Lateral Bracing

When he bridge is unloaded, the wind load acting on the bridge structure on windward and leeward girders P1 = 88.17 kN

Reaction at the end	=	44.08	kN

6 panels @ 5 m are provided in the top lateral bracing.

Lateral load at each intermediate panel point			
	=	88.17/6	kN
	=	14.69	kN
Lateral Load at end panel	=	7.35	kN
End strut carries maxi. Compression	=	44.08	kN
Shear force in end panel	=	44.08 - 7.35	
	=	36.74	kN
tan θ	=	1.4	
θ	=	50° 12˙	
sin θ = 0.768 , cosec θ	=	1.30	
Force in the end diagonal,	=	1.035 x 36.74	
	=	47.83	kN

End strut

Effective length	=	0.85x7000	
	=	5950	mm
Compression in end strut	=	44.08	kN

Assume allowable stress in axial compression for the slenderness ratio 120 and the steel having yield stress as 236 N/mm^2

λ = 120 σac	=	63	N/mm^2
Cross - sectional area required	=	(44.08 x1000)/63	
	=	699.73	mm^2

Maximum allowable slend. Ratio for compression members of wind bracing

Minimum radius of gyration r min	=	5950/140	mm
	=	42.5	

Provide 2 ISA 150 x 115 x 8 mm with 10 mm gusset plate

rx = 47.6 mm , ry	=	34.1	mm
Area	=	4116	mm^2
Slenderness ratio	=	5950/46.2	
	=	174.49	mm

From IRS allowable stress in axial compression for steel having yield stress as

σac	=	236	N/mm^2
σac	=	59.24	N/mm^2
Force carrying capacity of member	=	59.24x4116/1000	
	=	243.83	kN

Diagonal member

Force in diagonal	=	47.83	kN
Net area required	=	(47.94 x1000)/140	
	=	341.67	mm^2
Area of Rivets hole	=	235	mm^2
Gross area required	=	576.67	mm^2
Provided ISA 90 mm x 60 mm x 10			mm
Gross area provided	=	1401	mm^2
	=	576.67 < 1401	mm^2
			Ok

Bottom Lateral Bracing

Wind load on bottom chord when bridge is unloaded, P2

	=	140.67	kN

Wind load on bottom chord when bridge is loaded, P2

	=	87.92	kN
Wind load on moving train, P3	=	157.5	kN
Racking force @ 6.00 kN/m			
P4	=	6.00 x 30	
	=	180	kN
Total lateral load	=	P2+P3+P4	
	=	425.42	kN

The lateral load action in the plane of bottom lateral bracing is maximum in case the bridge is loaded. Therefore, the action lateral bracing is designed for lateral force

	=	425.42	kN

End reaction	=	212.71	kN
Number of panels	=	6	
Wind load at each intermediate panel point			
	=	70.90	kN
Wind load at end panel	=	35.45	kN
Shear force in end panel	=	212.71-35.45	
	=	177.26	kN
Cosec θ	=	1.302	
Force in diagonal member	=	177.26 x 1.035	
	=	230.78	kN
Allowable stress in axial tension,	=	140	N/mm^2
Net area required	=	(231.32 x1000)/140	mm^2
	=	1648.48	mm^2
Assuming area for rivet hole	=	640	mm^2
Gross area required	=	2288.48	mm^2

Provided 2ISA 150 mm x 115 mm x 10 mm.

Gross area provided	=	2 x 2552	
	=	5104	mm^2

APPENDIX - D RAILWAY LOADING TABLES

EUDL For Broad Gauge Standard Loading -1987. Broad Gauge 1676 mm					
Length	**Total Load Bending Moment.**		**Total Load for Shear**		**C D A**
Meter	**KN**	**t**	**KN**	**t**	$0.15+\frac{8}{6+L} \prec 1.0$
1	490	50	490	50	1
1.5	490	50	490	50	1
2	490	50	519	52.9	1
2.5	490	50	598	61	1
3	490	50	662	67.5	1
3.5	516	52.6	707	72.1	0.992
4	596	60.8	778	79.3	0.95
4.5	677	69	838	85.5	0.912
5	741	75.6	888	90.5	0.887
5.5	794	81	941	95.9	0.846
6	838	85.5	985	100.4	0.817
6.5	876	89.3	1024	104.4	0.79
7	911	92.9	1068	108.9	0.765
7.5	948	96.7	1111	113.3	0.743
8	981	100	1154	117.7	0.721
8.5	1010	102.9	1210	123.4	0.702
9	1040	106.1	1265	129	0.683
9.5	1070	109.1	1315	134.1	0.66
10	1101	112.3	1377	140.4	0.63
11	1282	130.7	1492	152.2	0.621
12	1377	140.4	1589	162	0.594
13	1475	150.4	1670	170.3	0.571
14	1558	158.9	1740	177.4	0.55
15	1631	166.3	1801	183.6	0.531
16	1695	172.8	1853	189	0.514
17	1751	178.5	1926	196.4	0.498
18	1820	183.6	1990	203.9	0.483
19	1886	192.4	2080	212.1	0.47
20	1964	200.3	2168	221.1	0.458
21	2039	207.9	2254	229.8	0.446
22	2123	216.5	2337	238.3	0.436
23	2203	224.7	2420	246.8	0.426
24	2280	232.5	2503	255.2	0.417
25	2356	240.2	2586	263.7	0.408
26	2431	247.9	2668	272.1	0.4
27	2506	255.5	2751	280.5	0.392
28	2580	263.1	2833	286.9	0.385
29	2654	270.6	2915	297.3	0.379
30	2727	278.1	2997	306.7	0.372
32	2874	293	3161	322.4	0.361

