1. Introduction

As discussed in earlier chapters the main advantages of structural steel over other construction materials are its strength and ductility. It has a higher strength to cost ratio in tension and a slightly lower strength to cost ratio in compression when compared with concrete. The stiffness to weight ratio of steel is much higher than that of concrete. Thus, structural steel is an efficient and economic material in bridges. Structural steel has been the natural solution for long span bridges since 1890, when the Firth of Forth cantilever bridge, the world's major steel bridge at that time was completed. Steel is indeed suitable for most span ranges, but particularly for longer spans. Howrah Bridge, also known as Rabindra Setu, is to be looked at as an early classical steel bridge in India. This cantilever bridge was built in 1943. It is 97 m high and 705 m long. This engineering marvel is still serving the nation, deriding all the myths that people have about steel. [See Figure 1]

![Howrah Bridge](image)

**Figure 1 Howrah Bridge**

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

- They could carry heavier loads over longer spans with minimum dead weight, leading to smaller foundations.
- Steel has the advantage where speed of construction is vital, as many elements can be prefabricated and erected at site.
- In urban environment with traffic congestion and limited working space, steel bridges can be constructed with minimum disruption to the community.
- Greater efficiency than concrete structures is invariably achieved in resisting seismic forces and blast loading.
- The life of steel bridges is longer than that of concrete bridges.
- Due to shallow construction depth, steel bridges offer slender appearance, which make them aesthetically attractive. The reduced depth also contributes to the reduced cost of embankments.
- All these frequently lead to low life cycle costs in steel bridges.

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

2 Steel used in Bridges

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

(i) **Carbon Steels:** This is the cheapest steel available for structural users where stiffness is more important than the strength. Indian steels have yield stress values up to 250 N/mm² and can be easily welded. The steel conforming to IS: 2062 - 1969, the American ASTM A36, the British grades 40 and Euro-norm25 grades 235 and 275 steels belong to this category.

(ii) **High Strength Steels:** They derive their higher strength and other required properties from the addition of alloying elements. The steel conforming to IS: 961 - 1975, British grade 50, American ASTM A572 and Euro-norm155 grade 360 steels belong to this category. Another variety of steel in this category is produced with enhanced resistance to atmospheric corrosion. These are called 'weathering' steels in Europe; in America they conform to ASTM A588 and have various trade names like ‘cor-ten’.

(iii) **Heat-treated Carbon Steels:** These are steels with the highest strength. They derive their enhanced strength from some form of heat treatment after rolling namely normalisation or quenching and tempering.

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

3 Classification of Steel Bridges

Steel bridges are classified according to

- the type of traffic carried
- the type of main structural system
- the position of the carriage way relative to the main structural system

These are briefly discussed in this section.

3.1 **Classification based on type of traffic carried**

Bridges are classified as

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

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

(i) **Girder Bridges** - Flexure or bending between vertical supports is the main structural action in this type. Girder bridges may be either solid web girders or truss girders or box girders. Plate girder bridges are adopted for simply supported spans less than 50 m and box girders for continuous spans up to 250 m. Cross sections of a typical plate girder and box girder bridges are shown in Figure 2(a) and Figure 2(b) respectively. Truss bridges [See Figure 2(c)] are suitable for the span range of 30 m to 375 m. Cantilever bridges have been built with success with main spans of 300 m to 550 m. In the next chapter girder bridges are discussed in detail. They may be further, sub-divided into simple spans, continuous spans and suspended-and-cantilevered spans, as illustrated in Figure 3.
(ii) **Rigid Frame Bridges** - In this type, the longitudinal girders are made structurally continuous with the vertical or inclined supporting member by means of moment carrying joints [Figure 4]. Flexure with some axial force is the main forces in the members in this type. Rigid frame bridges are suitable in the span range of 25 m to 200 m.

![Figure 3 Typical Girder Bridges](image1)

![Figure 4 Typical Rigid Frame Bridge](image2)

(iii) **Arch Bridges**

![Figure 5 Typical Arch Bridges](image3)

The loads are transferred to the foundations by arches acting as the main structural element. Axial compression in arch rib is the main force, combined with some bending.
Arch bridges are competitive in span range of 200 m to 500 m. Examples of arch bridges are shown in Figure 5.

(iv) **Cable Stayed Bridges** - Cables in the vertical or near vertical planes support the main longitudinal girders. These cables are hung from one or more tall towers, and are usually anchored at the bottom to the girders. Cable stayed bridges are economical when the span is about 150 m to 700 m. Layout of cable stayed bridges are shown in Figure 6.

(v) **Suspension Bridges** - The bridge deck is suspended from cables stretched over the gap to be bridged, anchored to the ground at two ends and passing over tall towers erected at or near the two edges of the gap. Currently, the suspension bridge is best solution for long span bridges. Figure 7 shows a typical suspension bridge. Fig. 7.8 shows normal span range of different bridge types.

3.3 **Classification Based on the Position of Carriageway**

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

(i) **Deck type Bridge** - The carriageway rests on the top of the main load carrying members. In the deck type plate girder bridge, the roadway or railway is placed on the top flanges. In the deck type truss girder bridge, the roadway or railway is placed at the top chord level as shown in Figure 8.
(ii) **Through Type Bridge** - The carriageway rests at the bottom level of the main load carrying members [Figure 8(b)]. In the through type plate girder bridge, the roadway or railway is placed at the level of bottom flanges. In the through type truss girder bridge, the roadway or railway is placed at the bottom chord level. The bracing of the top flange or lateral support of the top chord under compression is also required.

(iii) **Semi through Type Bridge** - The deck lies in between the top and the bottom of the main load carrying members. The bracing of the top flange or top chord under compression is not done and part of the load carrying system project above the floor level as shown in Figure 8(c). The lateral restraint in the system is obtained usually by the U-frame action of the verticals and cross beam acting together.

4 **Loads and Load Combinations**

4.1 **Loads on Bridges**

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

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

**Dead load** – The dead load is the weight of the structure and any permanent load fixed thereon. The dead load is initially assumed and checked after design is completed.

**Live load** – Bridge design standards specify the design loads, which are meant to reflect the worst loading that can be caused on the bridge by traffic, permitted and expected to pass over it. In India, the Railway Board specifies the standard design loadings for railway bridges in bridge rules. For the highway bridges, the Indian Road Congress has specified standard design loadings in IRC section II. The following few pages brief about the loadings to be considered.

