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Poutres de grandes dimensions à âme mince
Dünnwandige hohe Blechträger
Thin-Walled Deep Plate Girders

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DISCUSSION PRÉPARÉE / VORBEREITETE DISKUSSION / PREPARED DISCUSSION

Behaviour of Longitudinally Reinforced Plate Girders

Comportement de poutres avec raidisseurs longitudinaux

Das Verhalten des Vollwandträgers mit Längssteife

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1. INTRODUCTION

During the past two decades a great deal of research has been conducted on plate girders to determine the influence of the rigidity of longitudinal stiffeners upon the behaviour of plate girders. Since this work has been fully documented in earlier papers [1-3] and by Massonnet in the excellent paper he has presented at this conference, no detailed survey of published work will be given here and references will be confined to those papers of direct association with the study presented in this paper.

Massonnet [1,4-7] was one of the earliest researchers to examine the influence of stiffener rigidity upon the behaviour of plate girder webs. From his tests on double sided stiffeners, he noted that for stiffeners to behave effectively in the post buckled range it was necessary that they should have a rigidity $\gamma = k\gamma^*$, where γ^* is the theoretical rigidity which an ideal stiffener should have according to the linear theory of web buckling, in order to form a nodal line when the adjacent panels buckle. Massonnet then showed how k varied with the position of the longitudinal stiffeners and arrived at the values given below:-

Distance Between Horizontal Stiffener and Compression Flange	k
0.5d	3
0.33d	4
0.25d	6
0.20d	7

In earlier work on aluminium plate girders one of the Authors [2] observed that a value of k equal to 2 would be sufficient for single, double sided stiffeners placed at 0.2d from the compression flange.

These aluminium girders were of bolted construction, with relatively stiff flange assemblies and initially flat webs. Consequently, these girders behave differently in the immediate post buckled range ($W/W_{cr} > 1$) than a welded girder which is initially distorted and subjected to the deleterious influence of the residual stresses caused by the welding processes. Theoretical studies of the post buckled behaviour of stiffened web plates by Skaloud [8] and jointly by Massonnet, Donea and Skaloud [9-11] are in general agreement with the experimental observations referred to above.

Since 1961, a comprehensive study of many of the problems encountered with welded steel girders has been in hand at the University of Lehigh. Cooper [12,13], reporting on pure bending tests conducted on girders reinforced by a single longitudinal stiffener, commented that "the longitudinal stiffeners which were used in these tests had no significant effect upon the observed ultimate loads, except for specimen LB6 where a 11% increase in ultimate load was realised". However, the test data given by Cooper indicates that failure occurred at less than twice the theoretical buckling load calculated on the assumption that the web was simply supported along its boundary. Thus in this case one would not expect the influence of stiffener rigidity upon the post buckled behaviour of the web plate to be too significant.

In a more recent report, which has just become available to the writers, Cooper [14] presents a most extensive report on further tests conducted on girders having web plates reinforced by a single longitudinal stiffener; in this report he concludes that "if properly proportioned longitudinal stiffeners are provided, a significant increase in bending strength can result". As will be seen later, this conclusion is in general agreement with one of the findings of the present Authors.

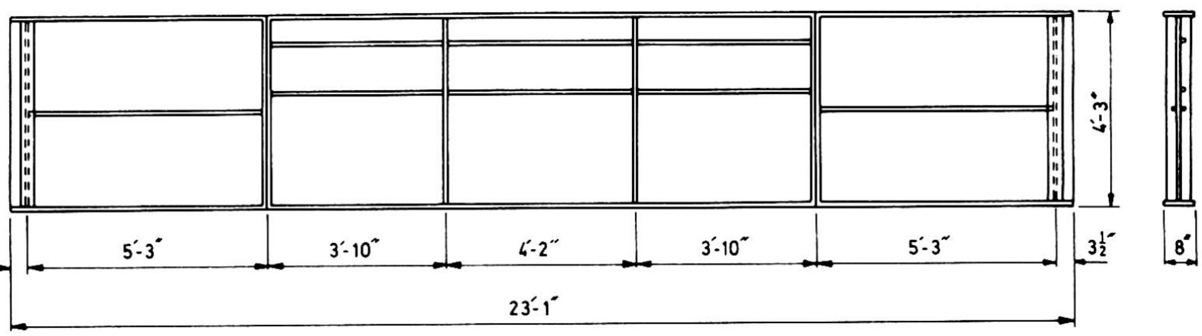
The present study was conducted to examine in particular the following:-

1. The behaviour of <u>single sided</u> longitudinal stiffeners on webs subjected to pure bending stresses. The study dealing with web plates reinforced by either one or two longitudinal stiffeners.

2. The influence of the ratio of γ/γ^* upon the collapse load of the girders.

II. <u>DESIGN OF GIRDERS</u>

The design details of the girders tested are given in <u>Figure 1</u> and <u>Table 1</u>.



<u>FIGURE 1 - DETAILS OF TEST GIRDERS</u>

Since the object of the investigation was to determine the behaviour of girders subjected to pure bending, only that portion of the girders loaded accordingly was of interest. Therefore, it was decided to use girders of the design shown in Figure 1, with the detachable end panels which could be repeatedly used. These end panels which were designed to ensure that failure always took place in the central panels, were attached to the central section by means of ¾ in. diameter, high tensile steel bolts distributed across the depth of the girder. In addition, it was found necessary to employ a short cover plate welded to the tension flange of the central section and bolted to the end panels, since during the testing of girder TG7-1 (the first test conducted) a weld fractured in this region, the tension flange breaking away from the vertical member.

