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

Band: 8 (1968)

Artikel: Influence of flange stiffness upon the load carrying capacity of webs in shear

Autor: Rockey, K.C. / Skaloud, M.

DOI: https://doi.org/10.5169/seals-8781

IIc

Influence of Flange Stiffness upon the Load Carrying Capacity of Webs in Shear

Influence de la rigidité des ailes sur la charge de rupture de cisaillement de l'âme

Einfluß der Flanschsteifigkeit auf die Traglast des Stehbleches unter Schub

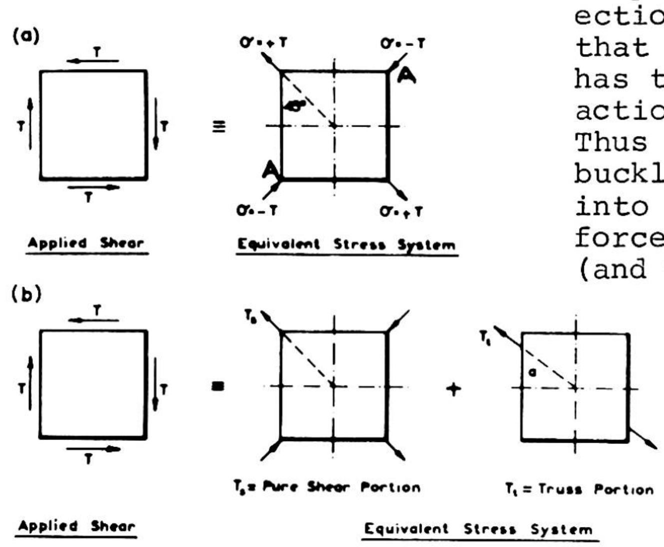
K.C. ROCKEY
M.Sc., Ph.D., C.Eng. M.I.C.E.
Professor of Civil and
Structural Engineering,
University of South Wales
and Monmouthshire, Cardiff

M. ŠKALOUD
C.Sc., Docent,
Czechoslovak Academy
of Science, Prague

I. INTRODUCTION

Previous studies, [1 - 5], have shown that the post buckled behaviour of a shear panel is greatly influenced by the rigidity of its boundary members. The present paper briefly reports on an extensive experimental study which has been carried out to determine the relationships which exist between flange stiffness and the ultimate shear load of the webs of welded plate girders.

A pure shear loading results in equal tensile and compressive principle stresses, acting as shown in Figure 1. After a shear panel buckles, it has no capacity to carry further compressive load in the direction AA, with the result that any additional shear load has to be carried by a 'truss action', see Figure 1(b). Thus the load carried by a buckled web can be divided into two parts, T_s the shear force carried in pure shear (and which is equal to the



(b) Applied Shear = Ts + Tt.

FIGURE 1 - STRESS SYSTEMS IN A WEB SUBJECTED TO SHEAR. FIGURE 1(a) SHOWS THE CONDITION FOR A PLANE WEB, WHILE FIGURE 1(b) SHOWS IT FOR A BUCKLED WEB

critical shear load), and a diagonal tensile load T_t. The distribution of the membrane stress in a web plate is greatly affected by the rigidity of the boundary members. If the framework is quite rigid, the diagonal tensile stresses will be uniformly distributed across the plate. If, however, the boundary members do not have sufficient stiffness, their deformations under load can be such as to significantly affect the form of the buckled web. In the extreme case, a very narrow diagonal buckle will form in the web plate and the resulting diagonal tensile membrane stress field will be far from uniform.

Some twelve years ago, one of the Authors [3] made a study of the influence of the flexural rigidity of flanges upon the immediate post buckled behaviour of shear webs. This study, which was carried out on web plates having length to width ratios of 2 and 3, showed that if the flange stiffness parameter I/b^3t did not equal or exceed 0.00035 $[P/Pcr - 1]$ then the web plate would develop excessively large lateral deformations. This earlier study was confined to a study of the elastic behaviour of buckled web plates and the present study has been carried out to examine the influence of flange rigidity upon the collapse load of a shear panel.

Reference 3 contained four diagrams which well illustrated the influence of flange rigidity upon the behaviour of shear webs and these are reproduced in Figures 2 to 5. During the testing of Girder TG4 (R), shown in Figures 2 and 3, it was noted that as the applied load was increased the buckles which formed in the web shifted and slipped into the diagonal form shown. Figure 6 shows the theoretical buckled pattern developed in a simply supported web plate of aspect ratio 2.5 at the onset of buckling. It will be noted that there are two buckles inclined at approximately 45° to the flanges. Thus it is seen that as a result of the flange deflecting, the half waves have altered both their shape and inclination, and developed into the form shown in Figure 3. The web plate of the girder had been coated with a brittle lacquer and the chalk marks in Figure 2 indicate the regions where yielding occurred. It is seen that the yield zone is restricted to a narrow diagonal strip. This prompted the investigator to heavily reinforce the flange of the next girder and to observe what happened.