For more details, the reader is referred to the particular standard.

**Railway bridges**: Railway bridges including combined rail and road bridges are to be designed for railway standard loading given in bridge rules. The standards of loading are given for:

- **Broad gauge** - Main line and branch line
- **Metre gauge** - Main line, branch line and Standard C
- **Narrow gauge** - H class, A class main line and B class branch line

The actual loads consist of axle load from engine and bogies. The actual standard loads have been expressed in bridge rules as equivalent uniformly distributed loads (EUDL) in tables to simplify the analysis. These equivalent UDL values depend upon the span length. However, in case of rigid frame, cantilever and suspension bridges, it is necessary for the designer to proceed from the basic wheel loads. In order to have a uniform gauge throughout the country, it is advantageous to design railway bridges to Broad gauge main line standard loading. The EUDLs for bending moment and shear force for broad gauge main line loading can be obtained by the following formulae, which have been obtained from regression analysis:

For bending moment:
\[ EUDL \text{ in kN} = 317.97 + 70.83l + 0.0188l^2 \geq 449.2 \text{ kN} \]  
(1)

For shear force:
\[ EUDL \text{ in kN} = 435.58 + 75.15l + 0.0002l^2 \geq 449.2 \text{ kN} \]  
(2)

Note that, \( l \) is the effective span for bending moment and the loaded length for the maximum effect in the member under consideration for shear. \( l \) should be in metres. The formulae given here are not applicable for spans less than or equal to 8 m with ballast cushion. For the other standard design loading the reader can refer to Bridge rules.

**Highway bridges**: In India, highway bridges are designed in accordance with IRC bridge code. IRC: 6 - 1966 – Section II gives the specifications for the various loads and stresses to be considered in bridge design. There are three types of standard loadings for which the
bridges are designed namely, IRC class AA loading, IRC class A loading and IRC class B loading.

IRC class AA loading consists of either a tracked vehicle of 70 tonnes or a wheeled vehicle of 40 tonnes with dimensions as shown in Figure 10. The units in the figure are mm for length and tonnes for load. Normally, bridges on national highways and state highways are designed for these loadings. Bridges designed for class AA should be checked for IRC class A loading also, since under certain conditions, larger stresses may be obtained under class A loading. Sometimes class 70 R loading given in the Appendix - I of IRC: 6 - 1966 - Section II can be used for IRC class AA loading. Class 70R loading is not discussed further here. Class A loading consists of a wheel load train composed of a driving vehicle and two trailers of specified axle spacings. This loading is normally adopted on all roads on which permanent bridges are constructed. Class B loading is adopted for temporary structures and for bridges in specified areas.

For class A and class B loadings, reader is referred to IRC: 6 - 1966 – Section II.

**Foot Bridges and Footpath on Bridges** – The live load due to pedestrian traffic should be treated as uniformly distributed over the pathway. For the design of footbridges or footpaths on railway bridges, the live load including dynamic effects should be taken as 5.0 kN/m² of the footpath area. For the design of footpath on a road bridges or road-rail bridges, the live load including dynamic effects may be taken as 4.25 kN/m² except that, where crowd loading is likely, this may be increased to 5.0 kN/m².

The live load on footpath for the purpose of designing the main girders has to be taken as follows according to bridge rules:

(i) For effective spans of 7.5 m or less - 4.25 kN/m²
(ii) The intensity of load is reduced linearly from 4.25 kN/m² for a span of 7.5 m to 3.0 kN/m² for a span of 30m
(iii) For effective spans over 30 m, the UDL may be calculated as given below:

$$P = \frac{1}{1} \left( 13.3 + \frac{4}{1} \right) \left( \frac{1-W}{14} \right) \text{kN/m}^2$$  \hspace{1cm} (3)
Where,

\[ P = \text{Live load in kN/m}^2 \]
\[ l = \text{Effective span of the bridge in m} \]
\[ W = \text{Width of the foot path in m} \]

Where foot-paths are provided on a combined rail-road bridge, the load on foot-path for purpose of designing the main girders should be taken as 2.0 kN/m².

**Impact load**

![Impact Percentage Curve for Highway Bridges for IRC class A and IRC class B loadings](Figure 11)

The dynamic effect caused due to vertical oscillation and periodical shifting of the live load from one wheel to another when the locomotive is moving is known as impact load. The impact load is determined as a product of impact factor, \( I \), and the live load. The impact factors are specified by different authorities for different types of bridges. The impact factors for different bridges for different types of moving loads are given in the Table 1. Figure 11 shows impact percentage curve for highway bridges for class AA loading. Note that, in the above table \( l \) is loaded length in m and \( B \) is spacing of main girders in m.

**Longitudinal forces** – Longitudinal forces are set up between vehicles and bridge deck when the former accelerate or brake. The magnitude of the force \( F \), is given by

\[
F = \frac{W \delta V}{g \delta t}
\]  

(4)

Where,

\( W \) – weight of the vehicle
\( g \) – acceleration due to gravity
\( \delta V \) – change in velocity in time \( \delta t \)

This loading is taken to act at a level 1.20 m above the road surface. No increase in vertical force for dynamic effect should be made along with longitudinal forces. The possibility of more than one vehicle braking at the same time on a multi-lane bridge should also be considered.
Table 1: Impact factors for different bridges

<table>
<thead>
<tr>
<th>BRIDGE</th>
<th>LOADING</th>
<th>IMPACT FACTOR (I)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Railway Bridges according to Bridge Rules</td>
<td>Broad Gauge and Meter Gauge</td>
<td>(a) Single track 20</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.14 + 1 ≤ 1.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(b) Main girder of double track</td>
</tr>
<tr>
<td></td>
<td></td>
<td>with two girders</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(c) Intermediate main girder of</td>
</tr>
<tr>
<td></td>
<td></td>
<td>multiple track spans</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(d) Outside main girders of</td>
</tr>
<tr>
<td></td>
<td></td>
<td>multiple track spans</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(e) Cross girders carrying two or more</td>
</tr>
<tr>
<td></td>
<td></td>
<td>tracks</td>
</tr>
<tr>
<td></td>
<td>Broad Gauge</td>
<td>Rails with ordinary fish plate joints</td>
</tr>
<tr>
<td></td>
<td>Meter Gauge</td>
<td>supported directly on sleepers or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>transverse steel troughing</td>
</tr>
<tr>
<td></td>
<td>Narrow Gauge</td>
<td></td>
</tr>
</tbody>
</table>

