Zeitschrift: IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht

Band: 8 (1968)

Artikel: On an improved theory for Dr. Basler's theory

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DOI: https://doi.org/10.5169/seals-8785

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On an Improved Thory for Dr. Basler's Theory

Essai d'amélioration de la théorie de Basler

Verbesserungsversuch der Basler-Theorie

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1. Introduction

It is thought to be reasonable to design plate girders based on the ultimate strength because of the large capacity of post-buckling strength. In view of this fact, Dr. Basler's ingenious theories are considered to be very worthy. It is the author's opinion, however, that there are some problems to be discussed in his theories, especially on the shear strength. The first problem is that the contribution of flange rigidity to the tension field action is neglected in Dr. Basler's theory which should not be considered negligible in many cases. As the results of this assumption the direction of the tension field derived by Dr. Basler always gives less slope than the diagonal of web panel. But, when flanges are sufficiently strong and web buckling stress is small, direction of tension field should approach to 45° to the flange as shown by Wagner[1]. The next problem is that Dr. Basler derived an equation of equilibrium of forces from Fig. 1, but he neglected the shear force brought about in the stiffener at section 0, which must be accounted for if a partial tension field is assumed. Moreover, the effect of compressive force brought about in flanges by the tension field action on the interaction curves under combined bending and shear is neglected in Dr. Basler's theory. This is unsafety side because the compressive force overlaps the compressive force caused by bending.

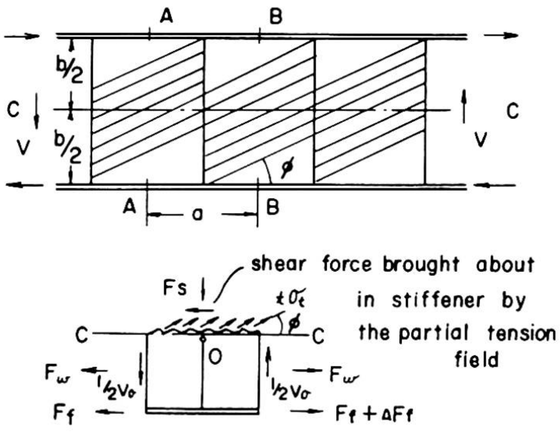


Fig. 1

The author tried to introduce a new approach of finding the shear strength of girders in post-buckling range with the above-mentioned points taken into consideration.

2. Theory(2)

In the following discussion, it is assumed that plate girders are so designed as not to give rise to lateral or local buckling of flanges, and that the stiffeners are designed sufficiently strong, too.

If a pure shearing stress field is assumed within a girder panel which is surrounded by upper and lower flanges and vertical stiffeners (Fig. 2), stresses in the web in the direction making an angle ϕ with the flange are given by the following equations,

$$\sigma_\xi = \tau \sin 2\phi, \quad \tau_{\xi\eta} = \tau \cos 2\phi,$$
$$\sigma_\eta = -\tau \sin 2\phi. \qquad (1)$$

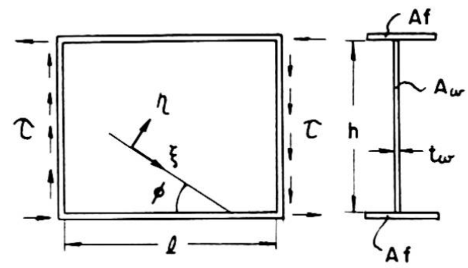


Fig. 2

Therefore, if the web buckling stress is denoted by τ_{cr}, the web stresses at the instant of buckling can be expressed as,

$$\sigma_{\xi cr} = \tau_{cr} \sin 2\phi,$$
$$\sigma_{\eta cr} = -\tau_{cr} \sin 2\phi, \qquad (2)$$
$$\tau_{\xi\eta cr} = \tau_{cr} \cos 2\phi.$$

In order to compute stresses after the web has buckled, the author assumes that the direction of the principal tensile stress σ_1 concides with that of waves of buckling and that the principal compressive stress σ_2 in the direction perpendicular to σ_1 is kept of the same stress value as that at the instant of the web buckling in the same direction. Once the action of the tension field comes out, stress σ_v and σ_w come into existence in the periphery of the panel to equilibrate with the tension field action as shown in Fig. 3.

