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Objekttyp: **Article**

Zeitschrift: **IABSE reports of the working commissions = Rapports des commissions de travail AIPC = IVBH Berichte der Arbeitskommissionen**

Band (Jahr): **11 (1971)**

PDF erstellt am: **26.05.2024**

Persistenter Link: https://doi.org/10.5169/seals-12052

Ultimate Strength of Stiffened Plate Girders Subjected to Shear

Résistance à la ruine des poutres à âme pleine raidies soumises au cisaillement

Tragfähigkeit ausgesteifter, schubbeanspruchter Blechträger

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I. INTRODUCTION

For many years it was recognized that the static strength of plate girder entirely depends upon the web buckling strength. However, it has been shown by Massonnet, Rockey, Basler and other authors having been in progress since 1957 that the slender-web plate girder has its considerable inherent post-buckling strength. In several countries, the current Design Specification for transversely stiffened plate girders is based on these findings.

The present effort is part of a continuing study about the behavior of slender-web plate girder stiffened by transverse and longitudinal stiffeners, and that is concerned with the predicting of the limit strength of its panel under shear. To insure the justification of this approach, a series of proof-tests was carried out at Osaka University in the summer of 1968.

Five 7m long welded girders with 3.33mm webs, the slenderness ratios of which were 200, 225, 250, were tested. The test panels of girders had one or two longitudinal stiffeners, dividing the web into two or three subpanels of equal depth, in addition to transverse stiffeners. The stiffeners were designed by taking Prof. Skaloud's recommendations so as to allow the sufficient development of incomplete diagonal tension field.

The behavior of longitudinally stiffened slender-web girder under shear will be discussed in this report. Then the essential data of the proof-tests were compared with the predicted values. These test results as well as ones measured by several authors show excellent agreement with theoretical values obtained according to proposed method.

Finally, design recommendations based on these static studies have been formulated for plate girders subjected to shear.

II. MODE OF FAILURE UNDER SHEAR

A pure shear loading causes some kinds of failure mode chiefly dependent on the relation between the slenderness of web and the strength of smaller flange. In the case of low slenderness, a shear carrying action called "beam action" resists at the neutral axis to the pure shear. With the attainment of shear yield stress at the same place, the failure starts and then the yielding phenomenon spreads all over the web by only small increasing of the shear force. To design the girder having a comparatively slender web according to the classical beam theory based on "beam action", we need transverse stiffener spaced

close enough so that instability due to shear is excluded.

On the other hand, it has been long recognized that the pure diagonal tension develops in such a extremely thin web as those being seen in aircraft. The action of a pure diagonal tension field is quite similar to that of a diagonal member of Pratt Truss.

In the case of median slenderness, a pure shear loading results in equal tensile and compressive principal stresses up to critical shear buckling. After buckling, only a diagonal tension can carry any additional shear load. Basler and his colleagues at Lehigh University have developed "Theory of modified incomplete diagonal tension field". In the state of incomplete diagonal tension, two kinds of stress situation always exist together in the web plate. One of them corresponds to a critical shear stress, another a diagonal tensile stress. The mode of failure in this stress condition is greatly afftected by the rigidity of the boundary members and the slenderness of web plate. If the boundary members have sufficient stiffness, the diagonal tensile stress will be uniformly distributed in the web plate. So the collapse of that panel certainly occurs due to yielding all over the web plate. If the boundary member, however, do not have sufficient rigidity, yield zone is restricted to a narrow diagonal strip. The diagonal strip of the web flows plastically with the development of plastic hinges in the flange. The residual flange and web deflections shown by Prof. Rockey make it quite clear that the mode of failure is not similar to that assumed by Baslar. It should be also noted that the position of plastic hinge varies with the flange stiffness, being quite differ from Fujii's assumption.

For high flange stiffness, the plastic hing is located in the middle point of the panel length. While, its position is quite near a corner of panel for relatively flexible flange.

III. COMPUTING PROCEDURE

In order to find the ultimate strength of a plate girder under shear, the girder may be assumed to behave according to beam theory up to the critical buckling stress of web plate and then in a diagonal tension field manner up to the yielding point. With the exception of extremely low slenderness of web, for example less than 90, or extremely flexible stiffness of flange, the panel may also be assumed to be rigidly clamped along the flange.

Fig. 1 shows the relation between the extension of diagonal tension field and the deformation of panel-frame after collapse. In the same figure, γ denotes the shearing displacement and v(x) the deflection at a point x of flange due to its bending. If the tension field develops with the inclination θ to girder axis after buckling, a tensile strain ε_{tu} will be induced in the direction P"R' parallel to that of tension field.

$$\varepsilon_{tu} = \left[\gamma - \frac{V_u(x)}{a - x}\right] \sin\theta \cdot \cos\theta \qquad (1)$$

, where a is the panel length.

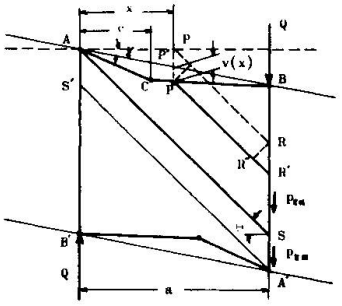


Fig. 1 Deformation of shear panel

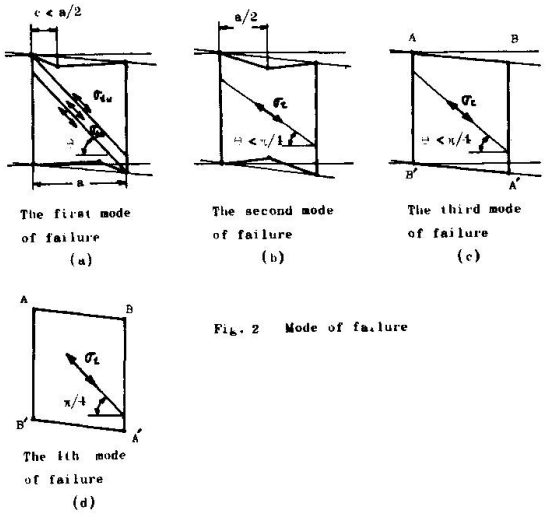


Fig. 2 Mode of failure

Corresponding tensile stress σ_{tu} will be also produced with the elastic modulus E_t.