Highway Bridges according to IRC regulations

<table>
<thead>
<tr>
<th>BRIDGE</th>
<th>LOADING</th>
<th>IMPACT FACTOR (I)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Highway Bridges according to IRC regulations</td>
<td>(i) Spans less than 9m</td>
<td>0.25 for spans up to 5m and linearly</td>
</tr>
<tr>
<td></td>
<td>(a) Tracked Vehicle</td>
<td>reducing to 0.10 to spans of 9m</td>
</tr>
<tr>
<td></td>
<td>(b) Wheeled Vehicle</td>
<td>0.25</td>
</tr>
<tr>
<td>IRC Class A loading and IRC Class B Loading</td>
<td>Spans between 3m and 45m</td>
<td>9</td>
</tr>
<tr>
<td></td>
<td></td>
<td>13.5 + 1 In accordance with the curve</td>
</tr>
<tr>
<td></td>
<td></td>
<td>indicated in Figure 11 for all spans</td>
</tr>
</tbody>
</table>

Foot Bridges

<table>
<thead>
<tr>
<th>BRIDGE</th>
<th>LOADING</th>
<th>IMPACT FACTOR (I)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No separate impact allowance is made</td>
<td></td>
</tr>
</tbody>
</table>

**Thermal forces** – The free expansion or contraction of a structure due to changes in temperature may be restrained by its form of construction. Where any portion of the structure is not free to expand or contract under the variation of temperature, allowance should be made for the stresses resulting from this condition. The coefficient of thermal expansion or contraction for steel is $11.7 \times 10^{-6}/^\circ C$

**Wind load** – Wind load on a bridge may act
- Horizontally, transverse to the direction of span
- Horizontally, along the direction of span
- Vertically upwards, causing uplift
- Wind load on vehicles

Wind load effect is not generally significant in short-span bridges; for medium spans, the design of sub-structure is affected by wind loading; the super structure design is affected by wind only in long spans. For the purpose of the design, wind loadings are adopted from the maps and tables given in IS: 875 (Part III). A wind load of 2.40 kN/m² is adopted for the unloaded span of the railway, highway and footbridges. In case of structures with opening the effect of drag around edges of members has to be considered.
**Racking force** – This is a lateral force produced due to the lateral movement of rolling stocks in railway bridges. Lateral bracing of the loaded deck of railway spans shall be designed to resist, in addition to the wind and centrifugal loads, a lateral load due to racking force of 6.0 kN/m treated as moving load. This lateral load need not be taken into account when calculating stresses in chords or flanges of main girders.

**Forces on parapets** - Railings or parapets shall have a minimum height above the adjacent roadway or footway surface of 1.0 m less one half the horizontal widths of the top rail or top of the parapet. They shall be designed to resist a lateral horizontal force and a vertical force each of 1.50 kN/m applied simultaneously at the top of the railing or parapet.

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

**Forces due to curvature** - When a track or traffic lane on a bridge is curved allowance for centrifugal action of the moving load should be made in designing the members of the bridge. All the tracks and lanes on the structure being considered are assumed as occupied by the moving load. This force is given by the following formula:

\[
C = \frac{W^2}{12.7R}
\]

Where,
- \(C\) - Centrifugal force in kN/m
- \(W\) - Equivalent distributed live load in kN/m
- \(V\) - Maximum speed in km/hour
- \(R\) - Radius of curvature in m

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

### 4.2 Load Combinations
Stresses for design should be calculated for the most sever combinations of loads and forces. Four load combinations are generally considered important for checking for adequacy of the bridge. These are given in Table 7.2 and are also specified in IS 1915 - 1961.

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Load Combination</th>
<th>Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Stresses due to normal loads</td>
<td>Dead load, live load, impact load and centrifugal force</td>
</tr>
<tr>
<td>2</td>
<td>2 Stresses due to normal loads + occasional loads</td>
<td>Normal load as in (1) + wind load, other lateral loads, longitudinal forces and temperature stresses</td>
</tr>
<tr>
<td>3</td>
<td>Stresses due to loads during erection</td>
<td>-</td>
</tr>
</tbody>
</table>
4  Stresses due to normal loads + occasional loads + Extra-ordinary loads like seismic excluding wind load

| Loads as in (2) + with seismic load instead of wind |

5  Analysis of Girder Bridges

As discussed above, bridge decks are required to support both static and moving loads. Each element of a bridge must be designed for the most severe conditions that can possibly be developed in that member. Live loads should be placed in such a way that they will produce the most severe conditions. The critical positions of live loads will not be the same for every member. A useful method for determining the most severe condition of loading is by using “influence lines”.

An influence line represents some internal force such as shear force, bending moment etc. at a particular section or in a given member of girder, as a unit load moves over the span. The ordinate of influence line represents the value of that function when the unit load is at that particular point on the structure. Influence lines provide a systematic procedure for determining how the force (or a moment or shear) in a given part of a structure varies as the applied load moves about on the structure. Influence lines of responses of statically determinate structures consist only of straight lines whereas this is not true of indeterminate structures. It may be noted that a shear or bending moment diagram shows the variation of shear or moment across an entire structure for loads fixed in one position. On the other hand an influence line for shear or moment shows the variation of that response at one particular section in the structure caused by the movement of a unit load from one end of the structure to the other. In the following section, influence lines only for statically determinate structures are discussed.

5.1  Influence lines for beams and plate girders

![Influence lines for shear and bending moment](image)

Figure 12 Influence lines for shear and bending moment

Figure 12(a) shows the influence line for shear at a section in a simply supported beam. It is assumed that positive shear occurs when the sum of the transverse forces to the left of a section is in the upward direction or when the sum of the forces to the right of the section is downward.
A unit force is placed at various locations and the shear force at sections 1-1 is obtained for each position of the unit load. These values give the ordinates of influence line with which the influence line diagram for shear force at sections 1-1 can be constructed. Note that the slope of the influence line for shear on the left of the section is equal to the slope of the influence line on the right of the section. This information is useful in drawing shear force influence line in all cases.

Influence line for bending moment at the same section 1-1 of the simple beam is shown in Figure 12(b). For a section, when the sum of the moments of all the forces to the left is clockwise or when the sum to the right is counter clockwise, the moment is taken as positive. The values of bending moment at sections 1-1 are obtained for various positions of unit load and influence line is plotted. The detailed calculation of ordinates of influence lines is illustrated for members of the truss girder in the following section.