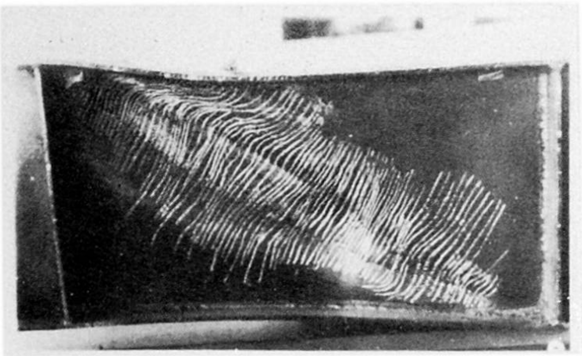


FIGURE 2 - STRESS DISTRIBUTION IN GIRDER TG4(R) OBTAINED USING A BRITTLE LACQUER

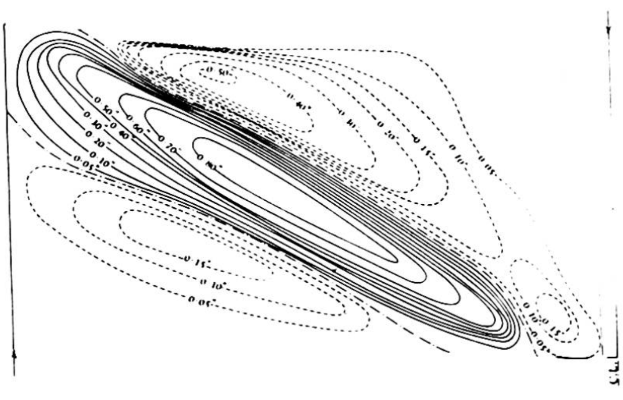


FIGURE 3 - RESIDUAL WEB DEFLECTIONS IN GIRDER TG4(R)

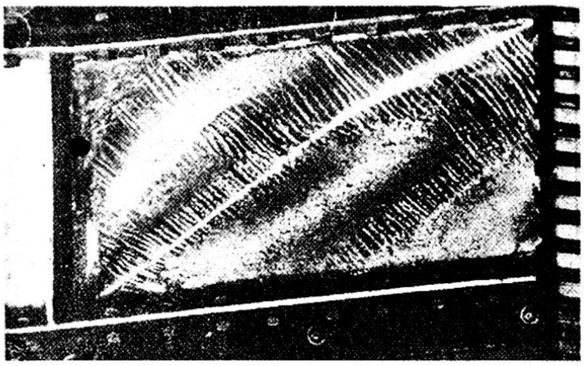


FIGURE 4 - STRESS DISTRIBUTION IN GIRDER TG3(R) OBTAINED USING A BRITTLE LACQUER.

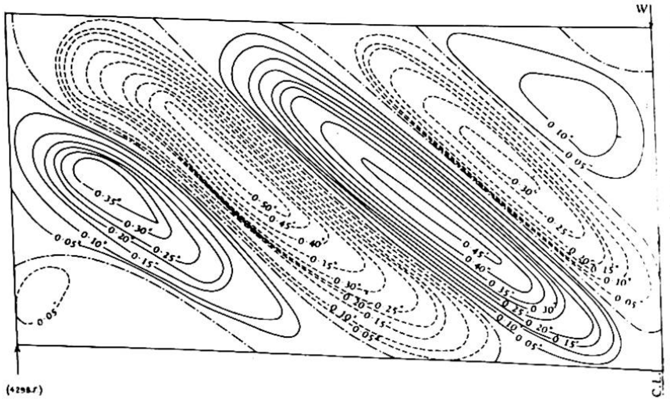


FIGURE 5 - RESIDUAL WEB DEFLECTIONS IN GIRDER TG3(R)

This is well illustrated in Figures 4 and 5. Not only did the half-waves maintain their inclination, but in accordance with accepted theory increased in number as the load was progressively increased beyond the buckling load. From the strain distribution given by the brittle lacquer, it will be seen that in this case, the distribution of stress is far more uniform. Figures 3 and 5 clearly show that both the magnitude of the buckles and the form of the buckled webs were greatly affected by a change in flange rigidity.

In addition, as a natural consequence of the more uniform stress distribution, the ultimate load of Girder TG3 (R) was 1.6 times that of Girder TG4 (R).

Since then, Basler and his colleagues [7 - 10] at Lehigh have developed an ultimate load method of design based on a modified incomplete diagonal tension field. This assumed that the flanges of most practical girders are so flexible that in the post buckled range, the buckles slip into the diagonal form shown in Figure 7.

Using this given structional mode, Basler obtained the following expression for the ultimate load of a shear panel:-

$$P_{ult} = \left[\tau_{cr}\, dt + \frac{\sqrt{3}\,\tau_y\, dt}{\sqrt{1+\alpha^2}} \left(1 - \tau_{cr}/\tau_y\right) \right] \quad \ldots \quad \ldots \quad \ldots \quad \ldots \quad (2)$$

Where T_{cr} = critical shear stress

T_y = shear yield stress

α = length/depth ratio of panel

d = depth of panel

t = thickness of panel

The results of this most interesting study forms the basis of the current design procedures used in the U.S.A. The present paper will show that the shear load which can be carried by a web plate is greatly influenced by the flexural stiffness of the flanges and that the collapse model assumed by Basler is not correct for a wide range of structural units.

