Structural Design of Plate Girder Railway Bridge

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CERTIFICATE

This is to certify that the work entitled, **"Structural Design of Plate Girder Railway Bridge"** submitted by Achin Aggarwal 051607, Sachin Jindal 051609 has been carried out under my supervision. This work has not been submitted partially or wholly to any other University or Institute for the award of this or any other degree or diploma.

Supervisor

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Mrs. Poonam Dhiman
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Certified the above mentioned project work has been carried out by the said group of students.

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Professor and Dean
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CANDIDATE'S DECLARATION

We here by certify that the work which is being presented in this report, **"Structural Design of Plate Girder Railway Bridge"** in partial fulfillment of the requirement for the award of B. tech degree, submitted in the department of Civil Engineering, Jaypee University of Information Technology, Waknaghat, is an authentic record of our own work carried out from July 2008 to May 2009 under the guidance of Mrs. Poonam Dhiman, Lecturer in department of Civil Engineering. We have not submitted the matter embodied in the report for the award of any other degree.

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ACKNOWLEDGMENT

This project comes near to the culmination of all the concepts assimilated while studying the subject. It has presented us with an opportunity to use the technical know how imparted to us a real life project.

Designing a Railway bridge involves the concepts of steel structure and mechanics. Our esteemed mentors Mrs. Poonam Dhiman, Dr. R.M. Vasan of Department of civil Engineering, Jaypee University of information Technology, not only cleared all our ambiguities but also generated a high level of interest in the subject. We are truly grateful to both of them.

The prospect of working in a group with a high level of accountability fostered a spirit of teamwork and created a feeling of unity which thus, expanded our range of vision, motivated us to perform to the best of our ability and create a report oh highest quality.

To do only the best quality work, with almost sincerity and precision has our constant endeavor.

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CONTENT

ABSTRACT

Aim of this project is to design a steel plate girder bridge. Plate girder bridges are used to support railroads, highways over the valley. In this project, a deck type plate girder bridge is designed which is intended to be used as a railway bridge. This bridge is first designed with IS 800-1984, which is based on Working Stress Method. Design loads such as live loads are taken from IRC. All the components of bridge such as stiffeners are also designed using IS: 800-1984. Then same bridge is again designed for the same loads using IS: 800-2007.IS: 800-2007 is revised code for general construction in steel, which is based on Limit State Method of design. After comparing the results of design, it was found that- LSM method gives a better section.

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INTRODUCTION

1.1 General

As far back as it can be seen in history, human beings have used new technology to solve problems and ease their physical burdens. The distinctiveness of humans as a species is defined by their use of tools, and bridges are technological tools that aim to solve the problem of crossing an obstacle in such a way as to cut down the effort and time needed to do so. The better a bridge is, the less attention the user will need to pay it. Bridge is a structure providing passage over an opening-gorge, road, railway, canal, river, creek etc without closing the same. The required passage may be for road, railway, canal, pipe line, or pedestrian. There is hardly a project, whether it is for the highways, railways or a combination thereof, which will not envisage the use of a bridge, large or small. One of the major concerns in planning transportation networks in the modern times is to ensure savings in time, apart of course, from financial economy. A bridge project could lead to this by connecting areas hitherto joined by lengthy routes, or by permitting high speed transportation. Expansion of transportation network will require construction of a large number of bridges with great speed. The traffic congestion on city roads, even in moderate sized cities is becoming intolerable. The answer lies in building flyovers or elevated road systems. The quest to modernize railway yards and the problem of tackling large volume of pedestrian movement at railway stations has thrown up the need for 'road-over' and 'foot-over' bridges. To cater the above need, bridges in all ranges of span, from small to large, have to be employed.

1.2 Introduction to project

In this project we are designing a deck type plate girder bridge for single track B.G main line loading for the following data:

1.) Effective span: 24 m

2.) Spacing of plate girders: 1.9 c/c

3.) Weight of stock rails: 440 N/m

4.) Weight of guard rails: 260 N/m

5.) Weight of fastenings etc: 280 N/m of track

6.) Timber sleepers: 250 mm × 150 mm × 2.8 m @ 0.4 m c/c

7.) Density of timber: 7.4 KN/m^3

1.3 Introduction to Bridges

There are following types of bridges depending on various parameters:

1.3.1 Classification based on the main structural system

A.) Arch Bridge

In the arch type of bridge, weight is carried outward along two paths, curving toward the ground. The points where the arch reaches the ground keep the bridge up by resisting the outward thrust. The roadway is located on top of the arch. Stone was an apt material for arch bridges due to is compressive strength. The arch is really a beam curved to form a semi circular shape, which is prevented from straightening and spreading sideways by strong abutments at either ends. The traditional shape of the arch is made from a series of block carefully cut to fit together perfectly. These voussoirs are wedge-shaped and gradually take the curve of the arch from the central and vertical keystone down to the outermost and horizontal footers.

As the keystone is pushed downwards by the load above it, its wedge shape means that it pushes outward onto the voussoirs to each side. Thus the forces are spread sideways, rather than downwards, and thence around the arch. Ultimately, the entire load (the weight both of the bridge itself and of any traffic crossing it) is transferred partly down into the ground and partly out to the elements of the bridge to either side of the keystone. This is the key feature by which the arch improves on the simple beam – the partial dissipation of vertical forces horizontally. This does mean, however, that for an arch to work there always needs to be substantial material to the sides, to stop the arch spreading and the central section collapsing inwards. This is what makes early arch bridges so massive in form. The road deck can be around the curve of the arch itself or it can rest on the arch. Arch bridges remained the design of choice for heavy traffic bridges for over 2000 years, but bridge engineers were constantly incorporating changing fashions and new materials. As we will see, the arch has gone through some extraordinary and often beautiful reincarnations.

One disadvantage of the arch is that it needs firm support from the sides. If there are no abutments or banks to build against, the arch will spread and collapse. This is why large cathedrals need 'buttresses' to provide the horizontal support force necessary to counteract the horizontal dissipation of the load from the roof and walls. The wider one makes an arch -- a wide arch is known as an 'elliptical arch' – the more it resembles a beam, and the higher the stresses are at the midpoint.

The larger arch is, in fact, ten times weaker - or ten times more prone to stress - than the smaller one. This is because the weight of the larger arch goes up by a factor of ten cubed, since it depends on the number of cubic meters of material in the bridge. Hence it weighs 1000 times as much as its smaller counterpart. Unfortunately, the strength of the arch depends on its cross-section, which only increases by a factor of ten squared. Hence the stress on each part of the bridge increases by a factor of 1000/100, or by a factor of ten. In other words, the larger bridge is ten times weaker, or ten times more

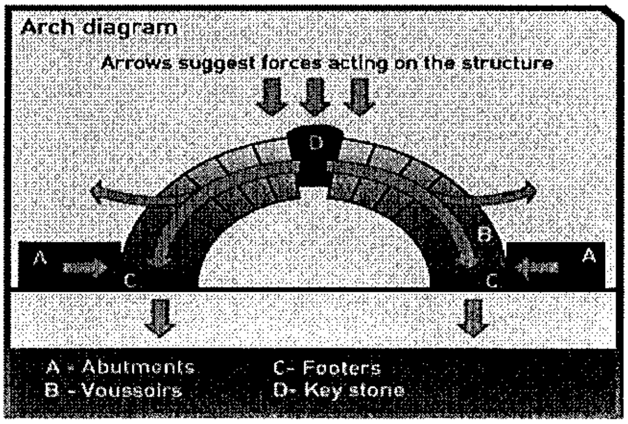


Fig1.1 Structural diagram of arch bridge

prone to stress, than the smaller one. Arch bridges can be made of both steel and concrete.

a.) Steel Arches

There are a few different methods used to construct steel arches. One design uses an arch structure with the traffic way passing through the arch. The traffic way rests on the part of the arch it passes through. Cables, suspended from the arch, pull the traffic way up so it does not sag.

That type of bridge is constructed through this process:

i. The anchors for the arch are constructed. The arch is started and is supported by temporary steel cables on either side securely anchored in rock. These cables will hold the arch until it is completed or very close to completion.

ii. Cranes are used to build the arch. They are placed on the existing fraction of the arch and lift the steel to its proper position, and then they erect it.

iii. When the arch is fully built, the temporary steel cables are disconnected and the cranes put the traffic way in place.

Another method used is one that is useful when the other type of steel arch bridge is not possible. It is built by building an arch (which is often supported by pillars until completed) and then building a traffic deck above the arch. The traffic deck is supported by pillars. It's almost like a beam-type bridge on top of a steel arch.

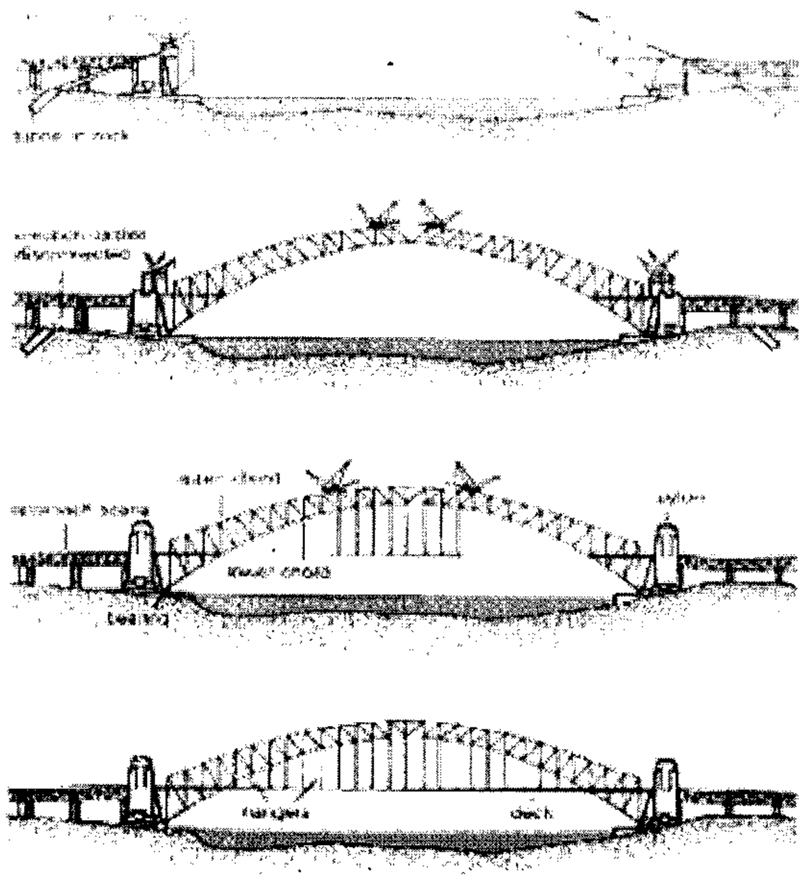


Fig1.2 Process of construction of steel arch bridge

B.) Suspension Bridge

Suspension bridges "bridge the unbridgeable". The deck (traffic way) of a suspension bridge is hung by cables which hang from towers. The cables transfer the weight to the towers, which transfer the weight to the ground. Suspension bridges become more economical to build than other types of bridges for spans greater than 1500 feet. Suspension bridges utilize cables that are attached to anchorages at either end, and draped over towers, to hold up the deck over which the traffic flows. One of the most difficult and dangerous aspects of building a suspension bridge is the laying of caissons or cofferdams, used in constructing the foundations. Cofferdams are built in place, whereas caissons are generally constructed elsewhere and moved into place. They are used to support the towers.