### 5.2 Influence lines for truss girders

Influence lines for support reactions and member forces for truss may be constructed in the same manner as those for beams. They are useful to determine the maximum force that may act on the truss members. The truss shown in Figure 13 is considered for illustrating the construction of influence lines for trusses.

The member forces in U3U4, U3L4 and L3L4 are determined by passing a section X-X and considering the equilibrium of the free body diagram of one of the truss segments.

![Figure 13 A Typical Truss](image)
When unit load is in between \( L_4 \) and \( L_8 \):

Then, there will not be rolling unit load in the left-hand side section.

\[
U_3 U_4 = \frac{4I}{h} \left( 1 - \frac{x}{L} \right)
\]

Note that the influence diagram gives force in the member \( U_3 U_4 \) directly, due to the unit load.

5.2.2 Influence line diagram for member \( U_3 L_4 \) (Inclined member) [Figure 14(b)]

![Influence line diagram](image)

Figure 14 Typical shapes of influence lines

Again consider the left-hand side of the section 1-1, and use the equilibrium equation for vertical forces i.e.

\[
\Sigma V = 0 \text{ where, } V \text{ represents the vertical force.}
\]

When unit load is in between \( L_0 \) and \( L_3 \):

\[
\frac{x}{L} + U_3 U_4 c = 0
\]

\[
U_3 U_4 = \frac{-x}{Lc} \theta
\]

Where, \( \theta = \tan^{-1} \left( \frac{1}{h} \right) \)

When unit load is in between \( L_4 \) and \( L_8 \):

\[
U_3 U_4 = -\left( 1 - \frac{x}{L} \right)
\]
When unit load is in between $L_3$ and $L_4$:

Since the variation of force in member $U_3L_4$ is linear as the unit load moves from $L_3$ to $L_4$ joining the ordinates of influence line at $L_3$ and $L_4$ by a straight line gives the influence line diagram in that zone. Note that, $U_3L_4$ represents the force in that member.

5.2.3 Influence line diagram for $U_3L_3$ (Vertical member) [Figure 14(c)]

Consider the left-hand side of the section 2-2 shown in Figure 13 for illustrating the construction of influence line for vertical member.

When unit load is in between $L_0$ and $L_3$:
By considering the equilibrium equation on the section left hand side of axis 2-2.

\[
U_3L_4 - \frac{x}{L} = 0
\]

\[
U_3L_4 = \frac{x}{L}
\]

When unit load is in between $L_4$ and $L_3$:

\[
U_3L_4 = -\left(1 - \frac{x}{L}\right)
\]

When unit load is in between $L_3$ and $L_4$:

Joining the ordinates of influence line at $L_3$ and $L_4$ by a straight line gives the influence line diagram between $L_3$ and $L_4$. $U_3L_3$ represents the force in that member.

Similarly influence line diagrams can be drawn for all other members. Typical shapes of influence line diagrams for the members discussed are shown in Figure 14. The design force in the member is obtained in the following manner. In this chapter, compressive forces are considered negative and tensile forces are positive.

Case (1): If the loading is Railway loading (UDL)
- Influence line diagram for force is drawn for that member
- The algebraic sum of areas of influence line under loaded length multiplied by magnitude of uniformly distributed load gives the design force.

Case (2): If the loading is Highway loading (Concentrated loading)
- Influence line diagram for force is drawn for that member
- The algebraic sum of the respective ordinates of influence line at the concentrated load location multiplied by concentrated loads gives design load of that member
- The series of concentrated loads are arranged in such a way that the maximum value of the desired member force is obtained.

6 Plate Girder Bridges

Plate girders became popular in the late 1800's, when they were used in construction of railroad bridges. The plates were joined together using angles and rivets to obtain plate girders of desired size. By 1950's welded plate girders replaced riveted and bolted plate girders in developed world due to their better quality, aesthetics and economy. Figure 15 shows the cross sections of two common types of plate girder bridges. The use of plate girders rather than rolled beam sections for the two main girders gives the designer freedom to select the most economical girder for the structure.
If large embankment fills are required in the approaches to the bridge, in order to comply with the minimum head-room clearance required, the half through bridge is more appropriate [Figure 15 (a)]. This arrangement is commonly used in railway bridges where the maximum permissible approach gradient for the track is low. In this case the restraint to lateral buckling of compression flange is achieved by a moment resisting U-frame consisting of floor beam and vertical stiffness, which are connected together with a moment resisting joint. If the construction depth is not critical, then a deck-type bridge, as shown in Figure 15 (b) is a better solution, in which case the bracings provide restraint to compression flange against lateral buckling.

6.1 Main Plate Girders

The design criterion for main girders as used in buildings, was discussed in chapters on Plate Girders. In the following sections some additional aspects that are to be considered in the design of plate girders in bridges, are discussed.

Generally, the main girders require web stiffening (either transverse or both transverse and longitudinal) to increase efficiency. The functions of these web stiffeners are described in the chapters on plate girders. Sometimes variations of bending moments in main girders may require variations in flange thickness to obtain economical design. This may be accomplished either by welding additional cover plates or by using thicker flange plate in the region of larger moment. In very long continuous spans (span> 50 m) variable depth plate girders may be more economical.

Initial design of main plate girder is generally based on experience or thumb rules such as those given below. Such rules also give a good estimate of dead load of the bridge structure to be designed. For highway and railway bridges, indicative range of values for various overall dimension of the main girders are given below:

Overall depth, D: \[
\frac{l}{18} \leq D \leq \frac{l}{12} \text{ (Highway bridges)}
\]
\[
\frac{l}{10} \leq D \leq \frac{l}{7} \text{ (Railway bridges)}
\]

Flange width, 2b: \[
D/4 \leq 2b \leq D/3
\]
Flange thickness, T: \[
b/12 \leq T \leq b/5
\]
Web thickness, t: \[t \approx \frac{D}{125}\]

Here, \(l\) is the length between points of zero moment. The detailed design process to maximize girder efficiency satisfying strength, stability, stiffness, fatigue or dynamic
criteria, as relevant, can be then carried out. Recent developments in optimum design methods allow direct design of girder bridges, considering minimization of weight/cost.