σ_v is transmitted to the stiffeners as a shearing stress in the flanges, while σ_w is kept in equilibrium with compressive forces brought about in the flanges. For simplifying calculation, another assumption is introduced that these stresses are uniformly distributed throughout the panel.

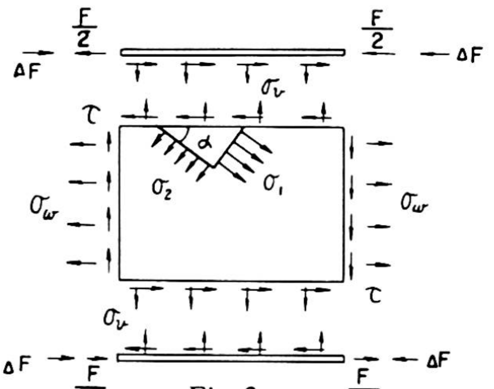


Fig. 3

Let α be an angle between the principal stress σ_1 and the flanges, then the formulas of equi-librium of forces on the boundary with the flanges are written in the forms,

$$\sigma_1 \sin^2\alpha + \sigma_2 \cos^2\alpha = \sigma_v, \qquad (3)$$

$$(\sigma_1 - \sigma_2) \sin\alpha \cos\alpha = \tau, \qquad (4)$$

Shearing force is,

$$V = h\, t_w \tau = A_w (\sigma_1 - \sigma_2) \sin\alpha \cos\alpha, \qquad (5)$$

where

h is depth of web,

t_w is thickness of web,

and $A_w = h\, t_w$ is sectional area of web.

σ_2 is given from the afore-mentioned assumption in the form,

$$\sigma_2 = - \tau_{cr} \sin 2\alpha \tag{6}$$

Suppose the web be yielded uniformly all over the panel under theses stress conditions. By using Tresca's yield condition.

$$\sigma_1 = \sigma_{wY} - \tau_{cr} \sin 2\alpha \tag{7}$$

where, σ_{wY} is yield stress of web.

By substituting Eqs (6) and (7) into Eq. (3)

$$\frac{\sigma_v}{\sigma_{wY}} = \sin^2\alpha - \frac{\nu_{cr}}{2}\sin 2\alpha, \tag{8}$$

where,

$$\nu_{cr} = \frac{\tau_{cr}}{\tau_{wY}},$$

By substituting Eqs. (6), (7) and (8) into Eq. (5) and by making it dimensionless, the following equation is obtained.

$$\nu = \frac{\left(1 - 2\frac{\sigma_v}{\sigma_{wY}}\right)\nu_{cr} + \sqrt{1 + \nu_{cr}^2 - \left(1 - 2\frac{\sigma_v}{\sigma_{wY}}\right)^2}}{1 + \nu_{cr}^2} \tag{9}$$

where,

$$\nu = \frac{V}{V_p}$$

$V_p = A_w \tau_{wY}$ is plastic shear force,

and $\tau_{wY} = \frac{\sigma_{wY}}{2}$ is yield shear stress of web.

The value of σ_v varies with the bending deformation of the flanges, but it reaches its maximum when the flanges start to collapse forming the plastic hinges at the both ends supported by vertical stiffeners and at the midspan of flanges (Fig. 4).

By applying the theory of simple plasticity to the flanges which are regarded as beams of rectangular cross section subjected to uniformly distributed load with both ends fixed, this maximum value is obtained by the following formula,

$$\frac{1}{4} l^2 t_w \sigma_{v\,max} = A_f t_f \sigma_{fY} \tag{10}$$

where,

$A_f = b_f t_f$ is sectional area of flange,

b_f is width of flange,

t_f is thickness of flange,

and σ_{fY} is yield stress of flanges.