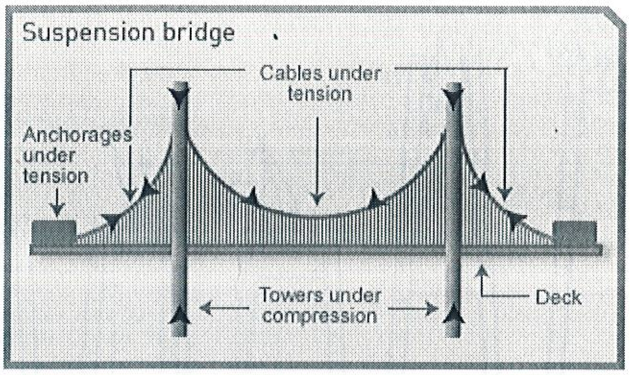


Fig 1.3 Structural diagram of a suspension bridge

In all modern suspension bridges, the roadway hangs from massive steel cables, which are draped over two towers and secured into solid concrete blocks, called 'anchorages', at both ends of the bridge. The vehicles push down on the roadway, as on any beam bridge, but because the roadway is suspended by 'hangers' from the two main cables, the cables transfer the entire onto the tops of the two towers. The two towers are constantly in compression, and transfer the forces to the ground on which they are built. The anchorages are usually attached to solid rock on either bank, and absorb the tension in the main cables.

There is a disadvantage to the suspension bridge design – namely its lack of the rigidity implicit in both beam and arch designs. This makes suspension bridges susceptible to the phenomenon of 'resonance', where vibrations build in magnitude due to some regular energy input.

a. Materials and Construction

The amount of towers on suspension bridges can vary, but a suspension bridge must have at least two towers. When possible, these towers are built on ground. However, there are methods for making "floating towers" that are secured. Towers are usually built with hollow steel boxes, but some are built with concrete.

b. How to make "floating towers"?

Floating towers are made by caissons. Caissons are hollow structures that are filled with concrete until they hit the ground. Gravity does the work on keeping the bridge up.

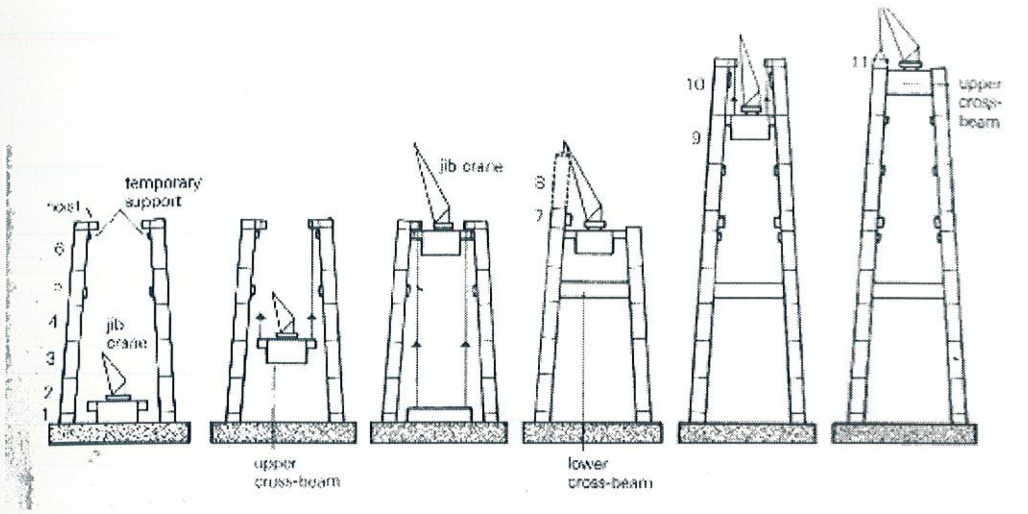


Fig 1.4 Floating tower

Here is one way that towers are built:

i. The first portion of the tower is built. Hoists and temporary supports are built also, so a crane (called a jib crane, which lies on the beam that will be at the top of the bridge) can be lifted up and be used to complete the tower.

ii. The crane is lifted up to the highest level on top of the upper cross-beam.

iii. The crane puts the lower cross-beam in place on its way up.

iv. The crane builds the tower up to its final height.

v. The upper cross-beam is finally laid in place.

vi. After the towers are completed, the cables must be put in place.

c. How to place cables?

A method called air-spinning is sometimes used. Here are the steps in the air-spinning method:

i. A continuous loop of rope is hung across the wheel, and two spinning wheels are attached to that rope at both ends.

ii. Wire is looped on to the spinning wheel from its reel, and the other end is anchored.

iii. When a reel of cable is exhausted, the end of the first wire is joined to a new reel of cabling.

iv. This process is repeated until the desired amount of strands has been spun. Then, the strands are packed together, covered with wrapping wire, and painted. Clamps are connected at certain intervals to carry the ropes that will suspend the deck.

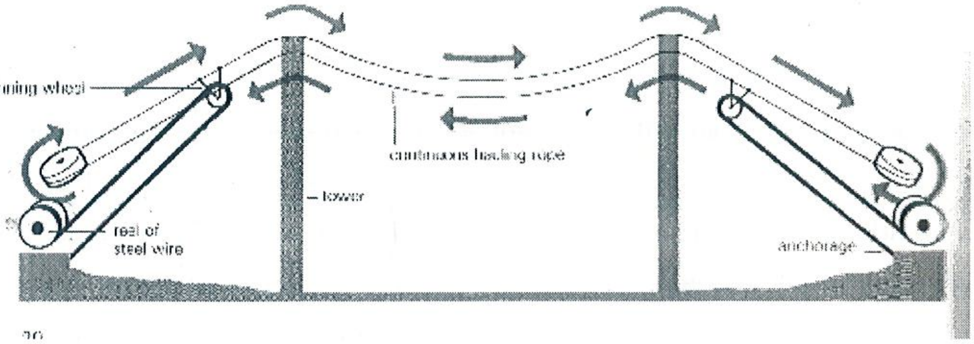


Fig1.5 Placing of cable in suspension bridge

C.) Cable stayed

Cable-stayed bridges have towers, but cables from the towers go directly to the road deck, instead of spanning from tower to tower. The cables may be attached in a radial pattern, extending from several points on the deck to the top of the tower, or in a parallel pattern, in which they are attached to different points on the towers, in parallel patterns. In the cable stay version of the suspension bridge, the deck is hung from diagonal cables that exert a force towards the towers as well as vertically. This makes the tension in the steel cables extremely high, and hence they are very stiff. In addition, the cables effectively stabilize the towers from both sides. Cable-stayed bridges can be constructed in a huge variety of designs. They are often used to span river mouths, sometimes themselves resembling giant sailing boats.

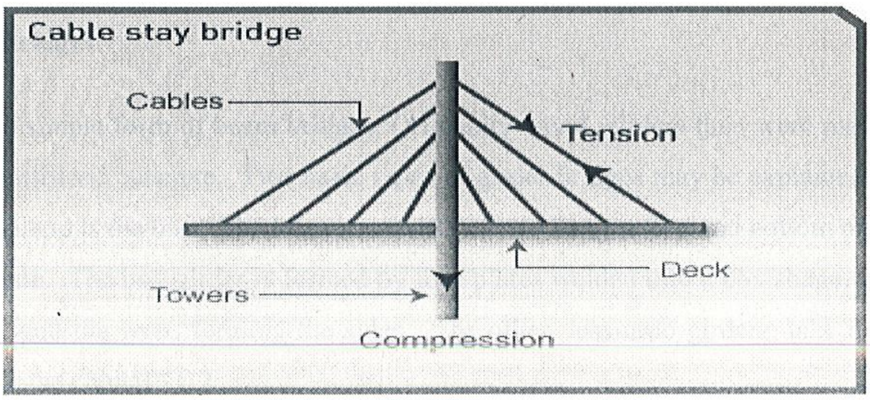


Fig1.6 Structural diagram of cable stayed bridge

There are two major classes of cable-stayed bridges: In a harp design, the cables are made nearly parallel by attaching cables to various points on the tower(s) so that the height of attachment of each cable on the tower is similar to the distance from the tower along the roadway to its lower attachment. In a fan design, the cables all connect to or pass over the top of the tower(s). The cable-stay design is the optimum bridge for a span length between that of cantilever bridges and suspension bridges.

Key advantages of the cable-stayed form are as follows:

i. much greater stiffness than the suspension bridge, so that deformations of the deck under live loads are reduced.
ii. can be constructed by cantilevering out from the tower - the cables act both as temporary and permanent supports to the bridge deck.
iii. for a symmetrical bridge (i.e. spans on either side of the tower are the same), the horizontal forces balance and large ground anchorages are not required.

A further advantage of the cable-stayed bridge is that any number of towers may be used. This bridge form can be as easily built with a single tower, as with a pair of towers. However, a suspension bridge is usually built only with a pair of towers.

D.) Girder bridges

The girder is the simple form of beam bridges. Originally wood, girders then were made from iron or steel, or even reinforced concrete. Two basic types of girder bridges may be explained. First is the I-beam, and the second is the box. The I-beam consists of two flanges (top and bottom plates) welded to the web (side plate. The box girder is formed by four plates welded into a box shape, with flanges on top and bottom, and the webs forming the sides. The other, less-used girders, take the shape of the Greek letter pi or the capital "T.".

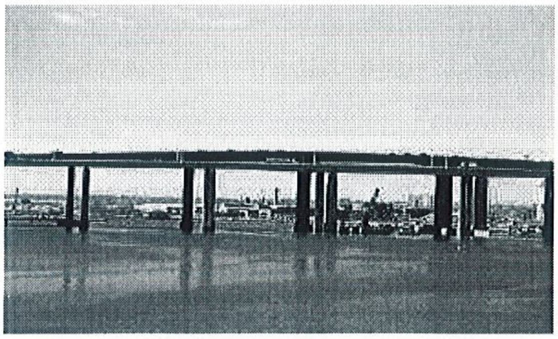


Fig 1.7 Diagram showing a girder bridge

E.) Beam Bridge

The beam type is the simplest type of bridge. The beam bridge could be anything as simple as a plank of wood to a complex structure. It is made of two or more supports which hold up a beam. Beam bridges are commonly built from concrete. Beam bridges are also made of steel, or a mixture of steel and concrete. They often are made in sections, or boxes, where they are attached at the site of the bridge. The boxes are made out of steel and concrete or just concrete. A single rigid 'beam', resting on supports at either end and unsupported in the middle. The weight of the beam, and of any traffic on it, is carried directly to the ground by the supports, often called 'piers' in the trade. The beam need not be of any particular shape and there are no other elements besides the piers to help dissipate the load. Hence the piers take the full weight of the load and are said to be in 'compression'. This means that they are being squashed by the forces at the top and bottom, and must be built from materials that can resist such forces without crumpling.

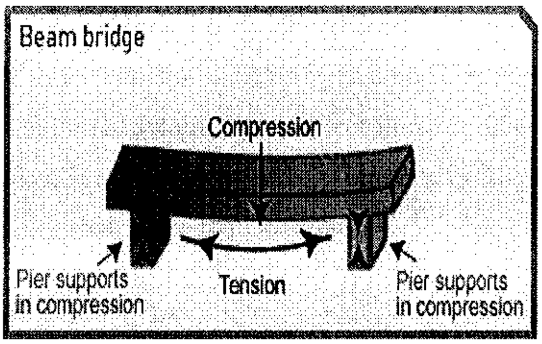


Fig1.8 Structural diagram of a beam bridge

The fact that the load tends to stress the beam itself - the top surface of the beam being shortened slightly and the bottom surface stretched - shows that the top is in 'compression' while the bottom is in 'tension'.

a. What are the disadvantages of a beam bridge?

All beams tend to 'sag' between the piers and 'hog' over the piers themselves. This results from the downward forces of the load and the upward forces at the pier supports. The greater the span or the load, the greater is the tendency towards sagging and hogging.

