16 PLATE GIRDERS – II

1.0 INTRODUCTION

This chapter describes the current practice for the design of plate girders adopting meaningful simplifications of the equations derived in the chapter on Plate Girders – I, as per provisions of BS 5950: Part – 1 for buildings.

It is important to choose appropriate sections for various components of the plate girder. In these girders, the bending moments are assumed to be carried by the flanges by developing compressive and tensile forces. To effect economy, the web depth '*d*' (See Fig. 1) is chosen to be large enough to result in low flange forces for the design bending moment.

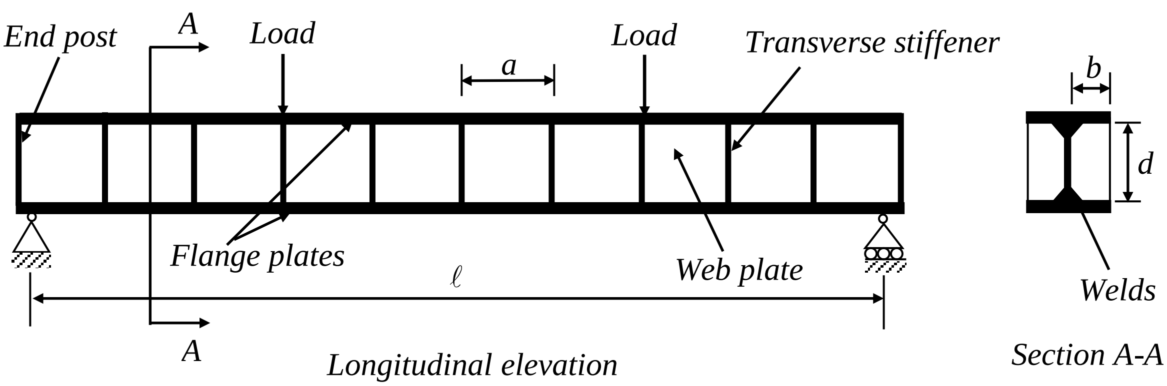


Fig. 1 A typical Plate Girder

1.1 Span to depth ratios

The recommended span / depth (ℓ/d) ratios for initial choice of cross-section in a plate girder used in a building is given below:

i.	Constant depth beams used in simply-supported composite and non-composite girders with concrete decking	$12 < \ell/d < 20$
ii.	Constant depth beams in continuous composite and non-composite girders	$15 < \ell/d < 25$
iii.	Simply-supported crane girders	$10 < \ell/d < 15$

1.2 Recommended proportions for web

When $d/t \leq 66.2\varepsilon$, where $\varepsilon = (250/f_y)^{1/2}$ the web plate will not buckle because the shear stress 'q' is less than critical buckling stress 'q_{cr}'. The design, in such cases, is similar to rolled steel beams. Here we consider plate girders having thin webs with $d/t > 66.2\varepsilon$. In the design of these webs, shear buckling should be considered. In general we may have an un-stiffened web, a web stiffened by transverse stiffeners (Fig. 1) or a web stiffened by both transverse and longitudinal stiffeners (Fig. 2).

By choosing a minimum web thickness 't', the self-weight is reduced. However, the webs are vulnerable to buckling and hence are stiffened if necessary. The web thicknesses recommended are:

i. For un-stiffened web $\qquad t \geq d/250$

ii. For stiffened web $\qquad$ with $a/d > 1$; $\qquad t \geq d/250$

and with $a/d \leq 1$; $\qquad t \geq (d/250)(a/d)^{1/2}$

where a is the horizontal spacing between the transverse stiffeners in a web of depth d and thickness t.

In practice, however, $a/d < 1$ is rarely used - if at all - in plate girders used in buildings and bridges.

To avoid flange buckling into web, BS 5950: Part - 1, specifies

i. For un-stiffened web $\qquad t \geq (d/294)(p_{yf}/250)^{1/2}$

where p_{yf} is the design stress of flange material.

