44 STEEL BRIDGES -II

1.0 INTRODUCTION

It was shown in the previous chapter, Steel Bridges - I, that many different types of steel bridges may be designed, depending upon the span length, type of loading and approach road conditions. In this chapter, the design of plate girder and truss girder bridges, which are the most common type, are discussed in detail. An example design of truss girder bridge is presented in appendix.

Design is an open-ended process, wherein different engineers attempting solution to the same problem come up with different designs. In the conceptual design stage, decisions regarding the structural systems to be used, span, spacing and configuration of the main and sub-members of the system are made. The decisions made at this stage have a major impact on the economy, efficiency and aesthetics of the bridge, although very little information is available at this stage. Therefore, experience of the designer is very important at this stage of design. An inexperienced designer may have to try different options and carry out the design of each option to a greater level of detail, before finalising one design for execution.

The Indian Standard Code of Practice for Steel Bridges (IS: 1915 - 1961) is based on the Working Stress Method. Since Limit States Method of design is more rational and leads to more efficient, economical design and uniform reliability, most international standards have adopted it. Indian standards are also in the process of such a change. Hence, the Limit States method has been followed in this chapter. Since Indian Codes on the Limit States Method are just now evolving wherever necessary BS: 5400 - Part 3: 1982 provisions have been followed.

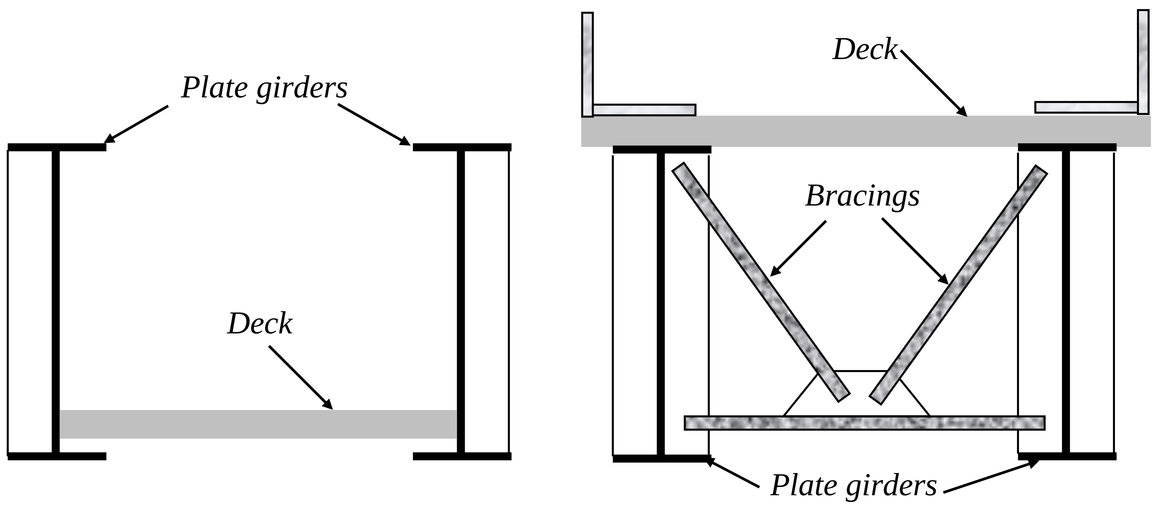
2.0 PLATE GIRDER BRIDGES

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. Fig. 1 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 [Fig. 1(*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 Fig. 1(*b*) is a better solution, in which case the bracings provide restraint to compression flange against lateral buckling.

2.1 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.



(a) Half through type plate girder bridge   *(b) Deck type plate girder bridge*

Fig. 1 Common types of plate girder bridge

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:	$\ell/18 \le D \le \ell/12$	(Highway bridges)
	$\ell/10 \le D \le \ell/7$	(Railway bridges)
Flange width, $2b$:	$D/4 \le 2b \le D/3$	
Flange thickness, T:	$b/12 \le T \le b/5$	
Web thickness, t:	$t \approx D/125$	

Here, ℓ is the length between points of zero moment. The detailed design process to maximise 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 minimisation of weight/cost.

