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A Study on Web Buckling Of Plate Girders

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***Abstract:* -- The plate girder is fabricated from plates and the designer has greater freedom to vary the section to correspond with changes in the applied forces. Thus variable depth plate girders have been increasingly designed in recent years. For a given bending moment the required flange areas can be reduced by increasing the distance between them.Thus for an economical design, it is advantageous to increase the distance between flanges. To keep the self-weight of the girder as the minimum, the web thickness should be reduced as the depth increases, but this leads to web buckling. Web buckling considerations being more significant in plate girders than in rolled beams since Rolled beam section are manufactured by keeping depth to thickness ration such that there will be no local bucking in the web. The web buckling of plate girder can be avoided by using the thicker web or by using stiffeners.In this paper feasibility of using stiffeners to reduce the dead load, material and fabrication cost of plate girders is studied. Use of transverse stiffeners lead to creating tension field in the plate girders prior to buckling and this tension field helps to increases the buckling resistance of web. In this paper as per IS: 800:2007 design of plate girder is done. The optimum section for the given load is found out by varying the thickness of web and number of stiffeners in different trials. Finally, the most suitable section for a given load is proposed in this paper.**

***Keywords* :— Web buckling, Tension field action, Shear strength.**

I. INTRODUCTION

In modern construction practices, the use of plate girders become an increasingly popular, especially where structural elements are subjected to heavy loads and clear span required is large. More moment of resistance is required if section is subjected to heavy load. This can be achieved by providing truss girder, built-up section and plate girders. The fabrication and erection cost of truss girder is more if it is used for light load and short span. Plate girder have moment of resistance between rolled I section and truss girder and it can be built to any desired proportion.

The flange provides more flexural strength therefore most of the steel must be concentrated in flange and flange is kept away from the neutral axis of girder, this results in deep and thin web. Such slender web is susceptible to problem like local instability. There is always a possibility that the thin web get buckled before yielding. The three types of buckling to be considered are: shear or diagonal buckling, bending or longitudinal buckling and bearing or vertical buckling. Slender webs in plate girders are prone to local and shear buckling and to prevent this, stiffeners are provided. Provisions of intermediate transverse stiffeners serve boundaries for tension field action in the web. To make the web more stable along with transverse stiffeners, sometime longitudinal stiffeners are also provided. Various Studies have been carried out in the past in order to determine the number, dimensions, and positioning of the stiffeners in a particular web panel for optimum performance of steel plate girders. However, much less effort has been devoted to optimisation of the section which seems to require more attention to get more economical section. Once safe section for the particular loading is achieved, It can be optimized by providing holes in the webs at certain location. This paper is intended to show that decrease in material and cost of plate girder may be achieved by accurate proportion of depth to thickness ratio of web, height to width ratio of web panel and by using the tension field action.

II. OUTLINE

The steel plate girders were designed based provisions given in IS: 800:2007. The design is done such that the girder would not fail in lateral torsional buckling. A number of trial is done for depth of girder (d), stiffeners spacing(c) and thickness of web (t). 20 m span beam is taken, over which 100kN/m uniformly distributed load is placed. Some basic dimensions were kept the same in all the girders. Only d/t and c/d ratios are varied in different trials. Flange width, bf

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=450 mm, flange thickness, tf =35 mm, depth of girder, df = 1470 mm.
Optimum depth of plate girder can be found out by assuming that the entire bending moment is resisted by flange.

$$M_z = f_y b_f t_f d \quad \text{........ (eq.1)}$$

The gross sectional area is given by-

$$A = 2b_f t_f + dt_w \quad \text{......... (eq.2)}$$

Eliminating, $b_f t_f$ in above equation we will get,

$$A = \frac{2M_z}{kt_w f_y} + kt_w^2 \quad \text{...... . (eq.3)}$$

Now, the optimum value of thickness may be obtained by differentiating the above equation w.r.t t_w and equating it to zero.

$$t_w = \left(\frac{M_z}{f_y k^2}\right)^{0.33}$$

The optimum value of d may be obtained by differentiating eq.3 w.r.t d and equating it to zero.

$$d = \left(\frac{M_z k}{f_y}\right)^{0.33}$$

As per IS : 800 :2007 critical buckling stress is given by

$$\tau_{cr,e} = k_v \frac{\pi^2 E}{12(1-\mu^2)\left(\frac{d}{t_w}\right)^2}$$

Where, $k_v =$ shear buckling coefficient
The elastic critical shear stress increases if the value of d/t is sufficiently low. The web will yield under shear before buckling.