$$\sigma_{tu} = E_t \left[\gamma - \frac{V_u(x)}{a - x} \right] \sin\theta \cdot \cos\theta \qquad (2)$$

Therefore, a distributed transverse internal force q_u acts on the flange all over the span AB.

$$q_u = E_t \left[\gamma - \frac{V_u(x)}{a - x} \right] \sin^3\theta \cdot \cos\theta \cdot t_w \qquad (3)$$

, where t_w is the thickness of web plate.
Whereas it may be assumed that both bounds A'S and AS' are scarcely deformed, a diagonal tensile stress σ_{tm} in central region may be written as follows;

$$\sigma_{tm} = E_t \gamma \sin\theta \cdot \cos\theta \qquad (4)$$

Now, two kinds of failure mode are cosidered according to the rigidity of flange. The first mode corresponding to a relatively flexible flange starts with bending collapse of flange, being accompanied by web-yielding in the central region such as shown in Fig.1. However, both upper region ABS and lower one A'B'S' remain elastic in spite of failure of girder. Neglecting the elastic deformation, the deflection of flange just after collapse may be represented by the following linear equation of coordinate x.

$$v(x) = \frac{x}{c}\Delta \qquad 0 \le x \le c$$
$$v(x) = \frac{a-x}{a-c}\Delta \qquad c \le x \le a \qquad (5)$$

Substituting v(x) into eq.(3), the transverse internal force q_u is written by

$$q_u = E_t \left(\gamma - \frac{x}{c} \cdot \frac{\Delta}{a-x} \right) \sin^3\theta \cdot \cos\theta \cdot t_w \qquad 0 \le x \le c$$

$$q_u = q_{u2} = E_t \left(\gamma - \frac{\Delta}{a-c} \right) \sin^3\theta \cdot \cos\theta \cdot t_w \qquad c \le x \le a \qquad (6)$$

, where Δ is the deflection at the point C.
Especially, at x=0

$$q_u = q_{u1} = E_t \gamma \sin^3\theta \cdot \cos\theta \cdot t_w \qquad (7)$$

For the sake of simplicity, the linear distribution of transverse internal force q_u along the coordinate axis x may be assumed as an approximation between A and C.

$$q_u = q_{u1} - (q_{u1} - q_{u2}) \frac{x}{c} \qquad (0 \le x \le c) \qquad (8)$$

By applying the principle of virtual displacement to the flange AB broken off by the transverse action q_u, a following relation can be obtained.

$$\frac{12a}{c(a-c)} M_p = cq_{u1} + (3a-c) q_{u2} \qquad (9)$$

, where M_p denotes the full plastic moment of smaller flange, being computed by following formula.

$$Mp = \frac{1}{4} \sigma_{fy} b_f t^2_f \qquad (10)$$

In above formula, σ_{fy} , b_f, and t_f denote yield stress, breadth, and thickness of flange, respectively

Substituting eqs.(6) and (7) into eq.(9), and putting

$$\bar{\eta} = E_t \Delta \sin^3\theta \cos\theta \cdot t_w / a$$

$$= \frac{q_{u1}(a-c) - \frac{4Mp}{c}}{a - \frac{c}{3}} \qquad (11)$$

Considering of equilibrium in vertical direction at the cross section AB gives a following relation,

$$Q = \int_0^a p_{zu}(x)dy(x) + \int_{a\tan\theta}^{b} p_{zm}(y)dy + \tau_{cr} bt_w \qquad (a)$$

, p_{zu} and p_{zm} are shear internal forces per unit length along the boundary at the upper and central region, respectively as shown in Fig.1. The third term represents the contribution of remaining beam action. From boundary conditions of the panel,

$$\left.\begin{aligned} p_{zu} &= \sigma_{tu} \sin\theta \cdot \cos\theta \cdot t_w \\ p_{zm} &= \sigma_{tm} \sin\theta \cdot \cos\theta \cdot t_w \end{aligned}\right\} \tag{12}$$

Then, using eqs.(12) in eq.(a),

$$Q = q_{ul} b \cdot \cot\theta - \bar{\eta}\left[1+\frac{c}{2(a-c)}\right] a + \tau_{cr} b t_w \tag{13}$$

Furthermore, the attainment of yielding in the central region causes the complete failure of plate girder, so that the diagonal tensile stress σ_{tm} can increase up to following value according to the yield criterion,

$$\sigma_{tm} = \sigma_{wy} - 2\tau_{cr} \sin 2\theta \tag{14}$$

, where σ_{wy} , τ_{cr} are yield stress and critical shear buckling stress of web, respectively.
So q_{ul} becomes as follows,

$$q_{ul} = (\sigma_{wy} - 2\tau_{cr} \sin 2\theta) \sin^2\theta \cdot t_w \tag{15}$$

Substituting eqs.(11) and (15) into eq.(13), the ultimate shear force can be finally obtained.

$$Q = \frac{1}{2}(\sigma_{wy} - 2\tau_{cr} \sin 2\theta)\left\{b \sin 2\theta - \alpha(1-\cos 2\theta)\right\} t_w + \frac{4 M_p \alpha}{c(a-c)} + \tau_{cr} b t_w \tag{16}$$

, where b and t_w are depth and thickness of web plate respectively, and

$$\alpha = \frac{a-c/2}{a-c/3} a \tag{17}$$

The inclination of diagonal tension field θ can be obtained under such condition as the shearing resistance should become maximum; that is,

$$\frac{\partial Q}{\partial \theta} = 0$$

, from which the following relation is deduced.

$$\sigma_{wy}(b \cos 2\theta - \alpha \sin 2\theta) + 2\tau_{cr}\left\{\alpha(\cos 2\theta - \cos 4\theta) - b \sin 4\theta\right\} = 0 \tag{18}$$

For this case, the inclination should be determined to satisfy above condition (18).