2.1.1 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 Fig. 2(*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 Fig. 3, where f_y is to be substituted in SI units (MPa).

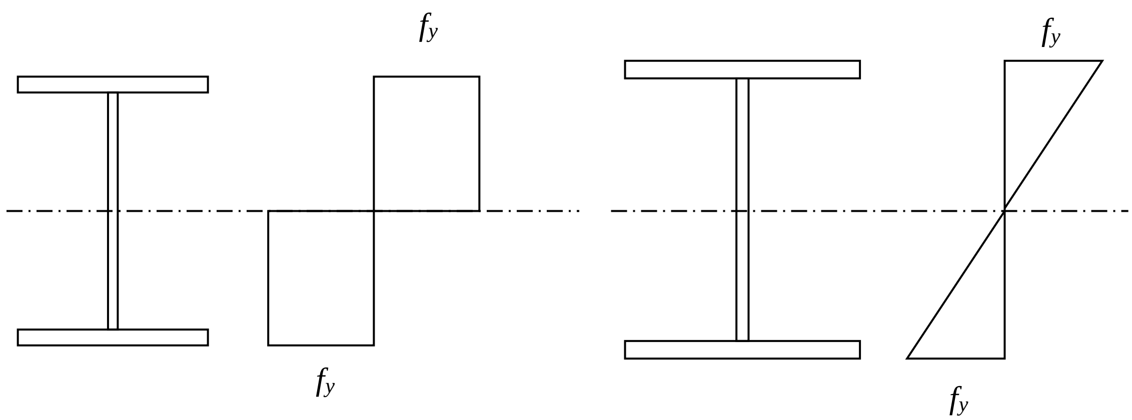
The section that does not fulfil the minimum thickness criterion of compact section is defined as non-compact section. A non-compact section may buckle locally before full section plastic capacity is reached. Therefore the design of such section is based on triangular stress block wherein yielding at the extreme fibre, as shown in Fig. 2(*b*), limit the design moment.

The moment capacity of the compact and non-compact cross sections can be evaluated by the following formulae:

$$M_u = Z_p \, f_y / \gamma_m \qquad \textit{for compact sections} \tag{1a}$$

$$M_u = Z \;\, f_y / \gamma_m \qquad \textit{for non-compact sections} \tag{1b}$$

where, f_y - yield stress
Z_p - plastic modulus
Z - elastic modulus
γ_m - partial safety factor for material strength (*1.15*)



