

Design and Study of Plate Girder by Tension Field Theory

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ABSTRACT: The steel plate girder is a widely used structural element in many fields of application mainly in long span railway bridges because of its numerous favourable properties. Plate girders are usually designed for bending and shear. Plate girders have a moment resisting capacity between rolled I-sections and truss girders. Plate girders are built up flexural members. Their bending capacity can be increased by increasing the distance between the flanges. This also increases the shear capacity as the web area increases. Economy can only be achieved if the self-weight of plate girder is minimum. To achieve this, the web thickness should be reduced, but this will lead to web buckling. Here web buckling considerations become more important for plate girders when compared with rolled beams. The rolled beams are manufactured with sufficient thickness of web so that it will not fail in local web buckling. The design of plate girder is like the design of ordinary beam design: the proportioning of a member with an adequate section modulus to resist bending, thickness of web capable of resisting shear and providing sufficient stiffness. This stiffness can be achieved by placing the vertical stiffeners at appropriate intervals throughout the length of the beam. In this paper, feasibility of using transverse vertical stiffeners to reduce the weight of plate girder and fabrication cost have been studied. The ratio of spacing between the transverse stiffeners to the depth of the web (c/d ratio) is varied for different types of railway bridge loadings. Creation of tension field effect in plate girders prior to web buckling is achieved by placing transverse stiffeners, thereby avoiding the failure of web buckling of web. In this paper, as per IS 800: 2007 design of plate girder is done. The optimum section is found for different types of bridge loadings by varying the thickness of web and c/d ratio in different trails. Finally, minimum usage of material for different loadings is proposed in this paper.

KEYWORDS: Plate girder, transverse stiffener, tension field theory, moment and shear capacity.

I. INTRODUCTION

A flexural member is designed usually for bending and occasionally for shear. Plate girders have a moment resisting capacity between rolled I-sections and truss girders. Plate girders are built up flexural members. Their bending resistance can be increased by increasing the distance between the flanges. This in turn also increases the shear resistance as the web area increases. Plate girders are generally provided in bridges and also very common in buildings where heavy concentrated loads (from upper storey columns etc.,) act on a long span beam. e.g., a dining hall floor beam of a restaurant requiring a clear span all through are perhaps the most unpredictable and devastating of all natural disasters.

Real difference between plate girder and beam is that, the designer has greater control over the dimensions of the section and may make the web thinner in proportion to the depth. But, web buckling is of great concern. The designer has the choice of making the web thick enough to avoid premature buckling. This often defeats the purpose of plate girder construction. Alternatively, placing the intermediate stiffeners along its length, thereby dividing it into small panels and reducing the span of plate girder, increasing its web buckling resistance.

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At the location of high shear in a girder web, usually near the support and at or near the neutral axis, the principal planes will be inclined with respect to the longitudinal axis of the member, and the principal stresses will be diagonal tension and diagonal compression. The diagonal tension poses no problem, but the diagonal compression causes the web to buckle. This problem can be addressed in one of the ways: Depth to thickness ratio of the web can be made small enough so that the problem is eliminated. Web stiffeners can be used to form panels with increased shear strength. Web stiffeners can be used to form panels that resist the diagonal compression through tension field theory. At present the steel plate girders are most widely used as the heavy load resisting system. Various studies have been carried out in the past in order to determine the dimensions, positioning and number of stiffeners in a particular web panel for optimum performance of steel plate girders. However, in the present study much less effort has been devoted for optimization. More attention is given to get more economical section.

II. RELATED WORK

Hassanein M.F and Kharoob O.F. (2010), studied about the shear behavior of transversely stiffened hollow tubular flange plate girders with rectangular tubes for both the compression and tension flanges. A parametric study is carried out on simply supported hollow tubular flange plate girders regarding the span length, the flange cross-section dimensions, and the thickness and the aspect ratio of the web. The results of the design equations specified by the European specifications are highly conservative compared to the results of the finite element models. The results proved that decreasing the web plate slenderness makes the plastic hinges nearer to the supports. The deformed shapes displayed how the top and bottom hollow tubular flanges deform in a way similar to a simple beam with a slight difference in the initial rotation between them; the top flange relatively rotates more than the bottom one. This differential rotation results in a shear deformation near the support, which finally leads to the formation of shear plastic hinges.

