7.6 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. Fig.7.15 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 [Fig.7.15 (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 Fig.7.15 (b) is a better solution, in which case the bracings provide restraint to compression flange against lateral buckling.

7.6.1 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: $l/18 \le D \le l/12$ (Highway bridges)

$l/10 \le D \le l/7$ (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: $t \approx D/125$

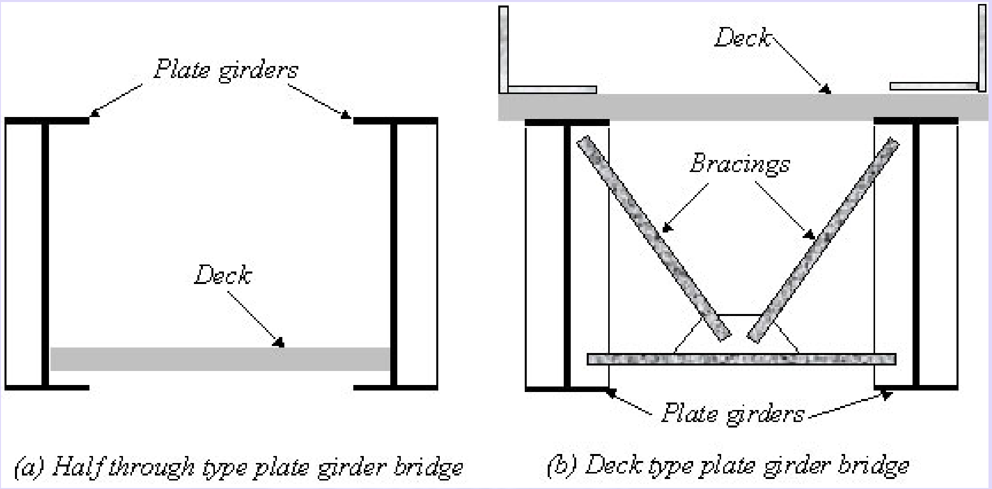

Fig.7.15 Common types of plate girder bridge

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.

7.6.1.1 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 Fig.7.16 (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 Fig.7.17, where f_y is to be substituted in SI units (MPa).