(a) Compact sections   *(b) Non-compact sections*

Fig. 2 Design stresses

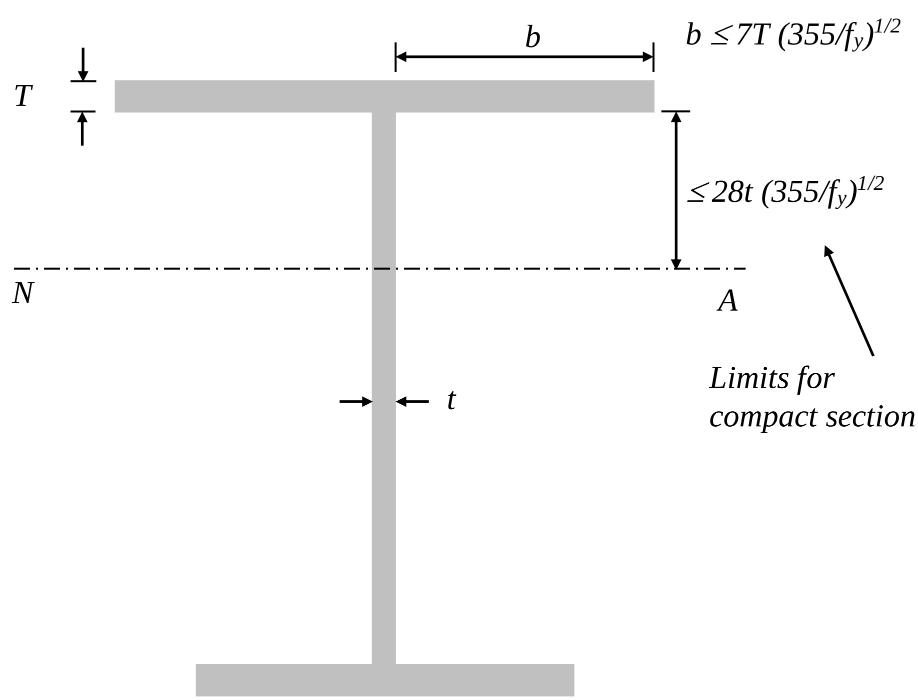


Fig. 3 Shape limitations for Plate girder

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 Fig. 4(*a*). Such a girder is susceptible to lateral torsional buckling. Fig. 4(*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 Fig. 4(*c*). A part of the beam between restraints can translate downwards and sideways and rotate about shear centre [Fig. 4(*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

$$q_c = k\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{d}\right)^2 \tag{3}$$

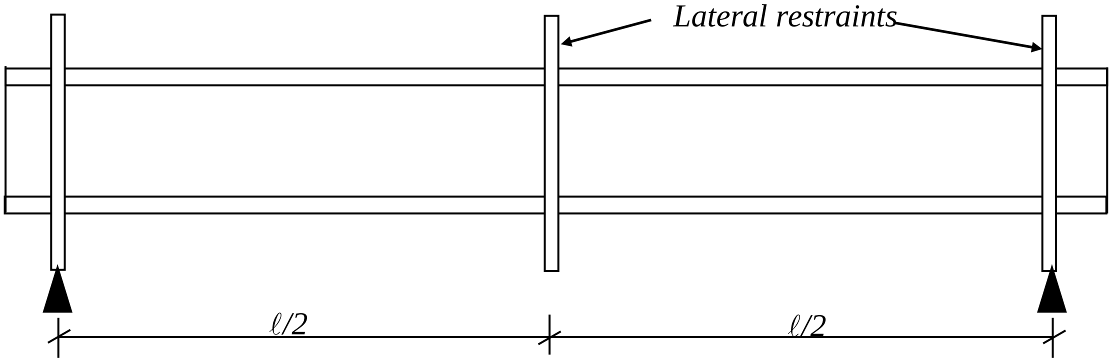
where,

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

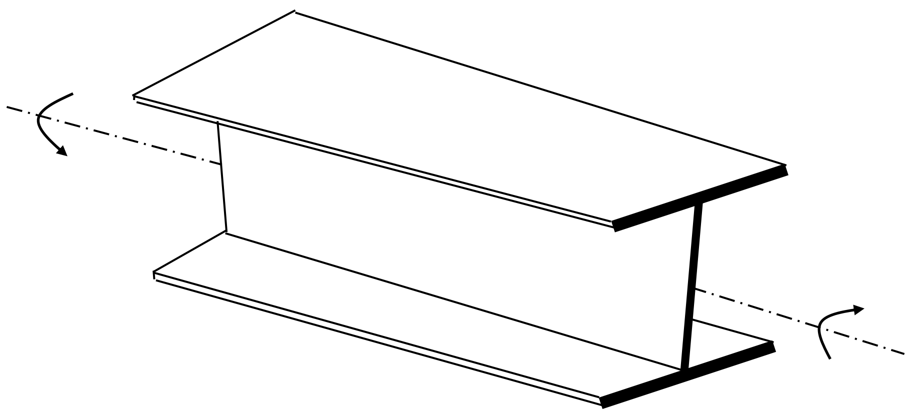
$$k = 4 + 5.34\left(\frac{d}{a}\right) \ \ when\ \frac{a}{d} < 1.0$$

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

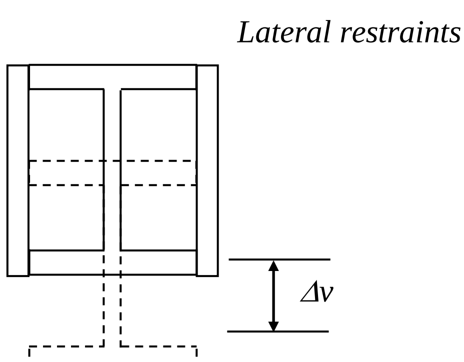
The elastic local buckling of the web in shear does not lead to collapse Limit State, since the web experiences stable post-buckling behaviour. In mode (ii), a tension field develops in the panel after shear buckling. In mode (iii) the maximum shear capacity is reached, when pure shear stress in mode (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.