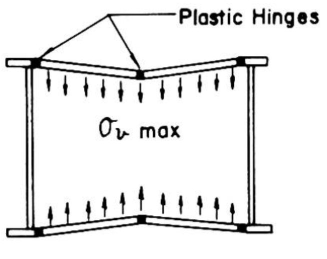


Fig. 4

Strictly speaking web portions adjacent to the flanges should be considered to act as a part of the flanges and the influences of axial force and shearing force in flanges on collapse should be taken into account.

However, it is considered that these influences are not significant, because the former and the latter influences cancel each other. Therefore, Eq. (10) will be exact for practical application.

The value of (σ_v/σ_{wY}) which makes Eq. (9) maximum will be obtained by putting $\partial\nu/(\sigma_v/\sigma_{wY}) = 0$ as follows.

$$\sigma_v / \sigma_{wY} = (1 - \nu_{cr})/2 \tag{11}$$

The maximum value of ν or ultimate shear force ν_u is derived from Eqs. (9), (10) and (11) in the following way.

If $-(1-\nu_{cr}) < \varepsilon$, by substituting Eq. (11) into Eqs. (8) and (9),

$$\nu_u = 1, \qquad \alpha = 45^\circ. \tag{12}$$

If $\varepsilon \leq (1-\nu_{cr})$, by substituting Eq. (10) into Eqs. (8) and (9),

$$\nu_u = \frac{(1-\varepsilon)\,\nu_{cr} + \sqrt{1+\nu_{cr}^2-(1-\varepsilon)^2}}{1+\nu_{cr}^2}$$

$$\tan\alpha = \frac{\nu_{cr} + \sqrt{1+\nu_{cr}^2-(1-\varepsilon)^2}}{2-\varepsilon} \tag{13}$$

If $\varepsilon = 0$ or the rigidity of the flanges can be neglected,

$$\nu_{u0} = 2\nu_{cr}/(1+\nu_{cr}^2)$$

$$\tan\alpha_0 = \nu_{cr} \tag{13'}$$

On the other hand, if the stiffener space is comparatively small, the portion where tension field action is directly anchored by the axial force of the stiffeners is formed in the web as indicated by hatched portion in Fig. 5.

This portion can bear higher tension than the neighboring triangular portions because the condition given in Eq. (3) need not be satisfied in this portion.

Therefore, this portion can be assumed to be under the yielded condition and the principal tensile stress σ_1' in this portion is given as

$$\sigma_1' = \sigma_{wY} - \tau_{cr}\sin 2\alpha \tag{14}$$

Eqs. (3), (4) and (6) are applicable to stresses in neighboring triangular portions.

From the equilibrium of forces,

$$V = A_w(1-\lambda\tan\alpha)\tau' + \lambda A_w \tan\alpha\cdot\tau$$
$$= A_w\{(1-\lambda\tan\alpha)(\sigma_1'-\sigma_2) + \lambda\tan\alpha\,(\sigma_1-\sigma_2)\}\sin\alpha\cdot\cos\alpha \tag{15}$$

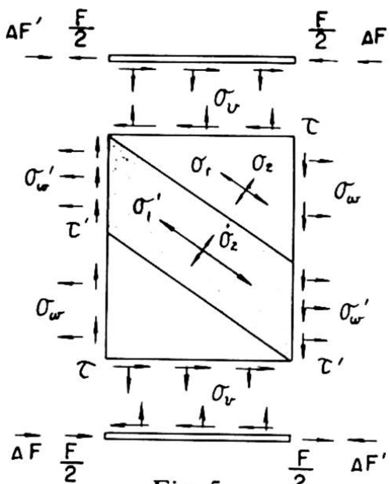


Fig. 5

When the girder collapses with a partial tension field formed nearly in the direction of the diagonal of the panel as mentioned above, the prerequisite is

$$\sigma_1 \leq \sigma_1' \tag{16}$$

and the flange must satisfy Eq. (10)