Bedynek A.et al. (2013), presented the paper on “studied a numerical study on the lateral–distortional buckling (LDB) behavior and ultimate strength of hollow tubular flange plate girders (HTFPGs) with slender unstiffened webs. Using shell finite element models, a comparative study between the HTFPGs with unstiffened webs and their equivalent I section plate girders (IPGs, with unstiffened and stiffened webs) was presented. A finite strip model was used to determine the length range in which the LDB mode of HTFPGs is relevant. Then, a simplified model with 4 dof was developed and analytical formulas were proposed. The most relevant parameters influencing the response of the HTFPGs were taken into account in this study:(i) the flange width-to-depth ratio (ii) web slenderness and (iii) span length (L). Nonlinear shell finite element analyses were performed to evaluate the ultimate strength of HTFPGs.

Darehshouri S. F. et al. (2017),studied about the ultimate shear capacity of steel-concrete composite plate girders in which one of the flanges of the steel girder is connected to reinforced concrete slab through shear connectors so that the steel girder and the slab act compositely as a single unit. The analysis of such girders becomes complex and it is necessary to have a simple method for design office use. Therefore, proposed an analytical method to determine the shear strength of a web panel in the girder. As a result, larger tension forces in the post-buckling stage can be carried out. It was noted that variations in load-deflection curves can be observed for all test girders. A sudden drop of applied load soon after the ultimate load was reached can be observed in all load-deflection curves were presented. Results for the collapse behavior have been presented for all the girders. In many cases, the girders swayed to one side with tremendous buckling at the web and top flanges.

Azmi M.R. et al (2017) presented the ultimate load behaviour of perforated steel plate girders with inclined stiffeners. Ten simply supported plate girders were tested to failure subjected to a single concentrated load applied at the centre of the girder span. The circular shape of web openings of various diameters, viz., 100 mm, 150 mm, 200 mm and 250 mm were located at the centre of every web panel in the girder specimens. The main focus of this study was to investigate the effects of different inclination angles of intermediate stiffeners on the post-buckling behaviour. The inclination angles of intermediate stiffeners measured from the bottom flange, viz., 90, 75, 60, 45 and 30 were considered in the test series. Due to the effects of such inclinations, the variations of strength, failure characteristic and load-

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Vol. 8, Issue 2, February 2019

deflection response were investigated. Test results showed significant increases in the ultimate strength to the extent of 92% as the angle of the inclined stiffeners reduced.

III. EXPERIMENT RESULTS

At present study, a simply supported beam of length 20 m is considered. The dimensions considered are as follows:

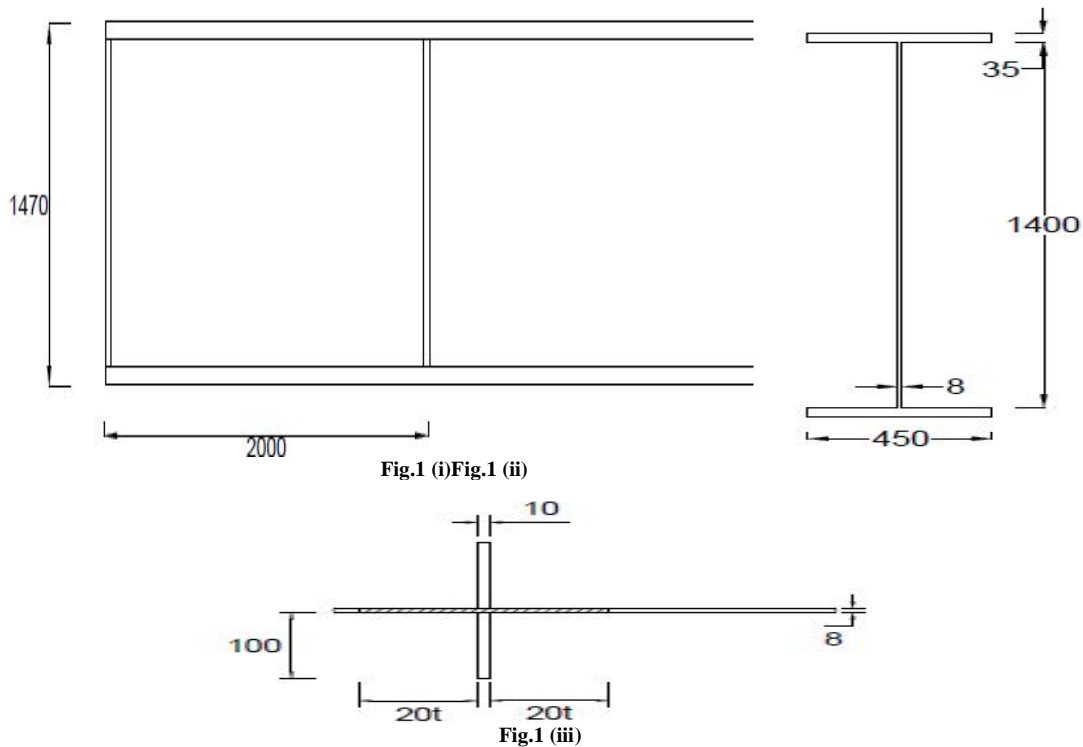


Fig.1 Dimensions of plate girder

Table.1 illustrates the dimensions of the plate girder considered

Plate girder span	20m
Web thickness	8mm
Web height	1400mm
Width of flange	450mm
Thickness of flange	35mm
Vertical stiffener thickness	10mm

COMPARISON OF DESIGN OF PLATE GIRDER WITH VERTICAL STIFFENERS (IS 800: 2007)

For this study different loading are taken from Bridge rules code. Three arbitrary loading are taken (i.e Modified broad gauge loading-1987, Heavy mineral loading, DFC loading). The loading for Modified broad gauge loading-1987,

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Vol. 8, Issue 2, February 2019

Heavy mineral loading, DFC loading are 108.4kN/m, 140.35 kN/m and 147.13kN/m respectively. These arbitrary loading are shown in table 5.

Table. 2 Different bridge loadings and corresponding shear force in considered beam

DIFFERENT BRIDGE LOADINGS	LOADING (kN/m)	SHEAR FORCE (kN)
Modified broad gauge loading(1987)	108.4	1084
heavy mineral loading	140.35	1403.5
DFC loading	147.13	1471.3

For the load of 108.4kN/m, 140.35 kN/m and 147.13kN/m the maximum shear force are 1084kN, 1403.5kN and 1471.3kN respectively. To resist the shear force without stiffeners, 12 mm thick web plate is required in case of for Modified broad gauge loading-1987; 16 mm thick web is required for Heavy mineral loading and DFC loading. But usually web with thick web is uneconomical therefore the plate girder with web thickness i.e 6mm, 8mm and 10mm and 12mm is also design with vertical stiffeners and finally the comparison is done on the basis of material required in each cases.

As per IS: 800:2007 there are two method for the design of plate girder with vertical stiffeners i.e simple post critical method and tension field method. For this study only tension field method is used. Each thickness of web is checked for different spacing of vertical stiffeners and the minimum spacing is calculated which resist the maximum shear force. The result is shown in table 3, table 4 and table 5.

Table 3: Shear Resistance of Web for Different c/d Ratio Using Tension field Method (Modified broad gauge loading-1987)

dy	tw	dy/tw	c	c/d ratio	K_x	f_{crx}	λ_{cr}	λ_{cr}	A_v	A_{cr}	ϕ	ψ	s	V_{cr}	f_v	$0.9w_{tf}f_v \sin\phi$	V_{cr}	BUCKLING
1400	8	175.0	2000	1.4	7.3	43.1	1.83	43.1	11200	483	35	61	107	122	185	410	893	Unsafe
		175.0	1900	1.4	7.5	44.4	1.80	44.4	11200	497	36	64	103	122	183	418	915	Unsafe
		175.0	1700	1.2	8.1	47.6	1.74	47.6	11200	533	39	70	96	122	176	432	965	Unsafe
		175.0	1500	1.1	8.8	52.1	1.66	52.1	11200	584	43	78	90	122	168	442	1026	Unsafe
		175.0	1300	0.9	10.2	60.2	1.55	60.2	11200	675	47	90	84	122	154	436	1111	Safe
	10	140.0	2100	1.5	7.1	65.7	1.48	65.7	11200	736	34	91	99	110	149	286	1022	Unsafe
		140.0	2000	1.4	7.3	67.4	1.46	67.4	11200	755	35	95	95	110	146	288	1043	Unsafe
		140.0	1900	1.4	7.5	69.4	1.44	69.4	11200	777	36	99	92	110	141	289	1066	Unsafe
		140.0	1800	1.3	7.8	71.7	1.42	71.7	11200	803	38	104	89	110	137	289	1092	Safe
	12	116.7	3500	2.5	6.0	79.6	1.35	79.6	11200	891	22	82	135	100	142	166	1057	Unsafe
		116.7	3300	2.4	6.1	80.6	1.34	80.6	11200	903	23	87	128	100	138	170	1072	Unsafe
		116.7	3200	2.3	6.1	81.2	1.33	81.2	11200	910	24	89	125	100	136	171	1081	Unsafe
116.7		3170	2.3	6.1	81.4	1.33	81.4	11200	912	24	90	124	100	135	172	1084	Unsafe	
116.7		3000	2.1	6.2	82.6	1.32	82.6	11200	925	25	95	118	100	131	174	1100	Safe	