ii. For stiffened web $\qquad$ with $a/d > 1.5$; $\qquad t \geq (d/294)(p_{yf}/250)^{1/2}$

and $\qquad$ with $a/d \leq 1.5$; $\qquad t \geq (d/337)(p_{yf}/250)^{1/2}$

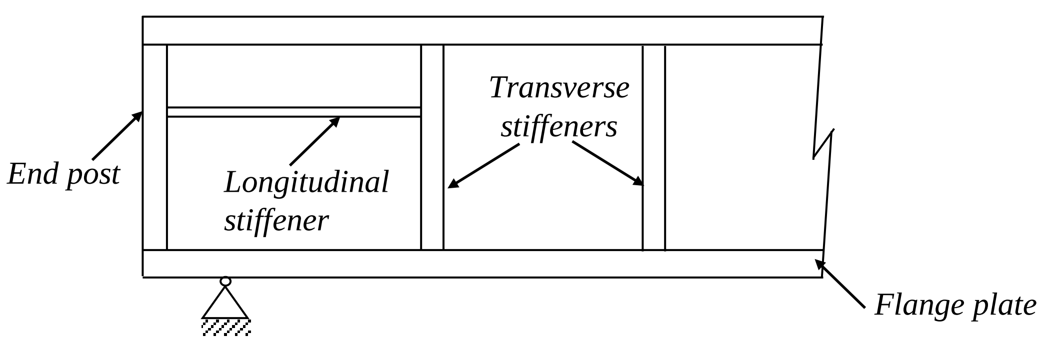


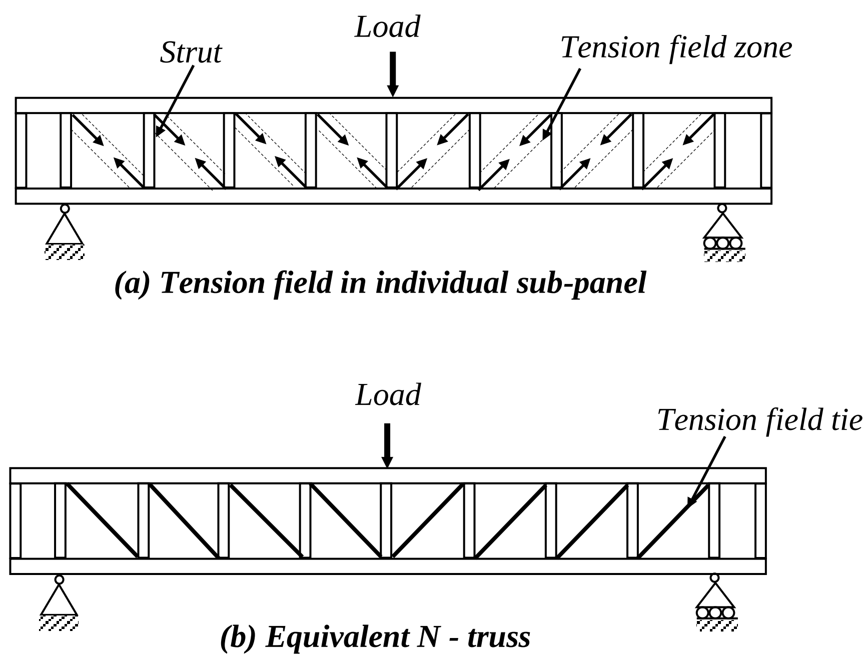
Fig. 2 End panel strengthened by longitudinal stiffener

1.3 Recommended proportion of flanges

Generally the thickness of flange plates is not varied along the spans for plate girders used in buildings. For non-composite plate girder the width of flange plate is chosen to be about *0.3* times the depth of the section as a thumb rule. It is also necessary to choose the breadth to thickness ratio of the flange such that the section classification is generally limited to plastic or compact sections only ($b/T \leq 8.9\varepsilon$). This is to avoid local buckling before reaching the yield stress. For preliminary sizing, the overall flange width-to-thickness ratio may be limited to *24*. For the tension flanges (i.e. bottom flange of a simply supported girder) the width can be increased by *30%*.

1.4 Stiffener spacing

Vertical stiffeners are provided close to supports to increase the bearing resistance and to improve shear capacity. Horizontal stiffeners are generally not provided in plate girders used in buildings. Intermediate stiffeners also may not be required in the mid-span region. When vertical stiffeners are provided, the panel aspect ratio *a/d* (see Fig.1) is chosen in the range of *1.2* to *1.6*. The web is able to sustain shear in excess of shear force corresponding to q_{cr} because of vertical stiffeners. Vertical stiffeners help to support the tension field action of the web panel between them. Where the end panel near support is designed without using the tension field action a smaller spacing of *a/d = 0.6 -1.0* is adopted. Sometimes double stiffeners are adopted near the bearing (see Fig. 3) and in such cases the overhangs beyond the supports are limited to *1/8* of the depth of the girder.