TABLE NO. 1

TEST GIRDERS	NO. OF LONGI-TUDINAL STIF-FENERS	POSITION OF STIFFENERS		ACTUAL DIMENSION OF STIFFENERS (IN.)		γ/γ^*	ULTIMATE LOAD TONS
		FIRST (TOP)	SECOND (BOTTOM)	FIRST (TOP)	SECOND (BOTTOM)		
TGO	0	-	-	-	-	-	48.2
TG1-1	1	0.2d	-	0.713x0.187	-	0.89	55.5
TG2-1	1	0.2d	-	0.872x0.187	-	1.63	57.15
TG4-1	1	0.2d	-	1.323x0.187	-	5.8	59.75
TG7-1*	1	0.2d	-	0.87 x0.187	-	3.28	44
TG1-2	2	0.123d	0.40d	0.765x0.187	0.758x0.187	0.64	54.75
TG2-2	2	0.123d	0.40d	1.008x0.187	1.014x0.187	1.51	52.5
TG3-2	2	0.123d	0.40d	1.262x0.187	1.261x0.187	2.94	62.5
TG4-2	2	0.123d	0.40d	1.503x0.187	1.531x0.187	5.11	64.0
TG5-2	2	0.123d	0.40d	1.741x0.187	1.746x0.187	7.75	64.75

* Double Sided Stiffener.

The central panel of one test girder, TG-O, was only reinforced by transverse stiffeners, this test thus providing a datum against which to judge the efficiency of the longitudinal stiffeners which were used on the other test panels. Four girders were fitted with a single longitudinal stiffener placed at one fifth of the clear web depth from the compression flange which is the optimum position when the flanges provide a simple support [15,16]. Three of these girders were fitted with single sided stiffeners, the fourth with a double sided stiffener.

The third test series consisted of five girders with two in number single sided longitudinal stiffeners whose dimensions are also given in Table 1. The stiffeners were again positioned such as to give the optimum buckling load for the girders [17].

The girders were of welded construction. On the recommendation of the British Welding Research Association, a staggered welding technique was used to weld the stiffeners to the web plate in an attempt to minimise initial deformations of the stiffeners and web plate. Unfortunately, even with this procedure, quite large initial deformations were formed, see Table 2.

The ratio of panel depth to web thickness was chosen to be 750:1, the web plate being 0.0666 in. thick and having a clear web depth between flanges of 50 in. This ensured that the ultimate load as calculated from simple plastic theory was approximately three times the critical load for girders with two lines of longitudinal stiffeners and six times the critical load for girders reinforced by a single longitudinal stiffener.

TABLE NO. 2

Maximum Initial Deformations (ins.)

	FIRST STIFFENER	SECOND STIFFENER	TOP PANEL	MIDDLE PANEL	LOWER PANEL
TGO	-	-	0.3	-	-
TG1-1	0.192	-	-0.241	-0.281	-
TG2-1	0.152	-	0.158	-0.193	-
TG4-1	0.056	-	0.166	-0.338	-
TG1-2	-0.032	0.37	-0.078	-0.113	-0.092
TG2-2	-0.110	0.138	-0.11	0.133	0.183
TG3-2	0.092	-0.05	-0.091	0.138	-0.073
TG4-2	0.020	0.06	-0.087	-0.155	-0.320
TG5-2	0.040	0.117	-0.089	+0.204	0.268

The dimensions of all stiffeners and those of the flanges were so chosen as to ensure that the failure of the girder should not be influenced by any local elastic buckling of the individual elements.

III. EXPERIMENTAL APPARATUS

The essential features of the testing rig can be seen in Figure 2. The girders were simply supported at their ends on case-hardened steel rollers and loaded equally at the junction of end panel and the central section by means of two hydraulically operated jacks of 100 tons capacity. The applied load was recorded by two load cells connected to the Elliott load indicators which can be seen in the right-hand side of Figure 2.

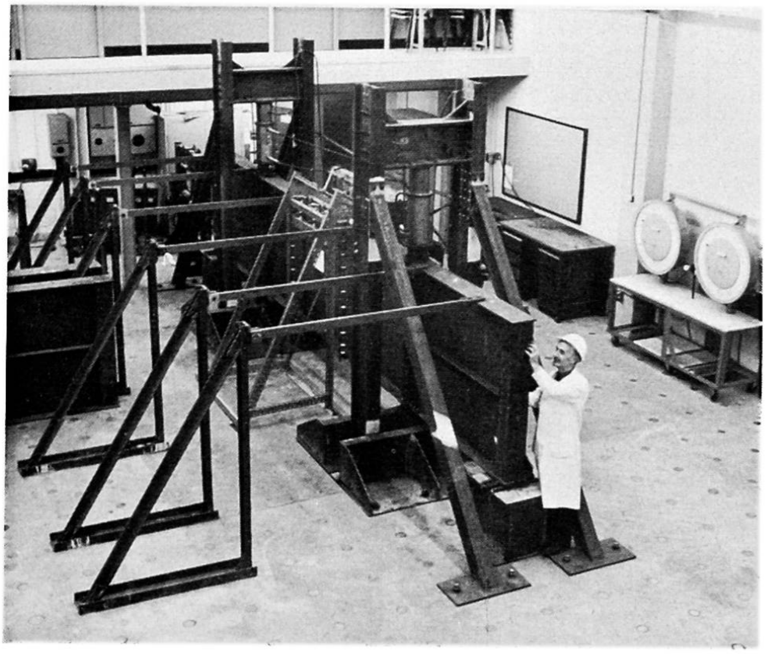


FIGURE 2
GENERAL VIEW OF A GIRDER UNDER TEST

The compression flange was restrained laterally by the use of the six stabilising trusses which are shown in Figure 2. Lever arms were pin connected at one end to these rigid trusses and at the other end to the compression flange of the girder. These lever arms, which were 6ft. long, allowed free vertical movement of the girder but restricted the lateral movement of the compression flange.

The rotation, vertical and lateral movements of the central panel of the test section, were recorded by means of the dial gauge system shown in Figure 3. The frame supporting the dial gauges was firmly bolted to the floor.

It was essential that the deflection of the longitudinal stiffeners was measured with extreme accuracy, and this was achieved by means of the dial gauge system shown in Figure 4. This apparatus was clamped to the transverse stiffeners by specially designed clamps, which ensured that the bar carrying the gauges remained unstressed during the loading of the girder. Eleven dial gauges were distributed along each stiffener. For the testing of the girder with no longitudinal stiffeners, TG-O, the same device, attached to the flanges, was used to check that the transverse stiffeners remained effectively rigid.