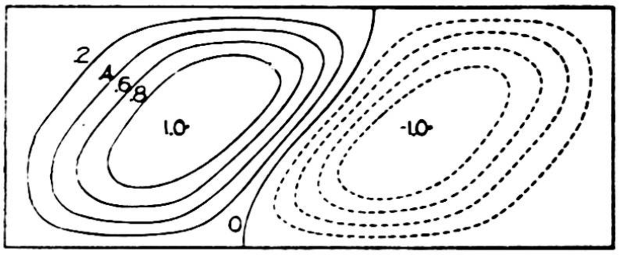


FIGURE 6 - THEORETICAL BUCKLE FORMATION IN SIMPLY SUPPORTED PLATE OF ASPECT RATIO 2.5

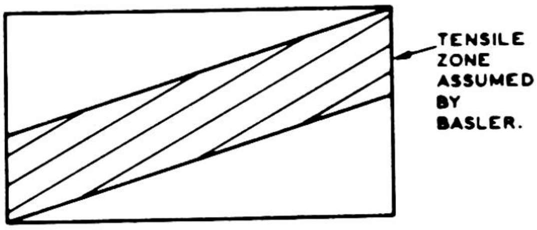


FIGURE 7

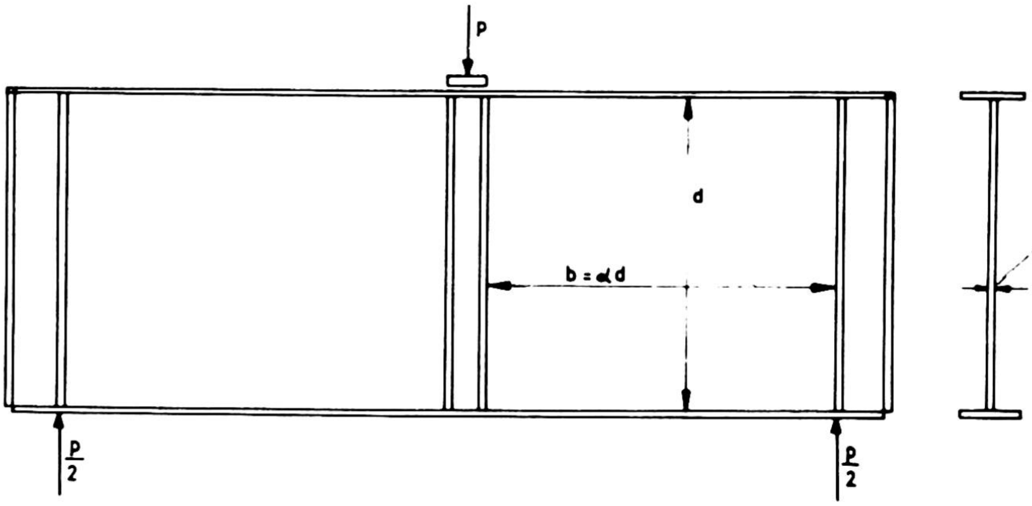


FIGURE 8 - DETAILS OF TEST GIRDERS

II. TEST PROGRAMME

II.(a) Apparatus

Details of the test girders are given in Figure 8 and Table 1. All girders were of welded construction and manufactured from normal mild steel, to B.S.15. It was decided to use web plates of constant nominal depth and thickness and to simply vary the stiffness of the flanges and the span of the girders. Including the initial pilot test, 24 girders were tested. Nine of these provided panels having an aspect ratio of 2.0, eight girders had panels of aspect ratio 1.5 and six girders had aspect ratio panels of 2.0.

Since it was essential to know the deflected form of the web of each girder, it was necessary to take sufficient deflection readings to enable contour plots of the deflected web to be constructed. This was achieved by marking a 3 inch square grid on each web plate and measuring the change in deflection of the web at each grid position, with the aid of a portable deflection bar carrying a dial gauge graduated in units of 0.001 inch.

The deflection of the flanges were measured using the apparatus shown in Figure 9. It consisted of a keyed bar on to which the individual blocks holding the dial gauges could be securely fixed. The end clamps, which were attached to the vertical stiffeners, were designed to ensure that any relative translational or rotational movement between the two stiffeners did not impose any bending on the bar, and so affect the dial gauge readings.

The strains in the web plate and flanges were measured using both electric resistance strain gauges and 2 inch 'demec' gauges. Figure 10 clearly shows both the diagonal grid of demec gauge points which was employed on most of the girders and the 3 inch grid for the deflection readings. From these demec gauge readings, it was possible to measure the distribution of the diagonal strain both prior to and after collapse of the girders.