6.1.1 Detailed Design of main Plate Girders in Bridges

The load effects (such as bending moment and shear force) are to be found using individual and un-factored load cases. Based on these, the summation of load effects due to different load combinations for various load factors are obtained. Since bridges are subjected to cyclic loading and hence are vulnerable to fatigue, redistribution of forces due to plastic mechanism formation is not permitted under BS 5400: Part - 3. The design is made based on Limit State of collapse for the material used considering the following:

- Shape limitation based on local buckling
- Lateral torsional buckling
- Web buckling
- Interaction of bending and shear
- Fatigue effect

Shape limitation Based on Local Buckling

Depending on the type of cross section (compact or non-compact) the variation of stress over the depth at failure varies. A compact section can develop full plastic moment i.e. rectangular stress block as shown in Figure 16 (a). Before the development of this full plastic moment, local buckling of individual component plates should not occur. Thus the compact section should possess minimum thickness of elements on the compression zone such that they do not buckle locally before the entire compression zone yields in compression. The minimum thickness of elements for a typical compact section is shown in Figure 17, where $f_y$ is to be substituted in SI units (MPa).

![Figure 16 Design Stresses](image-url)
The section that does not fulfill the minimum thickness criterion of compact section is defined as non-compact section. A non-compact section may buckle locally before full section plastic capacity is reached. Therefore the design of such section is based on triangular stress block wherein yielding at the extreme fibre, as shown in Figure 16(b), limit the design moment. The moment capacity of the compact and non-compact cross sections can be evaluated by the following formulae:

\[ M_u = \frac{Z_p f_y}{\gamma_m} \]  
for compact sections \hspace{1cm} (6a)

\[ M_u = \frac{Z f_y}{\gamma_m} \]  
for non-compact sections \hspace{1cm} (6b)

Where,  
- \( f_y \) - yield stress  
- \( Z_p \) - plastic modulus  
- \( Z \) - elastic modulus  
- \( \gamma_m \) - partial safety factor for material strength (1.15)

Even in the compact section, the use of plastic modulus does not imply that plastic analysis accounting for moment redistribution is applicable. BS 5400: Part - 3 precludes plastic analysis and does not allow any moment redistribution to be considered. This is to avoid repeated plastification under cyclic loading and the consequent low cycle fatigue failure. When non-compact sections are used the redistribution will not occur and hence plastic analysis is not applicable.

**Lateral Torsional Buckling**

A typical bridge girder with a portion of the span, over which the compression flange is laterally unrestrained, is shown in Figure 18(a). Such a girder is susceptible to lateral torsional buckling. Figure 18(b) shows a laterally buckled view of a portion of the span. The displacements at mid span, where the beam is laterally restrained, will be only vertical, as shown in Figure 18(c). A part of the beam between restraints can translate downwards and sideways and rotate about shear centre [Figure 18(d)]. Failure may then be governed by lateral torsional buckling. This type of failure depends on the unrestrained length of compression flange, the geometry of cross section, moment gradient etc. The procedure in detail for calculating the value of the limiting compressive stress is given in chapters on laterally unrestrained beams.
Web Buckling

The web of plate girders resist the shear in the three modes, namely (i) pure shear, (ii) tension field action and (iii) that due to formation of collapse mechanism. These are discussed in detail in the chapters on plate girders. They are presented briefly below:

The elastic critical shear strength of a plate girder is given by

$$q_c = k \frac{\pi^2 E}{12(1-\mu^2)} \left( \frac{t}{d} \right)^2$$

(7)

where,

$$k = 5.34 + 4 \left( \frac{d}{a} \right)^2 \quad \text{when} \quad \frac{a}{d} \geq 1.0$$

$$k = 4 + 5.34 \left( \frac{d}{a} \right) \quad \text{when} \quad \frac{a}{d} < 1.0$$

Where t, d and a are the web thickness, depth and distance between vertical stiffeners, respectively.

The elastic local buckling of the web in shear does not lead to collapse Limit State, since the web experiences stable post-buckling behavior. In mode (ii), a tension field develops in the panel after shear buckling. In mode (iii) the maximum shear capacity is reached, when
pure shear stress in mode (i) and the membrane stress, $p_t$ in mode (ii) cause yielding of the panel and plastic hinges in the flanges. This is discussed in detail in the chapters on plate girders.

The membrane tensile stress $p_t$ in terms of the assumed angle $\theta = \tan^{-1}(d/a)$ of the tension field with respect to neutral axis (NA) and the first mode shear stress $q_s$ is given by,

Thus the resistance to shear in the three-modes put together is given by,

$$\frac{p_t}{q_s} = \left[ 3 + \left( 2.25 \sin^2 \theta - 3 \right) \left( \frac{q_s}{q_y} \right)^2 \right]^{1/2} - 1.5 \frac{q_s}{q_y} \sin \theta$$

(8)

if

$$m_{fw} \leq \frac{1}{4\sqrt{3}} \frac{a^2}{q_y} p_t \sin^2 \theta$$

$$\frac{q_u}{q_y} = \left[ \frac{q_s}{q_y} + 5.264 \sin \theta \left( m_{fw} \frac{p_t}{q_y} \right)^{1/2} + \frac{p_t}{q_y} \left( \cot \theta - \phi \right) \sin \theta \right]$$

(9)

if

$$m_{fw} > \frac{1}{4\sqrt{3}} \frac{a^2}{q_y} p_t \sin^2 \theta$$

$$\frac{q_u}{q_y} = \left[ 4\sqrt{3} m_{fw} \left( \frac{d}{a} \right) + \frac{p_t}{2q_y} \sin^2 \theta + \frac{q_s}{q_y} \right]$$

Where, $m_{fw}$ is the non-dimensional representation of plastic moment resistance of the flange, given by

$$m_{fw} = \frac{M_p}{d^2 f_{yw}}$$

When tension field action is used, careful consideration must be given to the anchorage of the tension field forces created in the end panels by appropriate design of end stiffeners.

**Shear-Moment Interaction**

![Shear-Moment Capacity Interaction Diagram](image)
Bending and shear capacities of girders without longitudinal stiffeners can be calculated independently and then an interaction relationship as given in Figure 19 is employed. In Figure 19, MD and MR are the bending capacities of the whole section with and without considering contribution of the web, respectively. VD and VR are the shear capacities with tension field theory, considering flanges and ignoring the flanges, respectively. However, for girders with longitudinal stiffeners, combined effects of bending and shear is considered by comparing the stresses in the different web panels using the relevant critical buckling strengths of the panel.