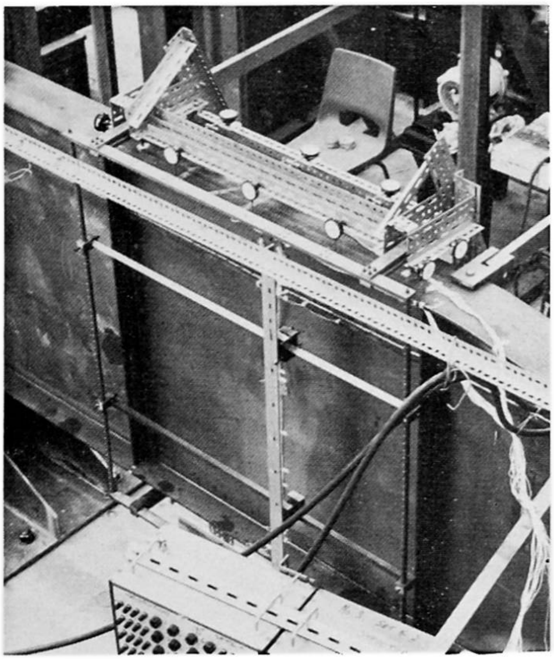


FIGURE 3

In order to be able to determine the effectiveness of the various stiffeners in restricting the lateral deformation of the web plate under load, the device shown in Figures 3 and 5 was used to measure web plate deflections over the whole of the centre panel. This framework was spring-loaded on to the compression and tension flanges in the plane of the web plate, small centering holes being used to accurately position the frame. Thus the apparatus was completely unaffected by any rotation induced in the flanges. Deflections were determined relative to the vertical aluminium bar which contained slots at convenient vertical intervals. This bar could freely slide along a pair of horizontal guides, thus allowing readings to be taken at any vertical section. Trial tests carried out with this apparatus established that very good reproducibility of results was obtained.

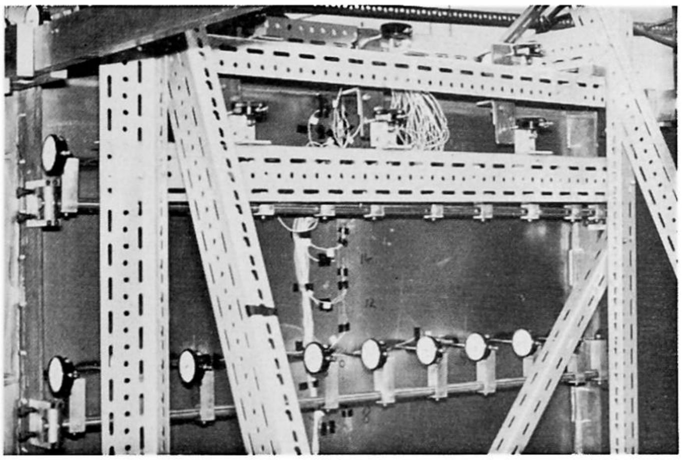


FIGURE 4
APPARATUS USED TO RECORD LATERAL DEFLECTIONS OF STIFFENERS

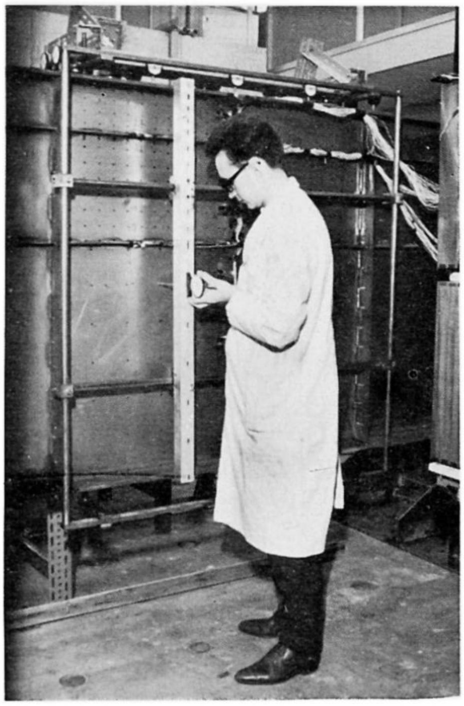


FIGURE 5
APPARATUS USED TO RECORD DEFLECTIONS OF WEB PLATE

Each girder was instrumented with electrical resistance strain gauges, ranging from over a hundred gauges for certain girders with two lines of longitudinal stiffeners to only twenty-five for girder TG-O. In each case gauges were attached to the longitudinal stiffeners as well as the web plate and flanges. Wherever possible orthogonal pairs of gauges were used to enable the evaluation of stress. Material tests were carried out on the web and flange material used in each girder and the synopsis of the results obtained is given in Table 3.

IV. PROCEDURE FOR TESTS

The strain gauge bridges were initially balanced to give zero readings and the initial readings of the various dial gauges were recorded. The lateral deflection of the web plate in the central panel was measured at a sufficient number of points to enable a contour plot of the initial deformed shape to be drawn. For the large panel adjacent to the tension flange, readings were taken over a 5 in. square mesh. For the panel adjacent to the compression flange in the case of those girders with one line of stiffeners and the panel bounded by the stiffeners in the case of the girders with two stiffeners, readings were taken on a 2 in. square grid. For the case of girders with two longitudinal stiffeners, readings in the panel adjacent to the compression flange were taken at 1½ in. intervals in the vertical direction and at 2 in. intervals in the longitudinal direction.

Load was applied to the girder in increments of 2½ tons, the strain gauge readings being noted at these intervals. In addition, in the initial stages, at each 10 ton load increment the stiffener and flange deflections were noted. However, as the ultimate load was approached, these readings were recorded more frequently.