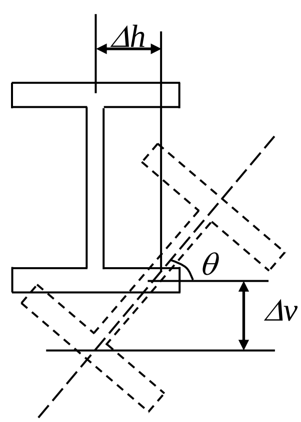
(a) Girder with lateral restraints at supports and mid-span section



(b) View showing lateral torsional buckling



(c) Section at restraint



(d) Section between restraints

Fig. 4 Distorsion caused by lateral torsional buckling

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

$$\frac{p_t}{q_y} = \left[3 + (2.25\, Sin^2\theta - 3)\left(\frac{q_c}{q_y}\right)^2 \right]^{1/2} - 1.5\, \frac{q_c}{q_y}\, Sin^2\theta \qquad (4)$$

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

$$If \quad m_{fw} \le \frac{1}{4\sqrt{3}}\left(\frac{a}{d}\right)^2 \frac{p_t}{q_y}\, Sin^2\theta$$

$$\frac{q_u}{q_y} = \left[\frac{q_c}{q_y} + 5.264\; Sin\theta \left(m_{fW} \;\; \frac{p_t}{f_y} \right)^{1/2} + \frac{p_t}{q_y}\,(Cot\theta - \phi)\, Sin^2\theta \right]$$

$$If \;\; m_{fw} > \frac{1}{4\sqrt{3}}\left(\frac{a}{d}\right)^2 \frac{p_t}{q_y}\, Sin^2\theta$$

$$\frac{q_u}{q_y} = \left[4\sqrt{3}\, m_{fw}\left(\frac{d}{a}\right) + \frac{p_t}{2q_y} Sin^2\theta + \frac{q_c}{q_y} \right] \qquad (5)$$

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

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

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

Bending and shear capacities of girders without longitudinal stiffeners can be calculated independently and then an interaction relationship as given in Fig. 5 is employed. In Fig. 5, M_d and M_R are the bending capacities of the whole section with and without considering contribution of the web, respectively. V_d and V_R 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.