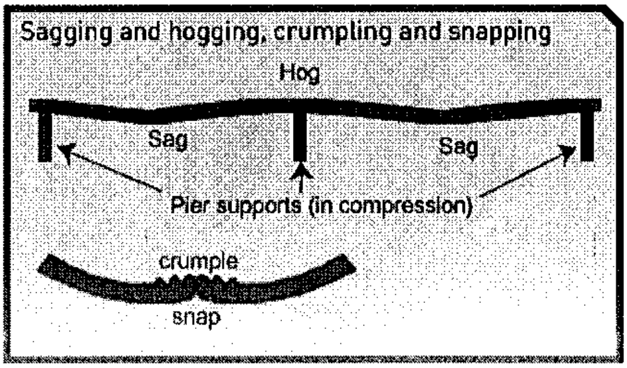


Fig1.9 Diagram showing defects can occur in beam bridge

The longer a beam is, the weaker it becomes. The greater length gives more weight and more leverage for that weight - increasing the 'bending moment'. This is why the bridge tends to sag more in the centre. It is rather as if you were holding a piece of A4 paper out towards someone else. The paper is much more likely to bend if you place a weight on it or hold it by its corner rather than at its centre. Both the weight and the unsupported length of an object make it more likely to sag. As the width of the span is increased, any beam bridge will eventually crumple on the top and snap on the underside. This is why beam bridges rarely span more than 80 meters.

F.) Cantilever Bridge

To solve the problem of increasing the span distance, other alternatives to beam and arch bridges included suspension and cantilever bridges. Cantilever bridges are a modified form of beam bridges, with the support being placed not at the end, but somewhere in the middle of the span. A cantilever is a structure or beam that is unsupported at one end but supported at the other, like diving boards. This configuration made longer spans possible and wider clearance beneath. In the cantilever type of bridge, two beams support another beam, which is where the deck or traffic way is. The two beams must be anchored, and this must be done well. Cantilever bridges depend on counterbalances. But what are counterbalances? Counterbalances are weights used to balance another weight. They consist of two or more (which many cantilever bridges have at least four) arms that equally balance each other, almost like a perfectly balanced see-saw. Often, the part of the bridge that leads to the first cantilever is just a beam bridge.

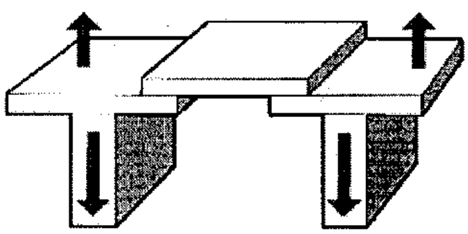
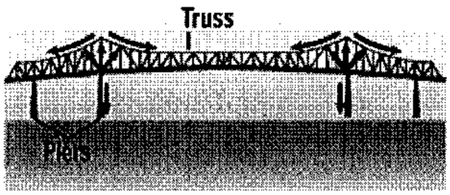


Fig1.10 Cantilever bridge

The advantage to a cantilever bridge is its ability to span wide spaces without the need of extensive and expansive support while under construction.

G.) Truss Bridge

A truss bridge is a bridge composed of connected elements (typically straight) which may be stressed from tension, compression, or sometimes both in response to dynamic loads. Truss bridges are one of the oldest types of modern bridges. A truss bridge is economical to construct owing to its efficient use of materials. It consists of an assembly of triangles. Truss bridges are commonly made from a series of straight, steel bars. Rigid arms extend from both sides of two piers. Diagonal steel tubes, projecting from the top and bottom of each pier, hold the arms in place. The arms that project toward the middle are only supported on one side, like really strong diving boards. These "diving boards," called cantilever arms, support a third, central span.



Figl.11 A truss bridge

1.3.2 Classification based on the materials

A.) Concrete bridges

Concrete offers many advantages compared to other materials in construction. From durability and versatility to beauty and economy, concrete is the material of choice for superior bridges. It has great strength in compression, and since its basic components are water and aggregates, it is extremely inexpensive. The use of concrete dates back to Roman times, but the modern practice of using Reinforced Concrete in construction was introduced in the last century. It is the most used construction material for bridges in the world. Using steel rebar embedded within a concrete beam, column, or slab utilizes the tensile strength of the steel in conjunction with the compressive strength of the concrete to make a stronger, safer structure.

Pre stressed concrete is used for longer spans. It can increase the span length of a bridge over 35% compared to reinforced concrete. It resists loads that cannot be resisted by normal strength concrete and improves the lateral stability of the bridge.

The properties of the concrete provide a sufficient solution to most of the designs combining architectural effects with structural capacity

1.) Economical: Reduced initial construction costs, lower maintenance and inspection costs.

2.) Durability: Concrete bridges have a service life of 100 years or more. They easily withstand extreme temperature changes and corrosive chemicals in a variety of conditions.

3.) Competitiveness: The value of concrete is repeatedly recognized in competitive bidding situations.

4.) Aesthetics: Dynamic, graceful, long-span bridges often become symbols of a city, a tourist attraction, which encourages economic development.

There are following types of concrete bridges:

1.) Arch bridges.

2.) Beam and slab bridges.

3.) Box girder bridges.

4.) Cantilevered.

B.) Steel bridges

Steel bridges are constructed both over moderate to high span as well as for heavy trafficular loads. In India, steel bridges are commonly used for railways for all types of spans because of following advantages:

1) Low weight of components.

2) Easy fabrication and simple installation.

3) Durability.

Classification of steel bridges:

1) According to type of structural arrangement:

   i. I - girder bridges
   ii. Plate girder bridges
   iii. Truss girder bridges
   iv. Suspension bridges

2) According to structural action:

   i. Simply supported span bridges
   ii. Cantilever bridges
   iii. Arch bridges

3) According to floor action:

   i. Deck type bridges
   ii. Through type bridges
   iii. Half through type bridges

4) According to type of connection:

   i. Riveted bridges
   ii. Welded bridges
   iii. Bolted (or pin connected) bridges

C.) Timber bridges

Timber bridges can help protect water quality and stream habitat during forestry operations. There are three basic types of temporary timber bridges. Log stringer bridges are built from trees felled in the area. Solid- sawn stringer bridges are made from new lumber, railroad ties, or large timbers removed from buildings that are being torn down. Panel bridges are constructed from stress-laminated, glued-laminated, dowel-laminated, or nail-laminated lumber. Timber bridges can be placed over small streams or channels with firm, stable banks.

a. **Advantages**

Operators can find materials for timber bridges at the site or purchase them locally or through commercial outlets. Little site preparation is needed. Structural characteristics are known and engineering specifications may be available for lumber or panels. Operators can remove and reuse timber bridges several times. Local water regulators generally favor timber bridges.

b. **Disadvantages**

Timber bridges may pose a safety hazard if operators don't get help from a licensed engineer or accurately assess the strength of logs and other construction materials. Logs, railroad ties, and demolition materials can have rot, knots, and other problems that affect strength. If operators don't use proper abutments, timber bridges may freeze into the ground during the winter. Surfaces may wear quickly during skidding operations.

1.3.3 Classification based on movement of structural parts of bridge:

A.) Fixed Bridges

A fixed type bridge is the one which always remains in one position. A majority of bridges are of this type. However, when bridge is constructed across a navigable channel, clearance under the bridge girder may not be sufficient for the passage of masted vessels or steamers. In such a circumstance, a movable bridge is the only answer.

B.) **Movable bridges**

Movable bridges enable the bridge to be moved to allow other traffic to pass. This is accomplished by vertically lifting the deck (a vertical lift bridge), by rotating the deck (a swing bridge), or by raising the deck at an angle through the use of weights (a bascule bridge). The original movable bridges were the drawbridges of medieval times, raised by ropes and pulleys. It is the one which can be opened either horizontally or vertically so as to allow the river or channel traffic to pass. So these were the different types of bridges. Our main concern in this project is with plate girder bridges.

1.4 Plate girder bridges.

A plate girder bridge is a bridge supported by two or more plate girders. The plate girders are typically I-beams made up from separate structural steel plates (rather than rolled as a single cross-section), which are welded (or occasionally bolted or riveted) together to form the vertical web and horizontal flanges of the beam. In some cases, the plate girders may be formed in a Z-shape rather than I-shape. Plate girder bridges are suitable for short to medium spans and may support railroads, highways or other traffic.

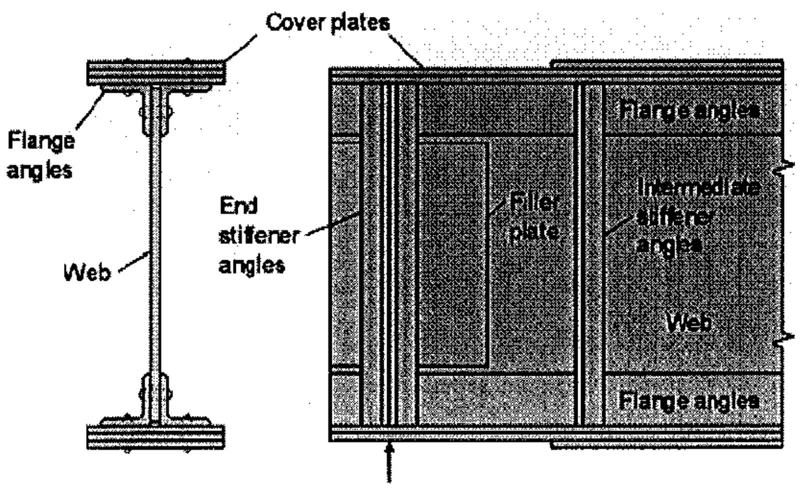


Fig 1.12 Typical component of riveted plate girder

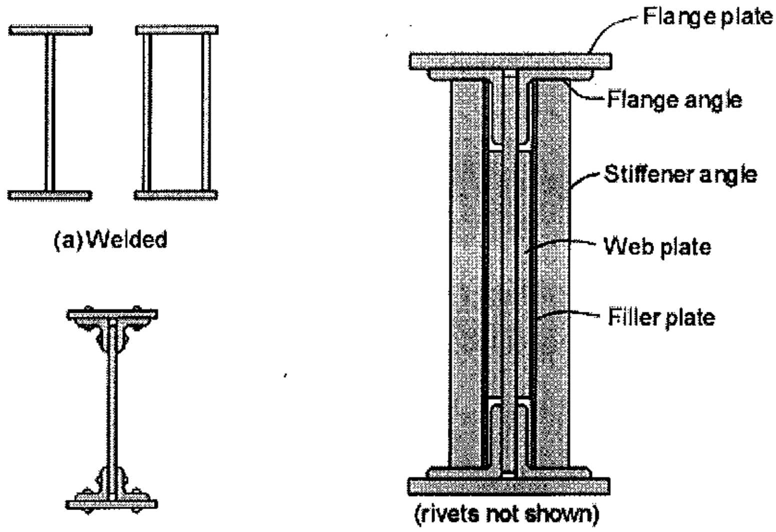


Fig 1.13 Plate girder cross sections

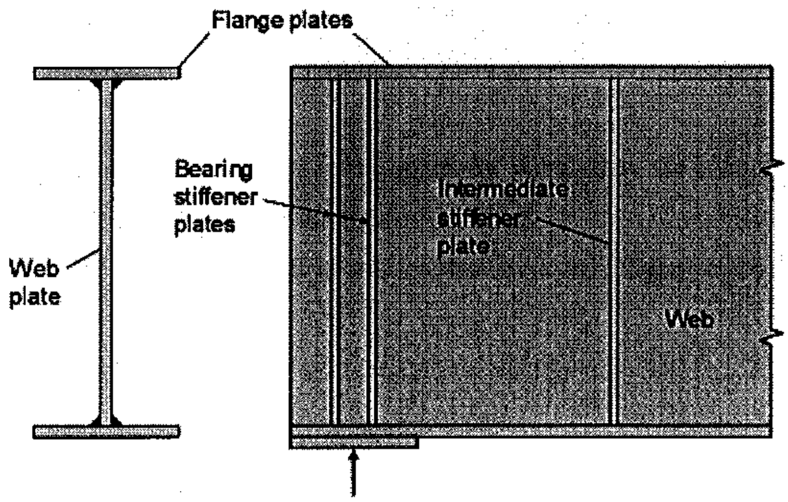


Fig 1.14 Typical component of welded plate girder

Two types of plate girder bridges are quite common:

1.4.1 Deck type plate girder bridges

In the deck-type bridge, a steel or reinforced concrete bridge deck is supported on top of two or more plate girders, and may act compositely with them. Additional beams may span across between the main girders, for example in the form of bridge known as ladder-deck construction. Also, further elements may be attached to provide cross-bracing and prevent the girders from buckling.