Lateral deflection readings to provide contour plots of the entire centre panel were also taken at zero load and at load values beyond the theorētical buckling load.

V. RESULTS

Typical failures are shown in Figure 6. The failure mechanism for the unstiffened girder TG-O can be seen in Figure 6, whilst Figure 7 shows girder TG2-1 after failure and Figure 8 shows girder TG5-2 after collapse.

Bar the sole exception of girder TG7-1 where collapse of girder was due to a weld fracture in the tension zone, all remaining girders failed by the inward buckling of the compression flange. This inward collapse of the compression flange occurred suddenly with very little lateral movement or twisting. Figure 9 shows girder TG4-2 after collapse and indicates that the compression flange was on the point of collapse along its complete length.

TABLE NO. 3

MATERIAL	LIMIT OF PROPORTIONALITY TONS/IN^2	LOWER YIELD STRESS TONS/IN^2	UPPER YIELD STRESS TONS/IN^2	ULTIMATE STRESS TONS/IN^2
WEB PLATE	8.68	13.66	13.98	21.31
FLANGE PLATE	12.36	15.66	15.98	27.37

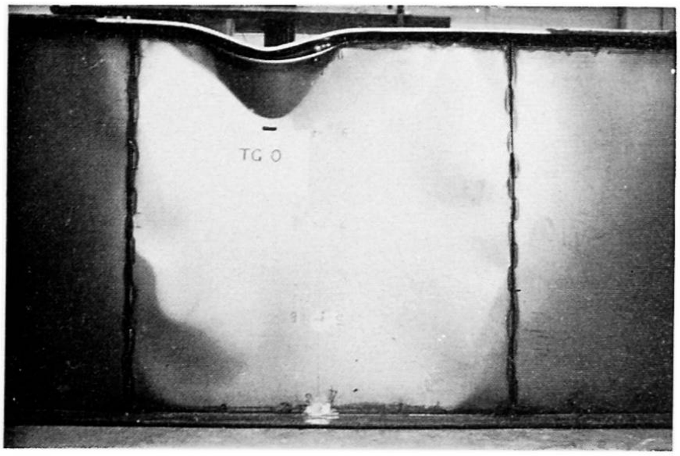


FIGURE 6

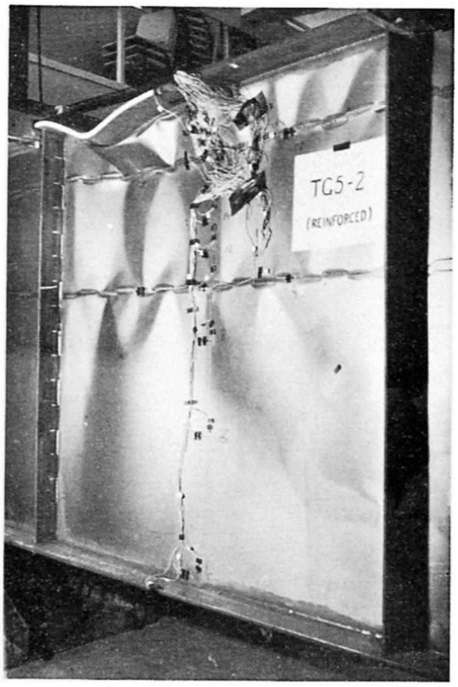


FIGURE 8

FIGURE 7

The collapse loads for the girders tested are given in Table 1, together with the ratio of the flexural rigidity of the longitudinal stiffeners to the theoretical optimum value for each girder. Figure 10 shows the percentage gain in load carrying capacity obtained by the use of stiffener systems with increasing flexural rigidity. The benefits of employing fully effective longitudinal stiffeners are clearly demonstrated. Also shown is the theoretical ultimate load for the girders as calculated from the simple plastic theory, ignoring the effects of strain hardening.

The growth of the maximum lateral deflection of the longitudinal stiffeners with increasing applied load is shown for the case of a single longitudinal stiffener in Figure 11 and corresponding diagrams for the case of two lines of longitudinal stiffeners in Figures 12 and 13. For the girders with two lines of longitudinal stiffeners it will be noted that the deflection of the top stiffener, i.e., the one nearer the compression flange significantly exceeds that of the lower stiffener. It will also be noted that there is a tendency for the stiffener deflection to decrease with increasing value in the ratio γ/γ^*.