From Eqs. (6), (3) and (10)

$$\frac{\sigma_1}{\sigma_{wY}} = \frac{\frac{\varepsilon}{2} + \frac{\nu_{cr}}{2}\sin 2\alpha\cos^2\alpha}{\sin^2\alpha} \tag{17}$$

By substituting Eqs. (6), (14) and (17) into Eq. (15) and making it dimensionless,

$$\nu = \varepsilon\lambda + \lambda\nu_{cr}\sin 2\alpha + (1-\lambda\tan\alpha)\sin 2\alpha \tag{18}$$

Eq. (18) takes its maximum value for a certain value of α which is obtained by putting $\partial \mathcal{V}/\partial\alpha = 0$, as follows,

$$\tan 2\alpha = (1 + \lambda \mathcal{V}_{cr})/\lambda \tag{19}$$

By substituting this value into Eq. (18) the ultimate shear force in this case will be

$$\mathcal{V}_u = \sqrt{\lambda^2 + (1+\lambda \mathcal{V}_{cr})^2} - (1-\varepsilon)\lambda \tag{20}$$

Particularly, if $\varepsilon = 0$,

$$\mathcal{V}_{uo} = \sqrt{\lambda^2 + (1+\lambda \mathcal{V}_{cr})^2} - \lambda \tag{20'}$$

The prerequisite condition under which the ultimate shear load is given by Eq. (20) is obtained by substituting Eqs. (14), (17) and (19) into Eq. (16) in the form

$$\varepsilon \leq 1 - \frac{\lambda + \mathcal{V}_{cr}(1+\lambda \mathcal{V}_{cr})}{\sqrt{\lambda^2 + (1+\lambda \mathcal{V}_{cr})^2}}, \tag{21}$$

or

$$\lambda \leq \frac{1}{\sqrt{\varepsilon + \mathcal{V}_{cr}^2}} \frac{\sqrt{1-\varepsilon} - \mathcal{V}_{cr}}{1 + \mathcal{V}_{cr}^2} \equiv \lambda_{cr} \tag{21'}$$

Ultimate shear forces are summarized from the above-mentioned results as shown in Table -1.

Values of $\mathcal{V}_{cr}$ are calculated by the following equations which are modified Johnson's column formula.

$$\mathcal{V}_{cr} = \frac{k_s \pi^2}{12(1-\nu^2)} \left(\frac{E}{\tau_{wY}}\right)\left(\frac{t_w}{h}\right)^2, \quad \mathcal{V}_{cr} \leq 0.5$$

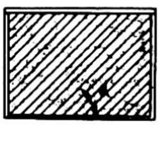
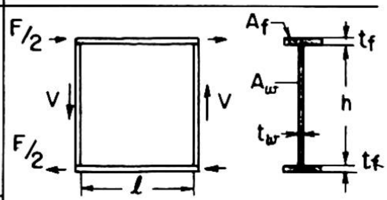
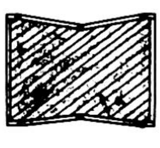
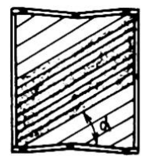
$$\mathcal{V}_{cr} = 1 - \frac{3(1-\nu^2)}{k_s \pi^2} \left(\frac{\tau_{wY}}{E}\right)\left(\frac{h}{t_w}\right)^2, \quad \mathcal{V}_{cr} > 0.5 \tag{22}$$

Where ν is Poisson's ratio and k_s is a buckling coefficient depend on λ and constraint conditions around a panel.

Since the stiffeners are usually almost equal in thickness to the web while flanges are much thicker than the web, it seems appropriate to consider that the web is fixed at the flanges and supported at the stiffeners.

And the theoretical values using the buckling coefficient calculated for a panel fixed at the flanges shows a good coincidence with the experimental values.