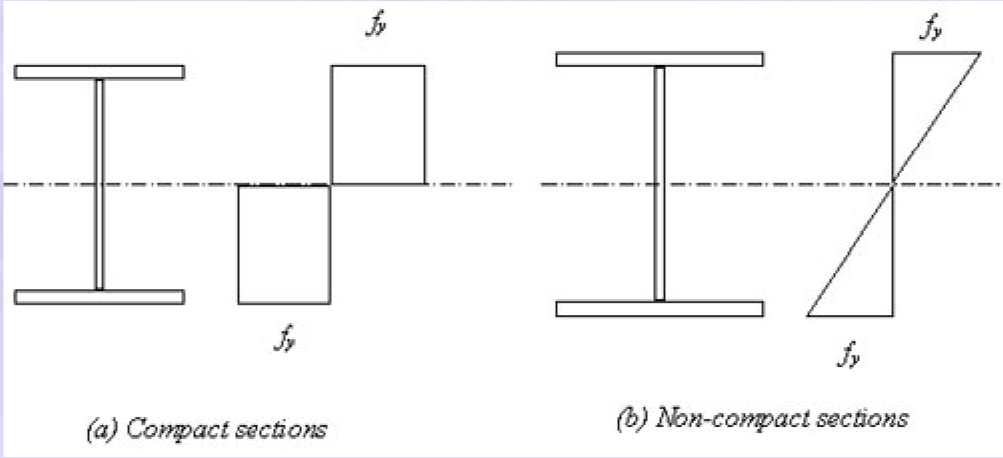

Fig.7.16 Design stresses

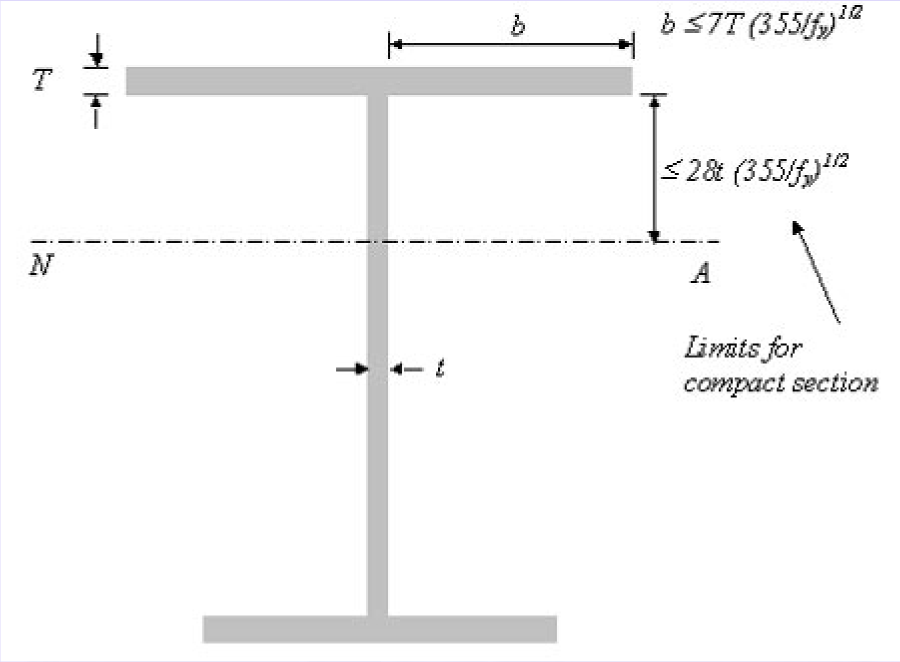

Fig.7.17 Shape limitations for plate girder

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 Fig.7.16 (b), limit the design moment.

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

$$M_u = Z_p f_y / \gamma_m \quad \text{for compact sections} \qquad \textit{(7.6a)}$$

$$M_u = Z f_y / \gamma_m \quad \text{for non-compact sections} \qquad \textit{(7.6b)}$$

Where, f_y - yield stress

Z_p - plastic modulus

Z - elastic modulus

γm - 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 Fig. 7.18(a). Such a girder is susceptible to lateral torsional buckling. Fig. 7.18(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 Fig. 7.18(c). A part of the beam between restraints can translate downwards and sideways and rotate about shear centre [Fig. 7.18(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

$$q_c = k\frac{\pi^2 E}{12\left(1-\mu^2\right)}\left(\frac{t}{d}\right)^2 \qquad \textbf{\textit{(7.7)}}$$

Where,

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

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

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

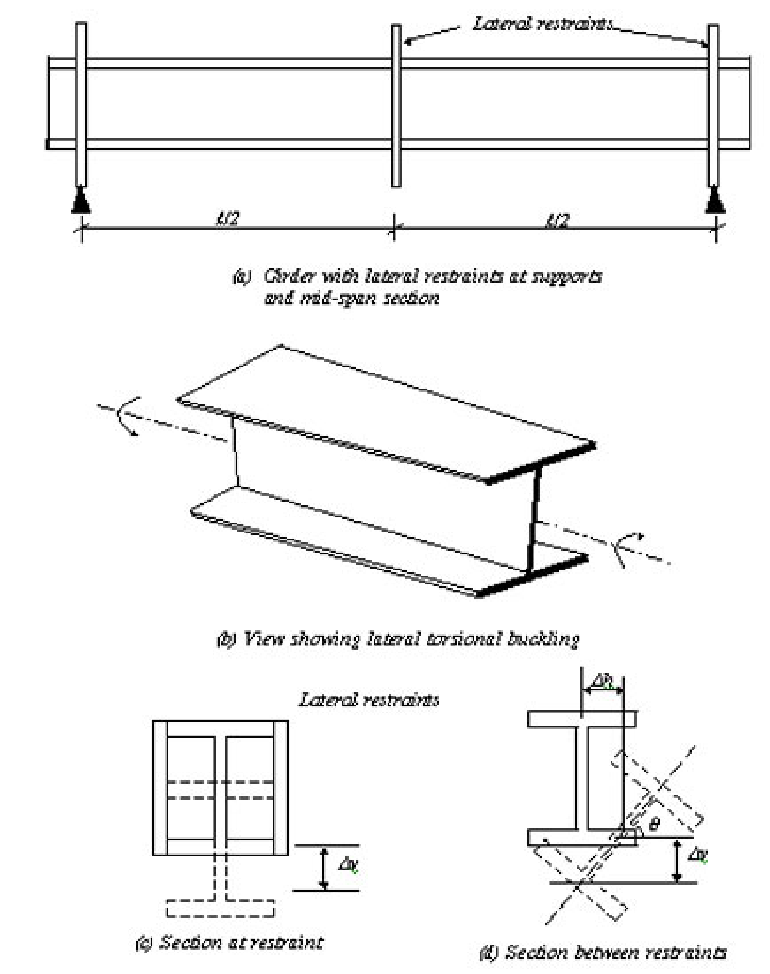

Fig.7.18 Distrosion caused by lateral torsional buckling

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, pt 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 pt 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,