FIGURE 9
VIEW OF COMPRESSION FLANGE AFTER COLLAPSE

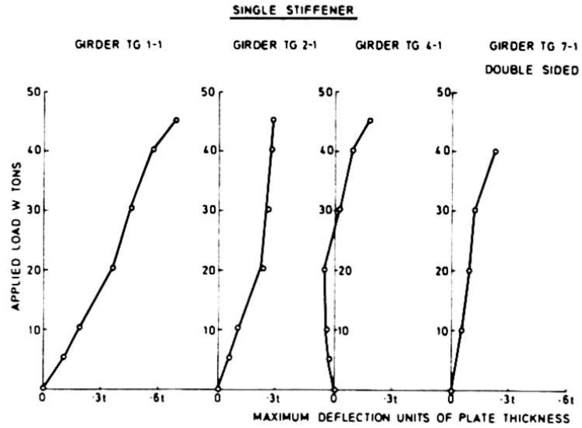


FIGURE 11
APPLIED LOAD, MAXIMUM LATERAL DEFLECTION PLOTS. SINGLE LONGITUDINAL STIFFENER

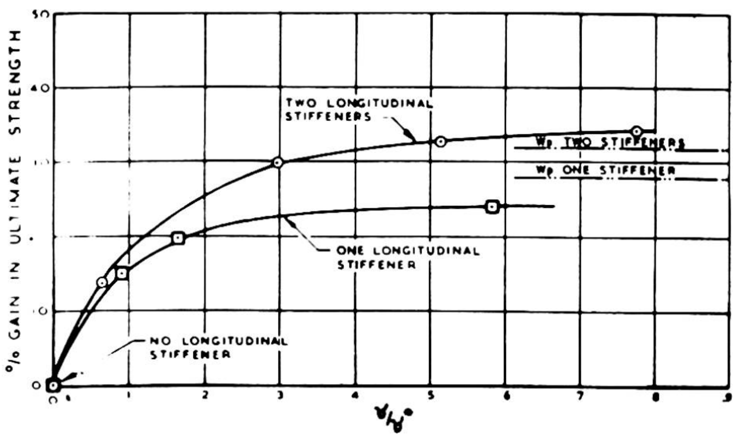


FIGURE 10

The web plate deflection readings enabled contour maps of the deformed surface to be drawn, a computer programme being used to determine the contour plots. For those girders reinforced by a single longitudinal stiffener, in each case the panel adjacent to the compression flange tended to produce the classical sine wave patterns usually associated with bending tests. These can be seen in Figure 14, which gives the additional web deflections recorded in Girder TG2-1. However, due to the increased buckling resistance, for those girders with two lines of stiffeners the upper two panels did not develop as well defined buckling patterns.

It also follows from the analyses of the deflected web plates that stiffeners having a flexural rigidity close to the theoretical optimum value tend to deflect with the web plates and do not form a nodal line in the post buckled range.

From the contour plots of the initial web deformation, the maximum positive and negative panel deflections could be found and these are given in Table 2, together with the corresponding values for the longitudinal stiffeners.

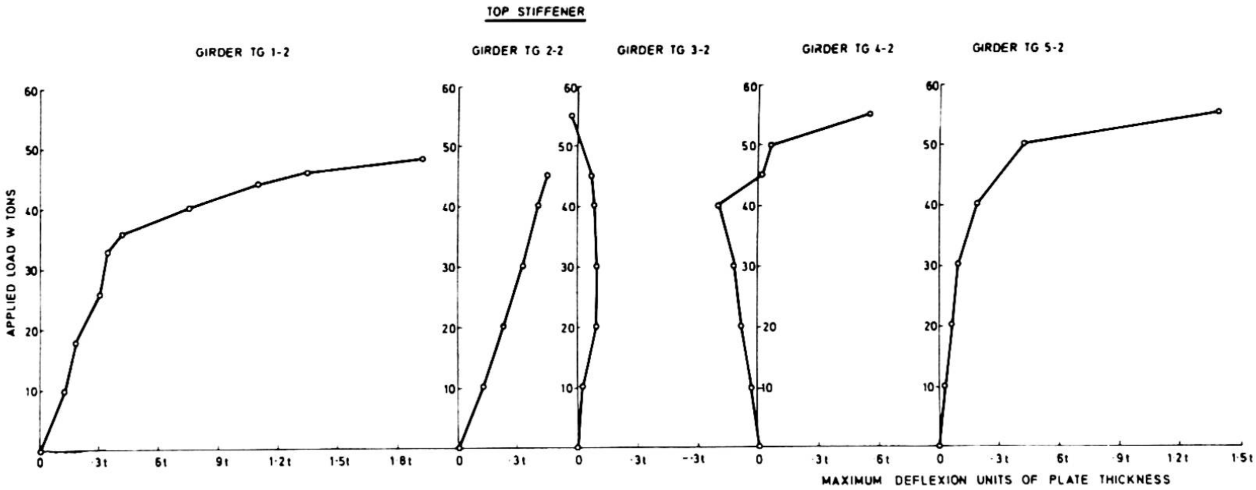


FIGURE 12
APPLIED LOAD/MAXIMUM DEFLECTION PLOTS FOR TOP STIFFENER AT 0.123d FROM COMPRESSION FLANGE. WEB REINFORCED BY TWO STIFFENERS

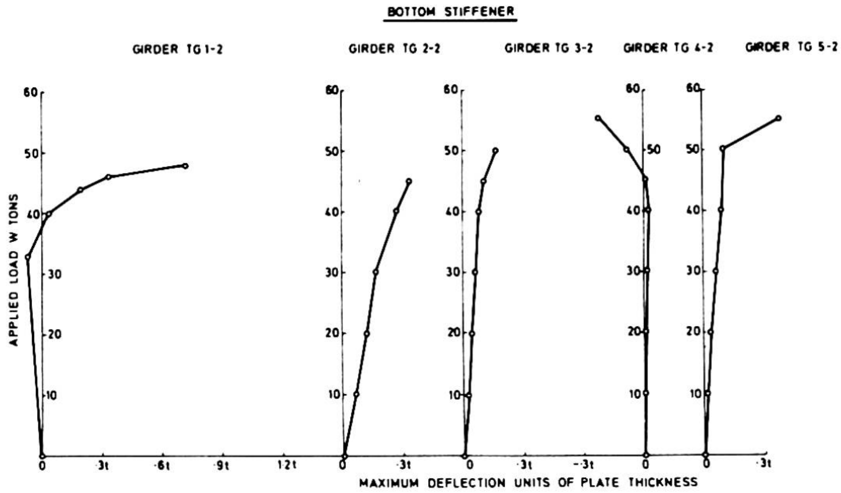


FIGURE 13
APPLIED LOAD/MAXIMUM DEFLECTION PLOTS FOR BOTTOM (SECOND) STIFFENER AT 0.4d FROM COMPRESSION FLANGE WEB REINFORCED BY TWO STIFFENERS

In Figure 15 the strain variation along the centre line of the girder TG5-2 is shown. The strains on both web plate faces are given for various load increments, together with the mean longitudinal strain values. This clearly demonstrates the increasing ineffectiveness of portions of the web plate in the compression zone in resisting stresses as the loading is increased.

The mid-plate longitudinal strains on the centre line section of the girder are shown for various load values in Figure 16. The shift in the position of the neutral axis towards the tension flange brought about by the ineffectiveness of parts of the web in the two upper panels, is well demonstrated.