Then the position of plastic hinge can be determined by a following empirical formula,

$$C = \frac{1.782 \dfrac{Mp}{\sigma_{wy}\, b^2 t_w} + 0.38}{2\left(1.782 \dfrac{M_p}{\sigma_{wy} b^2 t_w} + 1\right)}\, a \qquad (19)$$

For flexible flange, Mp value is so small that the factor $\bar{\eta}$ of plastic deflection in eq.(11) takes positive value.
In such a case, the first mode of collapse as shown in Fig.2(a) will be occured. If the rigidity and strength of flange are much enough to resist elastically against the transverse internal force q_u even after yielding of web, the third or fourth mode of failure will be deduced as shown in Fig.2(c) or (d).
In these modes, the diagonal tensile stress will be uniformly distributed all over the panel. The failure of girder will be induced by complete panel-yielding, when the diagonal tensile stress σ_t will attain to the yield point. Therefore

$$p_z = p_{zu} = p_{zm} = \sigma_t \sin\theta \cdot \cos\theta \cdot t_w$$

$$= \frac{1}{2}\left(\sigma_{wy} - 2\tau_{cr} \sin 2\theta\right) \sin 2\theta \cdot t_w \qquad (20)$$

Substituting eq.(20) into eq.(a), the ultimate shear force Q can be obtained as follows,

$$Q = \left\{ \frac{1}{2}\sigma_{wy} \sin 2\theta + \tau_{cr}(1 - \sin^2 2\theta) \right\} b t_w \qquad (21)$$

According to the condition of maximum value $\partial Q / \partial \theta = 0$ again,

$$\cos 2\theta \left(\sigma_{wy} - 4\tau_{cr} \sin 2\theta \right) = 0 \qquad (22)$$

If the slenderness of web is so large that the critical shear buckling stress has a very small value and an inequality

$$\sigma_{wy} - 4\tau_{cr} \geq 0 \qquad (23)$$

can be satisfied, the fourth mode of failure will be occured. Since the inclination θ of tension field should become $\pi/4$ in such a case, the ultimate shear force Q can be readily found.

$$Q = \tau_{wy}\, b\, t_w + 4Mp/a \qquad (24)$$

The last term represents the sum of the full pastic moments at the corners A, B, A' and B'.

This means so-called Wagner's complete tension field.

However, if the slenderness of web plate is not so large as to satisfy inequality(23), the inclination should be determined by a following equation.

$$\theta\ (=\theta_0) = \frac{1}{2}\sin^{-1}\frac{\sigma_{wy}}{4\tau_{cr}} \tag{25}$$

The ultimate force Q should be, of course, calculated by eq.(21). Consequently the third mode of failure will be occured in such a case.

As an intermediate case between the first and third mode, it will be considered that the plastic hinge at the middle point of flange will be built up simultaneously with the spreading of yielded scope in the web. For such a case, the mode of failure may be called the second mode and shown in Fig.2(b). In this case, $\bar{\eta}$ takes a negative value for θ satisfying condition(18), while it has positive one for θ taking $\pi/4$ in spite of satisfying inequality(23).

Since it must be complicated to analyze rigorously such stress situation, the following approach may be useful to find ultimate load. From the condition of simultaneous collapse, the following relation may be derived.

$$M_p = \frac{a^2}{16}\left(\sigma_{wy} - 2\tau_{cr}\sin 2\theta\right)\sin^2\theta \cdot t_w \tag{26}$$

When the plastic moment M_p of flange is given, the inclination θ for such mode of failure should satisfy above eq.(26).

Thus eq.(21) can be applied again to determine the ultimate shear force, because the combined stress will be attained to yield point in entire panel immediately after collapse of flange.

The practical computing process may be carried out in such a way as shown by a block diagram as shown in Fig.3.

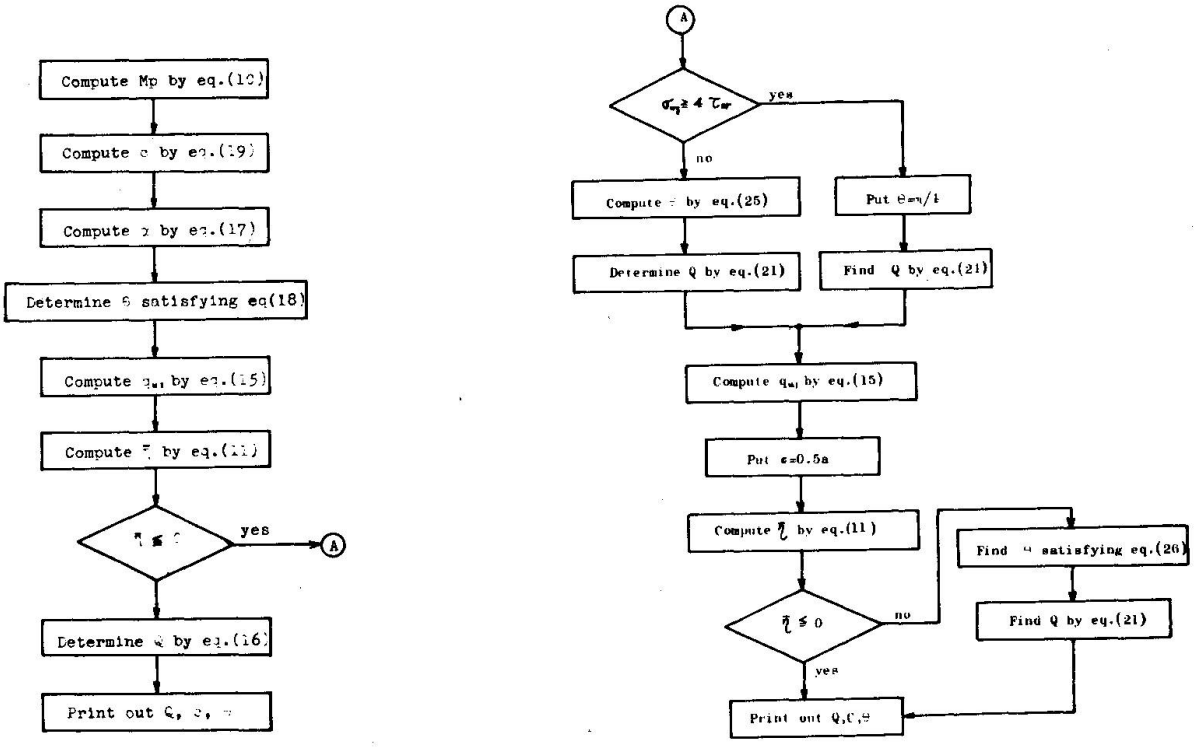


Fig. 3 Block diagram for ultimate shear force computation program

IV. PROOF TESTS

The primary purpose of the main tests was to justify the newly developed theory for the ultimate strength of unsymmetrical plate girder under shear.