1.4.2 Half through type plate girder bridges

In the half-through bridge, the bridge deck is supported between two plate girders, often on top of the bottom flange. The overall bridge then has a 'U'-shape in cross-section. As cross-bracing cannot normally be added, vertical stiffeners on the girders are normally used to prevent buckling. This form of bridge is most often used on railroads as the construction depth (distance between the underside of the vehicle, and the underside of the bridge) is much less. This allows obstacles to be cleared with less change in height.

1.5 Plate Girder Vs Trusses.

The usual practical alternatives to plate girders in the spans for which they are economical are trusses. However, plate girders have the following advantages over trusses.

(a) The cost of fabrication is lower compared to trusses; however it is higher compared to roll steel sections.

(b) Erection is faster and cheaper than trusses.

(c) Plate girders require smaller vertical clearances than trusses.

(d) Due to the compactness' of the plate girders, vibration and impact are not serious problems.

(e) Plate girders are safer than trusses. For example, if a truck runs into a bridge composed of plate girders, it would probably bend the steel plates a little; however a similar accident could cause the breaking of a member in a truss, which may even lead to the failure of the truss.

(f) Points where stresses may be critical are fewer in plate girders compared to trusses.

(g) A plate girder is more easily painted than a truss.

However, plate girders have the following disadvantages.

(a) They are usually heavier than trusses for the same span and loads.

(b) Larger number of connections are required between webs and flanges.

(c) Larger exposed wind area compared to a truss.

(d) Low torsional stiffness (box girders provide better torsional stiffness).

(e) Susceptibility to stability problems of the compression flange during erection.

1.6 Differences between Beams and Plate Girders

Beams are rolled in mills to standard sizes, whereas plate girders are assembled by welding of plates. In its simplest form, the plate girder is a built-up beam consisting of two flange plates welded to a web plate to form an I-section. Web stiffeners are not used commonly in beams and they are used widely in plate girders. Because they are more economical than their welded equivalents, rolled beams are used whenever possible. The principal difference between the design of a rolled beam and the design of a plate girder is that the structural designer has considerable freedom in proportioning a plate girder. Therefore, the designer might face several structural problems, which may not arise when rolled sections are used. The most important of these are the local buckling of the compression flange and the shear buckling of the web. In a plate girder, the primary function of the flange plates is to resist bending moments by developing axial compressive and tensile stresses. The web plate resists the shear. For making the cross section efficient in resisting the in-plane bending, it is required that maximum material is placed as far away from the neutral axis as possible. From this point of view it is economical to keep the flanges as far apart as possible. The axial forces in the flanges decrease, as the depth of the girder increases. Thus a smaller area of cross section would suffice than would be the Case if a smaller depth were chosen. However, this would also mean that the web would be thin and deep. In such a situation, premature failure of the girder due to web buckling in shear might occur. Here there is a choice between thin web provided with vertical and horizontal stiffeners, and a thicker web requiring no stiffening (and therefore avoiding costly fabrication). The choice between the two depends upon a careful examination of the full costs of both forms of construction.

CHAPTER 2

DESIGN OF BRIDGE AS PER IS 800: 1984

2.1 Design Problem

2.1.1 Computation of Loads, B.M and S.F. in plate girder.

a) Computation of loads:

Weight of main rails per meter run of track	$= 0.44 \times 2 = 0.88$ KN/m
Weight of guard rails per meter run of track	$= 0.26 \times 2 = 0.5$ KN/m
Weight of fastenings per meter run of track @ 0.28 kN/m	$= 0.28$ KN/m
Weight of sleepers per meter run of track	$= (2.8 \times 0.25 \times 0.15)/0.4$
	$= 1.94$ KN/m
Total dead load	$= 3.62$ KN/m
Self weight of girders, for B.G main line loading is w	$= 0.78L = 18.72$ KN/m
Total dead load per meter run of track	$= 3.62 + 18.72 = 22.34$ KN/m
Total dead load	$= 22.34 \times 24 = 536.2$ KN/m
Dead load per girder	$= 536.2/2 = 268.1$ KN

b) Computation of B.M:

EUDL for B.M, for BG for loaded length L	= 24 m is 2034 KN on each track.
CDA	$= 0.15 + 8/(6 + L) = 0.417$
Impact load for B.M	$= 2034 \times 0.417 = 848.2$ KN
Hench total per track	$= 536.2 + 2034 + 848.2$
	$= 3418.4$ KN
Hence total load per girder, for B.M	$= 3418.4/2 = 1709.2$ KN
$M = WL/8 = 5127.6$ kN/m	
B.M at distance x from mid span is	$= 5127.6 - 35.61\ x^2$

c) **Computation of S.F:**

The CDA will vary with loaded length L which depends upon the position of the section under consideration.

For end sections, L	$= 24$ m
Hence EUDL for shear	$= 2231$ KN (Table 28.1 A)

$$CDA = 0.15 + \frac{8}{6 + 24} = 0.417$$

Impact load, I.L	$= 2231 \times 0.417 = 930.3$ KN
Total (L.L. + I.L.)	$= 2231 + 930.3 = 3161.3$ KN
Hence, (L. L + I. L)	$= 1/2 \times 3161.3 = 1580.6$ KN
S: F due to (L.L. + I.L.)	$= 1/2 \times 1580.6 = 790.3$ KN
Dead load per girder, W_d	$= 268.1$ kN
Dead load shear per girder	$= 1/2 \times 268.1 = 134.1$ KN
Total S.F. at each end of a girder	$= 790.3 + 134.1 = 924.4$ KN

For any section x from one end, $L = I\text{-}x$

Let W_e be the total L.L + I.L. on length L for each girder.

$$F_x = R_A = \frac{W_e\,(I - x)}{2I}$$

For a section at 2 m from end, L	$= 24 - 2 = 22$ m
EUDL for shear	$= 2102$ KN (Table 28.1 A)
CDA = 0.436	
(L.L. + I.L)	$= 2102(1 + 0.436)$
	$= 3018.4$ KN (for both girders.)
(L.L. + I.L) for each girder = W_e	$= 1/2 \times 3018.4 = 1509.2$ KN

$$F = \frac{1509.2\,(24 - 2)}{2 \times 24} = 691.7 \text{ KN}$$

Also, S.F due to dead load $= \frac{W_d}{2} - \frac{W_d}{l}x = \frac{268.1}{2} - \frac{268.1}{24}x = 134.05 - 11.17x$

At $x = 2$ m, $F = 134.05 - 11.17 \times 2 = 111.7$ KN

Total shear = 691.7 + 111.7 = 803.4 KN

The computations for other sections are shown in following Table

Table 2.1 showing computation of S.F for other sections

Distance from end,(m)	L (m)	L.L (KN)	CDA	Total(L.L+I.L)=We per girder, (KN)	S.F per girder (KN)		
					Due to (L.L+I.L)	Due to D.L	Total
0	24	2231	0.417	1580.6	790.3	134.1	924.4
2	22	2102	0.436	1509.2	691.7	111.7	803.4
4	20	1953	0.458	1423.7	593.2	89.4	682.6
6	18	1777	0.483	1317.6	494.1	67.0	561.1
8	16	1619	0.514	1225.6	408.5	44.7	453.2
10	14	1482	0.550	1148.6	335.0	22.4	357.4
12	12	1359	0.594	1083.1	270.8	0	270.8

2.1.2 Design of central section of plate

$M_{max} = 5127.6$ KN-m.

From Table 28.13, $\sigma_c = 138$ N/mm^2 for $d_1/t > 85$ mild steel.

Let us keep thickness of web, $t_w = 10$ mm.

Economical depth d $= 1.1 \sqrt{\left(\frac{M}{t_w \cdot \sigma_b}\right)} = 2120$ mm

Minimum depth, d > L/12 > 24000/12 > 2000 mm. OK

Assuming depth of flange angles as 150 mm, $d_1 = 2120 - 2 \times 150 = 1820$ mm

d_l/t_w = 1820/10 = 182 > 175, which is not permissible.

Hence either we have to decrease d or else increase t_w. Choose t_w = 12 mm

$d = 1.1\sqrt{\dfrac{5127.6 \times 106}{12 \times 138}}$ = 1936 mm

Hence adopt d = 2000 mm and t_w = 12 mm.

Areas of web, $A_w = t_w \cdot d$ = 12 × 2000 = 24000 mm²

Net area of flange $A_{ft}' = \dfrac{M}{\sigma_{bt} \cdot d} = \dfrac{5127.6 \times 106}{138 \times 2000}$ = 18578 mm²

Net area of flange plates, $A_{p'} = 2/3A_{ft}' - 1/8A_w$ = 9386 mm²

Keep gross area of plates = 1.2 × 9386 = 11263 mm²

Provide two plates 500mm x 12mm each having A_p = 2 × 500 × 12 = 12000 mm²

Net area of flange angles, $A_{a'} = A_{ft}'/3$ = 18578/3 = 6193 mm²

Keep gross area of flange angles, A_a = 1.25 × 6193 = 7740 mm²

:. Gross area of each angle = 7740/2 == 3870 mm²

Choose 2 ISA 200 × 150 × 12 mm @ 31.8 kg/m, each having A = 4056 mm²,

I_{xx} = 793.2 × 10^4 mm⁴ and C_x = 36.0 mm

d_t = 2000 - 2 x 1500 = 1700 mm

:. d_t/t_w = 1700/12 = 141.7 < 175. Hence OK. Also, since d_l/t_w > 85, permissible bending stress

σ_{bt} = 138 N/mm², from Table 28.13 applicable for railway bridges.

Rivet gauge distance C for 200 mm leg of angle = 115 mm

from Table XXXI of ISI Hand Book Vol. I

:. Out stand of flange plate = 1/2 (500 - 2 × 115 - 12) = 129 mm

This is less than 16 *t* = (16 x 12 = 192 mm)

Hence the section is OK.

Let us use 20 mm dia rivets.

Table 2.2 Table showing total net area

Component	Gross Area (mm2)	Rivet hole deduction (mm2)	Net Area (mm2)
ISA 200×150×12	2×4056 = 8112	4×21.5×12 = 1032	7080
Plates 500 × 12	2×500×12 = 12000	4×21.5×12 = 1032	10968
Web equivalent	1/6 × 24000 = 4000	-----------	1/8 × 24000 = 3000
Total	Aft = 24112	-----------	Aft' = 21048

$I_{xx} = 1/12 \times 12 (2000)^3 + 4 [793.2 \times 104 + 4056 (1000 - 36)^2] + 2 \times 500 \times 24 (1000 + 12)^2 = 4.7688 \times 1010 \text{ mm}^4$

$\sigma_{bc,cal} = \frac{M}{I} \cdot Y_{max}$ = 110.1 N/mm < 138. Hence safe

It is to be noted that since the sleepers rest directly on the top flange, which is compression flange, the compression flange may be considered to be laterally supported.

Hence σ_{bc} = 138 N/mm² for $d_1 / t > 87$ for mild steel.

$\sigma_{bt,cal} = \sigma_{bc,cal} \cdot \frac{Aft}{Aft'}$ = 126.1 N/mm <138. Hence safe.

Shear stress,

$\tau_{va} = F_{max} / d.t_w$ = 385 N/mm² < 100 Hence safe.