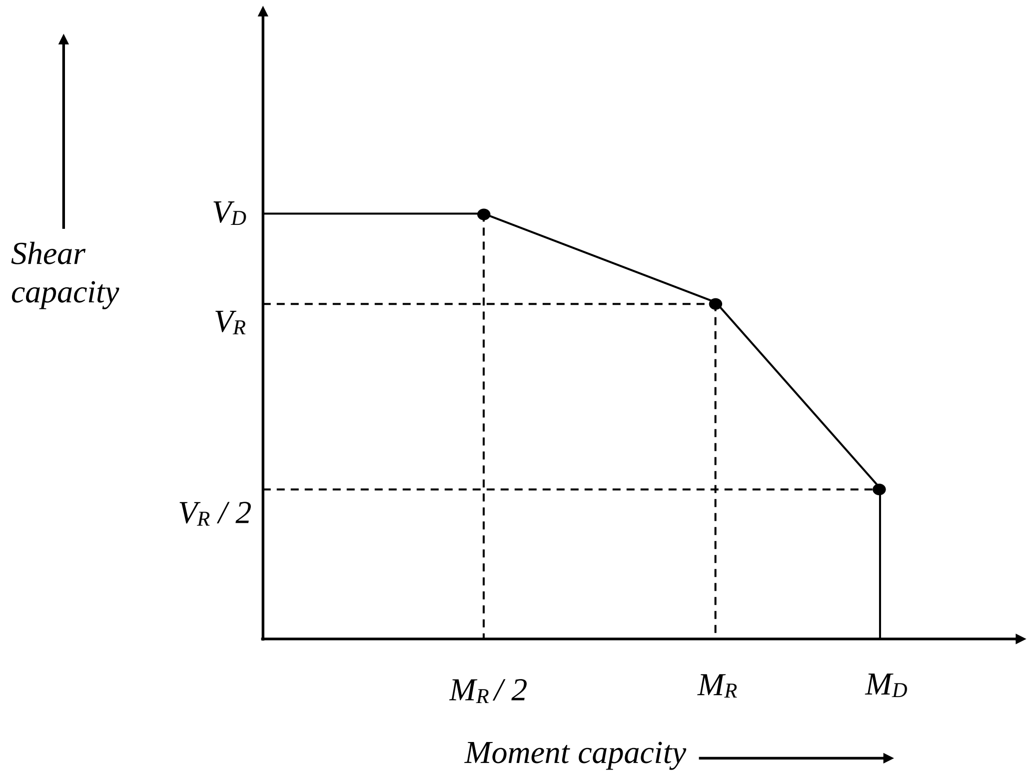


Fig. 5 Shear - Moment capacity interaction diagram

Fatigue effect

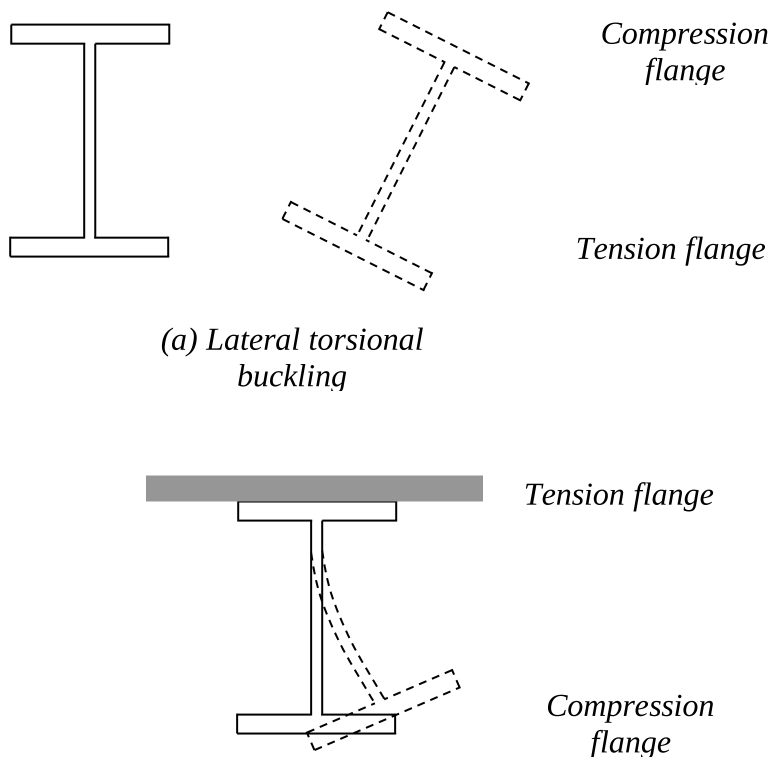
Under cyclic load, experienced by bridges, flaws in tension zone lead to progressively increasing crack and finally failure, eventhough stresses are well within the static strength of the material. It may be low cycle fatigue, due to stress ranges beyond yielding or high cycle fatigue, at stresses below 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 favourably increase design life of plate girders.

2.2 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 Fig. 6(*a*). Adequate resistance to such instability has to be provided during construction. In the completed structure, the compression flange is usually stabilised by the deck. If the unrestrained flange is in compression, distorsional buckling, Fig 6(*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.