Five girders could be loaded statically to fail for this purpose. Since it was unde sirable to fail in any panel other than the test-panel, the girder was designed so that the failure loads were remarkably different for adjacent panels.

4.1 Girder Specimens

Among the five girders tested, each of three girders had a one-sided longitudinal stiffener at the middle of web depth shown in Fig.4, the other had two one-sided longitudinal stiffeners on the internal trisector of the web. The test girders were connected to a rigid supporting girder by high tension bolts shown in Fig.5.

Moreover, two corner plates were welded to increase the ridigity of upper flange shown in Fig.6. The dimensions of test girders are given in Table 1.

4.2 Test Load

The load was applied by means of two oiljacks with total capacity 150tons. Five equal increments of load were taken up to two-thirds of predicted yield load, then the load was increased by 2tons at one time until failure.

4.3 Material Properties

The actual dimensions of the component plates of the test specimens were obtained from measurements on coupon plates cut from the various plates before fabrication. Table 2 shows the measured values of dimensions and the results of the tensile tests.

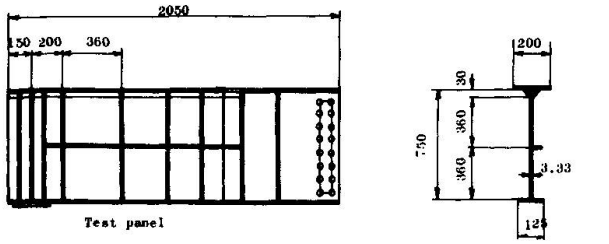


Fig. 4 Specimen A-2

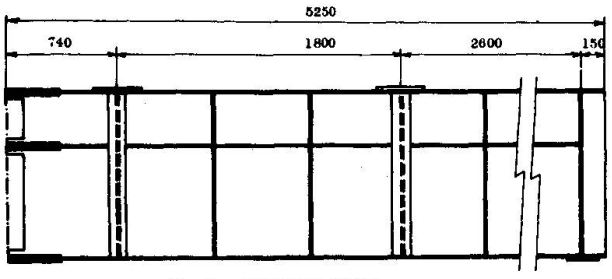


Fig. 5 Supporting girder

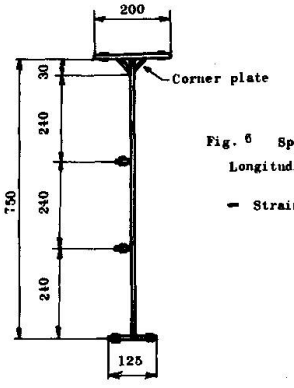


Fig. 6 Specimen A-3
Longitudinal stiffener section
35mm×1.5mm
— Strain gauge

Specimen	b cm	b_1 cm	t_w cm	b_f cm	t_f cm	a cm	n
A-1	67	32	0.333	12.5	1.0	32	1
A-2	75	36	0.333	12.5	1.0	36	1
A-3	75	24	0.333	12.5	1.0	36	2
A-4	83	40	0.333	12.5	1.0	40	1
A-5	83	26.7	0.333	12.5	1.0	40	2

b: depth of web
b_1: depth of subpanel
t_w: thickness of web
b_f: breadth of smaller flange
t_f: thickness of smaller flange
a : length of shear panel (distance between transverse stiffeners)
n : number of longitudinal stiffeners

Table 1 Dimensions of panel

Specimen	Size mm	Section area mm	Gauge distance mm	Yield load ton	Ultimate load ton	Yield stress kg/cm	Ultimate stress kg/cm	Stretch mm	Percentage of stretch %
A-1	39.93 9.93	396.505	200	15.00	21.11	3783.1	5399.7	44.0	22.0
A-2	39.95 9.85	393.508	200	14.89	21.06	3784.0	5352.0	45.0	22.5
A-3	40.00 9.65	386.000	200	14.50	20.69	3756.9	5360.1	41.0	20.5
B-1	40.07 10.69	428.348	200	15.93	24.02	3719.4	5608.2	41.0	20.5
B-2	40.07 10.79	432.355	200	15.98	24.16	3695.7	5588.1	41.0	20.5
B-3	39.87 10.61	423.021	200	16.00	23.80	3782.5	5626.2	41.0	22.5
C	50.03 14.09	704.923	200	25.70	40.10	3646.0	5688.8	44.0	22.0
D	25.25 4.44	112.110	50	5.50	6.46	4906.3	5762.2	10.5	21.0
E-1	23.17 3.33	77.156	50	3.50	4.35	4533.7	5637.9	16.5	33.0
E-2	23.20 3.35	77.72	50	3.29	4.19	4233.1	5391.1	15.5	31.0
E-3	23.17 3.33	77.156	50	3.27	4.19	4237.9	5430.6	16.0	32.0

A,B,C,D: SM50 for flange E : SM50 for web

Table 2 Tensile test of material

V. TEST RESULTS

In this paper, the typical data of many test results are shown as follows.

5.1 Deflection of Web

The lateral deflections at the middle point of each subpanel under shear were plotted against load as shown in Fig.7.

5.2 Strain of Flanges

The longitudinal strain of both flanges were plotted against load as shown in Fig.8. It can be considered that these strains were suddenly increased due to shear buckling of web plate. Therefore, actual critical shear buckling stresses may be observed from these diagrams. Thus, the experimental critical shear stresses could be seen to coincide with the theoretical ones of subpanel fixed at both longitudinal sides and simply supported at vertical sides.