2.1.3 Curtailment of flange plates.

$M = I_{xx}\, \sigma_b / (y_{max}) \times \frac{Aft'}{Aft.}$ = 5610 × 106 N-mm = 5610 KN-m

Let us curtail one plate from each of the flanges.

$I_{xx1} = 0.8 \times 1010 + 1.51 \times 1010 + 2 \times 500 \times 12 (1000 + 6)^2$ = 3.5244 × 1010 mm⁴

A_{ft1} = 24112 - 500 ×12 = 1812 mm²

A_{ft1}' = 21048 – 1/2 × 10968 = 15564 mm²

M_{R1} = 4130 KN-m

M_{x1} becomes equal to M_{R1} at distance x_1 given by

$5127.6 - 35.61\ x_1^2 = 4130$

This gives $x_1 = 5.29$ m from mid-span or at 12 - 5.29 = 6.71 m from ends. Similarly, with both plates curtailed from each of the flanges

I_{xx0} $= 0.8 \times 1010 + 1.51 \times 1010 = 2.31 \times 1010\ mm^4$

A_{flo} $= 24112 - 12000 = 12112\ mm^2$

$A_{flo'}$ $= 21048 - 10968 = 10080\ mm^2$

M_{Ro} = 2653 KN-m

This occurs at a section distant *Xo* from mid-span, given by

$5127.6 - 35.61xo^2 = 2653$ which gives $Xo = 8.34$ m

Compressive stress in the outer plate, at the point of cut off = $(M_{x1}/ I_{xx})\ y_{mean}$

$$\sigma_{bc} = \frac{4130}{4.7688}\ (1024-6) = 88.16\ N/mm^2$$

$$\sigma_{bt} = \sigma_{bc} \times \frac{Aft}{Aft'} = 101\ N/mm^2$$

:. Tensile force in the curtailed plate = 101 (10968/2) × 10 - 3 = 554 KN

Rivet value of 20 mm dia rivets in single shear = $\pi/4\ (21.5)^2 \times 100 \times 10^{-3} = 36.3$ KN

No of rivets required = 554/36.3=16.

These rivets are to be provided in two rows. Hence provide 8 rivets in each row. Keeping pitch equal to 3 times the rivet diameter and edge distance equal to 1.5 times the dia., the distance of actual point of cutoff = $5.29 + 8 \times 3 \times 21.5 \times 10^{-3} = 5.29 + 0.52 = 5.81$ m from mid span.

2.1.4 Design of riveted joint connecting flange angles with the web

Maximum axle load for main line of B.G. is 224.6 kN, and this axle load is assumed to be distributed on a length of 1.2 m. Impact factor may be taken as 1.0,· corresponding to a span of zero.

$$\text{(L.L. + I.L) per girder, per mm run} = \frac{1/2 \times 224.6}{1200} \cdot (1 + 1) = 0.187\ KN/mm$$

Dead load of track, per girder = $1/2 \times 3.62$ KN/m = 1.81×10^{-3} KN/mm Total load w on compression flange = 0.187 + 0.00181 0.189 KN/mm a.) For compression flange: Pitch is given by

$$p = R/\sqrt{\left[\frac{V.\ \ A_f}{d_e(A_f + A_w/6)}\right]^2 + w^2}$$

At supports, V = 924.4 kN.

Since one flange plate is available, A_f = 8112 + 12000/2= 14112 mm².

Take d_e=d = 2000 mm. Strength of 20 mm dia.

Rivets in double shear = 2 (π/4) $(21.5)^2 \times 100 \times 10^{-3}$ = 72.61 KN

Strength in bearing on 12 mm = 21.5 × 12 × 232 × 10^{-3} = 59.86 KN

:. Rivet value R = 59.86 KN.

p = 147.2 mm

Max permissible' pitch = 12 × *t* = 12 × 12 = 144 mm. Hence provide rivets at a pitch of 140 mm c/c. at the supports. At other locations, S.P. *V* goes on decreasing and hence pitch increases. Since the maximum permissible pitch is 144 mm, it is advisable to provide a uniform pitch of 140 mm throughout the length.

(b) For tension flange : Pitch is given by

$$P = \frac{Rd_e}{V}\ \frac{A_f' + 1/8A_w}{A_f'}$$

At supports, flange plates are not available. Hence A_f' = 7080

p = 184.4 mm

Max. permissible pitch= 16 × t = 16 × 12 = 192 mm. Hence provide uniform pitch of 180 mm throughout the length.

2.1.5 Design of riveted joint connecting flange plates to flange angles

(a) Connection of flange plate to flange angle in compression flange, at ends A_t ends, V = 924.4 kN and one plate is available in top flange.

Hence A_f= 8112 + 12000/2= 14112 mm². The rivets will be in single shear. Hence using 20 mm dia rivets, strength of rivets in single shear= (π/4) $(21.5)^2 \times 100 \times 10^{-3}$ = 36.3 KN

while that in bearing = 21.5 × 12 × 232 × 10^{-3} = 59.86KN

Thus, R =36.3 kN

$$P = Rde \frac{Af + 1/6\ Aw}{V.\ Ap} = 237 \text{ mm}$$

Since rivets are provided in two rows, actual pitch will be = 2 x 237 = 474 mm. However, max permissible pitch = 12 t = 12 × 12 = 144 mm. Hence provide these at a pitch of 140 mm. Since at other locations, V will be less, p will be more. Hence provide a uniform pitch of 140 mm throughout the length of the compression flange.

There is no flange plate in the tension flange of the end section

(b) Connection of flange to flange angle in tension flange at 3.66 m from end.

Here, only one flange plate is available. Hence Ar = 7080 + 10968/2= 12564 mm²

while $1/8A_w$ = 3000 mm². Also, V= 703 KN

p = 293 mm

Since rivets are provided in two rows, actual pitch = 2 × 293 = 586 mm.

Max permissible pitch in tension= 16 t = 16 × 12 = 192 mm. Hence keep uniform pitch of 190 mm throughout in the tension flange to connect the flange plate(s) to flange angles.

2.1.6 Design of end stiffener

End reaction = 924.4 KN;

Permissible bearing stress = 185 N/mm².

:. Bearing area required $= \frac{3/4 \times 924.4 \times 10^3}{185}$ = 3748 mm²

Let us provide four angles, each having a thickness ta. Let us try 150 ×75 mm size angles having a root radius of 10 mm.

:. 4 t_a (150 - 10) = 3748 which gives ta = 6.7 mm

However provide 150 × 75 × 8 mm angles having $I_x = 407.2 \times 10^4$ mm^4

A = 1742 mm²

And C_{xx} = 52.3 mm. Use filler plates of 12 mm thickness.

Keep width of stiffener equal to 300 mm

$I_{xx} = 4\,[407.2 \times 104 + 1742\,(52.3 + 12 + 6)^2] + 1/12 \times 300\,(12)^3 = 5077 \times 10^4$ mm^4

$A = 4 \times 1742 + 300 \times 12 = 10568$ mm^2

$r = \sqrt{\frac{I}{A}}$ = 69.3 mm.

Actual length of stiffener = $2000 - 2 \times 12 = 1976$ mm

Effective length of stiffener = $3/4 \times 1976 = 1482$ mm

$l/r = 21.4$

Hence permissible p_{ac} = 132.65 N/mm^2 (from Table 28.15)

:. Load carrying capacity = $132.65 \times 10568 \times 10^{-3} = 1402$ KN > 924.4

Design of riveted connection for the end stiffener

Strength of 20 mm dia. rivets in double shear = $2 \times \pi/4(21.51 \times 100 \times 10^{-3}) = 72.6$ KN Strength of rivets in bearing on web= $21.5 \times 12 \times 233 \times 10^{-3} = 59.86$ KN

:. Rivet value R = 59.86 KN

No. of rivets required = 924.4/59.86 = 16

Hence provide 8 rivets, in each row, thus making a total of 16 rivets.

2.1.7 Design of intermediate stiffeners

Since d_1/t_w is greater than 75, intermediate stiffeners will be required. Clear depth (d_1) of web= 1700 mm and thickness of web= 12 mm

At the ends, V =924.4 KN

.. Shear stress, $f_s = \frac{924.4 \times 10^3}{2000 \times 12}$ = 38.5 N/mm = 3.93 kg/mm²

Hence for d = 1700 mm, t_w = 12 mm and fs = 3.93 kg/mm^2, we get max spacing of stiffeners near supports = 500 mm

For overall depth of girder equal to 2032 mm. $I = 63 \times 10^5$ mm^4 while for overall depth of 2438 mm, $I = 150 \times 10^5$ mm^4. Hence for the present case, for overall depth of 2048 mm, required minimum $I = 670 \times 10^4$ mm^4

Try 2 ISA 125 × 75 × 8 mm, each having $I_{xx} = 245.5 \times 104$ mm^4, A = 1538 mm^2 and C_{xx} = 41.5 mm.

Length of outstand of angle from face of web = 125 mm which is less than

16 t (= 16 ×8 = 128 mm) and hence satisfactory.

I_{xx} $= 2\ [245.5 \times 104 + 1538\ (41.5 + 6)^2]$

$= 1085 \times 10^4\ mm^4 > 670 \times 10^4$

At 4 m from ends, V = 682.6 kN

:. fs $= \frac{682.6 \times 103}{2000 \times 12} = 28.4\ N/mm^2 = 2.9\ kg/mm^2$

Hence, we get max. spacing = 1900 mm.

However, maximum permissible spacing = 1830 mm as per Railway Bridge code.

2.1.8 Design of top lateral bracing

Consider the bridge to be loaded. Following lateral loads will act

(i) Wind load on both girders @ 1.5 kN/m^2 = 1.5 (1 + 0.25) 2.5 = 4.688 kN/m

(ii) Wind load on train @ 1.5 kN/m^2 = 1.5 × 3.5 = 5.25 kN/m

(iii) Racking force @ 600 kg/m = $600 \times 9.81 \times 10^{-3}$ = 5.886 kN/m

Total horizontal force = 15.824 kN/m

This force is resisted by top lateral bracing. Let us divide the span into 12 panels, with length of each panel 24/12 = 2 m, thus forming approximate squares. We will provide double diagonal system. For the analysis, it is assumed that the diagonal members carrying tension remain active, while other diagonals (shown dotted) remain dummy for one direction of wind. With the change of direction of wind, the dummy diagonals become active while the active diagonals become dummy.

Lateral load on each intermediate panel point = 15.824 × 2 = 31.648 KN. Lateral load on end panel point = 15.824 KN

End reaction = 15.824 × 24/2 = 189.888 KN

Tan θ = 1.9/2.0 = 0.9048; θ = 42.1376°; sinθ = 0.6709; cosec θ = 1.4905

Compressive force in end strut = 189.888

Shear in end panel = 189.888 - 15.824 = 174.064

Tensile force in end diagonal = 174.064 cosecθ= 259.4 KN

Design of end strut

P = 189.888 KN

Let us provide two angles connected to the same side of gusset plate. Hence effective length A = 80.

Hence from Table 28.15, p_{ac} = 99 N/mm^2.

Required area = 189.888 × 103/99 = 1918 mm^2

Provide 2 ISA 80 × 80 × 8 each having a = 1221 mm^2 and r_{xx} = 24.4 mm

:. Actual λ = 1900/24.4= 77.9, corresponding to which σ_{ac} = 100.8 N/mm^2

Load capacity = 100.8 × 2242 × 10^{-3} = 246.1 kN. Hence OK.

The rivets will be in single shear

:. No. of 20 mm dia rivets = 189.888/36.3 = 6

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

Design of diagonal member

P = 259.4 KN (tensile)

Permissible tensile stress = 138 N/mm^2 (Table 28.13)

$$\text{Required } A_{nel} = \frac{259.4 \times 10^3}{138} = 1880 \text{ mm}^2$$

Try 2 ISA 80 × 80 × 8, each having a = 1221 mm^2• Using 20 mm dia. rivets, deduction for rivet holes = 21.5 x 8 = 172 mm^2 for each angle.