Table 1

Failure mode	Condition	Ultimate shear force	Notations
	$(1-\mathcal{V}_{cr}) < \varepsilon$	$\mathcal{V}_u = 1$ $\alpha = 45°$	
	$1 - \frac{\lambda + \mathcal{V}_{cr}(1+\lambda \mathcal{V}_{cr})}{\sqrt{\lambda^2+(1+\lambda \mathcal{V}_{cr})^2}} < \varepsilon \leq (1-\mathcal{V}_{cr})$	$\mathcal{V}_u = \frac{(1-\varepsilon)\mathcal{V}_{cr} + \sqrt{1+\mathcal{V}_{cr}^2-(1-\varepsilon)^2}}{1+\mathcal{V}_{cr}^2}$ $\tan\alpha = \frac{\mathcal{V}_{cr} + \sqrt{1+\mathcal{V}_{cr}^2-(1-\varepsilon)^2}}{2-\varepsilon}$	$\lambda = l/h$: aspect ratio α : inclination of tension field σ_{wY}: yield stress of web σ_{fY}: 〃 〃 flange τ_{cr}: web buckling stress
	$\varepsilon \leq 1 - \frac{\lambda + \mathcal{V}_{cr}(1+\lambda \mathcal{V}_{cr})}{\sqrt{\lambda^2+(1+\lambda \mathcal{V}_{cr})^2}}$	$\mathcal{V}_u = \sqrt{\lambda^2 + (1+\lambda \mathcal{V}_{cr})^2} - (1-\varepsilon)\lambda$ $\tan 2\alpha = \frac{1+\lambda \mathcal{V}_{cr}}{\lambda}$	$\varepsilon = \frac{8}{\lambda^2} \frac{t_f}{h} \frac{A_f \sigma_{fY}}{A_w \sigma_{wY}}$ $\mathcal{V}_{cr} = \tau_{cr}/\tau_{wY}$ $\mathcal{V}_u = V_u / A_w \tau_{wY}$

3. Comparison with the test results

The results of the test conducted on girders G-6, G-7, G-8, G-9, by Dr. Basler et al.,[3] girders H-1, H-2 by Cooper et al.,[4] a girder B by Dr. Konishi et al., [5] are compared with the author's and Dr. Basler's theories. The experimental and the theoretical values are summarized in Table -2.

For the author's theory, the values calculated for the web with simply-supported periphery are also shown in round brackets for comparison with the values calculated for the web fixed along the flanges. The values of τ_{u0} which were obtained by neglecting the effect of flange stiffeness on tension field are also shown.

The flanges of girders H 1, H 2, G 1[8] and G 2[8] were provided with doublers of cover plates as shown in Fig. 6. In these cases, values of ε which gives the effect of rigidity of flanges were calculated by the following formula, presuming that the flanges and the cover plates act as independent simple beams.

$$\varepsilon = \frac{8}{\lambda^2} \frac{(t_f^2 b_f \sigma_{fY} + t_c^2 b_c \sigma_{cY})}{h A_w \sigma_{wY}} \tag{23}$$

where t_c is thickness of cover plate,

b_c is width of cover plate,

and σ_{cY} is yield stress of cover plate.

As was indicated in Section 1, Dr. Basler's theoretical formula was derived from equilibrium condition of forces, the shear force acting in the stiffeners being neglected. If this shear force be taken into account, the ultimate shear force is given by the following equation in stead of Eq. (12) of ref. (8),

$$V_u = V_p \left[\frac{\tau_{cr}}{\tau_Y} + \frac{\sqrt{3}}{2} \left(1 - \frac{\tau_{cr}}{\tau_Y} \right) \left(\sqrt{1 + \lambda^2} - \lambda \right) \right] \tag{24}$$

Theoretical values corrected by Eq. (24) are by 10~40% lower than the original values.

In Table-2, Dr. Basler's theoretical values in column (9) are the original values, and the values corrected by Eq. (24) are excluded. In column (10) are given ratios of the experimental values to the original and the corrected theoretical values, the latter ratios being given in square brackets for comparison with the former ratios. It is observed that the differences between the original theoretical values of V_u and the experimental ones exceed 10% in ten girders, nearly half of the specimens, and that the corrected theoretical values become considerably smaller than experimental values.