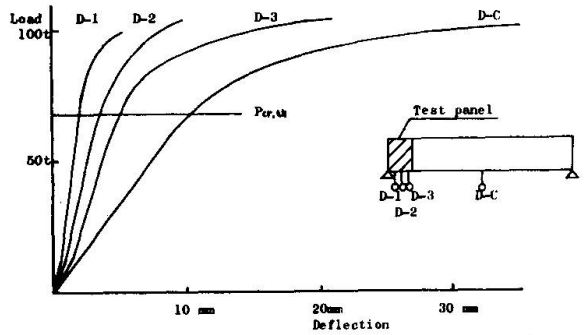


Fig. 9 Vertical deflection of girder A-2

5.3 Deflection of Girders

The deflection of several points measured by dialgages were plotted against loads as shown in Fig.9. Most curves tend to deviate from a straight line just after predicted critical shear buckling load $P_{cr,th}$ as shown in the same figures. From these facts, the theoretical values for shear buckling can be considered to be acceptable.

5.4 Strain of Longitudinal Stiffeners

The strains of longitudinal stiffeners were plotted against applied load as shown in Fig.10. After buckling of web, the longitudinal stiffeners of all test girders, except A-4, were bent upward. This behavior was caused by the fact that the membrane were pull up towards the upper flange which was more rigid than the lower one. Moreover, the longitudinal strains at the opposite side to stiffened surface of web were compressive in the case of one stiffener, or compressive at upper stiffener and tensile at lower one in the case of two stiffeners.

5.5 Strain of Transverse Stiffener

The strains of transverse stiffeners were plotted against applied load as shown in Fig.11. The strain scarcely induced in any transverse stiffener up to web buckling. The strain of transverse stiffeners as well as flanges, however, abruptly increase after buckling of web, because the frame consisting of flanges and stiffeners must bear the internal forces transfered from the buckled web.