Total net area provided = 2 [1221 - 172] = 2098 mm^2• Hence OK.

No of 20 mm dia rivets = 259.4/36.3 = 7.14. Hence 4 rivets on each angle. Similarly the struts and diagonals of intermediate panels can be designed.

2.1.9 Design of bottom lateral bracing

Bottom lateral system is similar to the top lateral bracing, except that the forces in the members will be 25% of the forces in corresponding members of top bracing.

Hence force in end strut = 1/4 × 189.888 = 47.472 KN

Try 2 IS'A 50 × 50 × 6 mm, each having a= 568 mm^2 and, = 15.1 mm

λ = 1900/15.1 = 125.8 corresponding to which p_{ac} = 58 N/mm^2

Load capacity = 2 × 568 × 58 × 10^{-3} = 65.9 kN.

Hence OK.

Similarly tensile force in diagonal member = 1/4 x 259.4 = 64.85 KN

:. Required A_{net} = $64.85 \times 10^3/138 = 470$ mm^2

Try 2 ISA 50 × 50 × 6, each having a = 568 mm^2

Using 22 mm dia rivets, $A_{net} = 2\,[568 - 21.5 \times 6] = 878$ mm^2• Hence OK.

Use one rivet on each angle, for connecting this member

2.1.10 Design of end cross-frames

Cross-frames are provided in the vertical planes, between the windward and leeward girders, at all panel points of horizontal truss bracing. An end cross frame, consisting of two diagonal members and top and bottom members. Only that diagonal is considered effective which carries tension. The cross-frame is subjected to a load P equal to the end reaction of the horizontal truss bracing.

P = 189.888 KN

Length of diagonal member = $\sqrt{(1.864)^2 + (1.976)^2} = 2.716$ mm

Cos θ = 1.864/2.716 = 0.6861

Tensile force in diagonal member = $\dfrac{189.888}{0.6861} = 276.8$ KN

Allowable stress in axial tension = 138 N/mm^2 (Table 28.13)

:. Required A_{net} = $276.8 \times 10^3/138 = 2005$ mm^2

Try 2 ISA 80 × 80 × 8, each having a = 1221 mm^2• Using 20 mm dia rivets,

Deduction for rivet holes = 21.5 × 8 = 172 mm^2 for each angle.

Total net area provided = 2 [1221 - 172] = 2098 mm^2• Hence OK

No. of 20 mm dia rivets = 276.8/36.3 = 7.63. Hence provide 2 rivets for each angle. The top horizontal member is the end strut of top lateral bracing, while the bottom horizontal member is the end strut of bottom lateral bracing.

2.1.11 Design of intermediate cross-frames

Use 2 ISA 50 × 60 × 6 mm for diagonals and connect the ends by 1 rivet of 20 mm dia for each angle.

2.1.12 Computation of wind effect

Let us assume intensity of wind pressure as 2.4 kN/m^2, at the bridge location. This wind pressure is to be taken only when the bridge is unloaded. For bridge loaded condition, maximum permissible wind pressure is 150 kg/m^2 (=1.5 kN/m^2) only, as per railway code. 1

(1) Bridge unloaded

Total depth of girder, $D = 2048$ mm = 2.048 m

Let us take the total depth of girder + sleeper + rails = 2048 + 150 + 300 = 2498 mm = 2500 mm

Spacing S = 1.9 m

Since this ratio is greater than 0.5 but lesser than 1, factor R=0.25.

Hence wind force $P_{WI} = p_w (1 + k) h_1 L = 2.4 (1 + 0.25)\, 2.5 \times 24 = 180$ kN.

(a) Overturning effect

$$P_{WI} \cdot \frac{h_1}{2} = (2R \cdot S)$$

$$2R = 118.42 \text{ kN}$$

$$\text{Extra B.M. } M_w = \frac{(2R)L}{8} = \frac{118.4 \times 24}{8} = 355.3 \text{ kN-m}$$

$$\text{Hence } \frac{M_w}{M} = 0.069 = 6.9\ \%$$

This increase is less then 16⅔ %. Hence ok.

(Note: the girder section has been designed for B.M. M = 5127.6 KN-m)

(b) Horizontal truss effect

The horizontal wind load P_{WI} is resisted by horizontal truss bracing, provided between the compression flanges of the two plate girders. The wind load P_{WI} includes B.M.

Mw Given by

$$M_w = \frac{P_{WI} \cdot L}{8}$$

This induces equal and opposite axial forces F in the compression flanges of two girders

Such that $$F.S. = \frac{P_{wI} \cdot L}{8}$$

F= 284.2 KN

This force is tensile in the compression flange of leeward girder.

Hence decrease in stress in compression flange of leeward girder $= \frac{284.2 \times 1000}{24112} = 11.79$ N/mm²

2. Bridge loaded

Maximum $P_w = 1.5$ kN/m² for broad gauge. Let us assume that the train occupies the whole span.

$P_{WI} = P_W (1 + k) h_1 L = 1.5 (1 + 0.25) 2.5 \times 24 = 112.15$ KN

Acting at $h_1/2 = 2.5/2 = 1.25$ above bottom.

$P_{WI} = 1.5 \times 3.5 \times 24 = 126$ KN acting at 4.85 m above bottom.

(a) Overturning effect

$112.5 \times 1.25 + 126 \times 4.85 = 2\,R \times 1.9$

From pitch 2 R = 395.6 KN

Hence additional downward load on the leeward girder = 395.6

Total DL + LL + IL on a girder =1/2 × 3418.4 = 1709.2 KN

Increase in load $= \frac{395.6 \times 100}{1709.2} = 23.1\%$

Since this increase in more than the permissible increase of 16⅔% in stresses (as per Railway Bridge Code), further checking in necessary.

∴ Max stress, including wind, by proportion $= \frac{395.6 + 1709.2}{1709.2} \times 126.1 = 153.3$ N/mm²

Permissible Stress under wind load = 1.167 × 138 = 161 N/mm². Hence safe

(b) Horizontal truss effect

Total horizontal force, $P = P_{w1} + P_{w2} = 112.5 + 126 = 238.5$ KN

$$\therefore F = \frac{PL/8}{S} = \frac{238.5 \times 24.}{8 \times 1.9} = 376.6 \text{ KN}$$

Hence decrease in stress in compression flange of leeward girder = F/A = 376.6 × 1000/24112

= 15.6 N/mm^2

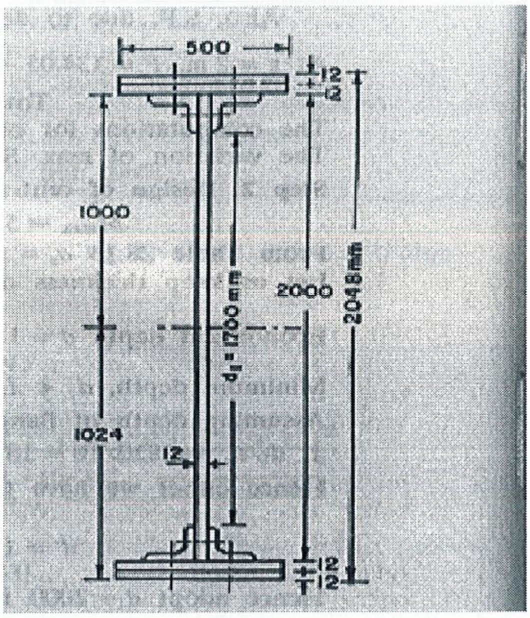


Fig 2.1 Full section

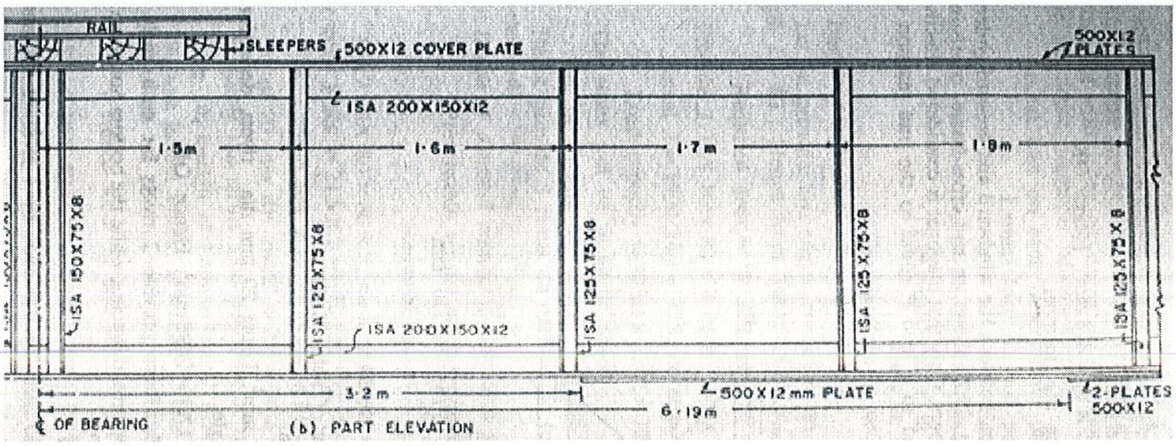


Fig 2.2 Plan of bridge

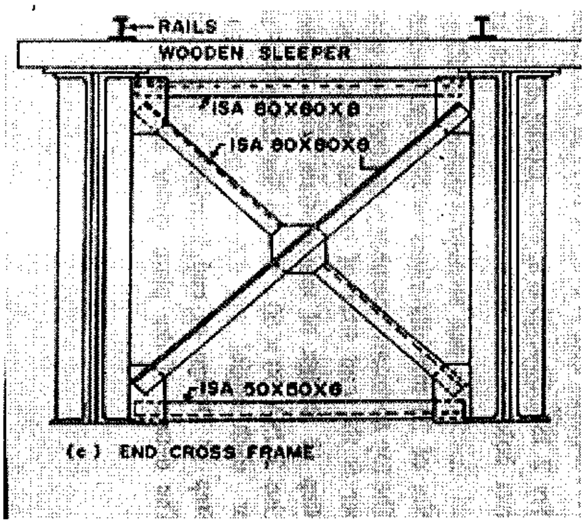


Fig 2.3 Cross section of bridge

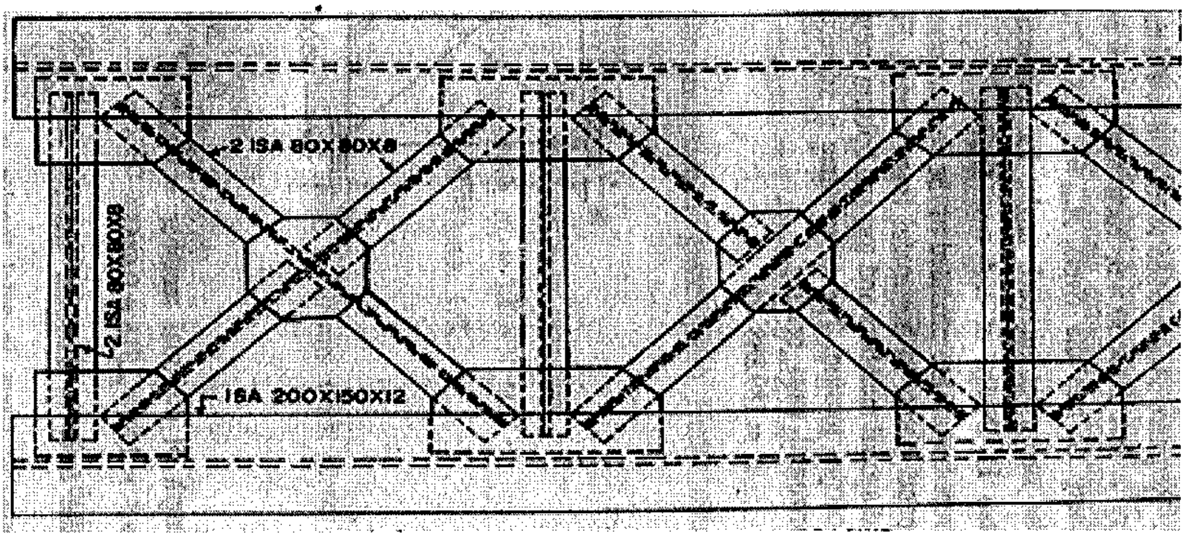


Fig 2.4 Top plan showing lateral bracing

CHAPTER 3
DIFFERENCE BETWEEN LSM AND WSM

3.1 Limit state method.

The limit state method of design is based on the behavior of structure at different limit states ensuring adequate safety against each limit states. In this the structure is designed by considering the stresses at different limit states. Here we design a structure by considering such probabilities and possibilities of collapse load (to such limit where building may get unsafe). The two principal limit states are the ultimate limit state and the serviceability limit state. The ultimate state is reached when the structure collapses. It requires that the structure must withstand the load for which it is designed with adequate factor of safety without collapse. There may be several other serviceability limit states such as durability, fire resistance, excessive vibration etc. The load considered for each limit state is known as the design load and is obtained from characteristic loads with appropriate partial factor of safety (γ_f). Design load = characteristic load × partial safety factor. This method includes uncertainty in design like uncertainty of loads, material strength and member dimensions. Due to these uncertainties structure must be designed for any possibility of overloading.