(a) Tension field in individual sub-panel

(b) Equivalent N - truss

Fig. (3) Tension field action and the equivalent N - truss

2.0 PROVISIONS FOR MOMENT AND SHEAR CAPACITY AS PER BS 5950-PART 1

Any cross section of the plate girder will have to resist shear force and bending moment. The design may be based on any one of the following assumptions:

1) The moment is resisted by the flanges and the shear is resisted by the web only.
2) The moment is resisted by the entire section while the web is designed to resist shear and longitudinal stresses due to bending.
3) A combination of (1) and (2) above by approximating a percentage of the shear to the web and remaining to the entire section. This method is rarely used.

The assumptions made in method (1) leads to mathematical simplification and a good and simple visualisation of load transfer mechanism. This method is used for computing the moment and shear capacity of a section as indicated below.

2.1 Moment Capacity

Moment capacity M_c is computed from the plastic capacity of the flanges. Thus,

$$M_c = p_{yf}\ Z_{pf} \qquad (1)$$

where, p_{yf} = The design stress of the flange steel ($= f_{yf}/\gamma_m$)
Z_{pf} = Plastic section modulus of flanges about the transverse axis of the section.
γ_m = Material safety factor for steel (= 1.15)

2.2 Shear Capacity

Thin webs are designed either with or without stiffeners. These two cases are described individually below.

2.2.1 Webs without intermediate stiffeners

The shear capacity of un-stiffened webs is limited to its shear buckling resistance. Hence,

$$V_{cr} = q_{cr}\, d\, t \qquad (2)$$

The elastic pre-buckling behaviour was described in section 2.1.1 of the previous chapter. Based on this theory the code gives the following values for q_{cr}, for webs, which are not too slender. These values depend on the slenderness parameter λ_w defined as

$$\lambda_w = (0.6\, p_{yw} / q_e)^{1/2} \qquad (3)$$

where, q_e = Elastic critical shear strength values to be used in design for different values of *a/d* and *d/t* are tabulated in Table - 1.

p_{yw} = Design strength of web ($= f_{yw}/\gamma_m$)

γ_m = Material safety factor for steel (= *1.15*)

The elastic critical stress (Refer Table 1 of the previous chapter) has been simplified and given based on *a/d* and *t/d* as given in Table - 1.

Table 1: Elastic critical stress related to aspect ratio

Aspect ratio	***Elastic critical stress***
$a/d \le 1$	$q_e = [0.75 + 1/(a/d)^2]\ [1000/(d/t)]^2$
$a/d > 1$	$q_e = [1 + 0.75/(a/d)^2]\ [1000/(d/t)]^2$

where, *a* is the stiffener spacing

d is the depth of web

Table - 2 gives the values of q_{cr} recommended by the code for design purposes.

Table 2: Elastic critical stress for design purposes

$\lambda_w \le 0.8$	$0.8 < \lambda_w < 1.25$	$\lambda_w \ge 1.25$
$q_{cr} = 0.6\ p_{yw}$	$q_{cr} = 0.6\ p_{yw}[1 - 0.8(\lambda_w - 0.8)]$	$q_{cr} = q_e$

Note that for very slender webs q_{cr} is limited to elastic critical shear stress. In other cases the value of q_{cr} is a function of design stress of web steel, p_{yw}.

2.2.2 Webs with intermediate stiffeners

Design of the plate girders with intermediate stiffeners, as indicated in Fig. 2, can be done by limiting their shear capacity to shear buckling strength. However this approach is uneconomical and does not account for the mobilisation of the additional shear capacity, as indicated earlier. The shear resistance is improved in the following two ways.

i) Increase in buckling resistance due to reduced *a/d* ratio.

ii) The web develops tension field action and thus resists considerably larger stress than the elastic critical strength of web in shear.

Fig.3 shows the diagonal tension fields anchored between top and bottom flanges and against transverse stiffeners on either side of the panel. With the stiffeners acting as struts and the tension field acting as ties the plate girder behaves similar to an N-truss [Fig. 3(*b*)]. As indicated in the previous chapter, the failure occurs when the web yields and plastic hinges form in flanges, *2* at top and *2* at bottom, developing a sway mechanism.