**Fatigue effect**

Under cyclic load, experienced by bridges, flaws in tension zone lead to progressively increasing crack and finally failure, even though stresses are well within the static strength of the material. It may be low cycle fatigue, due to stress ranges beyond yielding or high cycle fatigue, at stresses below the elastic limit. IS: 1024 gives the guide line for evaluating fatigue strength of welded details, that may be used to evaluate the fatigue strength.

Stress concentration may lead to premature cracking near bracing stiffener and shear connector welds. Proper detailing of connections is needed to favorably increase design life of plate girders.

### 6.2 Lateral Bracing for Plate Girders

Plate girders have a very low torsional stiffness and a very high ratio of major axis to minor axis moment of inertia. Thus, when they bend about major axis, they are very prone to lateral-torsional instability as shown in Figure 20(a). Adequate resistance to such instability has to be provided during construction. In the completed structure, the compression flange is usually stabilized by the deck. If the unrestrained flange is in compression, distorsional buckling, Figure 20(b), is a possible mode of failure and such cases have to be adequately braced. Thus, lateral bracings are a system of cross frames and
bracings located in the horizontal plane at the compression flange of the girder, in order to increase lateral stability.

Loads that act transverse on the plate girders also cause the lateral bending and the major contribution is from wind loads. Since plate girders can be very deep, increase in girder depth creates a larger surface area over which wind loads can act. This, in addition to causing lateral bending, contributes to instability of compression flange of the girder. Hence, design of lateral bracing should take account of this effect also.

Triangulated bracing as shown in Figure 15(b) is provided for deck type of plate girder bridges to increase lateral stability of compression flange. But, it cannot be adopted for the half-through or through girder bridges because it interferes with functions of the bridge. In these cases, the deck is designed as a horizontal beam providing restraint against translation at its level and the flange far away from the deck is stabilized by U-frame action as shown in Figure 15(a). The degree of lateral restraint provided to the compression flange by U-frame action depends upon the transverse member, the two webs of the main girder (including any associated vertical stiffener) and their connections. In this case, the effective length of a compression flange is usually calculated similar to the theory of beams on elastic foundations, the elastic supports being the U-frames.

**Plate Girder Bridges**
Truss Bridges

Truss Girders, lattice girders or open web girders are efficient and economical structural systems, since the members experience essentially axial forces and hence the material is fully utilized. Members of the truss girder bridges can be classified as chord members and web members. Generally, the chord members resist overall bending moment in the form of direct tension and compression and web members carry the shear force in the form of direct tension or compression. Due to their efficiency, truss bridges are built over wide range of spans. Truss bridges compete against plate girders for shorter spans, against box girders for medium spans and cable-stayed bridges for long spans. Some of the most commonly used trusses suitable for both road and rail bridges are illustrated in Figure 21.

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

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

7.1.1 Optimum depth of truss girder

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

7.1.2 Design of compression chord members

Generally, the effective length for the buckling of compression chord member in the plane of truss is not same as that for buckling out-of-plane of the truss i.e. the member is weak in one plane compared to the other. The ideal compression chord will be one that has a section with radii of gyration such that the slenderness value is same in both planes. In other words, the member is just likely to buckle in plane or out of plane. These members should be kept as short as possible and consideration is given to additional bracing, if economical.

The effective length factors for truss members in compression may be determined by stability analysis. In the absence of detailed analysis one can follow the recommendations given in respective codes. The depth of the member needs to be chosen so that the plate dimensions are reasonable. If they are too thick, the radius of gyration will be smaller than it would be if the same area of steel is used to form a larger member using thinner plates. The plates should be as thin as possible without losing too much area when the effective section is derived and without becoming vulnerable to local buckling.

Common cross sections used for chord members are shown in Figure 22. Trusses with spans up to 100 m often have open section compression chords. In such cases it is desirable to arrange for the vertical posts and struts to enter inside the top chord member, thereby providing a natural diaphragm and also achieving direct connection between member thus minimizing or avoiding the need for gussets. However, packing may be needed in this case. For trusses with spans greater than about 100 m, the chords will be usually the box shaped such that the ideal disposition of material to be made from both economic and maintenance view points. For shorter spans, rolled sections or rolled hollow sections may be used. For detailed design of compression chord members the reader is referred to the chapter on Design of axially compressed columns.

7.1.3 Design of tension chord members

Tension members should be as compact as possible, but depths have to be large enough to provide adequate space for bolts at the gusset positions and easily attach cross beam. The width out-of-plane of the truss should be the same as that of the verticals and diagonals so that simple lapping gussets can be provided without the need for packing. It should be possible to achieve a net section about 85% of the gross section by careful arrangement of the bolts in the splices. This means that fracture at the net section will not govern for common steel grades.
In this case also, box sections are preferable for ease of maintenance but open sections may well prove cheaper. For detailed design reader is referred to the chapter on Design of Tension members.

![Figure 22 Typical cross-section for truss members](https://example.com/image.png)

7.1.4 Design of vertical and diagonal members
Diagonal and vertical members are often rolled sections, particularly for the lightly loaded members, but packing may be required for making up the rolling margins. This fact can make welded members more economical, particularly on the longer trusses where the packing operation might add significantly to the erection cost.

Aesthetically, it is desirable to keep all diagonals at the same angle, even if the chords are not parallel. This arrangement prevents the truss looking over complex when viewed from an angle. In practice, however, this is usually overruled by the economies of the deck structure where a constant panel length is to be preferred. Typical cross sections used for members of the truss bridges are shown in Figure 22.

7.2 Lateral bracing for truss bridges
Lateral bracing in truss bridges is provided for transmitting the longitudinal live loads and lateral loads to the bearings and also to prevent the compression chords from buckling. This is done by providing stringer bracing, braking girders and chord lateral bracing. In case of highway truss bridges, concrete deck, if provided, also acts as lateral bracing support system.
The nodes of the lateral system coincide with the nodes of the main trusses. Due to interaction between them the lateral system may cause as much as 6% of the total axial load in the chords. This should be taken into account.

Figure 23 shows the two lateral systems in its original form and its distorted form after axial compressive loads are applied in the chords due to gravity loads. The rectangular panels deform as indicated by the dotted lines, causing compressive stresses in the diagonals and tensile stresses in the transverse members. The transverse bracing members are indispensable for the good performance of St. Andrew’s cross bracing system.

