Unit – III

Design of simple beams, Built-up beams, Plate girders and gantry girders.

Plate girder bridges (Riveted and welded)

Introduction

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 ranges of values for various overall dimensions of the main girders are given below: Overall depth, D: $l/18 \leq D \leq l/12$ (Highway bridges) $l/10 \leq D \leq l/7$ (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: $t \approx D/125$

(a) Half through type plate girder bridge   (b) Deck type plate girder bridge
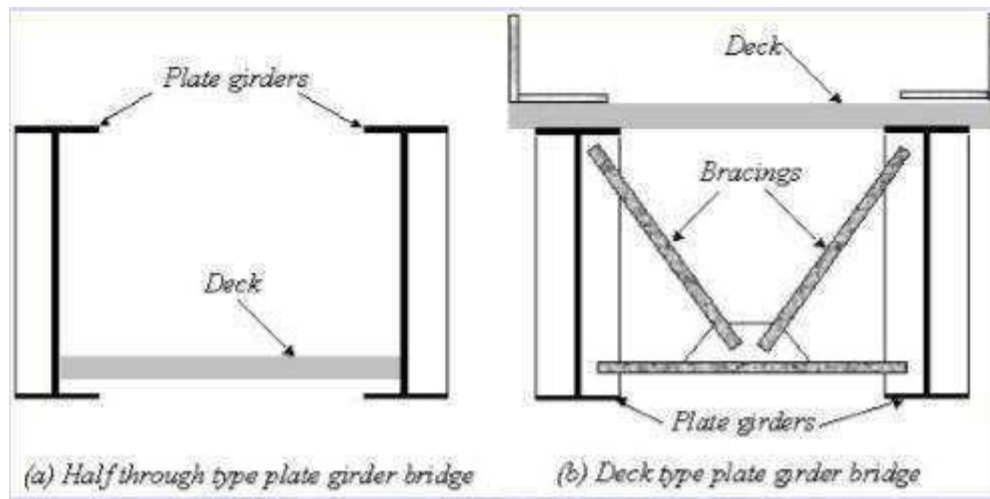
Common types of plate Girder Bridge

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.

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 is 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

fy is to be substituted in SI units (MPa).

(a) Compact sections   (b) Non-compact sections
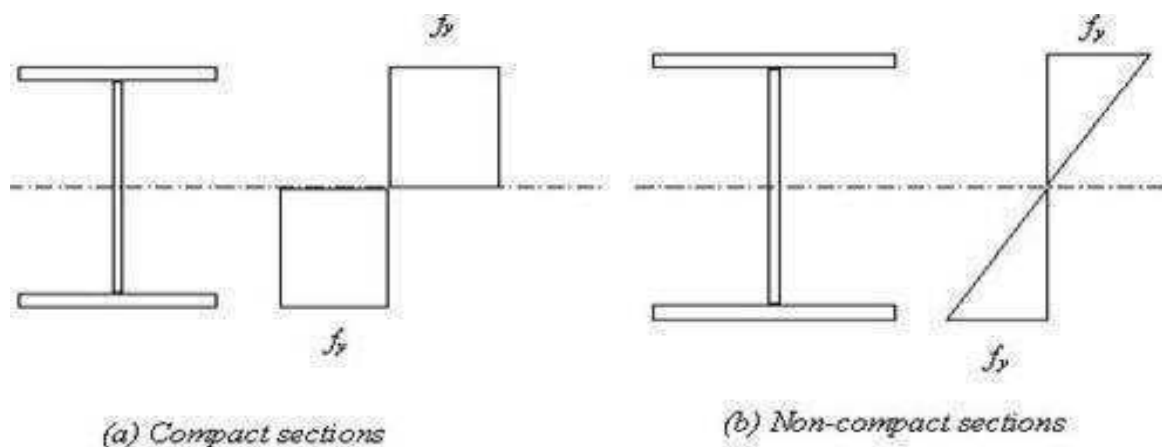
Design stresses
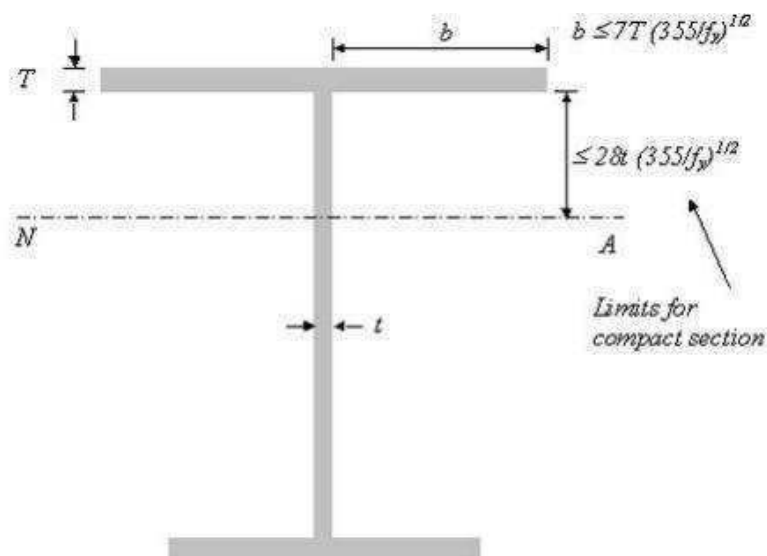
Shape limitations for plate girder