The minimum c/d ratio for which the web is safe against buckling is 0.9 for 8mm thick web, 1.3 for 10 mm thick web, 2.1 for 12 mm thick web as shown in table 6

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Vol. 8, Issue 2, February 2019

Table 4: Shear Resistance of Web for Different c/d Ratio Using Tension field Method (for Heavy mineral loading)

dw	tw	dw/tw	c	c/d ratio	K _{sv}	τ _{cr,e}	λ _{sv}	τ _b	A _v	A _{sv,tb}	φ	ψ	s	V _{tf}	f _v	0.9w _{tf,tv} f _v sinφ	V _{tf}	BUCKLING
1400	8	175.0	1300	0.9	10.2	60.2	1.55	60.2	11200	675	47	90	84	122	154	436	1111	Unsafe
		175.0	1200	0.9	11.3	66.6	1.47	66.6	11200	746	49	99	81	122	144	422	1168	Unsafe
		175.0	1100	0.8	12.7	74.8	1.39	74.8	11200	837	52	109	78	122	131	397	1235	Unsafe
		175.0	1000	0.7	14.5	85.5	1.30	85.5	11200	958	54	121	75	122	114	357	1315	Unsafe
		175.0	900	0.6	16.9	100.0	1.20	100.0	11200	1120	57	136	73	122	90	291	1411	Safe
	10	140.0	1400	1.0	9.4	86.2	1.29	86.2	11200	966	45	129	77	110	109	266	1232	Unsafe
		140.0	1300	0.9	10.2	94.1	1.24	94.1	11200	1054	47	141	75	110	95	241	1295	Unsafe
		140.0	1200	0.9	11.3	104.0	1.18	104.1	11200	1165	49	154	72	110	78	204	1369	Unsafe
		140.0	1100	0.8	12.7	116.8	1.11	116.8	11200	1308	52	170	70	110	55	148	1456	Safe
	12	116.7	1900	1.4	7.5	99.9	1.20	99.9	11200	1119	36	143	84	100	87	163	1282	Unsafe
		116.7	1800	1.3	7.8	103.2	1.18	103.2	11200	1156	38	150	81	100	80	155	1311	Unsafe
		116.7	1700	1.2	8.1	107.1	1.16	107.1	11200	1199	39	158	79	100	72	145	1344	Unsafe
		116.7	1600	1.1	8.4	111.7	1.14	111.7	11200	1251	41	166	76	100	63	131	1383	Unsafe
		116.7	1500	1.1	8.8	117.3	1.11	117.3	11200	1314	43	176	73	100	53	113	1427	Safe

The minimum c/d ratio for which the web is safe against buckling is 0.6 for 8 mm thick web, 0.8 for 10 mm thick web, 1.1 for 12 mm thick web as shown in table 7.

Table 5: Shear Resistance of Web for Different c/d Ratio Using Tension field Method (for DFC loading)