In diamond type of lateral bracing system the nodes of the lateral system occur midway between the nodes of the main trusses [Figure 23(c)]. They also significantly reduce the interaction with main trusses. With this arrangement, “scissors-action” occurs when the chords are stressed, and the chords deflect slightly laterally at the nodes of the lateral system. Hence, diamond system is more efficient than the St. Andrew’s cross bracing system.

It is assumed that wind loading on diagonals and verticals of the trusses is equally shared between top and bottom lateral bracing systems. The end portals (either diagonals or verticals) will carry the load applied to the top chord down to the bottom chord. In cases, where only one lateral system exists (as in Semi-through trusses), then the single bracing system must carry the entire wind load.
Truss bridges

Suspended central span

Continuous span
Connection gusset

8.1 Examples:

Design a through type single lane truss bridge for broad gauge main line loading. The effective span length of the bridge is 50 m. Consider $\gamma_m = 1.15$.

(1) Truss arrangement [See Figure E1]:
Effective Span of truss girder = 50 m.
Assume 10 panels @ 5 m interval.

Height and truss girder:
For economical considerations, height = $\frac{1}{8}$ to $\frac{1}{10}$ of span
Assume, height = 6m. (1/8.33 of span) Hence, O.K.
(2) Influence line diagrams:

(i) ILD for $L_0U_1$ (Diagonal member):

(a) If, unit load is in between $L_1$ and $L_{10}$ (i.e. $5 \leq x \leq 50$)

$$\sum V = 0$$

$$L_0U_1 \sin \theta = 1 - \left( \frac{x}{50} \right) \Rightarrow L_0U_1 = \frac{1}{\sin \theta} \left( 1 - \frac{x}{50} \right)$$

(b) If, unit load is in between $L_0$ and $L_1$ (i.e. $0 \leq x \leq 5$)

$$L_0U_1 = -\frac{9x}{\sin \theta \cdot 50}$$

Then, we can get ILD as shown in Figure E3.

(ii) ILD for $L_1U_1$ (Vertical member): [See free body diagram Figure E4]

(a) If, unit load is in between $L_0$ and $L_1$ (i.e. $5 \leq x \leq 50$)

$$\sum ML_0 = 0.$$
\[ 5L_1U_1 = x \]
\[ L_1U_1 = x / 5 \]

(b) If, unit load is in between \( L_2 \) and \( L_{10} \)

\[ L_1U_1 = 0 \]

(iii) ILD for \( U4U5 \) and \( L4L5 \): (Top and Bottom chord members respectively)

(a) If, the unit load is in between \( L_0 \) and \( L_4 \) (i.e. \( 0 \leq x \leq 20 \))

\[ \sum M_{L5} = 0 \]
\[ 6U_4U_5 + (25 - x) * 1 = 25 * [1 - (x/50)] \]
\[ U_4U_5 = \frac{1}{6} \left[ 25 \left( 1 - \frac{x}{50} \right) - (25 - x) \right] \]
\[ \sum M_{U4} = 0 \]
\[ 6L_4L_5 = (20 - x) * 1 = 20 * [1 - (x/50)] \]
\[ L_4L_5 = \frac{1}{6} \left[ 20 \left( 1 - \frac{x}{50} \right) - (20 - x) \right] \]
(b) If, unit load is in between \( L_5 \) and \( L_{10} \) (i.e \( 25 \leq x \leq 50 \))

\[
U_4U_5 = \frac{1}{6} \left[ 25 \left( 1 - \frac{x}{50} \right) \right]
\]

Then,

\[
L_4L_5 = \frac{1}{6} \left[ 20 \left( 1 - \frac{x}{50} \right) \right]
\]

ILDs for \( U_4U_5 \) and \( L_4L_5 \) are shown in Figure E7 and Figure E8 respectively.

![Figure E7 ILD for \( U_4U_5 \)](image)

![Figure E8 ILD for \( L_4L_5 \)](image)

(3) **Loads:**

(i) **Dead load** - Dead loads acting on truss girder are as follows:

- Weight of rails = 2 x 0.6 = 1.2 kN/m.
- * Weight of sleepers = 0.25 x 0.25 x 7.5 / 0.4 = 2.34 kN/m.
- Weight of fastenings (assumed) = 0.25 kN/m.
- Weight of stringers (assumed) = 3.0 kN/m.
- ** Self-weight of truss by Fuller’s Formula = 13.0 kN/m
- Total dead load per track = 24.8 kN/m.
- Therefore, Total dead load per girder = 24.8 / 2 = 12.4 kN/m.
- *[Assume 250 mm 250 mm 2m wooden sleepers @ 400 mm apart and weight of 7.5 kN/m3]*

**[Fuller's Formula = \( \frac{15l + 550}{100} = \frac{15*50 + 550}{100} = 13.0 \text{kN/m} \)]**

(ii) **Live load**

(a) Areas of Influence line diagrams for truss members discussed:

- Area of influence line for \( L_0 U_1 = \frac{1}{2} \times 50 \times 1.17 = -29.3 \) (Compression)
- Area of influence line for \( L_1 U_1 = \frac{1}{2} \times 10 \times 1.0 = +5.0 \) (Tensile)
- Area of influence line for \( U_4 U_5 = \frac{1}{2} \times 50 \times 2.08 = -52 \) (Compression)
- Area of influence line for \( L_4 L_5 = \frac{1}{2} \times 50 \times 2 = +50.0 \) (Tensile)

(b) Live loads and impact loads from IRS Bridge Rules - 1982:

Live loads and impact factors for each loaded length are found from IRS Bridge Rules - 1982. For maximum forces in chord members, the whole of the span should be loaded and
Live load is determined corresponding to maximum B.M. For other diagonal and vertical members, part of the span as indicated by influence line diagrams, should be loaded and the live load is determined corresponding to S.F. The impact factor is found corresponding to loaded length.

For maximum force in members L₄L₅ and U₄U₅:
Load length = 50 m
Live load for B.M. = 3895.2 kN
Impact factor = 0.15 +8/ (6+l) = 0.15+8/ (6+50) = 0.293
(LL+ IL) per m per girder = \( \frac{3895.2*(1+0.293)}{2*50} = 50.36kN/m \)

For maximum force in members L₀U₁ and L₁U₁:

L₀U₁
Load length = 50 m
Live load for B.M. = 4184.6 kN
Impact factor = 0.15 +8/ (6+l) = 0.15+8/ (6+50) = 0.293
(LL+ IL) per m per girder = \( \frac{4184.6*(1+0.293)}{2*50} = 54.1kN/m \)

L₁U₁:
Load length = 10 m
Live load for S.F. = 1227.8 kN
Impact factor = 0.293
(LL+ IL) per m per girder = \( \frac{1227.8*(1+0.65)}{2*10} = 101.3kN/m \)

(c) Longitudinal Loads from IRS Bridge Rules - 1982
Assume, there exist rail expansion joints in the bridge and prevent the transfer of longitudinal loads to approaches. It may be noted that for broad gauge bridges up to a loaded length of 44 m, the tractive effort is more than the braking force and for loaded lengths more than 44 m the braking force is more than the tractive effort.