The full shear buckling resistance is calculated as,

$$V_b = [\, q_b + q_f (k_f)^{1/2} \,]\, d\, t \quad but \le 0.6\, p_y\, dt \qquad (4)$$

The first term q_b comprises of critical elastic stress q_{cr} and the tension field strength of the panel i.e.,

$$q_b = q_{cr} + \frac{y_b}{2\left[\frac{a}{d} + \sqrt{1+\left(\frac{a}{d}\right)^2}\right]} \qquad (5)$$

where, $y_b = (p_{yw}^{\,2} - 3q_{cr}^{\,2} + \phi_t^{\,2})^{1/2} - \phi_t$

$$\phi_t = \frac{1.5 q_{cr}}{\sqrt{1+\left(\frac{a}{d}\right)^2}}$$

a – spacing of transverse stiffeners

d – depth of girder

t – web thickness

p_{yw} – design web strength ($= f_{yw}/\gamma_m$)

Note that this is a simplified version of equation (4) derived in the previous chapter based on yield criteria.

The term $q_f(k_f)^{1/2}$ represents the contribution of the flanges to the post buckling strength and depends on plastic moment capacity of the flanges M_{pf} [Equation (8) of the previous chapter]

The flanges support the pull exerted by the tension field. When the flanges reach their ultimate capacity they form hinges. k_f is a parameter that relates to the plastic moment capacity of the flange M_{pf}, and the web M_{pw}, described later.

The flange-dependent shear strength is simplified and given as

$$q_f = 0.6\, p_{yw} \left[4\sqrt{3} \left(\frac{y_b}{p_{yw}} \right)^{1/2} \sin\frac{\theta}{2} \right] \qquad (6)$$

where, $\theta = tan^{-1}(d/a)$

When the girder is to resist pure shear, then

$$M_{pf} = \frac{2b}{4} T^2 \, p_{yf} \qquad (7)$$

However in presence of overall bending moment, the contribution of flange to shear resistance will be reduced by the longitudinal stress f induced because of overall bending moment, by the factor $(1 - f/p_{yf})$. When f approaches p_{yf} at maximum moment region, the factor nearly becomes zero and hence the contribution of flanges to shear resistance will become negligible.

The plastic moment capacity of the web, M_{pw}, is given by

$$M_{pw} = 0.25\, d^2\, t\, p_{yw} \qquad (8)$$

and

$$k_f = M_{pf} / 4\, M_{pw} \qquad (9)$$

Fig. 4 shows typical variations of shear strength with web stiffness as contributed by

a) critical shear strength
b) post buckling strength due to web tension field adequately resisted by transverse vertical stiffeners
c) the plastic moment capacity of the flanges.

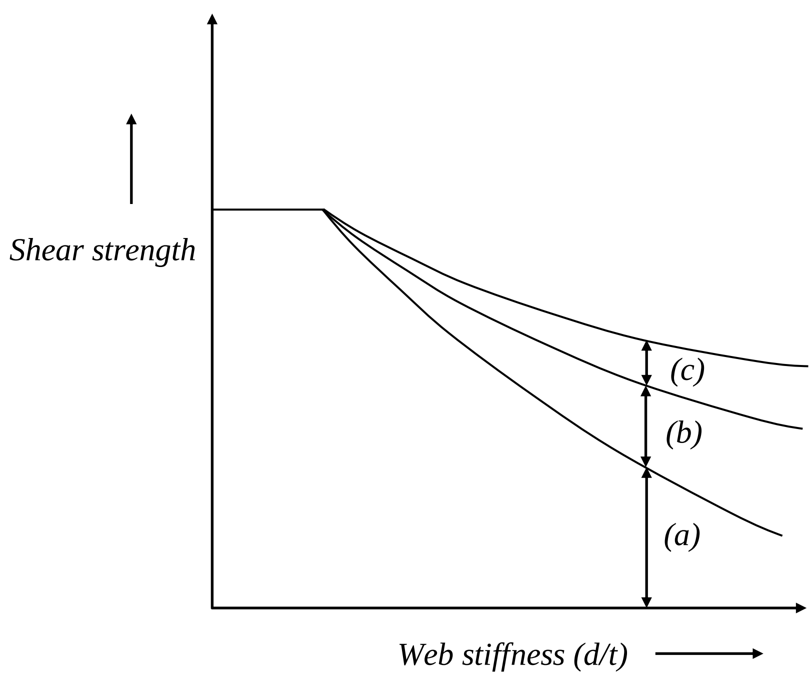


Fig. 4 Typical variation of shear strength with web stiffness as contributed by shear strength of panel, post buckling strength due to tension field and flange mechanism