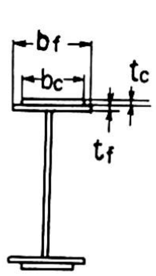


Fig. 6
Notations for cover plates

The author's theoretical values, which include the contribution of flange stiffeness on tension field and on the boundary condition of the web, as the author wishes to propose, the author's theoretical values coincide well with the experimental values.

For the 25 girders examined, differences between the theoretical and the experimental values were within 10%, only one exception being 22% for the girder G 6.

Table-2 Comparison of the theoretical values with the experimental values.

Ref. No.	Girder	Experimental values							Theoretical values						
		Web		Flange					Basler		Author				
		$h \times t_w$	σ_{wY}	$b_f \times t_f$ $b_c \times t_c$	σ_{fY} σ_{cY}	λ	$\frac{h}{t_w}$	V_u^{ex}	V_u^{th}	$\frac{V_u^{ex}}{V_u^{th}}$	V_p	v_{cr}	v_{u0}	v_u	$\frac{V_u^{ex}}{v_u V_p}$
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
		in in	k.s.i.	in in	k.s.i.			kips	kips		kips				
(3)	G6-T1	50 x 0.193	36.7	12.13 x 0.778	37.9	1.5	259	116	112	1.04 [1.58]	177	0.237 (0.155)	0.552 (0.441)	0.606 (0.525)	1.08
	G6-T2	"	"	"	"	0.75	"	150	157	0.95 [1.28]	"	0.355 (0.293)	0.722 (0.685)	0.877 (0.849)	0.97
	G6-T3	"	"	"	"	0.5	"	177	180	0.98 [1.15]	"	0.592 (0.551)	0.889 (0.871)	1.00 (1.00)	1.00
	G7-T1	50 x 0.196	"	12.19 x 0.768	37.6	1.0	255	140	142	0.98 [1.41]	180	0.275 (0.210)	0.620 (0.570)	0.740 (0.690)	1.05
	G7-T2	"	"	"	"	"	"	145	142	1.02 [1.46]	"	0.275 (0.210)	0.620 (0.570)	0.740 (0.690)	1.09
	G8-T1	50 x 0.197	38.2	12.00 x 0.750	41.3	3.0	254	85	76	1.12 [1.70]	188	0.211 (0.121)	0.416 (0.295)	0.456 (0.334)	0.99
	G9-T1	50 x 0.131	44.5	12.00 x 0.750	41.8	"	382	48	51	0.94 [1.63]	146	0.080 (0.048)	0.246 (0.211)	0.298 (0.263)	1.10
	G9-T2	"	"	"	"	1.5	"	75	85	0.89 [1.49]	"	0.093 (0.058)	0.383 (0.335)	0.486 (0.438)	1.06
(4)	H1-T1	50 x 0.393	108.1	18.06 x 0.980	106.4	3.0	127	630	473	1.33 [1.89]	1060	0.299 (0.190)	0.550 (0.383)	0.620 (0.473)	0.96
	H1-T2	"	"	18.06 x 0.980 17.03 x 0.982	106.4 105.8	1.5	"	769	710	1.08 [1.56]	"	0.338 (0.221)	0.626 (0.506)	0.790 (0.684)	0.92
	H2-T1	50 x 0.390	110.2	18.06 x 1.006 17.09 x 1.008	105.5 108.8	1.0	128	917	875	1.05 [1.45]	1075	0.369 (0.283)	0.695 (0.627)	0.929 (0.711)	0.92
	H2-T2	"	"	"	"	0.5	"	1125	1143	0.98 [1.09]	"	0.689 (0.687)	0.935 (0.934)	1.00 (1.00)	1.05
		mm mm	kg/mm^2	mm mm	kg/mm^2			ton	ton		ton				
(5)	B	1200 x 4.5	50.0	240 x 12	50.0	1.0	267	76	91.4	0.83 [1.18]	135	0.130 (0.100)	0.510 (0.485)	0.553 (0.528)	1.02
(6)	G1-1	1200 x 6.6	49.6	250 x 23	51.0	3.0	182	99	81.5	1.21 [1.83]	196	0.222 (0.141)	0.433 (0.319)	0.471 (0.357)	1.07
	G1-2	"	"	250 x 23 250 x 13	51.0 46.0	1.5	"	129	126	1.03 [1.57]	"	0.251 (0.164)	0.535 (0.450)	0.633 (0.548)	1.04
	G2-1	950 x 6.6	"	250 x 19	53.0	3.0	144	98	73.3	1.34 [1.83]	155.5	0.355 (0.226)	0.630 (0.439)	0.658 (0.480)	0.96
	G2-2	"	"	250 x 19 250 x 13	53.0 46.0	1.5	"	125	107	1.17 [1.63]	"	0.402 (0.262)	0.695 (0.547)	0.802 (0.668)	1.00
(7)	G1	440 x 8	44.0	160 x 30	42.0	2.61	55	82	96.7	0.85	77.5	0.910 (0.860)	0.996 (0.989)	1.00 (0.999)	1.06
	G2	"	"	200 x 30	"	"	"	84	"	0.87	"	"	"	1.00 (1.00)	1.08
	G3	560 x 8	"	160 x 30	"	2.63	70	99	102.2	0.97 [1.01]	98.5	0.854 (0.773)	0.988 (0.963)	0.997 (0.985)	1.01
	G4	"	"	250 x 30	"	3.57	"	97	99.1	0.98 [1.01]	"	0.849 (0.759)	0.987 (0.963)	0.995 (0.980)	0.99
	G5	"	"	"	"	2.68	"	107	102.2	1.05 [1.09]	"	0.854 (0.772)	0.988 (0.968)	0.999 (0.990)	1.09
	G6	"	"	"	"	1.25	"	120	113	1.06	"	0.875 (0.818)	0.992 (0.980)	1.00 (1.00)	1.22
	G7	"	"	"	"	2.68	"	107	102.2	1.07 [1.09]	"	0.854 (0.772)	0.988 (0.968)	0.999 (0.990)	1.09
	G8	720 x 8	"	160 x 30	"	2.78	90	93		failured by lateral buckling of flange					
	G9	"	"	250 x 30	"	"	"	118	98.5	1.20 [1.32]	127	0.758 (0.622)	0.962 (0.897)	0.979 (0.931)	0.95