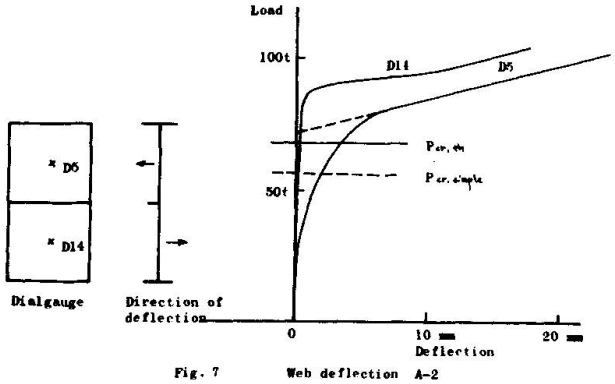


Fig. 7 Web deflection A-2

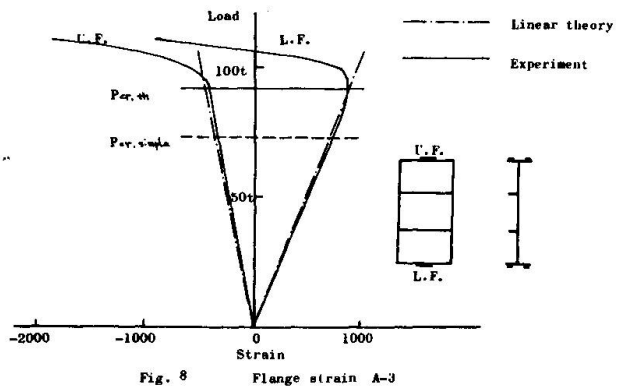


Fig. 8 Flange strain A-3

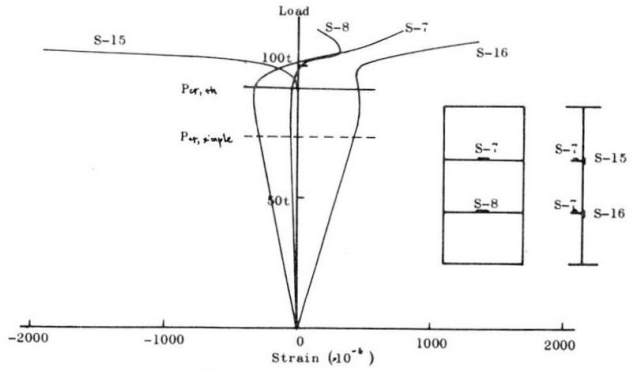


Fig. 10 Strain of longitudinal stiffeners

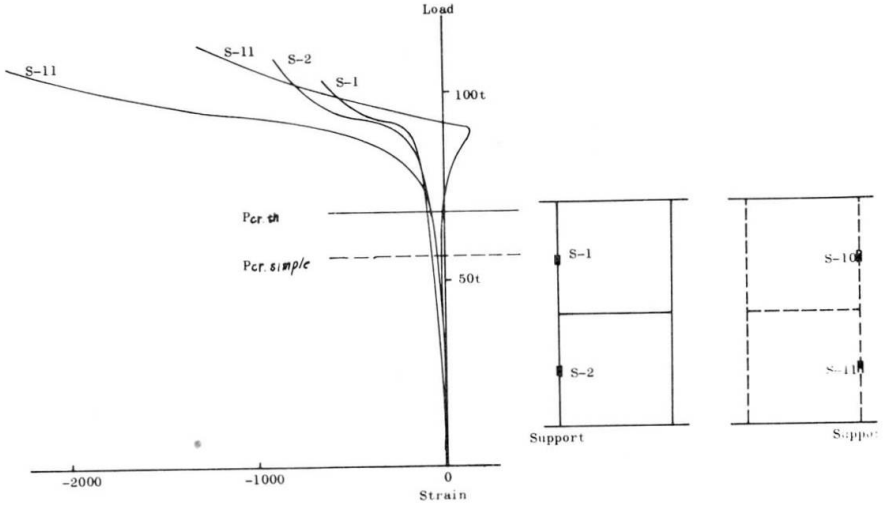
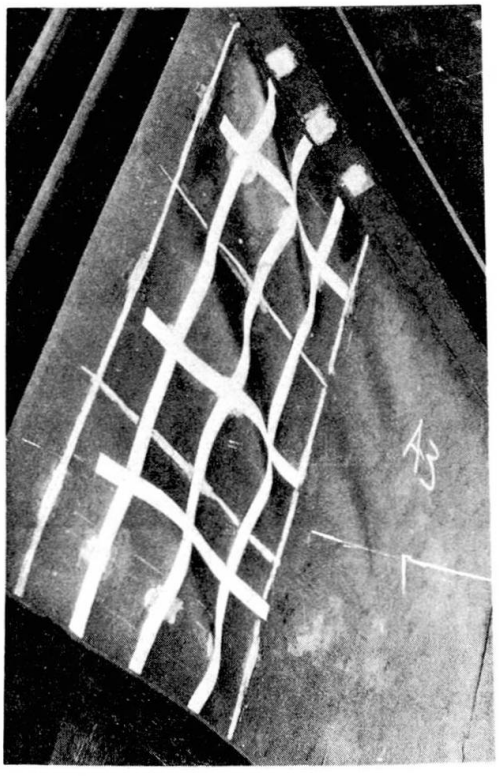
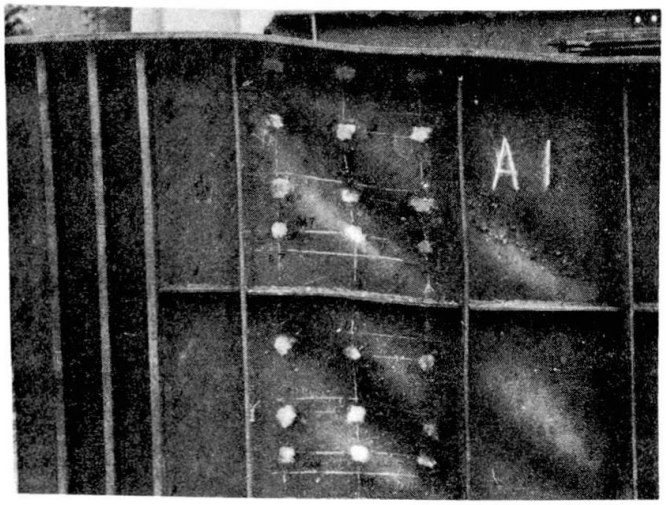


Fig. 11 Strain of traseverse stiffner A-2





VI. COMPARISON BETWEEN PREDICTED VALUES AND EXPERIMENTAL RESULTS

The experimentally obtained ultimate load Q_{ex} were compared with the values Q_{th} based on the author's theory as shown in Table 3. It can be seen that the fairly agreement between them could be obtained.

Moreover, the strength of every stiffeners except A-4 girder may be recognized to be so enough as to resist elastically diagonal tensile stress.

In Table 3, the experimental results given by other authors also agree with author's theoretical values.

VII. REQUIREMENT FOR FLANGE STRENGTH

In order to establish a well balanced design of girder under shear, how large flange or stiffener should be adopted? Concerning with this problem, it is rational to choose such a flange as to make the girder fail in the second mode for given dimension of web plate.

Hence, for the web satisfying inequality (23), the maximum ultimate shear force Q_{max} can be given when θ is equal to $\pi/4$, that is

$$Q_{max} = \tau_{wy}\, b\, t_w + 4M_p/a \tag{27}$$

Herein, the required plastic moment $M_{p,req}$ of smaller flange may be given from eq(26) as follows,

$$M_{p,req} = \frac{a^2}{32}\left(\sigma_{wy} - 2\tau_{cr}\right) t_w \tag{28}$$

However, if inequality(23) cannot be satisfied for given web, the inclination $\theta = \theta_0$ should be decided by eq.(25). Therefore, Q_{max} takes the value given by substituting $\theta = \theta_0$ into eq.(21). In this case, a required plastic moment $M_{p,req}$ of flange should be found by a following formula.

$$M_{p,req} = \frac{a^2}{16}\left(\sigma_{wy} - 2\tau_{cr}\sin 2\theta_0\right)\sin^2\theta_0 \cdot t_w \tag{29}$$

The required full plastic moments $M_{p,req}$ of desirable flange for girder specimens can be compared with the actual ones $M_{p,act}$ in Table 3. The strength of flange in the girder at which the 3rd mode of failure was occured was too large to be appropriately and economically designed for designated web. While, in the girder collapsing in the 1st mode, the strength of flange was too small to use up the full strength of girder.

CONCLUSIONS

An approach to the limit design of thin-web plate girder with transverse and longitudinal stiffeners under shear was proposed in this paper. It was shown by analysis that there were four modes of failure any one of which would

be induced in accordance with the relative strength of flange to the web. This calculating method of ultimate strength includes both the finding of the mode of failure for designated dimension of member and the determination of ultimate strength.

It was shown that the test results given by several authors fairly agreed with the theoretical ones.

The rational design method of shear panel, especially the determination of desirable dimension of flange was indicated here.

The stiffness of longitudinal stiffener has only to be sufficient to increase the critical buckling stress as much as possible and make the stiffener remain straight up to the collapse load of girder.