3.0 END PANELS

For tension field action to develop in the end panels, adequate anchorage should be provided all around the end panel. The anchor force H_q required to anchor the tension field force is

$$H_q = 0.75\, d\, t\, p_y \left(1 - \frac{q_{cr}}{0.6 p_y}\right)^{1/2} \tag{10}$$

The end panel, when designed for tension field will impose additional loads on end post and hence it will become stout [Fig. 5(*a*)]. For a simple design it may be assumed that the capacity of the end panel is restricted to V_{cr} so that no tension field develops in it [Fig. 5 (*c*)]. In this case, end panel acts as a beam spanning between the flanges to resist shear and moment caused by H_q and produced by tension field of penultimate panel.

This approach is conservative, as it does not utilise the post-buckling strength of end panel especially where the shear is maximum. This will result in the *a/d* value of end panel spacing to be less than that of other panels. The end stiffener should be designed for compressive forces due to bearing and the moment, M_{tf}, due to tension field in the penultimate panel.

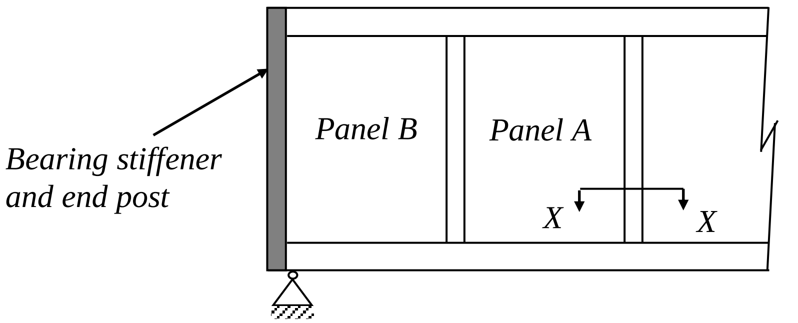
In order to be economical the end panel also may be designed using tension field action. In this case the bearing stiffener and end post are designed for a combination of stresses resulting from compression due to bearing and a moment equal to *2/3* caused due to tension in the flanges, M_{tf}. The stiffener will be stout. Instead of one stout stiffener we can use a double stiffener as shown in Fig. 5(*d*). Here the end post is designed for horizontal shear and the moment M_{tf}.

4.0 STIFFENERS

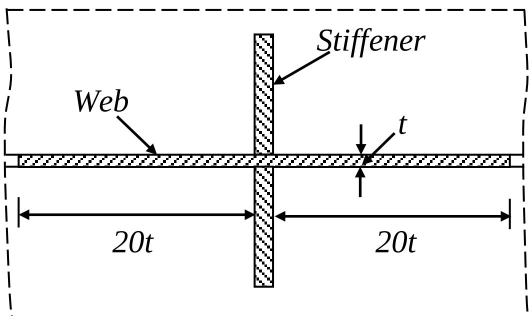
Stiffeners are provided to transfer transverse concentrated compressive force on the flange into the web and are essential for desired performance of web panels. These are referred to as bearing stiffeners. Intermediate web stiffeners are provided to improve its shear capacity. Design of these stiffeners is discussed below.

4.1 Load bearing stiffeners

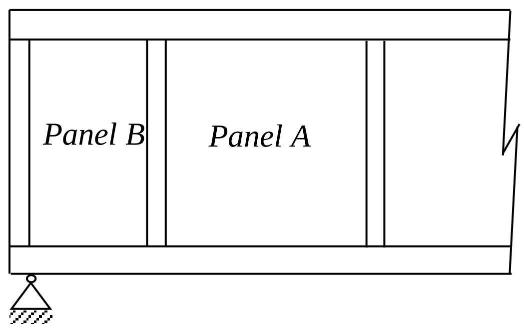
Whenever there is a risk of the buckling resistance of the web being exceeded, especially owing to concentrated loads, load-bearing stiffeners are provided. Normally a web width of *20 t* on both sides as shown in Fig. 5 (*b*) is assumed to act along with the stiffener provided to resist the compression as an equivalent cruciform shaped strut of effective length 0.7 times its actual length between the top and bottom flanges. The bearing stress in the stiffener is checked using the area of that portion of the stiffener in contact with the flange through which compressive force is transmitted.