dw	tw	dw/tw	c	c/d ratio	K _{sv}	τ _{cr,e}	λ _{sv}	τ _b	A _v	A _{sv,tb}	φ	ψ	s	V _{tf}	f _v	0.9w _{tf,tv} f _v sinφ	V _{tf}	BUCKLING
1400	8	175.0	1200	0.9	11.3	66.6	1.47	66.6	11200	746	49	99	81	122	144	422	1168	Unsafe
		175.0	1100	0.8	12.7	74.8	1.39	74.8	11200	837	52	109	78	122	131	397	1235	Unsafe
		175.0	1000	0.7	14.5	85.5	1.30	85.5	11200	958	54	121	75	122	114	357	1315	Unsafe
		175.0	900	0.6	16.9	100.0	1.20	100.0	11200	1120	57	136	73	122	90	291	1411	Unsafe
		175.0	800	0.6	20.4	120.3	1.10	120.3	11200	1348	60	155	71	122	52	176	1523	Safe
	10	140.0	1300	0.9	10.2	94.1	1.24	94.1	11200	1054	47	141	75	110	95	241	1295	Unsafe
		140.0	1200	0.9	11.3	104.0	1.18	104.1	11200	1165	49	154	72	110	78	204	1369	Unsafe
		140.0	1100	0.8	12.7	116.8	1.11	116.8	11200	1308	52	170	70	110	55	148	1456	Unsafe
		140.0	1000	0.7	14.5	133.6	1.04	133.6	11200	1496	54	190	67	110	22	63	1559	Safe
	12	116.7	1800	1.3	7.8	103.2	1.18	103.2	11200	1156	38	150	81	100	80	155	1311	Unsafe
		116.7	1700	1.2	8.1	107.1	1.16	107.1	11200	1199	39	158	79	100	72	145	1344	Unsafe
		116.7	1600	1.1	8.4	111.7	1.14	111.7	11200	1251	41	166	76	100	63	131	1383	Unsafe
		116.7	1500	1.1	8.8	117.3	1.11	117.3	11200	1314	43	176	73	100	53	113	1427	Unsafe
		116.7	1400	1.0	9.4	124.2	1.08	124.2	11200	1391	45	186	71	100	39	88	1479	Safe

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Vol. 8, Issue 2, February 2019

The minimum c/d ratio for which the web is safe against buckling is 0.6 for 8 mm thick web, 0.7 for 10 mm thick web, 1.0 for 12 mm thick web as shown in table 8

Design of end stiffeners and intermediate stiffeners.

The shear force is maximum at the end panels and also because of end anchorage force it is necessary to check the safety of end panel and end stiffeners number of trials has been done and the minimum section required to make stiffeners safe is provided table 6(i), 6(ii) & 6(iii) and table 7 shows the minimum cross section required for end stiffeners and intermediate stiffeners.

Table 6(i): End stiffeners cross section (using tension field method)

t	c	M_{tf}	additional compressive force due to bending F _{tf}	Area mm ²	B _f	t _f	A _{eff}	Single flat area	λ	p _d	Bearing capacity	Buckling safety
8	1500	226.1034	101.0	5250	175	35	5632	2240	16.75	1384.6	1136.36	safe
10	2200	283.7323	86.4	5250	175	35	6720	2160	14.38	1655	1500	safe
12	4500	341.6291	50.9	5250	175	35	6720	2160	14.38	1655	1500	safe

Table 6(ii): shear capacity of end stiffeners (using tension field method)

t	vp	v _{cr}	H _q	R _{tf}	capacity	safety
8	1616.6	584	1615.0	807.5	1469.7	safe
10	2020.8	720	2026.7	1013.3	1837.1	safe
12	2424.9	853	2440.2	1220.1	2204.5	safe

Table 6(iii): moment capacity of end stiffeners (using tension field method)

t	H _q	M _{tf}	I _s x10 ⁶ (mm ⁴)	c	y	M _q	safety
8	1615	226	2250	1500	750	682	safe
10	2027	284	8873	2200	1100	1833	safe
12	2440	342	91125	4500	2250	9205	safe

Table 7: Intermediate stiffeners cross sections (using tension field method)

I _s (required) x10 ⁶ (mm ⁴)	Width (mm)	Thickness (mm)	I _s provided x10 ⁶ (mm ⁴)	λ	Buckling force (kN)	F _{qd} (kN)	Buckling safety
1.07	200	16	42.6	10.21	878.79	925.81	Safe
3.63	160	14	19.11	22.18	573.46	950.59	Safe
1.28	90	6	1.45	54.68	326.17	886.46	Safe
7.68	140	14	12.28	26.30	850.59	893.23	Safe

Discussion

Fig. shows that the material required for a plate girder under different loading condition and different web thickness. It is seen that 12 mm and 16mm thick web for which no vertical stiffeners are required is uneconomical as shown in table 8.