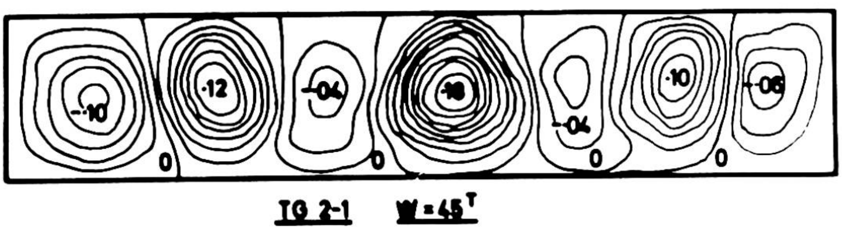


FIGURE 14
ADDITIONAL DEFLECTION PATTERN DEVELOPED IN PANEL ADJACENT TO COMPRESSION FLANGE. ONE LONGITUDINAL STIFFENER

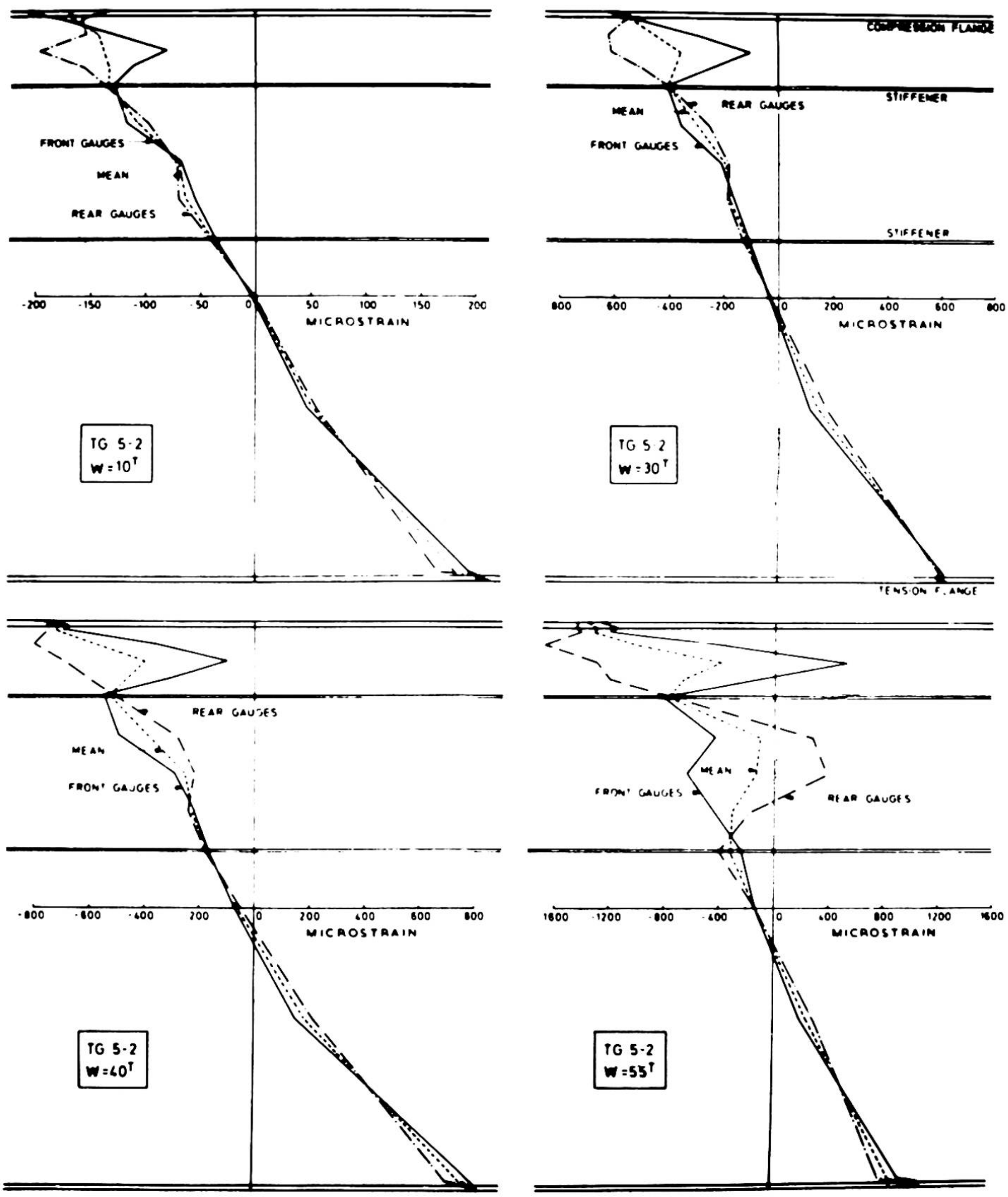


FIGURE 15
SURFACE AND MEAN STRAIN ACROSS DEPTH OF
GIRDER TG5-2 AT MIDSPAN

The effective section of a longitudinally stiffened web plate when loaded in the post buckled range is diagramatically illustrated in Figure 17. The loss of effective width results, as shown above, in the lowering of the neutral axis, a reduction in the effective sectional modulus and a corresponding increase in the compression strains. The loss of effective width is a function of the load ratio W/Wcr [18]. Thus the lower the buckling load, the greater the loss in effective width of those panels loaded in compression. It is for this reason that the girders reinforced by a single longitudinal stiffener have lower ultimate loads than those girders reinforced by a double line of stiffeners.