$$k_v = \begin{cases} 4 + \dfrac{5.35}{\left(\frac{c}{d}\right)^2} & for\ \dfrac{c}{d} < 1.0 \\ 5.35 + \dfrac{4.0}{\left(\frac{c}{d}\right)^2} & for\ \dfrac{c}{d} \geq 1.0 \\ 5.35\ when\ transvers\ stiffners \\ \ at\ sup\,port\ are\ provided. \end{cases}$$

III. DESIGN OF PLATE GIRDER

Keeping the depth constant, the minimum thickness of web is found out which can resist the load without intermediate stiffeners. Table-1 shows the results of different trials. For the given loading maximum shear force is 1000 kN and the un-stiffened web will be safe against buckling if 12 mm thick web is provided.

Table-1

tw (mm)	d/tw	$\tau_{cr,e}$ (kN/mm^2)	Resisting Force (KN)	BUCKLING
6	233.34	17.77	149.32	UNSAFE
8	175	31.6	353.95	UNSAFE
10	140	49.38	691.32	UNSAFE
12	116.66	71.1	1194.6	SAFE

Now, if 12 mm thick web is provided in the plate girder, the girder will be safe against buckling but the whole section may not be economical. Therefore the 6mm,8mm,10mm thick web with stiffeners should also be checked.

3.1. Spacing of Stiffeners.
If 6mm, 8mm or 10mm web is provided the section will not safe in buckling therefore web for thickness less than 12 mm should be stiffened. the spacing of stiffeners is decided by number of trials using simple post critical method.
If 6mm, 8mm and 10mm web is used then the minimum spacing is required to make section safe against local buckling is Shown in table -2.

Table-2

tw	c	c/d ratio	Kv	Shear Force (KN)	BUCKL-ING
6	840	0.6	18.86	526.44	UNSAFE
6	560	0.4	37.43	943.59	UNSAFE
6	518	0.37	43.07	1014.41	SAFE
6	490	0.35	47.67	1062.52	SAFE
8	980	0.7	14.91	987	UNSAFE
8	910	0.65	16.66	1102.41	SAFE
10	1820	1.3	7.71	997.17	UNSAFE
10	1792	1.28	7.79	1006.8	SAFE

Therefore optimum spacing of stiffeners, for 6 mm thick web 518 mm, 8mm web 910 mm and 10 mm thick web 1792mm.

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Now, once the spacing is fixed than end panel is design and the thickness of end stiffeners and intermediate stiffeners is decided.

3.2.End panel and stiffeners design

End panel is designed for the minimum value of c, for which the web is safe. Table -3 shows the result or end panel design. Here it is observed that if 6mm thick web is use with stiffeners at 512mm, the anchorage force in end panel is very large and the end panel fails because of moment due to anchorage force. The stiffeners are not designed as load

Table-3

tw	Vp	Rtf	Anchorag force	Mtf	Mq .(kNm)	moment due to anchorage force
6	1212	306.2	safe	85.7	60.9	unsafe
8	1617	569.8	safe	159.	188.2	safe
10	2021	894.6	safe	250.	729.8	safe

Carrying element. The minimum thickness of stiffeners required is calculated in number of trial.
Table 4 represents the value of minimum thickness of stiffeners for the particular web thickness.

Table-4

Thickess Of web	Is(req) (10^5) mm4	b of ISF	t of ISF	Iprovided (10^5) mm4	Buckling Force (kN)	Resisting Eqd
6	33.4	100	10	33.33	406.1	482.1
8	22.1	100	8	26.66	299.39	661.3
10	12.3	90	6	14.5	326.1	886.4

It is observed that for the given condition most economical section is web of 10mm thickness with intermediate stiffeners.

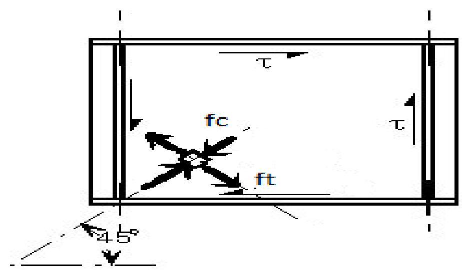
Table-5

thickness (mm)	No. Of stiffeners	total material (m^3)	Weight (kg)
6	39	0.92	7230.5
8	22	0.91	7210.3
10	11	0.95	7416.4
12	-	0.97	7583.1

Table 5, shows the number of stiffeners required per face of plate girders, total weight and total material required. The material and weight are calculated excluding the fabrication work.