Assume truss under consideration is simply supported by a hinge at L₀ and a roller at L₁₀. The longitudinal force in a member can be tensile or compressive depending on the direction of movement of train.

Panel L₄L₅:
Loaded length = 30 m
Tractive effort = 637.4 kN
Force per chord = 637.4/2 = ±318.7 kN

Unfactored loads:

<table>
<thead>
<tr>
<th>Member</th>
<th>Area of ILD</th>
<th>Load in kN/m</th>
<th>Forces in members (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>DL</td>
<td>LL+IL</td>
<td>DL</td>
</tr>
<tr>
<td>L₀U₁</td>
<td>-29.3</td>
<td>12.4</td>
<td>54.1</td>
</tr>
<tr>
<td>L₁U₁</td>
<td>+5.0</td>
<td>12.4</td>
<td>101.3</td>
</tr>
<tr>
<td>U₄U₅</td>
<td>-52.0</td>
<td>12.4</td>
<td>50.36</td>
</tr>
<tr>
<td>L₄L₅</td>
<td>+50.0</td>
<td>12.4</td>
<td>50.36</td>
</tr>
</tbody>
</table>
Use following Partial safety factors for the loads:
\(\gamma_{DL} = 1.35; \gamma_{LL} = 1.50; \gamma_{LongL} = 1.50\)

**Factored loads:**

<table>
<thead>
<tr>
<th>Member</th>
<th>Factored Forces in members (kN)</th>
<th>Total load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(DL)</td>
<td>(LL+IL)</td>
</tr>
<tr>
<td>(L_0U_1)</td>
<td>-490.4</td>
<td>-2377.6</td>
</tr>
<tr>
<td>(L_1U_1)</td>
<td>+83.7</td>
<td>+759.8</td>
</tr>
<tr>
<td>(U_4U_5)</td>
<td>-870.5</td>
<td>-3928</td>
</tr>
<tr>
<td>(L_4L_5)</td>
<td>+837</td>
<td>+3777</td>
</tr>
</tbody>
</table>

**Note:** Negative sign represents compression and positive sign represents tension.

(4) Design for truss members:

(i) **Design of diagonal member \((L_0U_1)\):** Note that in this illustration of this Member, the portal effect and fatigue are not considered.

Length of the chord, \(L_0U_1 = l = 7810\) mm
Assume, effective length, \(l_e = 0.7*l = 5467\) mm
Try a built up member with two ISHB350 spaced @ 300 mm

![Diagram of ISHB350 and ISMB350 elements](image)

\[A = 18442 \text{ mm}^2\]
\[r_x = 146.5 \text{ mm}\]
\[r_y = 158.8 \text{ mm}\]
\[\lambda_x = \frac{5467}{146.5} = 37.3\]
Then, \(\sigma_c = 221.8 \text{ N/mm}^2\)
[See chapter on axially compressed columns using curve c]
Axial capacity = \((221.8/1.15) * 18442/1000 = 3556.5 \text{ kN} > 2868 \text{ kN}\)
Hence, section is safe against axial compression

(ii) **Design of vertical member \((L_1U_1)\):**
Maximum tensile force = 843.4 kN
Try ISMB 350 @ 0.524 kN/m shown.
\[A = 6671 \text{ mm}^2\]
Axial tension capacity of the selected section = \(6671 * 250/1.15 = 1450 \text{ kN} > 843.4 \text{ kN}\)
Hence, section is safe in tension.
[Note: Welded connection assumed]

(iii) **Design of top chord member \((U_4U_5)\):**
Member length, \( l = 5000 \text{ mm} \)
Assume, effective length = \( 0.85l = 4250 \text{ mm} \)
Try the section shown.
\[ A = 25786 \text{ mm}^2 \]
\[ r_x = 165.4 \text{ mm} \]
\[ r_y = 210 \text{ mm} \]
\[ \lambda_x = 4250/165.4 = 25.7 \]
Then, \( \sigma_c = 239 \text{ N/mm}^2 \)

[See chapter on axially compressed columns using column curve c]
Axial capacity = \( (239/1.15)*25786/1000 = 5359 \text{ kN} > 4798.5 \text{ kN} \)
Hence, section is safe against axial compression

(iv) **Bottom chord design (L_4L_5):**
Maximum compressive force = 478 kN
Maximum tensile force = 5092 kN
Try the box section shown.

\[ A = 25386 \text{ mm}^2 \]
\[ r_x = 144 \text{ mm} \]
\[ r_y = 210 \text{ mm} \]

Axial tension capacity of the selected section = \( 25386* 250/1.15 = 5518 \text{ kN} > 5092 \text{ kN} \)
Hence, section is safe in tension.

Maximum unrestrained length = \( l = 5000 \text{ mm} \)
\[ \lambda_x = 5000/144 = 34.7 \]
Then, \( \sigma_c = 225 \text{ N/mm}^2 \)
Axial capacity = \( (225/1.15)* 25386/1000 = 4967 \text{ kN} > 478 \text{ kN} \)
Hence, section is safe against axial compression also.

The example is only an illustration. The following have to be taken into consideration:
- Design of lacings/batten
- Design of connections and effect of bolt holes on member strength
- Secondary bending effects
- Design for fatigue
9 Summary

After brief introduction, the steel used in bridges and its properties were discussed. The broad classification of bridges was mentioned and various loads to be considered in designing railway and highway bridges in India were discussed. Finally analysis of girder bridges was discussed using influence line diagrams.

This chapter deals with the design of steel bridges using Limit States approach. Various types of plate girder and truss girder bridges were covered. Basic considerations that are to be taken into account while designing the plate girder bridges are emphasised. Practical considerations in the design of truss members and lateral bracing systems are discussed briefly.

10 References


7. ESDEP, Group 15B, Volume 25: Structural systems - Steel Bridges, SCI, UK.