International Journal of Innovative Research in Science, Engineering and Technology

(A High Impact Factor, Monthly, Peer Reviewed Journal)

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Vol. 8, Issue 2, February 2019

Table 8: Material comparison for different loading conditions

Modified broad gauge			Heavy Mineral Loading			DFC loading		
t_w	C/d ratio	Material (Kg/m)	t_w	C/d ratio	Material (Kg/m)	t_w	C/d ratio	Material (Kg/m)
6	0.5	7618.51	8	0.49	7714.98	8	0.45	7695.63
8	0.65	7507.48	10	0.86	7710.584	10	0.81	7732.56
10	1.28	7551.49	12	2	7917.196	12	1.7	7924.23
12	-	7583.1	16	-	8462.3	16	-	8462.2

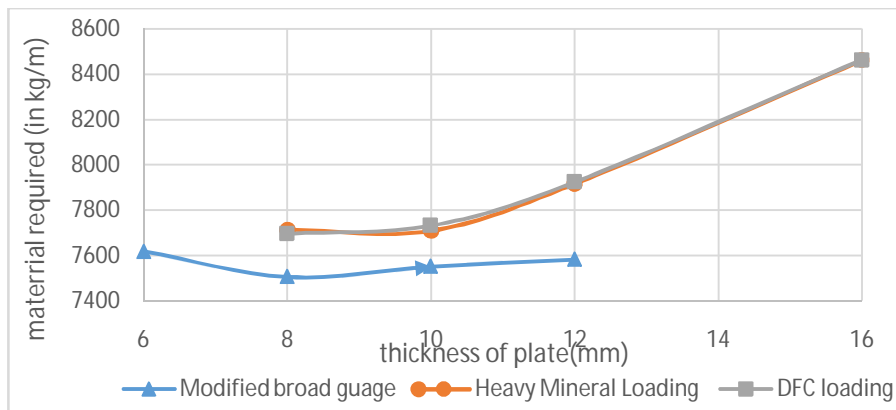


Fig. 2: Material comparison for different loading and web thickness

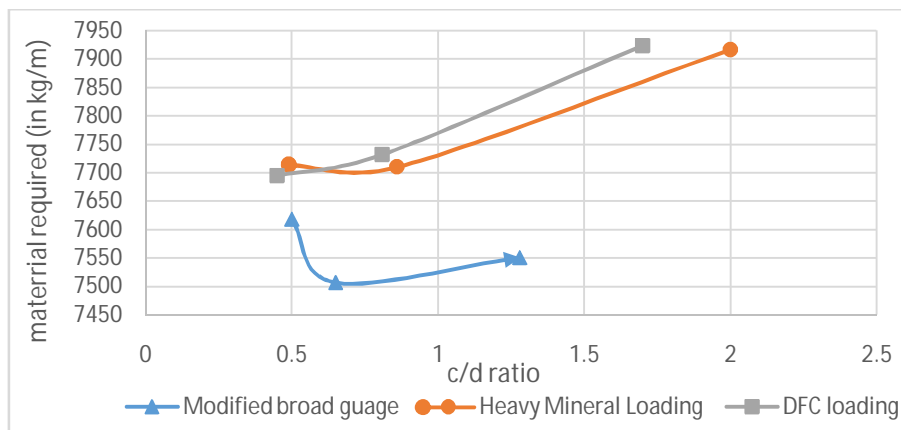


Fig.3: Material comparison for different loading and c/d ratio

.Fig. 3 show the material required for a plate girder under different loading condition and different web c/d ratio. It is also observed that the girder for which the c/d ratio is more than 1 is uneconomical whereas the thickness for which the c/d ratio is less than one or 1 is seems to be more economical. The thickness for which c/d ratio is less than 0.5, although the material required is less the fabrication cost and maintenance cost is very high.

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Vol. 8, Issue 2, February 2019

IV. CONCLUSION

Plate girder is designed for different loading conditions. In this paper, only depth of plate girder is kept constant and number of trails have been done by changing the thickness of web and spacing of stiffeners. By discussion it is observed 8 mm thick intermediate stiffener is economical for all types of loading if the advantage of tension field theory is not taken, 10 mm thick intermediate stiffener is economical for all types of loading if the advantage of tension field theory is taken. It can be concluded that using tension field theory, we can obtain most economical section for plate girder.

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