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

$$\frac{p_t}{q_y} = \left[3 + \left(2.25\,\text{Sin}^2\theta - 3\right)\left(\frac{q_c}{q_y}\right)^2\right]^{1/2} - 1.5\frac{q_c}{q_y}\,\text{Sin}^2\theta \qquad \textbf{\textit{(7.8)}}$$

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

$$\frac{q_u}{q_y} = \left[\frac{q_c}{q_y} + 5.264\,\text{Sin}\theta\left(m_{fw}\frac{p_t}{f_y}\right)^{1/2} + \frac{p_t}{q_y}\left(\text{Cot}\theta - \phi\right)\text{Sin}^2\theta\right] \qquad \textbf{\textit{(7.9)}}$$

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

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

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

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

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

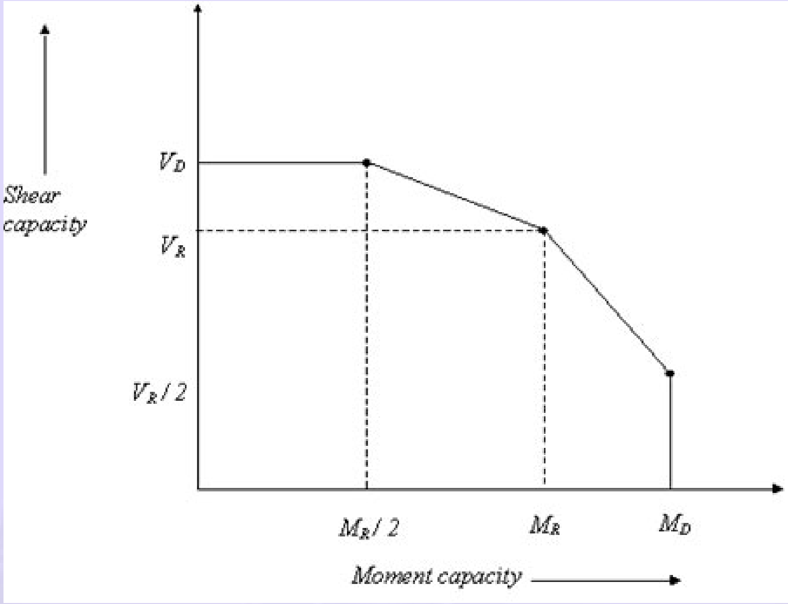

Fig.7.19 Shear-moment capacity interaction diagram

Bending and shear capacities of girders without longitudinal stiffeners can be calculated independently and then an interaction relationship as given in Fig. 7.19 is employed. In Fig. 7.19, 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.

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 favourably increase design life of plate girders.

7.6.2 Lateral bracing for plate girders

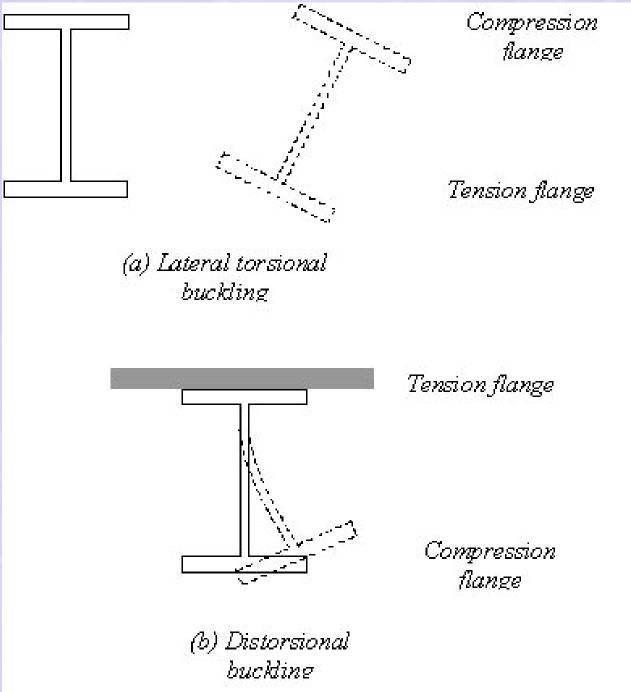

Fig.7.20 Modes of instability of plate girders

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.7.20 (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 7.20(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 Fig. 7.15(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. 7.15(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 grider bridges