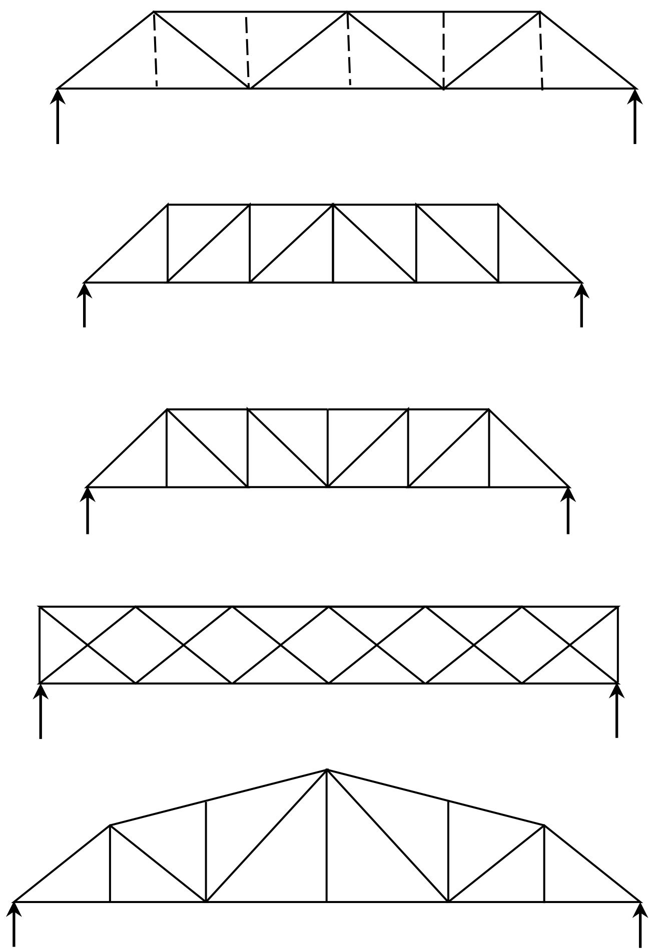
(b) Distorsional buckling

Fig. 6 Modes of instability of plate girders

Triangulated bracing as shown in Fig. 1(*b*) is provided for deck type of plate girder bridges to increase lateral stability of compression flange. But, it can not 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 stabilised by U-frame action as shown in Fig. 1(*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.

3.0 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 utilised. 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 Fig. 7.



(a) Warren truss

(b) Howe truss

(c) Pratt truss

(d) Double Warren truss

(e) Varying depth Warren truss

Fig. 7 Some of the trusses that are used in steel bridges

For short and medium spans it is economical to use parallel chord trusses such as Warren truss, Pratt truss, Howe truss, etc. to minimise 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 midspan.

As far as configuration of trusses is concerned, an even number of bays should be chosen in Pratt and modified Warren trusses to avoid a central bay with crossed diagonals. The diagonals should be at an angle between 50^o and 60^o 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.

3.1 General design principles

3.1.1 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 centres of main girders. The spacing between main truss depends upon the railway or road way clearances required.

3.1.2 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.

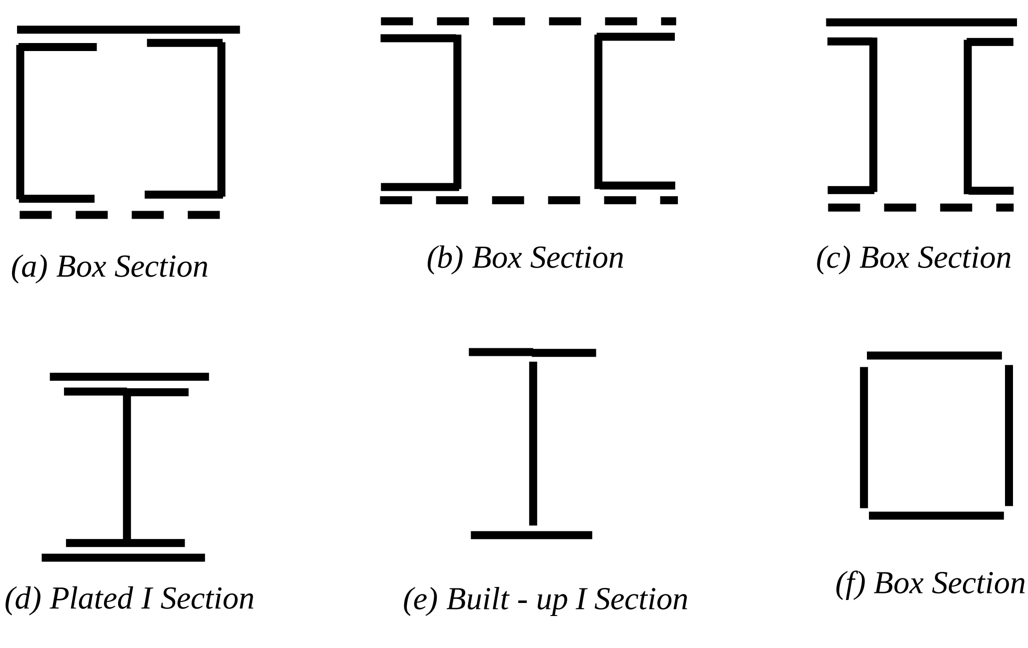
The effective length factors for truss members in compression may be determined by stability analysis. In the absence of detailed analysis one can follow the recommendations given in respective codes. The depth of the member needs to be chosen so that the plate dimensions are reasonable. If they are 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 Fig. 8. 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 minimising 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.