IV. TENSION FIELD ACTION

Fig.1 illustrates the concept of tension field action. At the time of buckling, web losses its ability to support the diagonal compression. The vertical component of this diagonal stress resisted by stiffeners and the horizontal component is resisted by flange. The web only resist the diagonal tension, hence it is termed as tension field. IS 800:2007 gives the following expression for computing the shear resistance of web due to tension field action if intermediate and end stiffeners are provided. Tension field comes into picture only when the web begins to buckle. Therefore the total strength will be the post –buckling strength



Now, the above problem is solved by using tension field action.

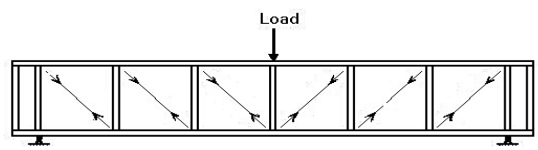


Fig. 1 Tension field action.

$$V_n = V_{tf}$$

$$V_{tf} = (A_v \tau_b + 0.9 W_{tf} t_w f_v \sin\phi) \le V_p$$

$$f_v = \sqrt{(f_{yw}^2 - 3\tau_b^2 + \psi^2)} - \psi$$

$$\psi = 1.5\tau_b \sin 2\phi$$

$$\phi = \tan^{-1}\left(\frac{d}{1.5}\right).$$

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$$\tau_b = buckling\ strength\ as\ obtained\ in\ simple\ post\ critical\ method$$

$$f_v = yield\ strength\ of\ tension\ field$$

Table-6 Check for buckling stress for different c/d ratio using tension field method.

dw	tw	c	c/d ratio	Kv	Wtf	fv	ϕ	Nf	Mfr	S	Vtf (KN)	BUCKLING
1400	6	1120	0.8	12.36	138.9	197.00	51.34	3484.32	1808697	88.94	460.35	UNSAFE
1400	6	840	0.6	18.86	138.9	180.44	59.04	3484.32	1808697	80.99	642.49	UNSAFE
1400	6	560	0.4	37.44	138.9	159.48	68.20	3484.32	1808697	74.80	1054.65	SAFE
1400	6	518	0.37	43.08	138.9	158.50	69.70	3484.32	1808697	74.05	1125.91	SAFE
1400	8	1120	0.8	12.36	120.3	165.02	51.34	3484.32	1808697	77.02	929.30	UNSAFE
1400	8	980	0.7	14.92	120.3	154.94	55.01	3484.32	1808697	73.42	1096.94	SAFE
1400	10	2170	1.55	7.01	107.6	176.70	32.83	3484.32	1808697	99.23	999.22	UNSAFE
1400	10	2128	1.52	7.08	107.6	175.67	33.34	3484.32	1808697	97.88	1008.54	SAFE
1400	10	1820	1.3	7.72	107.6	167.21	37.57	3484.32	1808697	88.23	1095.89	SAFE

Where, ϕ is the inclination of tension field, Mfr is the reduced plastic moment capacity of the respective flange after account the axial force Nf in the flange, due to overall bending and any external axial force in the cross section. Wtf is width of tension field, Vtf is the nominal shear resistance

Table-7 End panel design result using tension field method

tw	c	c/d ratio	d/tw	Kv	Vp	Vcr	Hq	Rtf	achorage force	Mtf	Mq ,(kNm)	moment due to anchorage force
6	518	0.37	233.33	43.08	1212.4	1014.4	612.4	306.2	safe	85.75	60.98	unsafe
8	1050	0.75	175.00	13.51	1616.5	893.90	1351.8	675.5	safe	189. 5	334.09	safe
10	2142	1.53	140.00	7.06	2020.7	912.13	1870.0	935.4	safe	261.9	1737.94	safe

Table-8 End stiffeners design using tension field method.