   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:

<center>M Z f / for compact sections u py m = γ</center>

<center>M Zf / for non-compact sections u ym = γ</center>

Lateral Torsional buckling

   A typical bridge girder with a portion of the span, over which the compression flange is laterally

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unrestrained, is shown in Fig. Such a girder is susceptible to lateral Torsional buckling. Fig. 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. A part of the beam between restraints can translate downwards and sideways and rotate about shear centre. 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 resists the shear in the three modes, namely

  (i)   **pure shear,**

  (ii)   **tension field action and**

  (iii)   **That due to formation of collapse mechanism.**

(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
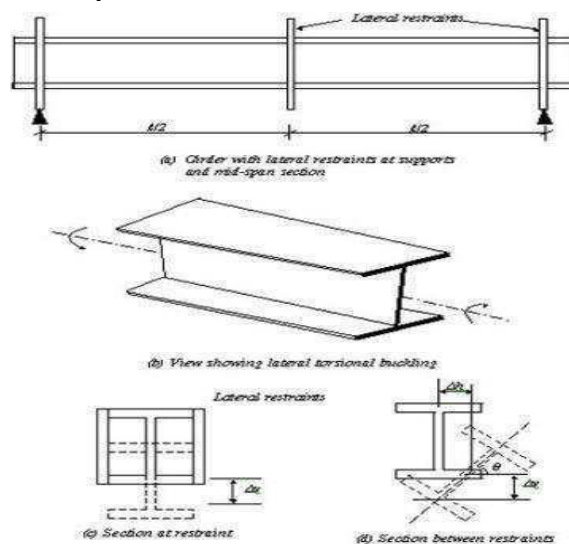
Distortion caused by lateral Torsional buckling

The elastic local buckling of the web in shear does not lead to collapse Limit State, since the web experiences stable post-buckling 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, 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.
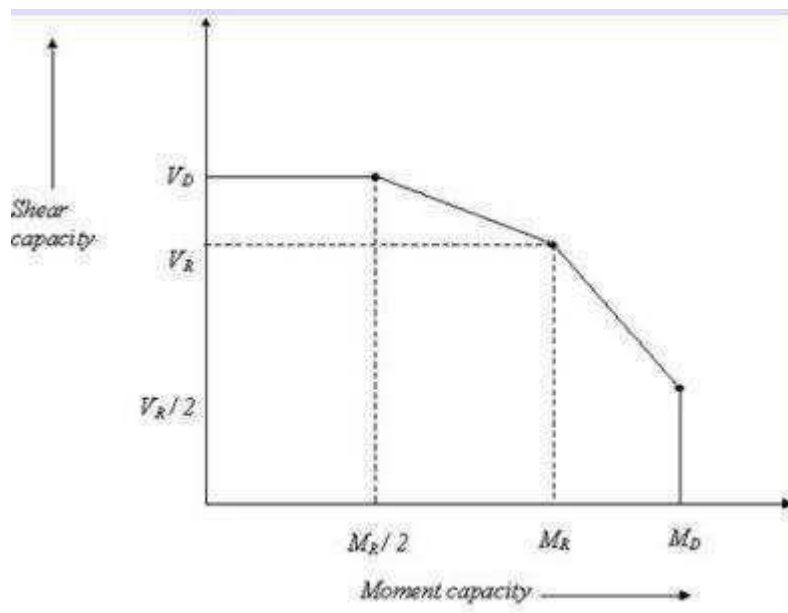
Shear-moment capacity interaction diagram

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

Lateral bracing for plate girders

(a) Lateral torsional buckling
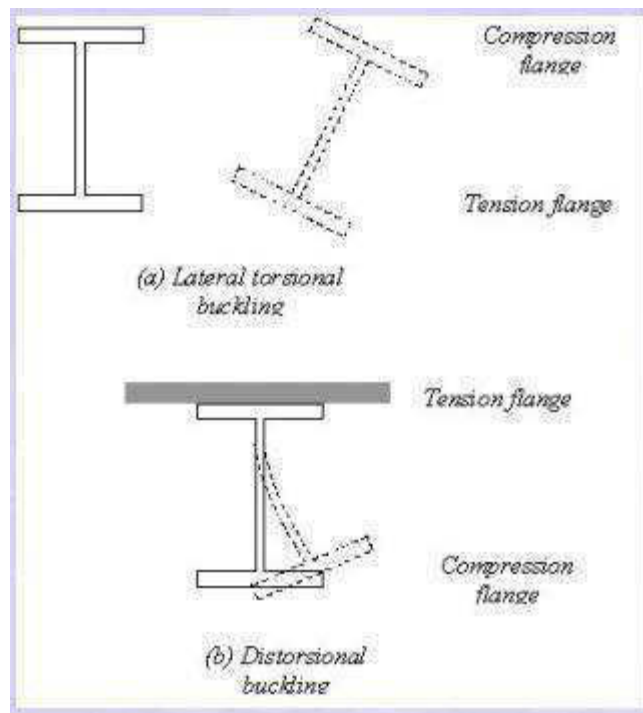
(b) Distorsional buckling

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 stabilized by the deck.