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	Specimen	Web			Flange			a	Plastic hinge C	Mode of failure	cr, th
		b (b_1)	tw	Vwy	bf	tf	Vfy				
1	A-1	67 (32)	0.333	4,534	12.5	1.0	3,783	32	None	3rd	1758
	A-2	75 (36)	0.333	4,233	12.5	1.0	3,784	36	None	3rd	1555
	A-3	75 (24)	0.333	4,235	12.5	1.0	3,756	36	None	3rd	1842
	A-4	83 (40)	0.333	4,395	12.5	1.0	3,738	40	20.0	2nd	1450
	A-5	83 (27)	0.333	4,238	12.5	1.0	3,738	40	None	3rd	1769
2	TG-1	100	0.25	2,037	16	0.506	2,862	100	19.0	1st	145.8
	TG-1'	100	0.25	2,037	16	0.526	2,862	100	19.0	1st	145.8
	TG-2	100	0.25	2,037	20	1.008	2,862	100	19.2	1st	145.8
	TG-2'	100	0.25	2,037	20	1.012	2,862	100	19.2	1st	145.8
	TG-3	100	0.25	2,037	20	1,643	2,862	100	50.0	2nd	145.8
	TG-3'	100	0.25	2,037	20	1.642	2,862	100	50.0	2nd	145.8
	TG-4	100	0.25	2,037	20	2.016	2,862	100	50.0	2nd	145.8
	TG-4'	100	0.25	2,037	20	2.013	2,862	100	50.0	2nd	145.8
	TG-5	100	0.25	2,037	25	2.975	2,862	100	None	4th	145.8
	TG-5'	100	0.25	2,037	25	2,972	2,862	100	None	4th	145.8
3	G6-T1	127	0.49	2,580	30.8	1.975	2,665	190.5	36.6	1st	306
	G6-T2	127	0.49	2,580	30.8	1.975	2,665	95.2	47.6	2nd	435
	G6-T3	127	0.49	2,580	30.8	1.975	2,665	63.4	None	3rd	741
	G7-T1	127	0.498	2,580	30.95	1.95	2,645	127	24.4	1st	358
	G7-T2	127	0.498	2,580	30.95	1.95	2,645	127	24.4	1st	358
	G8-T1	127	0.50	2,680	30.45	1.902	2,910	381	73.2	1st	283
	G9-T1	127	0.333	3,130	30.45	1.902	2,940	381	73.4	1st	125
	G9-T2	127	0.333	3,130	30.45	1.902	2,940	190.5	36.7	1st	145
4	B	120	0.45	5,000	24.0	1.2	5,000	120	22.9	1st	328
5	F10-P1	127	0.665	2,405	39.5	3.20	1,915	190.3	95.2	2nd	579
	F10-P5	127	0.653	2,720	40.75	2.54	2,025	152.4	76.2	2nd	590
	F11-P3	240(193)	0.665	2,405	35.9	3.20	1,915	241	46.0	1st	262
	F11-P1	241(193)	0.66	2,405	36.0	3.18	1,738	335	64.0	1st	236
6	LS3-T1	127(85)	0.46	2,690	36	3.84	2,095	190	95	2nd	556

	Specimen	Longitudinal stiffener	Q_{ex} x 10^3	Q_{th} x 10^3	Q_{ex}/Q_{th}	Mp, req.	Mp, act
1	A-1	1	56.5	55.5	1.02	5.692×10^{3}	1.182×10^{4}
	A-2	1	57.5	56.8	1.01	7.630×10^{3}	1.183×10^{4}
	A-3	2	59.0	61.2	0.96	5.191×10^{3}	1.174×10^{4}
	A-4	1	63.0	63.0	1.00	1.271×10^{4}	1.168×10^{4}
	A-5	2	63.5	66.4	0.96	7.025×10^{3}	1.168×10^{4}
2	TG-1	0	15.54	14.09	1.10	1.364×10^{5}	2.931×10^{3}
	TG-1'	0	11.85	14.15	0.84 } (0.97)*	1.364×10^{5}	3.167×10^{4}
	TG-2	0	16.3	16.98	0.96	1.364×10^{5}	1.454×10^{4}
	TG-2'	0	14.15	17.01	0.83 } (0.90)*	1.364×10^{5}	1.466×10^{4}
	TG-3	0	19.4	19.32	1.00	1.364×10^{5}	3.863×10^{4}
	TG-3'	0	19.35	19.32	1.00 } (1.00)*	1.364×10^{5}	3.858×10^{4}
	TG-4	0	22.3	21.86	1.02	1.364×10^{5}	5.816×10^{4}
	TG-4'	0	21.1	21.86	0.97 } (1.00)*	1.364×10^{5}	5.799×10^{5}
	TG-5	0	31.47	31.78	0.99	1.364×10^{5}	1.583×10^{5}
	TG-5'	0	30.5	31.78	0.96 } (0.98)*	1.364×10^{5}	1.580×10^{5}
3	G6-T1	0	52.5	50.6	1.03	1.005×10^{6}	8.004×10^{4}
	G6-T2	0	68.0	70.4	0.97	2.373×10^{5}	8.004×10^{4}
	G6-T3	0	80.4	81.0	0.99	4.038×10^{5}	8.004×10^{4}
	G7-T1	0	63.5	65.9	0.96	4.677×10^{5}	7.782×10^{4}
	G7-T2	0	65.8	65.9	1.00	4.677×10^{5}	7.782×10^{4}
	G8-T1	0	38.6	36.5	1.06	4.796×10^{6}	8.014×10^{4}
	G9-T1	0	21.8	21.4	1.02	4.349×10^{6}	8.096×10^{4}
	G9-T2	0	34.0	36.2	0.94	1.072×10^{6}	8.096×10^{4}
4	B	0	76.0	78.7	0.97	8.798×10^{5}	4.320×10^{4}
5	F10-P1	0	83.5	88.1	0.95	9.389×10^{5}	1.936×10^{5}
	F10-P5	0	86.2	92.8	0.93	7.303×10^{5}	1.331×10^{5}
	F11-P3	1	127.5	128.9	0.99	2.271×10^{6}	1.760×10^{5}
	F11-P1	1	111.9	105.5	1.06	4.472×10^{6}	1.582×10^{5}
6	LS3-T1	1	63.5	71.3	0.89	8.190×10^{5}	2.780×10^{5}