TABLE 1

GIRDER NO.	ASPECT RATIO	t THICKNESS IN.	d IN.	I/b^3t UNITS OF 10^{-6}	ULTIMATE LOAD TONS	% GAIN OVER MINIMUM VALUE OBTAINED WITH PARTICULAR ASPECT RATIO	Ps	$\frac{Pexp}{Ps}$	BASLER ULTIMATE LOAD Pb (EQUATION)	$\frac{Pb}{Pexp}$
TG1	1.0	0.107	24	7.47	22.6	0	47.30	0.478	33.44	1.48
TG1'	1.0	0.107	24	7.47	24	6	47.30	0.507	33.44	1.39
TG2	1.0	0.107	24	13.5	25.2	11.5	47.30	0.533	33.44	1.33
TG2'	1.0	0.107	24	13.5	23.5	4	47.30	0.497	33.44	1.42
TG3	1.0	0.108	24	27.9	28.5	26	47.74	0.597	33.84	1.49
TG3'	1.0	0.108	24	27.9	27	19.5	47.74	0.565	33.84	1.25
TG4	1.0	0.107	24	54.5	31.8	41	47.30	0.672	33.44	1.05
TG4'	1.0	0.107	24	54.5	30.3	34	47.30	0.641	33.44	1.10
TG13	1.0	0.103	24	224	41.7	84	52.23	0.798	35.98	0.86
TG5'	1.5	0.103	24	7.35	23.4	0	52.23	0.448	29.18	1.25
TG5	1.5	0.103	24	7.35	26.0	11	52.23	0.498	29.18	1.12
TG6	1.5	0.103	24	34	28.4	21	52.23	0.544	29.18	1.03
TG6'	1.5	0.103	24	34	26.7	14	52.23	0.511	29.18	1.09
TG7	1.5	0.103	24	129	35.5	52	52.23	0.680	29.18	0.82
TG7'	1.5	0.103	24	129	38.6	65	52.23	0.789	29.18	0.76
TG8	1.5	0.103	24	254	40.3	72.5	52.23	0.771	29.18	0.72
TG8'	1.5	0.103	24	254	41.4	76.5	52.23	0.793	29.18	0.70
TG9'	2.0	0.103	24	3.1	24.05	0	52.23	0.460	24.65	1.02
TG9	2.0	0.103	24	3.1	24.55	2	52.23	0.470	24.65	1.00
TG10	2.0	0.103	24	14.3	25.7	7	52.77	0.487	24.65	0.96
TG11	2.0	0.103	24.25	98.4	35.5	47.5	52.23	0.680	24.65	0.69
TG12	2.0	0.103	24	384.	45.7	90	52.23	0.875	24.65	0.54
TG12'	2.0	0.103	24	384.	29.2	104	52.23	0.942	24.65	0.50

TABLE 2

M A T E R I A L	YIELD STRESS TONS/IN^2	ULTIMATE STRESS TONS/IN^2
Sheet thickness 0.103 in.	18.3	22.06
Sheet thickness 0.107 in.	15.95	20.6

II.(b) Testing Procedure

Prior to testing, the initial deformations in the web plate were accurately determined using the systems mentioned above. The testing procedure simply consisted of taking dial gauge and strain readings at frequent load intervals.