If the unrestrained flange is in compression, distortional 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. 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 Fig. 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.

Beams

One of the frequently used structural members is a beam whose main function is to transfer load principally by means of flexural or bending action. In a structural framework, it forms the main horizontal member spanning between adjacent columns or as a secondary member transmitting floor loading to the main beams. Normally only bending effects are predominant in a beam except in special cases such as crane girders, where effects of torsion in addition to bending have to be specifically considered.

The type of responses of a beam subjected to simple uniaxial bending are shown in Table 6.1. The response in a particular case depends upon the proportions of the beam, the form of the applied loading and the type of support provided. In addition to satisfying various strength limits as given in the Table, the beam should also not deflect too much under the working loads i.e. it has to satisfy the serviceability limit state also.

Recently, IS: 800, the structural steel code has been revised and the limit state method of design has been adopted in tune with other international codes of practice such as BS, EURO, and AISC. This chapter attempts to throw light on the provisions for bending members in this code.
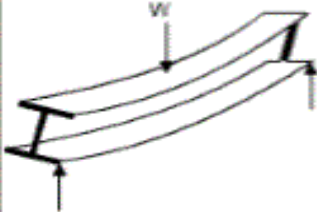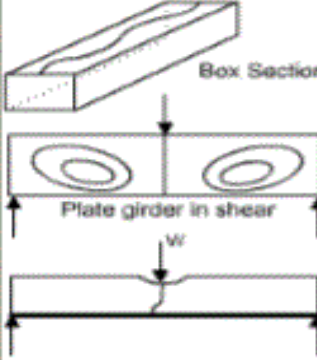
Limit state design of beams
In the working stress or allowable stress method of design, the emphasis is on limiting a particular stress in a component to a fraction of the specified strength of the material of the component. The magnitude of the factor for a structural action depends upon the degree of safety required. Further, elastic behavior of the material is assumed. The main objection to the permissible stress method is that the stress safety factor relating the permissible stress to the strength of the material is not usually the same as the ratio of the strength to the design load. Thus it does not give the degree of safety based on collapse load.

In the limit state method, both collapse condition and serviceability condition are considered. In this method, the structure has to be designed to withstand safely all loads and deformations likely to occur on it throughout its life. Designs should ensure that the structure does not become unfit for the use for which it is required. The state at which the unfitness occurs is called a limit state. Special features of limit state design method are:
   • It is possible to take into account a number of limit states depending upon the particular instance
   • This method is more general in comparison to the working stress method. In this method, different safety factors can be applied to different limit states, which is more rational than applying one common factor (load factor) as in the plastic design method.
   • This concept of design is appropriate for the design of structures since any new knowledge of the structural behavior, loading and materials can be readily incorporated.

The limit state design method is essentially based on the concept of probability. Its basic feature is to consider the possibility and probability of the collapse load. In this respect, it is necessary to consider the possibility of reduced strength and increased load.

Main failure modes of hot-rolled beams

Category	Mode		Comments
1	Excessive bending triggering collapse		This is the basic failure mode provided (1) the beam is prevented from buckling laterally,(2) the component elements are at least compact, so that they do not buckle locally. Such "stocky" beams will collapse by plastic hinge formation.
2	Lateral torsional buckling of long beams which are not suitably braced in the lateral direction.(i.e. "un restrained" beams)		Failure occurs by a combination of lateral deflection and twist. The proportions of the beam, support conditions and the way the load is applied are all factors, which affect failure by lateral torsional buckling.
3	Failure by local buckling of a flange in compression or web due to shear or web under compression due to concentrated loads	Box Section / Plate girder in shear	Unlikely for hot rolled sections, which are generally stocky. Fabricated box sections may require flange stiffening to prevent premature collapse. Web stiffening may be required for plate girders to prevent shear buckling. Load bearing stiffeners are sometimes needed under point loads to resist web buckling.

The object of design is to keep an acceptable level the probability of any limit state not being exceeded. This is achieved by taking account of the variation in strength and properties of materials to be used and the variations in the loads to be supported by the structure, by using the characteristic values of the strength of materials as well as the loads to be applied. The deviations from the characteristic values in the actual structures are allowed by using their design values. The characteristic values should be based on statistical evidence where necessary data are available; where such data are not available they should be based on an appraisal of experience. The design values are derived from the characteristic values through the use of partial safety factors, one for material strengths and the other for loads and load effects.