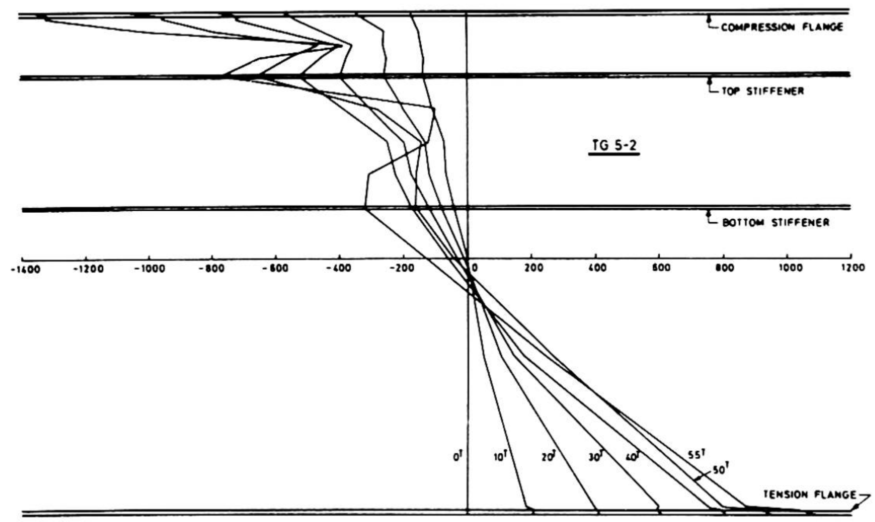


FIGURE 16
MEAN STRAINS ACROSS DEPTH OF GIRDER
TG5-2 SHOWING SHIFT OF NEUTRAL AXIS

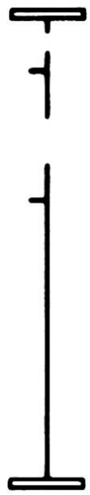


FIGURE 17
REDUCED EFFECTIVE SECTION
DUE TO BUCKLING

Thus it will be seen that by providing suitable longitudinal reinforcement in the compression zone it will be possible to design girders so that they develop ultimate loads very close to the plastic collapse load, given by simple plastic theory ignoring the effect of web buckling. A design procedure based on these studies will be presented in a forthcoming article [19].

VI. CONCLUSION

The present study has shown that (a) longitudinal stiffeners considerably influence the buckled pattern of the web, the stress distribution and the ultimate load of the girders, the beneficial influence of the longitudinal stiffeners being a function of their relative flexural rigidity, and (b) by reinforcing a web plate by suitably proportioned longitudinal stiffeners it is possible to achieve the full plastic collapse strength of a plate girder.

VII. ACKNOWLEDGEMENT

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VIII. SYMBOLS

t thickness of web plate

d clear depth of web plate

E Young's Modulus

μ Poisson's Ratio

$D = Et^3/12(1-\mu^2)$ flexural stiffeners of unit width of plate

EI flexural rigidity of a longitudinal stiffener

$\gamma = \frac{EI}{Dd}$

W applied load

W_{cr} theoretical load to cause buckling of an ideally stiffened plate without initial deformations

W_p plastic collapse load

all other symbols are defined as they appear in the text

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SUMMARY

The paper presents the results of ultimate load tests carried out on 10 plate girders having an overall depth of 51 in. The test section of the girders was subjected to pure bending stresses and the web was reinforced by either one or two longitudinal stiffeners. The influence of stiffener rigidity upon both the pre and post buckled behaviour of the stiffened web was examined and the results obtained have provided relationships between the collapse load of the girder and the flexural rigidity of the longitudinal stiffeners.

RÉSUMÉ

Le présent article fait état des résultats de tests de charge maximum portant sur dix poutres à âme pleine, de 127.5 cm. (51 in.) d'épaisseur hors tout. La section des poutres sur lesquelles ont porté les tests a été soumise à des tensions visant uniquement à les courber, l'âme de chaque poutre étant renforcée par une ou par deux pièces de renfort longitudinales. L'effet de la rigidité additionnelle due aux pièces de renfort sur le comportement de l'âme des poutres, avant et après la flexion, a fait l'object d'une étude, et les résultats obtenus ont permis de préciser les rapports entre la charge provoquant la rupture des poutres et le degré de flexibilité des supports longitudinaux.

ZUSAMMENFASSUNG

Dieser Beitrag zeigt die Ergebnisse der Traglastversuche an 10 Vollwandträgern mit 127.5 cm Höhe. Der Messquerschnitt war reiner Biegung unterworfen und der Steg entweder durch eine oder zwei Längssteifen verstärkt. Der Einfluss der Stegsteifen wurde untersuoht und die erhaltenen Ergebnisse haben eine Beziehung zwischen Traglast des Vollwandträgers und der Biegesteifigkeit der Längssteifen gezeigt.

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

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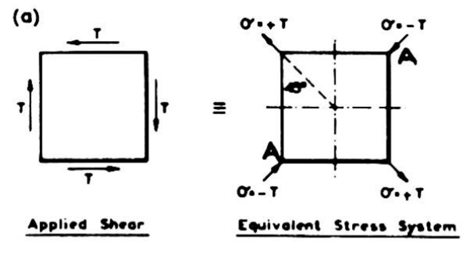
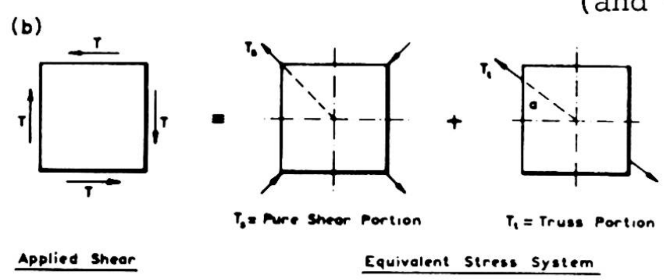
I. INTRODUCTION

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.