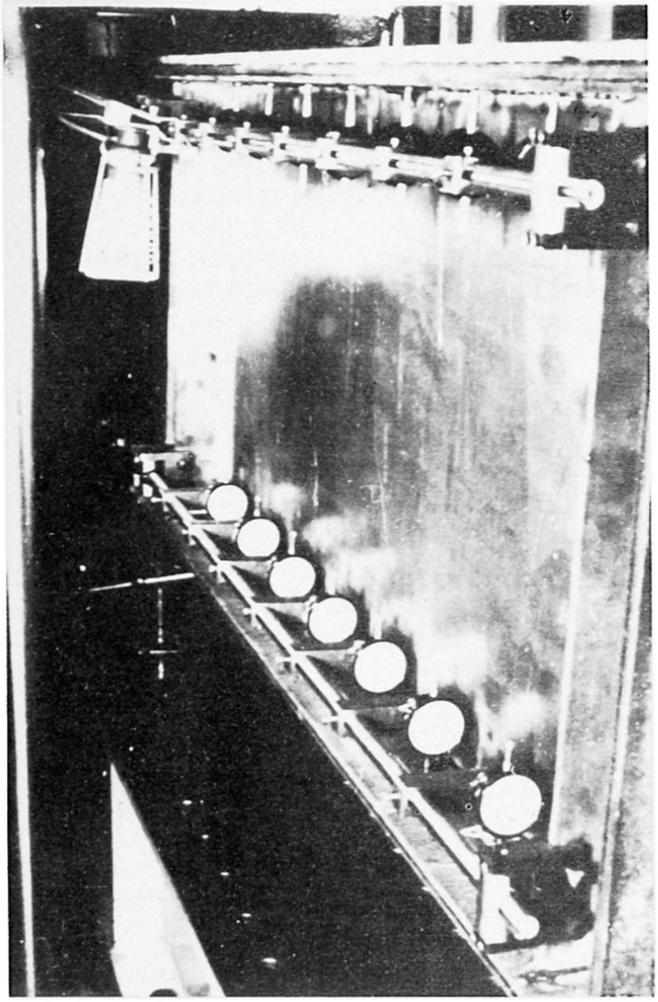


FIGURE 9

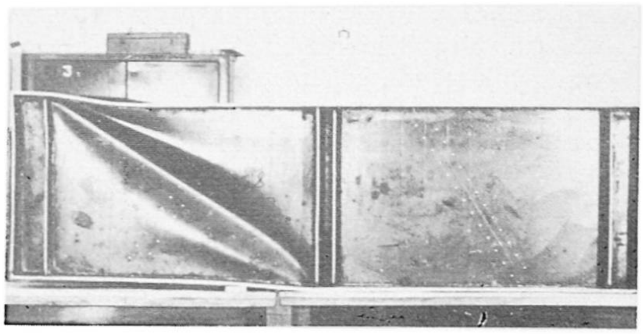


FIGURE 10 - GIRDER TG5 AFTER TEST TO FAILURE

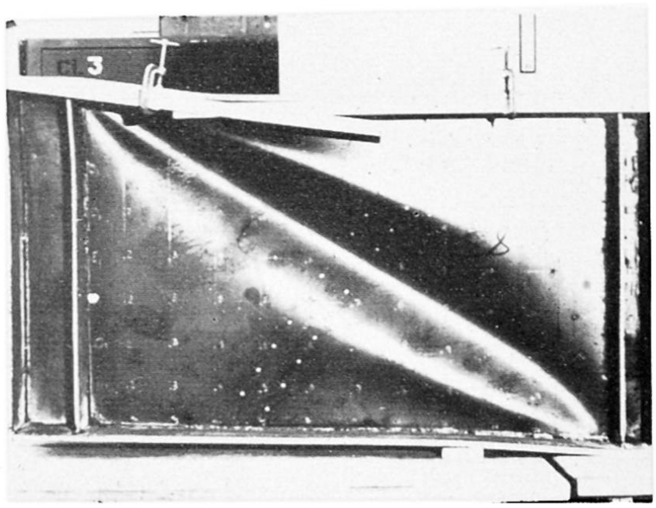


FIGURE 11 - GIRDER TG5

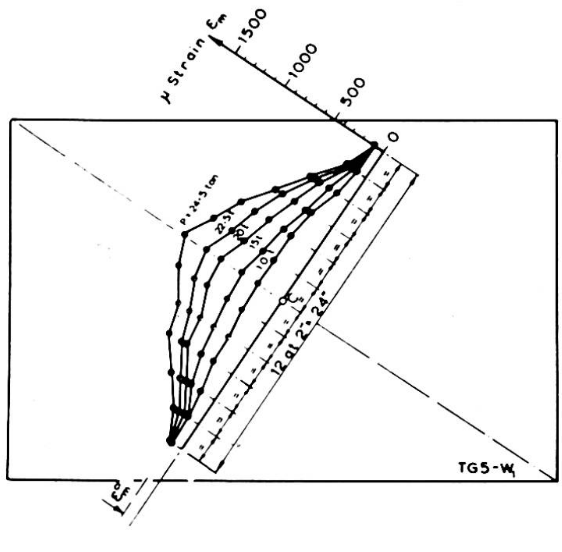


FIGURE 12 - STRAIN DISTRIBUTION IN PANEL W1 OF GIRDER TG5

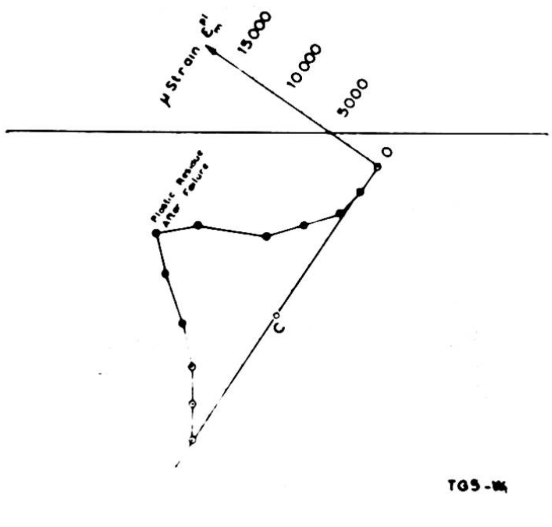


FIGURE 13 - RESIDUAL STRAINS IN PANEL W1 OF GIRDER TG5

To obtain the maximum elastic load which could be carried by each girder, at relatively high loads, the girders were unloaded after each small increment of load and the residual strains and deflections determined. At the end of each test deflection readings were taken of both the residual flange and web deflections.

III. DISCUSSION OF RESULTS

The limited scope of the present paper does not permit one to deal with all aspects of the investigations. Attention will therefore be restricted to an examination of the actual collapse loads of the girders. The development of the post buckled behaviour will be presented in forthcoming publications.