bl : Depth of main subpanel

C : Distance between plastic hinge and the nearest corner of panel

Mp,req.: Required plastic moment of desirable flange for given web

Mp,act : Existing plastic moment of flange

(1) Komatsu

(2) Skaloud

(3) Basler

(4) Konishi

(5) Patterson

(6) Cooper

* mean value of twin specemens

unit : cm, kg

Table 3 Comparison between experimental and theoretical ultimate load.

```
      REAL NU,KS,MU,MP,MS
      E=2.1*10.0**6
      NU=0.3
      PAI=3.141592
      DO 100 I=1,21
      READ(5,1000) B,B1,TW,SWY,BF,TF,SFY,A
 1000 FORMAT(8F8.0)
      READ(5,1001) KS,SK
 1001 FORMAT(2F8.0)
      ASP=A/B
      TWY=0.5*SWY
      WRITE(6,2000) B,TW,SWY,BF,TF,SFY,A
 2000 FORMAT(1H ,4H  B=F8.2,4H TW=F8.2,5H SWY=F8.2,4H BF=F8.2,4H TF=F8.2
     1,4H SF=F8.2,4H  A=F8.2)
      TCS=SK*PAI**2*E/12.0/(1.0-NU**2)*(TW/B1)**2
      IF(TCS-0.5*TWY) 20,20,21
   21 TCS=TWY*(1.0-3.0*(1.0-NU**2)/SK/PAI**2/E*TWY*(B1/TW)**2)
   20 CONTINUE
      TCR=KS*PAI**2*E/12.0/(1.0-NU**2)*(TW/B1)**2
      IF(TCR-0.5*TWY) 10,10,11
   11 TCR=TWY*(1.0-3.0*(1.0-NU**2)/KS/PAI**2/E*TWY*(B1/TW)**2)
   10 WRITE(6,2001) TCR,TCS
 2001 FORMAT(1H ,4HTCR=F8.2,13X,4HTCS=F8.2)
      MP =0.25*SFY*BF*TF**2
      C=(0.38+1.782*MP/SWY/B**2/TW)*0.5/(1.0+1.782*MP/SWY/B**2/TW)*A
      CA =C/A
      ALP=(A-C/2.0)/(A-C/3.0)*A
      II=1
   52 T  =0.10
      JJ=1
      BB=0.0
      STEP=0.01
      NC =1
    1 T  =T+STEP
      GO TO (2,3),II
    2 AA =SWY*(B*COS(2.0*T)-ALP*SIN(2.0*T))+2.0*TCR*(ALP*(COS(2.0*T)-COS
     1(4.0*T))-B*SIN(4.0*T))
      GO TO 4
    3 AA =MP-A**2/16.0*(SWY-2.0*TCR*SIN(2.0*T))*SIN(T)**2*TW
    4 CC =AA*BB
      IF(CC.LT.0.0) GO TO 5
      BB =AA
      NC =NC+1
      IF(NC.GT.100) GO TO 7
      GO TO 1
    5 T  =T-STEP
      JJ =JJ+1
      IF(JJ.EQ.3) GO TO 6
      STEP=STEP/10.0
      GO TO 1
    6 T  =T+0.5*STEP
    7 CONTINUE
      TH =T
      GO TO (18,19),II
   18 CONTINUE

   50 QU1=(SWY-2.0*TCR*SIN(2.0*TH))*SIN(TH)**2*TW
   51 ETB=(QU1*(A-C)-4.0*MP/C)/(A-C/3.0)
      IF(ETB) 12,12,13
   13 CONTINUE
      WRITE(6,2004)
 2004 FORMAT(1H ,11X,8H1ST MODE)
      Q  =0.5*(SWY-2.0*TCR*SIN(2.0*TH))*(B*SIN(2.0*TH)-ALP*(1.0-COS(2.0*
     1TH)))*TW
      Q  =Q+4.0*MP*ALP/C/(A-C)+TCR*B*TW
C     OUT PUT
   53 TUL=Q/B/TW
      MU =TUL/TCR
      TH=TH*180.0/PAI
      IF(SWY-4.0*TCR) 22,23,23
   22 X  =SWY/4.0/TCR
      TT =0.5*ATAN(X/SQRT(1.0-X**2))
      STM=(SWY-2.0*TCR*SIN(2.0*TT))
      PP    =A**2/16.0*STM*SIN(TT)**2*TW
      GO TO 60
   23 STM=SWY-2.0*TCR
      PP    =A**2/32.0*STM*TW
   60 CONTINUE
      WRITE(6,2002) MP,PP,ALP,QU1,ETB
 2002 FORMAT(1H ,3HMP=E14.7,8H MP,REQ=E14.7,4X,4HALP=E15.7,6X,4HQU1=E15.
     17,4X,4HETB=E15.7)
      WRITE(6,2005) Q,C,TH,TUL,MU
 2005 FORMAT(1H ,2HQ=E15.7,5X,2HC=E15.7,5X,3HTH=E15.7,5X,5HTULT=E15.7,5X
```

```
     1,3HMU=F15.7///)
 78   WRITE(6,2003)
 79 2003 FORMAT(///)
 80   GO TO 56
 81  12 IF(SWY-4.0*TCR) 14,15,15
 82  14 X  =SWY/4.0/TCR
 83   TH =0.5*ATAN(X/SQRT(1.0-X**2))
 84   Q  =(0.5*SWY*SIN(2.0*TH)+TCR*(1.0-SIN(2.0*TH)**2))*B*TW
 85   GO TO 54
 86  15 TH =PAI/4.0
 87   Q  =TWY*B*TW+4.0*MP/A
 88  54 QU1=(SWY-2.0*TCR*SIN(2.0*TH))*SIN(TH)**2*TW
 89   C  =0.5*A
 90   ETB=(QU1*(A-C)-4.0*MP/C)/(A-C/3.0)
 91   IF(ETB) 16,16,17
 92  16 IF(TH.EQ.PAI/4.0) GO TO 55
 93   WRITE(6,2006)
 94 2006 FORMAT(1H ,11X,8H3RD MODE)
 95   GO TO 53
 96  55 WRITE(6,2007)
 97 2007 FORMAT(1H ,11X,8H4TH MODE)
 98   GO TO 53
 99  17 II=2
100   GO TO 52
101  19 Q  =(0.5*SWY*SIN(2.0*TH)+TCR*(1.0-SIN(2.0*TH)**2))*B*TW
102   WRITE(6,2008)
103 2008 FORMAT(1H ,11X,8H2ND MODE)
104   GO TO 53

105  56 CONTINUE
106 100 CONTINUE
107   STOP
108   END
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