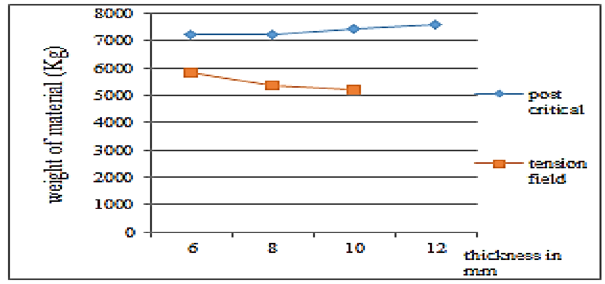
Ftf	Area,(mm2)	width of stiffeners	thickness of stiffeners	Aeff	single flate area	(Pd) load on stiffeners	buckling safety	bearing capacity	
1110.4	3908	140.0	16.00	4672	2240	1149.9	safe	1136.4	safe
1120.1	3943	140.0	16.00	4736	2240	1165.7	safe	1136.4	safe
1081.5	3807	140	16	4800	2240	1181.5	safe	1136.4	safe

V. RESULT COMPARISON

Table 10

Thickness (mm)	No. Of stiffeners	total material (m^3)	Weight (kg)
6	39	0.74	5834.81
8	19	0.68	5356.75
10	9	0.66	5185.30

Above table shows material and number of stiffeners are required if tension field is used.



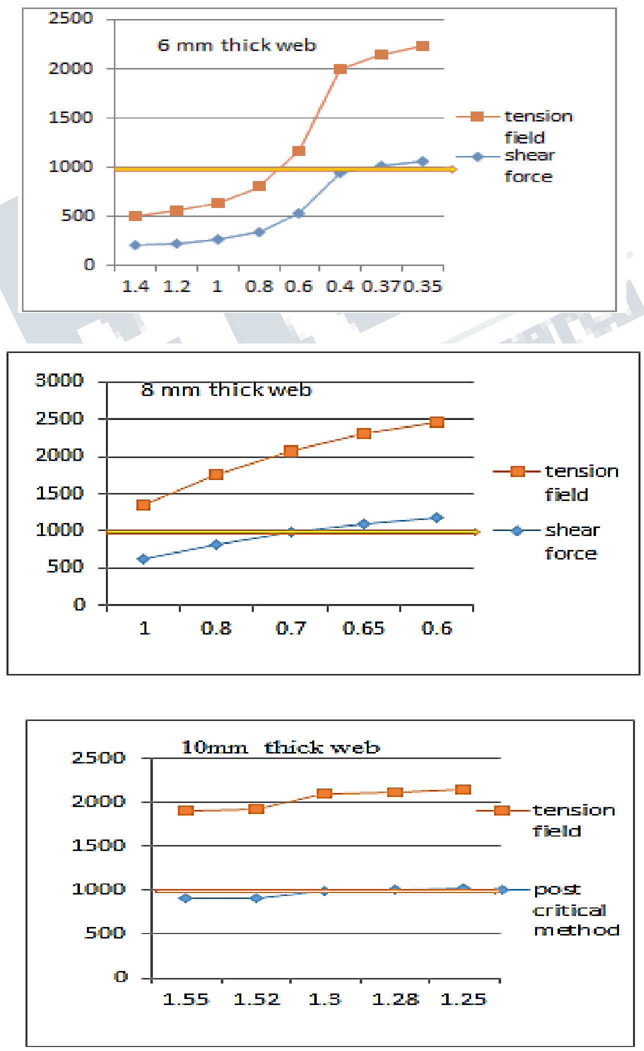


Fig.3 shear resistance vs c/d ratio graph shows the comparison of the result obtained from tension field method and simple post method for 6mm, 8mm and 10mm thick.

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VI. CONCLUSION

Plate girder is designed for a particular load. The depth of girder kept constant and Number of trial has been done by changing thickness of web and spacing of stiffeners. By result it is observed that if action of tension field is ignored than the most economical section is girder of 8mm thick web with intermediate stiffeners. If advantage of tension field action is taken then section with 10mm thick web is more economical. Whereas overall the most economical section comes out to be 10 mm thick web with intermediate stiffeners. It can be concluded that using tension field method we can obtain most economical section for plate girder.

REFERENCES

1. IS : 800 : 2007, Indian standard general construction in steel-code of practices.

2. Basler.K (1961), Strength of plate girders in shear. ASCE 128 (2) (1961).

3. Rahal.K.N,(2010) "Transversely Stiffened Girder Webs Subjected to Shear Loading" Part. 1:Design,Proceedings of In situation of Civil Engineers,Part2, 1989,pp.67–87. Alinia M.M .(2003) "Shear failure characteristics of steel plate girders". Thin-Walled Structure 2009;47:1498–50