Figures 10 and 11 show girder TG5 after it had been tested to failure. From these photographs it is seen that a deep diagonal wave has formed in the web plate and that in addition a partial plastic hinge has formed in each flange. Figures 12 and 13 show how the diagonal strains developed in panel 1 of girder TG5. It will be noted that the distribution of both the pre-collapse and residual strains are reasonably symmetric about the diagonal. Figures 14 and 15 give the residual deflection plots for girders TG1 and TG2. Also plotted in Figure 14 are the diagonal membrane strains developed; again it will be noted that the strains are symmetrical about the diagonal. These failures were very typical of the mode of failure obtained with all of the girders and which is shown diagramatically in Figure 16. All of the girders failed by the same mechanism in which a diagonal strip of the web flowed plastically with the development of plastic hinges in the flanges. These illustrations make it quite clear that the mode of failure is not similar to that assumed by Basler, see Figure 7.

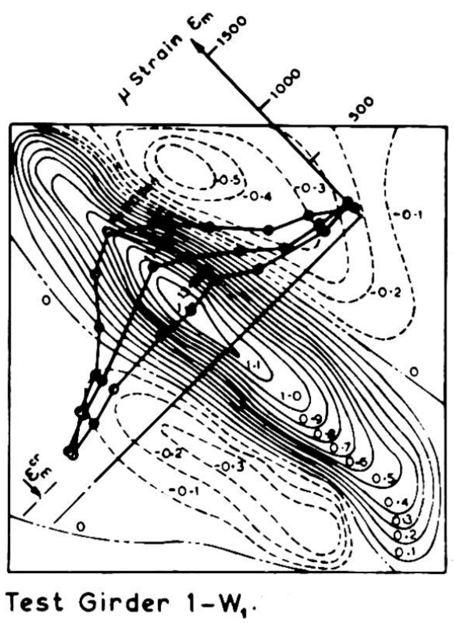


FIGURE 14 - RESIDUAL DEFLECTION (INS) IN PANEL W1 OF GIRDER TG1. ALSO SHOWN ARE DIAGONAL MEMBRANE STRAINS.

It was noted that the position of the plastic hinge varied with the flange stiffness parameter I/b^3t. For high values of the flange stiffness parameter, C is nearly equal to half the panel length b and for relatively flexible flanges is quite small. Figure 17 shows how the dimension C varies with the I/b^3t parameter whilst Figure 18 shows how the ultimate load also increases with