3.1.3 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.



(a) Box Section *(b) Box Section* *(c) Box Section*

(d) Plated I Section *(e) Built - up I Section* *(f) Box Section*

Fig. 8 Typical cross-sections for truss Members

3.1.4 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 Fig. 8.

3.2 Lateral bracing for truss bridges

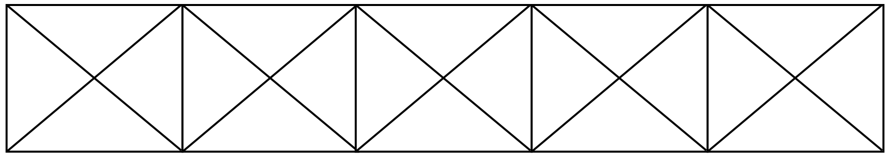
Lateral bracing in truss bridges is provided for transmitting the longitudinal live loads and lateral loads to the bearings and also to prevent the compression chords from buckling. This is done by providing stringer bracing, braking girders and chord lateral bracing. In case of highway truss bridges,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

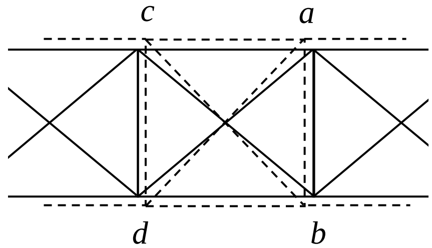
Fig. 9 shows the two lateral systems in its original form and its distorted form after axial compressive loads are applied in the chords due to gravity loads. The rectangular panels deform as indicated by the dotted lines, causing compressive stresses in the diagonals and tensile stresses in the transverse 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 [Fig. 9(*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.

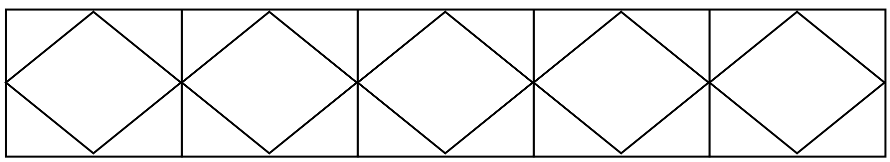
It is assumed that wind loading on diagonals and verticals of the trusses is equally shared between top and bottom lateral bracing systems. The end portals (either diagonals or verticals) will carry the load applied to the top chord down to the bottom chord. In cases, where only one lateral system exists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



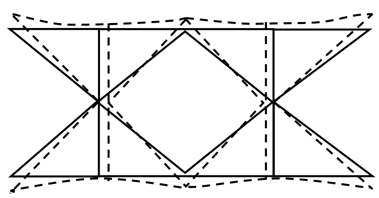
(a) St. Andrew's cross system



(b) Deformed Shape of (a)



(c) Diamond System



(d) Deformed shape of (c)

Fig. 9 Lateral Bracing Systems

4.0 SUMMARY

This chapter dealt 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. A worked example on through type truss girder Railway Bridge is given in the appendix.

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