Limit state: It is a state beyond which structure ceases to perform its intended function satisfactorily.

There are two limit states:

- Ultimate limit state
  - It deals with strength (yielding, buckling, transformation into mechanism)
  - Stability against overturning and sway
  - Fracture due to fatigue
  - Brittle failure
- Serviceability limit state
  - Deals with deformation and deflection
  - Vibration due to wind and earthquake
  - Corrosion and durability
  - Fire

Attainment of one or more ULS conditions may be regarded as inability to sustain loads where as attainment of SLS condition may be regarded as a need for repair and remedial actions. So ULS should be avoided and SLS is undesirable.

3.2 Working stress method.

The working stress method of design is based on the behavior of structure at working load. In this a structure is designed by considering the stresses in its working condition. In this method we design only for specific load. The stress distribution in concrete and steel at working load is assumed to be linear. Hence the design in made by assuming linear stress-strain relationship ensuring that the stresses in concrete and steel do not exceed their permissible values at service load which is taken as the fixed proportion of the ultimate or yield strength of material. IS 800: 1984 used working stress method for designing steel members. WSM is a traditional method and it limits the working stress up to elastic limit and structure is assumed to behave elastically. Stresses induced due to loads are checked against permissible stress.

Permissible stress = yield stress / factor of safety

Working stress in designed members should be less than permissible stress. Factor of safety is 1.67 for beams and tension members. It is 1.92 for short columns. For connection it is 2.5 to 3. Safety criteria used in WSM is as follows:

i. Stresses due to (DL + LL) < permissible stress
ii. Stresses due to (DL + LL + WL) < 1.3 (permissible stress)

Shortcomings of WSM

1. The assumption of material behaving linearly elastic and other assumption that working stress shall remain under permissible stress is not realistic because of local stress concentration, creep and shrinkage and other secondary effects. Steel has ability to tolerate high elastic stresses by yielding locally and redistributing the stresses.
2. WSM does not use realistic factor of safety.
3. WSM is not able to estimate the loads which can act simultaneously. This sometimes may lead to error in design but most of the structures designed by WSM perform satisfactory due to high factor of safety but sometimes WSM gives over design and over sizing of members.

CHAPTER 4

DESIGN OF BRIDGE AS PER LATEST IS 800: 2007

4.1 Salient features of IS: 800-2007

IS:800-2007 is the basic code for general construction in steel structures and is the prime document for any structural design and has influence on many other codes governing the design of other special steel structures. Realizing the necessity to update the standard to the state of the art of the steel construction technology and economy, the current revision of the standard was undertaken. Consideration has been given to the developments taking place in country and abroad and necessary modifications and additions have been incorporated to make the standard more useful. The revised standard will enhance the confidence of designers, engineers, contractors, technical institutions, professional bodies and the industry will open a new era in safe and economic construction in steel.

In this revision the following major modifications have been affected.

- The standard is based on Limit state method, reflecting the latest developments and the state of the art.
- In the view of development and production of new varieties of medium and high tensile structural steels in the country, the scope of the standard has been modified permitting the use of any variety of structural steel provided the relevant provision of the standard are satisfied.

1. a) The major Chapters of the new addition:
    - General design requirements
    - Methods of structural analysis
    - Limit state design
    - Design of tension members
    - Design of compression members
    - Design of members subjected to bending
    - Members subjected to combined forces
    - Connections
    - Working stress design

- Design and detailing of earthquake loads
- Fatigue
- Durability
- Fire resistance
- Design assisted by testing
- Fabrication and erection

b) It also includes the following Annexure

- List of referred Indian standards
- Analysis and design methods
- Design against floor vibration
- Determination of effective length of column
- Elastic lateral torsional buckling
- Connections
- General recommendations for steel work tenders and contracts
- Plastic properties of beams

c) General Design Requirement:

- The new edition of IS: 800 clearly classify cross section as to, plastic, compact, semi compact or slender. Separate design procedures have been laid down for each type of classification.
- The classification has been made based on each element of the section involved and depends on the ratio of the major and minor dimension of the element i.e. limiting width to thickness ratio.

d) Limit States Method of Design

- Separate Partial Safety Factors for different loads and combinations are considered based on the probability of occurrence of the loads. Similarly different safety factors for materials are also considered depending on perfection in material characteristics and fabrication erection tolerances.

- Different permissible deflections considering different material of construction have also been proposed.

e) Tension Members:

- Tension members have been designed by considering not only failure of the net cross section (after taking Shear Lag) but also considering yielding of the gross cross section and rupture of the section at the joint.

f) Compression Members:

- Design of Compression members considers the appropriate buckling curve out of total four numbers depending on the type of section and the axis of buckling. Earlier version of the Working Stress Method of design considered only one buckling curve for all types of members irrespective of the nature of buckling.

g) Members subjected to bending:

- Reduction in Flexure capacity due to high Shear Force has been elaborated in detail.
- New version introduces tension field design of plated steel girders.

h) Members subjected to combined forces:

- Moment Gradient across a member or element considered in detail, while designing against combined action of axial force and bending moment in an element of a structure.

i) Working Stress method of design:

- Working Stress Method (WSM) of Design has been kept in a separate chapter with minor modifications (compared to the earlier code) and in tune with the specifications of the new code to ensure smooth transition from WSM to LSM for practicing engineers and academics whosoever desires.

j) Design against fatigue:

- o Design against fatigue has been introduced for the first time. The state of the art concept of stress range has been introduced for the first time in this code, this code automatically supersedes IS: 1024 for steel structures which considered the stress ratio method.

k) Earthquake Resistance:

- o Response Reduction factor has been introduced and elaborated in the new edition for the first time.

4.2 Design Problem

4.2.1 Computation of Loads, B.M and S.F. in plate girder.

a) Computation of loads:

Weight of main rails per meter run of track	= 0.44 × 2 = 0.88 kN/m
Weight of guard rails per meter run of track	= 0.26 × 2 = 0.5 kN/m
Weight of fastenings per meter run of track @ 0.28 kN/m	= 0.28 kN/m
Weight of sleepers per meter run of track	= (2.8×0.25×0.15)/0.4
	= 1. 94 kN/m
Total dead load	= 3.62 kN/m
Self weight of girders, for B.G main line loading is w	= 0.78L= 18.72 kN/m
Total dead load per meter run of track	= 3.62+18.72 = 22.34 kN/m
Total dead load	= 22.34 × 24 = 536.2 kN/m
Factored Load	= 536.2 × 2 = 804.3 kN/m
Dead load per girder	= 804.3/2 = 402.15 KN

b) Computation of B.M:

EUDL for B.M, for BG for loaded length L	= 24 m is 2034 KN on each track.
CDA	= 0.15 + 8/(6+L) = 0.417
Impact load for B.M	= 2034 ×0.417 × 1.5 =1272.26 KN

Hench total per track	= 804.3 + 2034 + 1272.26
	= 4110.56 KN
Hence total load per girder, for B.M	= 4110.56/2 = 2055.28 KN

M = WL/8 = 6165.8 kN-m

c) Computation of S.F:

The CDA will vary with loaded length L which depends upon the position of the section under consideration.

For end sections, *L*	= 24 m
Hence EUDL for shear	= 2231 KN (Table 28.1 A)

$$CDA = 0.15 + \frac{8}{6 + 24} = 0.417$$

Impact load, I. L	= 2231 x 0.417 = 930.3 KN
Total (L.L. + I.L.)	= (2231 + 930.3)1.5 = 4741.95KN
Hence, (L. L + I. L)	= 1/2 x 4741.95 = 2370.97 KN
SF due to (L.L. + I.L.)	= 1/2 × 2370.97 = 1185.48 KN
Dead load per girder, W_d	= 402.15 KN
Dead load shear per girder	= 1/2 x 402.15 = 201.07 KN
Total S.F. at each end of a girder	= 201.07 + 1185.48 = 1386.55 KN

4.2.2 Design of central section of plate

$$M_{max} = 6165.8 \text{ kN-m.}$$

Optimum depth of plate girder, $d = (Mk/f_{yf})^{0.33} = (6165.8 \times 10^6 \times 180 / 250)^{0.33} = 1526.21$ mm

Assume, d = 1500 mm

Optimum value of thickness of web, $t_w = (M/k^2 f_{yf})^{0.33} = (6165.8 \times 10^6 / 180^2 \times 250)^{0.33} = 9$mm

There are two design requirements regarding the minimum web thickness for the condition of no intermediate stiffeners (clause 8.6.1.1 and 8.6.1.2 of code)

$d/t_w < 200\varepsilon$ (for serviceability)

$d/t_w \leq 345\varepsilon^2$ (to avoid flange buckling)

As the web is deliberately made thick, i.e, $d/t < 67\varepsilon$ (clause 8.4.2.1), these requirements are automatically met. Thus, the minimum web thickness should be as follows:

$t_w > 1500/67 = 22.3$ mm

Provide $t_w = 25$ mm

In order to maximize the moment capacity, the cross section of the plate girder should be so proportioned that it satisfies the requirements of plastic/ compact section. Thus b_f/t_f should be less than 8.4ε or 9.4ε for plastic and compact sections respectively (see Table 2 of the code). Assuming b_f as 0.3 times the depth of web $b_f = 0.3 \times 1500 = 450$ mm; $t_f = b_f/8.4 = 53.57$ mm, provide $t_f = 50$ mm.

Use 1500mm x 25mm web plates with flange plates of 450mm x 50mm.

4.2.3 Shear Capacity

As per clause 8.4 of the code

$$V \leq V_d$$

$$V_d = V_n / \gamma_{mo}$$

Nominal plastic shear resistance, $V_n = V_p = A_v F_{yw} / \sqrt{3}$

As per clause 8.4.1.1 of the code, for welded section, $A_v = dt_w$

$$V_n = dt_w F_{yw} / \sqrt{3} = 1500 \times 25 \times 250 / (\sqrt{3} \times 1.1 \times 10^3) = 4920 \text{ kN} > 1386.5 \text{ kN}$$

Safe in shear.

4.2.4 Moment Capacity

As per clause 8.2.1.2 of the code, design bending strength,

$M_d = \beta_p Z_p F_y/\gamma_{mo} \leq 1.2\, Z_e\, F_y/\, \gamma_{mo}$

Plastic modulus, $Z_p = 2b_f t_f (D-t_f)/2 + t_w d^2/4$

$$= 2 \times 450 \times 50(1600\text{-}50) + 25 \times 1500^2/4 = 83.81 \times 10^6 \text{ mm}^3$$

$M_d = 1.0 \times 83.81 \times 10^6 \times 250 / 1.1 \times 10^6 = 19047.4$ kN-m > 6165.8 kN-m

Hence safe to carry the applied moment.