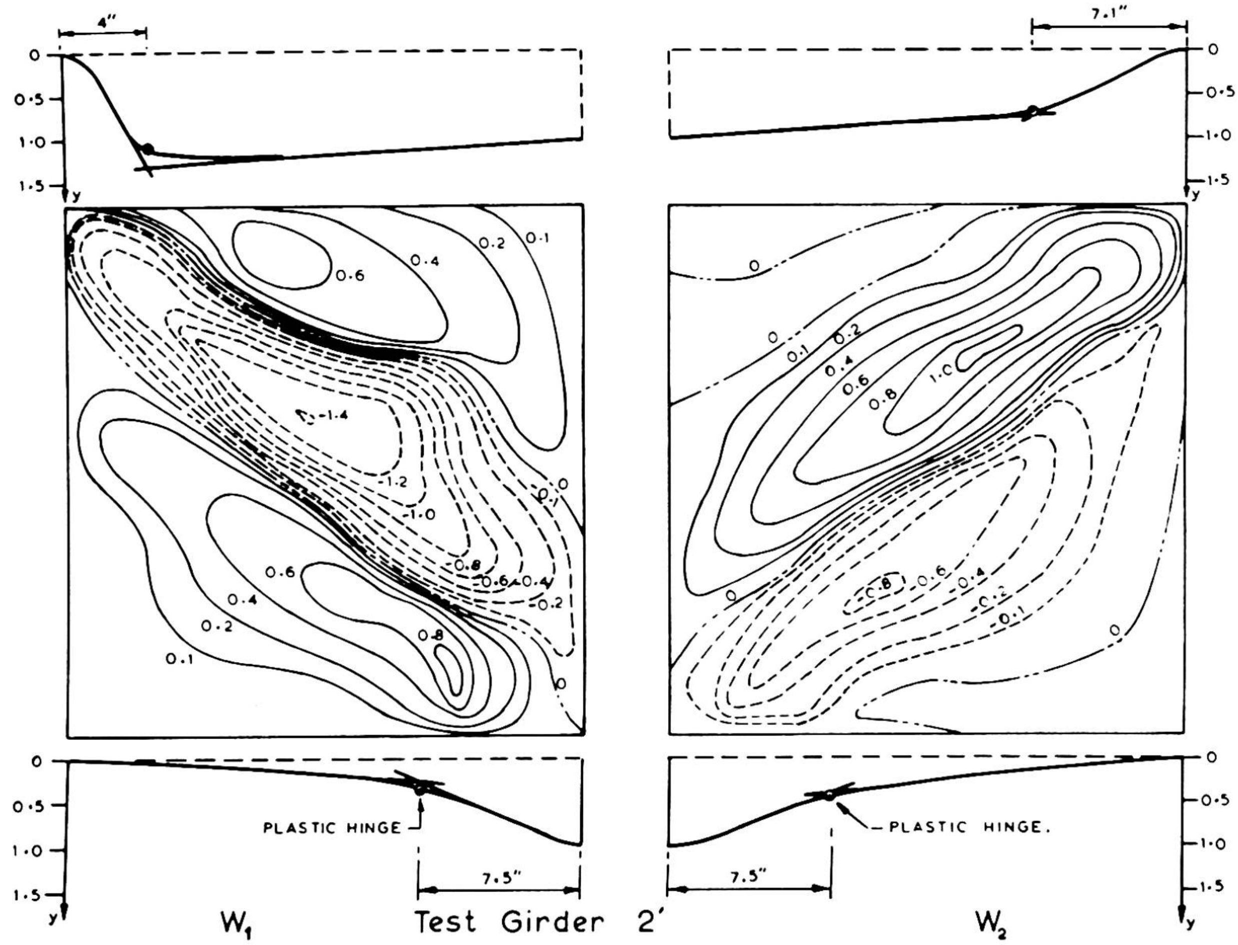


FIGURE 15 - RESIDUAL FLANGE AND WEB DEFLECTIONS (INS) FOR GIRDER TG2'

the I/b^3t parameter. This is due to the increase in the effective width of the diagonal strip with an increase in the value of C. The ultimate loads for the various girders are given in Table 1, from which for example it will be noted that girder TG12' carried 104% greater shear load than girder TG9' which had the same depth and thickness of web plate. Table 1 thus clearly demonstrates how, by employing stiffer flange assemblies, it is possible to obtain a higher shear load carrying capacity. Massonnet [11], in his excellent paper presented to this Conference, has already commented on the benefits which are obtained by the use of tubular flanges such as those shown in Figure 2 of his paper.

It is also worth noting that when the flanges were quite flexible the failure was a relatively sudden phenomenon, whereas in the case of the girders with the heavier flanges the collapse process was quite slow and therefore not such a dangerous type of failure.

Table 1 also gives the ultimate load of the shear panels calculated using Equation (2) as derived by Basler and his colleagues. The extreme right-hand column of Table 1 gives the ratio of the ultimate load (P_b) calculated by Equation (2) to the experimental value P_{exp}. It will be noted that for square panels having very flexible flanges, the Basler load is some 48% greater than the experimental value, whilst the values of $\alpha = 1.5$ and 2.0, the Basler load seriously underestimates the true load carrying capacity for higher values of the flange flexibility parameter I/b^3t. Thus it is seen that it is necessary to allow for the influence