34	3034	309.3	3325	339.1	0.35
36	3191	325.3	3489	355.8	0.34
38	3345	341.1	3652	372.4	0.332
40	3498	356.7	3815	389.1	0.324
42	3649	372.1	3978	405.1	0.317
44	3798	387.3	4141	422.3	0.31
46	3947	402.4	4304	438.9	0.304
48	4094	417.4	4467	455.5	0.298
50	4253	433.7	4630	472.1	0.293
55	4658	474.9	5036	513.6	0.281
60	5051	515.1	5442	550	0.271
65	5436	554.3	5848	596.4	0.263
70	5831	594.6	6254	637.7	0.255
75	6220	634.3	6660	679.1	0.249
80	6603	673.3	7065	720.4	0.243
85	6986	712.4	7470	761.8	0.238
90	7361	753.7	7876	803.1	0.233
95	7796	795	8281	844.4	0.229
100	8201	836.2	8686	885.7	0.225
105	8606	877.5	9091	927	0.222
110	9011	918.8	9496	968.3	0.219
115	9415	960.1	9901	1009.6	0.216
120	9820	1001.4	10306	1050.9	0.213
125	10225	1012.7	10711	1092.2	0.211
130	10630	1083.9	11115	1133.5	0.209

Allowable average shear stress in stiffened webs of steel conforming to IS: 226

d/t	**Shear stress (N/mm^2) for different distance between stiffeners**						
	0.4d	**0.6d**	**0.8d**	**d**	**1.2d**	**1.4d**	**1.5d**
110	87	87	87	87	87	87	87
130	87	87	87	87	87	84	82
150	87	87	87	85	80	77	75
170	87	87	83	80	76	72	70
190	87	87	79	75			
200	87	85	77				
220	87	80	73				
240	87	77					

Allowable working stress σbc for Different value of critical Stress CS (IRC24)		
Cs	**σbc for steel conforming to IS: 226 (mild steel with σy = 236 N/mm^2)**	**σbc for steel conforming to IS: 961 (high tensile steel with σy=299, 331, 362 N/mm^2)**
30	15	15
40	20	20
50	25	25
60	30	30
70	35	35
80	38	38
90	42	42
100	46	46
120	53	54
140	60	62
160	67	70
180	72	77
200	76	84
220	80	90
240	84	96
260	88	102
280	92	108
300	96	114
350	105	127
400	112	137
450	119	146
500	124	153
550	129	159
600	133	165
650	136	171
700	139	174
800	144	188
900	149	194
1500	158	212
2150	158	224

Allowable average shear stress in stiffened webs of steel conforming to IS: 226							
d/t	**Shear stress (N/mm^2) for different distance between stiffeners**						
	0.4d	**0.6d**	**0.8d**	**d**	**1.2d**	**1.4d**	**1.5d**
110	87	87	87	87	87	87	87
130	87	87	87	87	87	84	82
150	87	87	87	85	80	77	75
170	87	87	83	80	76	72	70
190	87	87	79	75			
200	87	85	77				
220	87	80	73				
240	87	77					

Allowable Working Stress σac in N/mm2 on Effective Cross Section for Axial Compression				
λ=(L/r)	**σy = yield Stress of Steel (N/mm^2)**			
	236	**299**	**331**	**362**
0	140	171.2	191.5	210
20	136	167	186	204
40	130	157	174	190
60	118	139	151.6	162
80	101	113.5	120.3	125.5
100	80.5	87	90.2	92.7
120	63	66.2	68	69
1410	49.4	51.2	52	52.6
160	39	40.1	40.7	41.1