( ) : calculated values supposing simply supported along the flanges

[ ] : V_u^{th} calculated by Eq. (24)

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SUMMARY

While extremely ingenious, and accepted generally to be well applicable to the design of plate girders, Dr. Basler's theories would appear to the present author to possess certain weak points, particularly with respect to the determination of shear strength. The present author has attempted a new approach to the question of finding the ultimate shear strength of plate girders. The improvement thus introduced has resulted in better agreement between theoretical values and those obtained empirically in experiments conducted at the Lehigh University and in Japan.

RÉSUMÉ

Bien qu'extrêmement ingénieuses et acceptées généralement comme bien valable pour les calculs de poutres à âme pleine, les théories du Dr. Basler, selon l'avis du présent auteur, comportent quelques points faibles surtout concernant la résistance au cisaillement. L'auteur a essayé d'aborder d'une direction nouvelle la question de trouver la résistance extrême au cisaillement des poutres à âme pleine. L'amélioration ainsi introduite a apportée une meilleur concordance entre les valeurs théoriques et celles empiriques mesurées aux essais effectuées à l'Université de Lehigh et au Japon.

ZUSAMMENFASSUNG

Obwohl ausgezeichnet und allgemein anerkannt für die Anwendung zur Erstellung von Vollwandträgern, erscheint es dem Verfasser doch angebracht, auf einige schwache Punkte, besonders im Hinblick auf die Bestimmung der Scherfestigkeit, hinzuweisen. Die vorgeschlagene Verbesserung ergab eine bessere Uebereinstimmung zwischen den theoretischen Werten und jenen, die durch Versuche sowohl an der Lehigh Universität als auch in Japan erzielt wurden.

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