4.2.5 Check for bearing stiffeners.

a) At the supports

Assume that the width of supports is 300 mm and that the minimum stiff bearing provided by the support $b_l = 300/2 = 150$ mm

Dispersion length (1:2.5), $n_2 = 2.5 \times 50 = 125$ mm

Local capacity of the web, $F_w = (b_1 + n_2)t_w \times f_{yw} / \gamma_{mo}$

$= (150 + 125)25 \times 250/1.1 \times 10^3 = 1562.5$ kN > 1386.5 kN (total shear)

b) At position of moving wheel loads.

$F_w = (b_1 + n_2)t_w \times f_{yw} / \gamma_{mo}$

Assuming, $b_1 = 0$

$F_W = (0 + 2.5 \times 2 \times 50)25 \times 250 / 1.1\times 10^3$

$= 1420.45$ KN > 402.15 KN

The associated buckling resistance F_{qd} is dependent on the slenderness of the unstiffened web (clause 8.7.1.5).

Slenderness ratio of the web = $L_e/r_y = 0.7l/r_y$ [with $r_y = t/2\sqrt{3}$]

$= 2.5\ d/t = 2.5 \times 1500/25 = 150$

From table 9c of code $f_{cd} = 59.1$ N/mm^2 and as per clause 8.7.1.3 of the code, stiff bearing length = 0

45^0 dispersion length (to the level of half the depth of beam) = 1600/2 = 800 mm

$F_{qd} = (0 + 800)25 \times 59.1/10^3 = 1182$ KN > 402.15 KN

The web is adequate at both supports and positions of concentrated loads. Hence, there is no need to provide bearing stiffeners.

Thus, by using thick webs, the use of load bearing stiffeners may be eliminated which will minimize the fabrication.

4.2.6 The design of weld connection.

Assuming fillet weld on both side of web,

$q_w = VA_f\, y / 2I_Z$

$I_Z = b_f D^3/12 - (b_f - t_w)\, d^3/12$

$= 450 \times 1600^3/12 - (50\text{-}25)1500^3/12 = 14656.8 \times 10^6$ mm^4

$q_w = 1386.5 \times 450 \times 50 \times 800/(2 \times 14656.8 \times 10^6) = 0.851$ KN/mm

Provide 7mm fillet weld (from table 6.6)

4.2.7 Design of top lateral bracing

Consider the bridge to be loaded. Following lateral loads will act

(i) Wind load on both girders @ 1.5 kN/m^2 = 1.5 (1 + 0.25) 2.5 = 4.688 kN/m

(ii) Wind load on train @ 1.5 kN/m^2 = 1.5 x 3.5 = 5.25 kN/m

(iii) Racking force @ 600 kg/m = 600 x 9.81 x 10^{-3} = 5.886 kN/m

Total horizontal force = 15.824 kN/m

Total factored force = 15.824 × 1.5 = 23.736 kN/m

This force is resisted by top lateral bracing. Let us divide the span into 12 panels, with length of each panel 24/12 = 2 m, thus forming approximate squares. We will provide double diagonal system. For the analysis, it is assumed that the diagonal members carrying tension remain active, while other diagonals (shown dotted) remain dummy for one direction of wind. With the change of direction of wind, the dummy diagonals become active while the active diagonals become dummy.

Lateral load on each intermediate panel point = 23.736 x 2 = 47.472 kN. Lateral load on end panel point = 23.736 KN

End reaction = 23.736 x 24/2 = 284.83 KN

Tan θ = 1.9/2.0 = 0.9048; θ = 42.1376° ; sinθ = 0.6709 ; cosec θ = 1.4905

Compressive force in end strut = 284.83 KN

shear in end panel = 284.83 – 23.736 = 261.096

Tensile force in end diagonal = 261.096 cosecθ= 389.16 KN

Design of end strut

P = 284.83 KN

Let us provide two angles connected to the same side of gusset plate. Hence effective length A. = 80

Hence from Table 28.15, p_{ac} = 99 N/mm^2.

Required area = 284.83 x 10^3/99 = 2877.07 mm^2

Provide 2 ISA 80 x 80 x 8 each having a = 1221 mm^2 and r $_{xx}$ = 24.4 mm

Design of diagonal member

P = 389.16 KN (tensile)

Permissible tensile stress = 138 N/mm^2 (Table 28.13)

Required A_{net} $= \frac{389.16 \times 10^3}{138} = 2820$ mm^2

Try 2 ISA 80 x 80 x 12, each having a = 1781 mm^2.

4.2.8 Design of bottom lateral bracing

Bottom lateral system is similar to the top lateral bracing, except that the forces in the members will be 25% of the forces in corresponding members of top bracing.

Hence force in end strut = 1/4 x 284.83 = 71.20 KN

Try 2 ISA 60 x 60 x 8 mm, each having a = 896 mm^2 and a = 18 mm

Similarly tensile force in diagonal member = 1/4 x 389.16 = 97.29 KN

:. Required A_{net} = 97.29 x 10^3/138 = 705 mm^2

Try 2 ISA 65 x 65 x 6, each having a = 744 mm^2

4.2.9 Design of end cross-frames

Cross-frames are provided in the vertical planes, between the windward and leeward girders, at all panel points of horizontal truss bracing. An end cross-frame, consisting of two diagonal members and top and bottom members. Only that diagonal is considered effective which carries tension. The cross-frame is subjected to a load P equal to the end reaction of the horizontal truss bracing.

P = 284.83 KN

Length of diagonal member $= \sqrt{(1.864)^2 + (1.976)^2} = 2.716$ mm

Cos θ = 1.864/2.716 = 0.6861

Tensile force in diagonal member $= \frac{284.83}{0.6861} = 415.14$ KN

Allowable stress in axial tension = 138 N/mm^2 (Table 28.13)

:. Required A_{net} = 415.14 × 10^3/138 = 3008 mm^2

Try 2 ISA 100 x 100 x 8, each having a = 1539 mm^2.

4.2.10 Design of intermediate cross-frames

Use 2 ISA 50 x 60 x 6 mm for diagonals.

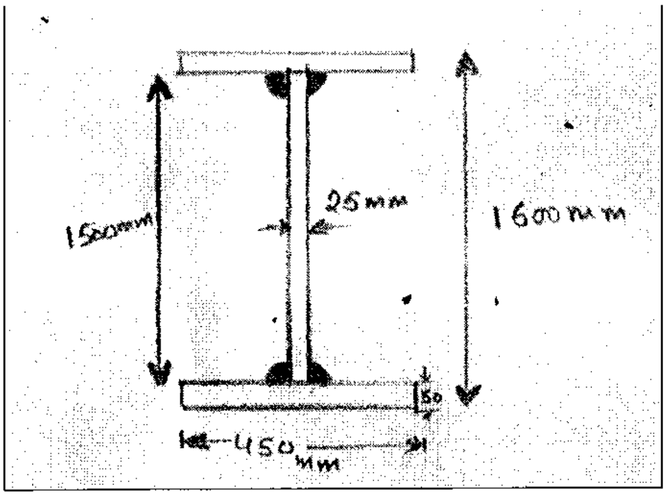


Fig 4.1 Full Section

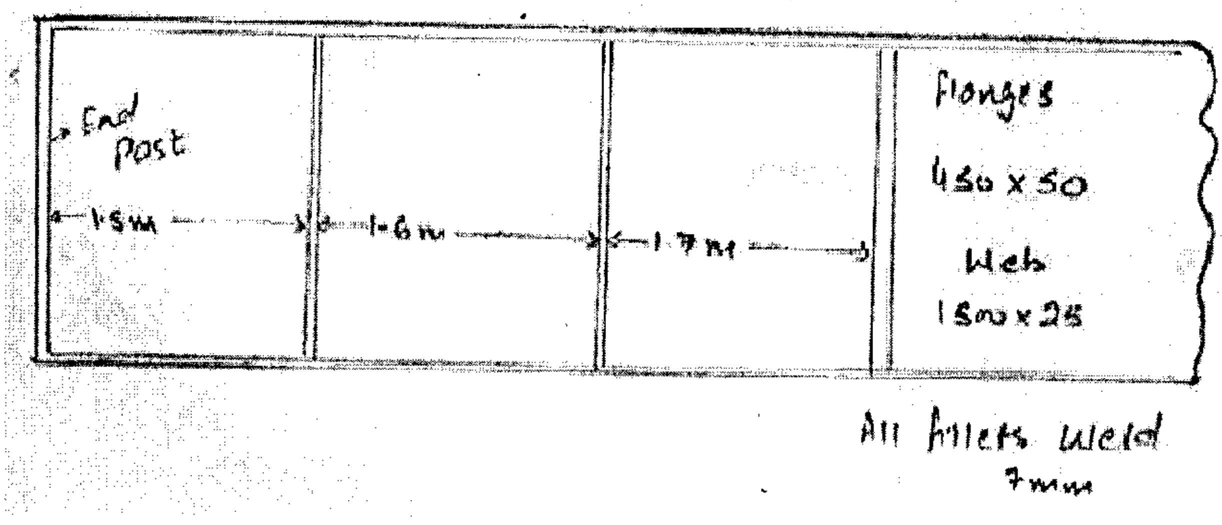


Fig 4.2 Plan of bridge

CHAPTER 5

COMPARISON AND CONCLUSION OF DESIGNS

5.1 Comparison of Result

The following table shows comparison of results of design.

Table 5.1

	Entity	As per IS: 800-1984	As per IS: 800-2007
Forces	DL + LL	268.1 KN	402.15 KN
	BM	5127.6 KN/m	6165.8 KN/m
	SF	924.4 KN	1386.5 KN
Central Section	Depth	2000 mm	1500 mm
	Thickness of web	12 mm	25 mm
Connection	....	Riveted	Welded
Stiffeners	....	Used of stiffeners	No stiffeners
Top Lateral Bracing	End strut	2 ISA 80 × 80 × 8 mm	2 ISA 80 × 80 × 8 mm
	Diagonal Element	2 ISA 80 × 80 × 8 mm	2 ISA 80 × 80 × 12 mm
Bottom Lateral Bracing	End strut	2ISA 60 × 60 × 6 mm	2ISA 60 × 60 × 8 mm
	Diagonal Element	2ISA 50 × 50 × 6 mm	2ISA 65 × 65 × 6 mm
End Cross Frames	....	2 ISA 80 × 80 × 8 mm	2 ISA 100 × 100 × 8 mm
Intermediate Cross Frames	....	2 ISA 50 × 60 × 6 mm	2 ISA 50 × 60 × 6 mm

5.2 Discussion and Conclusion

1.) Loads: Loads are multiplied by partial safety factor of 1.5 in case of Limit State Design. So values are higher in case of IS: 800-2007.

2.) Central Section: Due to change in formula for economical depth in IS: 800-2007, depth and thickness changes.

3.) Connection: IS: 800-2007 doesn't support the riveted connection that's why welded connection has been used.

4.) Stiffeners: As the thickness of web is thick enough, there is no need of stiffeners.

5.) Top and bottom lateral bracing: Due to change in loads, angles used in bracing are different from the design as per IS: 800-1984.

6.) End and intermediate cross frames: Due to change in loads, angles used in bracing are different from the design as per IS: 800-1984.

From above discussion, it can be concluded that design of plate girder bridge as per IS: 800-2007 is better in terms of strength, economy, serviceability and safety.

REFFERENCES

- IS 800: 1984, Indian Standard, *Code of Practice for General Construction in Steel,* Bureau of Indian Standards
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- B. C. Punmia, *Comprehensive Design of Steel Structures,* Laxmi Publication (P) Ltd
- N. Subramanian, *Design of Steel Structures,* Oxford University Press