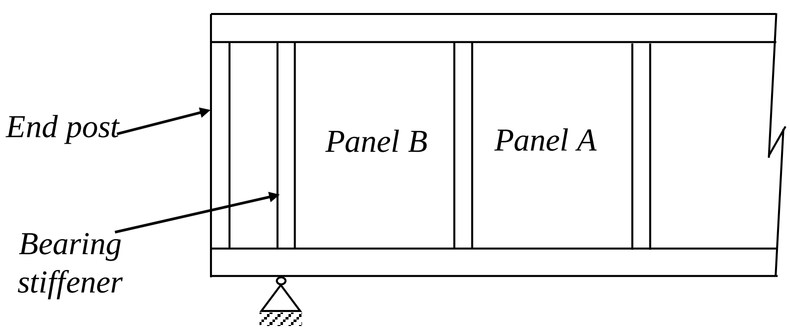
(a) End panel designed using tension field action and end post designed for both bearing and to resist tension field



(b) Section at X-X



(c) End panel designed without using tension field action



(d) End panel designed using tension field strengthened by additional stiffener (Double stiffener)

Fig. 5 Various treatments for end panel

4.2 Intermediate stiffeners

The intermediate stiffeners are provided to prevent out of plane buckling of web at the location of stiffeners. The buckling resistance P_q of the stiffener acting as a strut (with a cruciform section as described earlier) should be not less than $(V_t - V_s)$ where V_t is the maximum shear force in the panel and V_s is the buckling resistance of web without considering tension field action. In its limit V_s will be equal to V_{cv} of the web without stiffeners.

Sometimes the stiffeners are provided for more than one of the above purposes. In such cases stiffeners are considered for their satisfactory resistance under combined load effects. Such combined loads are common.

4.3 Longitudinal stiffeners

Longitudinal stiffeners are hardly used in building plate girders, but sometimes they are used in highway bridge girders for aesthetic reasons. They are not as effective as transverse stiffeners. Nowadays, the use of longitudinal stiffeners is rare due to welding problems.

For design of longitudinal stiffeners there are two requirements:

- A moment of inertia to ensure adequate stiffness to create a nodal line along the stiffener
- An area adequate to carry axial compression stress while acting integrally with the web.

5.0 CURTAILMENT OF FLANGE PLATES

For a plate girder subjected to external loading, the maximum bending moment occurs at one section usually, e.g. when the plate girder is simply supported at the ends, and subjected to the uniformly distributed load, then, maximum bending moment occurs at the centre. Since the values of bending moment decreases towards the end, the flange area designed to resist the maximum bending moment is not required at other sections. Therefore the flange plates may be curtailed at a distance from the centre of span greater than the distance where the plate is no longer required as the bending moment decreases towards the ends. It gives economy as regards to the material and cost. At least one flange plate should be run for the entire length of the girder.

6.0 SPLICES

6.1 Web splices

A joint in the web plate provided to increase its length is known as web splice. The plates are manufactured up to a limited length. When the maximum manufactured length of the plate is insufficient for full length of the plate girder, web splice becomes essential. It also becomes essential when the length of plate girder is too long to handle conveniently during transportation and erection. Generally, web splices are not used in buildings. They are mainly used in bridges.

Splices in the web of the plate girder are designed to resist the shear and moment at the spliced section. The splice plates are provided on each side of the web.

6.2 Flange splices

A joint in the flange element provided to increase the length of flange plate is known as flange splice. The flange splices should be avoided as far as possible. Generally, the flange plates can be obtained for full length of the plate girder. In spite of the availability of full length of flange plates, sometimes it becomes necessary to make flange splices. Flange joints should not be located at the points of maximum bending moment.

7.0 CONCLUSIONS

This chapter has outlined the procedure for design of plate girders as specified in BS 5950: Part - I for buildings. It shows how the reserve strength due to post buckling behaviour explained in Plate Girder I chapter can be advantageously used by the designer without performing mathematically involved calculations.

8.0 REFERENCES

1. Narayanan. R: Plate Girders, Steel Designer's Manual (Fifth Edition). The Steel Construction Institute UK 1992.

2. Evans H.R: Introduction to Steelwork Design to BS 5950 Part 1 SCI Publication 069. The Steel Construction Institute UK 1988.