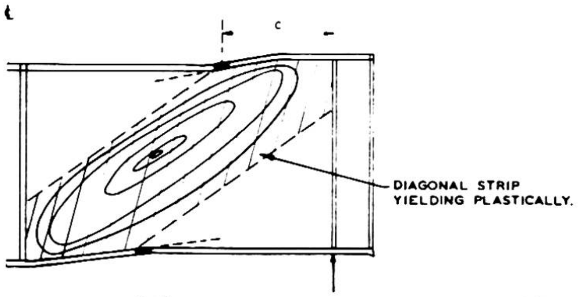


FIGURE 16 - DIAGRAMATIC ILLUSTRATION OF COLLAPSE MECHANISM

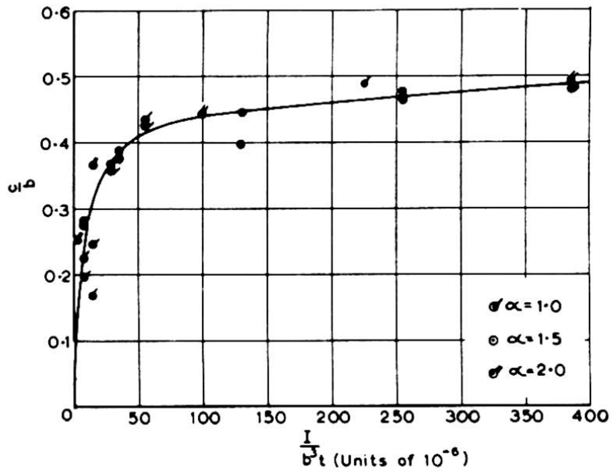


FIGURE 17 - VARIATION OF HINGE POSITION WITH I/b^3t PARAMETER

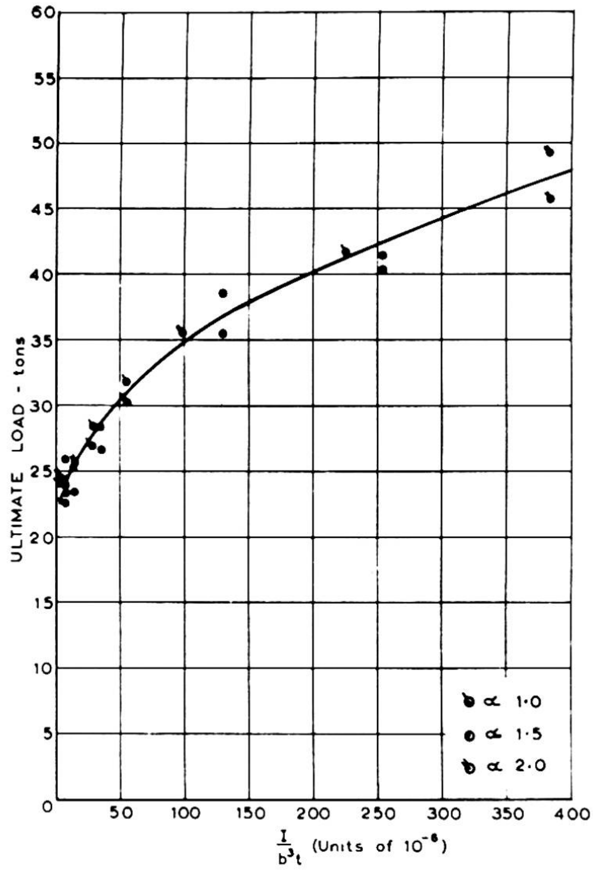


FIGURE 18 - VARIATION OF ULTIMATE LOAD WITH I/b^3t PARAMETER

of the flange stiffness parameter I/b^3t when designing a shear panel.

With the aid of the concept shown in Figure 16 and the relationship shown in Figure 17, a new design procedure is being developed by the Authors which should result in higher design stresses for girders having stiff flange assemblies such as obtained with closed sectioned flanges.

CONCLUSION

The paper has shown that the flexural stiffness of a flange has a significant effect upon the load carrying capacity of webs loaded in shear and indicates how the ultimate load of a shear panel varies with the stiffness of the flanges.

ACKNOWLEDGEMENT

The Authors wish to thank C.I.R.I.A. and B.C.S.A. for sponsoring this investigation.

BIBLIOGRAPHY

1. D.M.A. LEGGETT and H.G. HOPKINS. 'The Effect of Flange Stiffness on the Stress in a Plate Web Spar under Shear'. H.M. Stationery Office R. &. M. No. 2434.

2. BERGMAN, S.G.A. 'Behaviour of Buckled Rectangular Plates under the Action of Shearing Forces'. Book Stockholm. 1948.

3. ROCKEY, K.C. 'The Influence of Flange Stiffness upon the Post-buckled Behaviour of Web Plates subjected to Shear'. 'Engineering'. Vol. 184, pp. 788-792. Dec. 20th 1957.

4. P. KUHN, J. P. PETERSON and L.R. LEVIN. 'A Summary of Diagonal Tension'. Parts I and II. N.A.C.A. Tech. Notes 2661 and 2662. May 1952.

5. YAMAKI, Nuborv. 'Postbuckling Behaviour of a Simply Supported Infinite Strip under the Action of Shearing Forces'. ZAMM. 44(1964). Heft 3, Seite 107-117.

6. STEIN, Manuel and NEFF, John. 'Buckling Stresses of Simply Supported Rectangular Flat Plates in Shear'. N.A.C.A. Tech. Note 1222, March 1947.

7. BASLER, k., YEM, B.T., MUELLER, J.A., THRULIMANN, B. 'Web Buckling Tests on Welded Plate Girders. Welding Research Council Bulletin No. 64 September 1960.

8. BASLER, K. 'Strength of Plate Girders in Shear'. Fritz. Engineering Lab. Report No. 251-20. 1960.

9. BASLER, K. 'Strength of Plate Girders in Shear'. Trams. A.S.C.E. Vol. 128. Part II. 1963. pp. 683.

10. COOPER, P.B. 'Bending and Strength of Longitudinally Stiffened Plate Girders'. Fritz. Engineering Lab. Report No. 304.6 September 1965.

11. MASSONNET, Ch. 'Thin Walled Deep Plate Girders'. International Association for Bridge and Structural Engineering". Preliminary Publication pp. 157-177. 8th Congress, New York, 1968.

LIST OF SYMBOLS

t Thickness of web plate

d Depth of web plate

b Width of web plate

$\alpha = b/d$

P Applied load

P_{cr} Critical load in shear

P_{ult} Ultimate load of the girder

P_b Collapse load calculated according to Equation (2) (Basler load)

P_{exp} Experimental collapse load.

P_s Collapse load in pure shear = $(\tau_y dt)$

τ_y Shear yield stress

τ_{cr} Critical shear stress

I Moment of Inertia of flange about an axis through its centroid and perpendicular to the plane of the web.

SUMMARY

The paper examines the influence of flange rigidity upon the ultimate load behaviour of shear webs. The tests show that the ultimate load carrying capacity of a shear web is greatly influenced by the stiffness of the flange members. An improved collapse mechanism for welded shear panels is presented.

RÉSUMÉ

Le rapport étudie l'influence de la rigidité des ailes sur le comportement de l'âme sous charge de rupture de cisaillement. Les tests montrent que la charge de rupture dépend fortement de la rigidité des ailes. En outre, le rapport présente un mécanisme de rupture amélioré pour âmes soudées.

ZUSAMMENFASSUNG

In diesem Bericht wird der Einfluss der Flanschsteifigkeit auf das Traglastverhalten der schubbeanspruchten Stehbleche geprüft. Die Versuche zeigen, dass die Traglast des Stehbleches durch die Flanschsteifigkeit in erheblichem Masse beeinflusst wird. Ein verbesserter Gelenkmechanismus wird für den geschweissten Steg angegeben.

Leere Seite
Blank page
Page vide