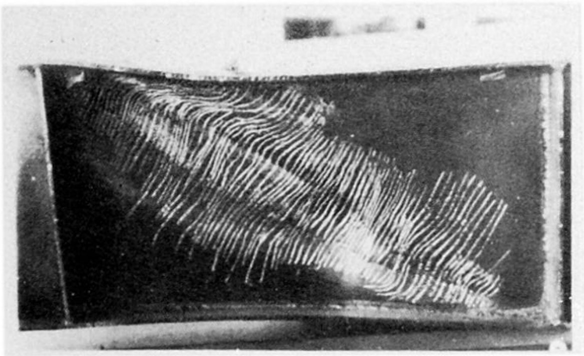


FIGURE 2 - STRESS DISTRIBUTION IN GIRDER TG4(R) OBTAINED USING A BRITTLE LACQUER

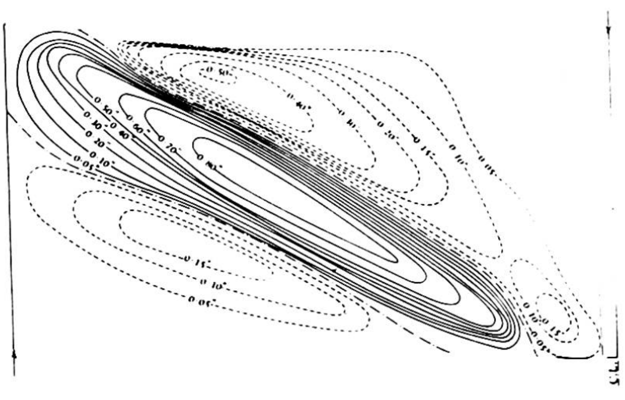


FIGURE 3 - RESIDUAL WEB DEFLECTIONS IN GIRDER TG4(R)

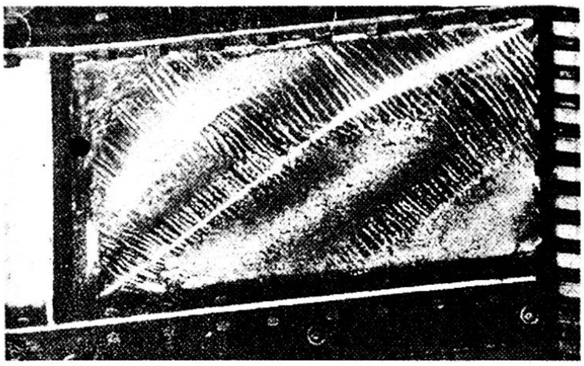


FIGURE 4 - STRESS DISTRIBUTION IN GIRDER TG3(R) OBTAINED USING A BRITTLE LACQUER.

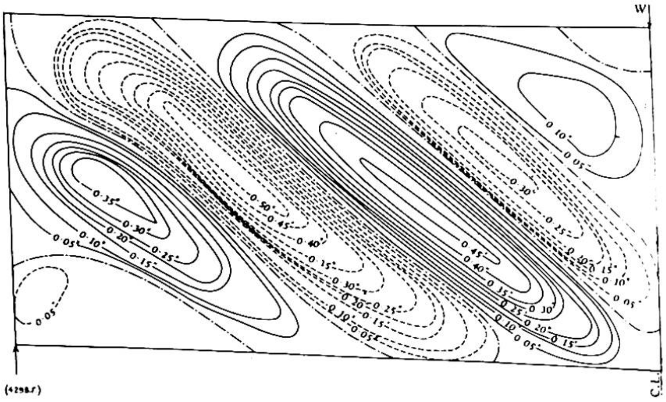


FIGURE 5 - RESIDUAL WEB DEFLECTIONS IN GIRDER TG3(R)

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.

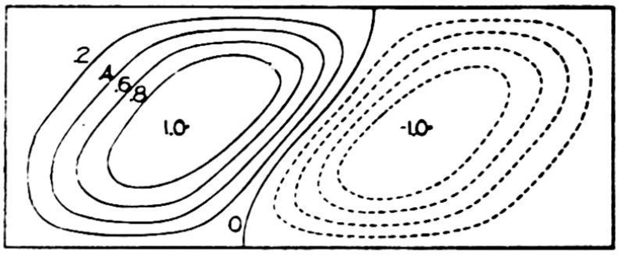


FIGURE 6 - THEORETICAL BUCKLE FORMATION IN SIMPLY SUPPORTED PLATE OF ASPECT RATIO 2.5

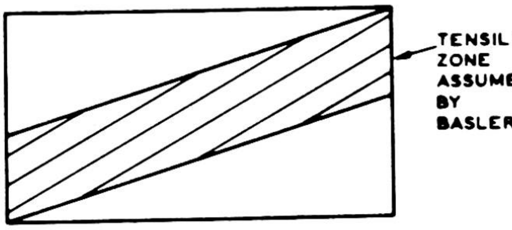


FIGURE 7

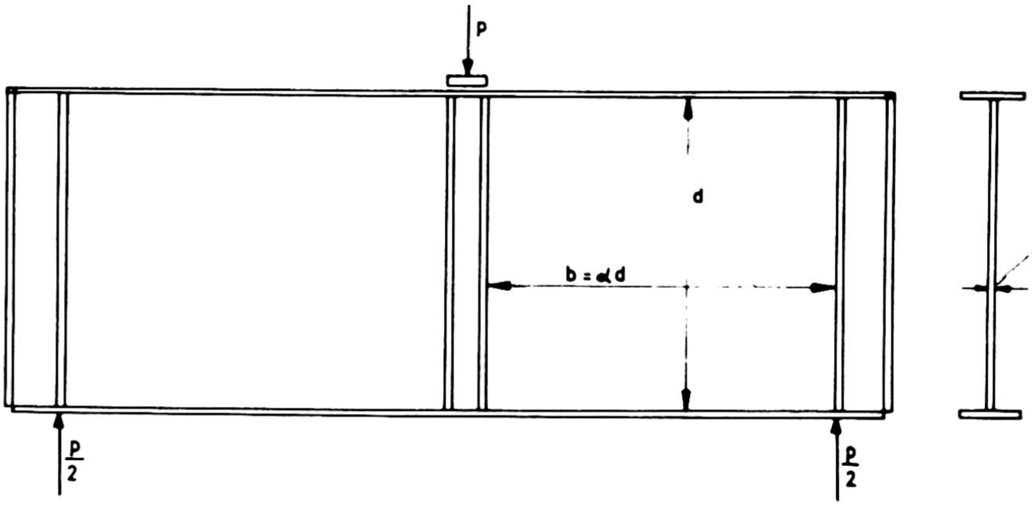


FIGURE 8 - DETAILS OF TEST GIRDERS

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.

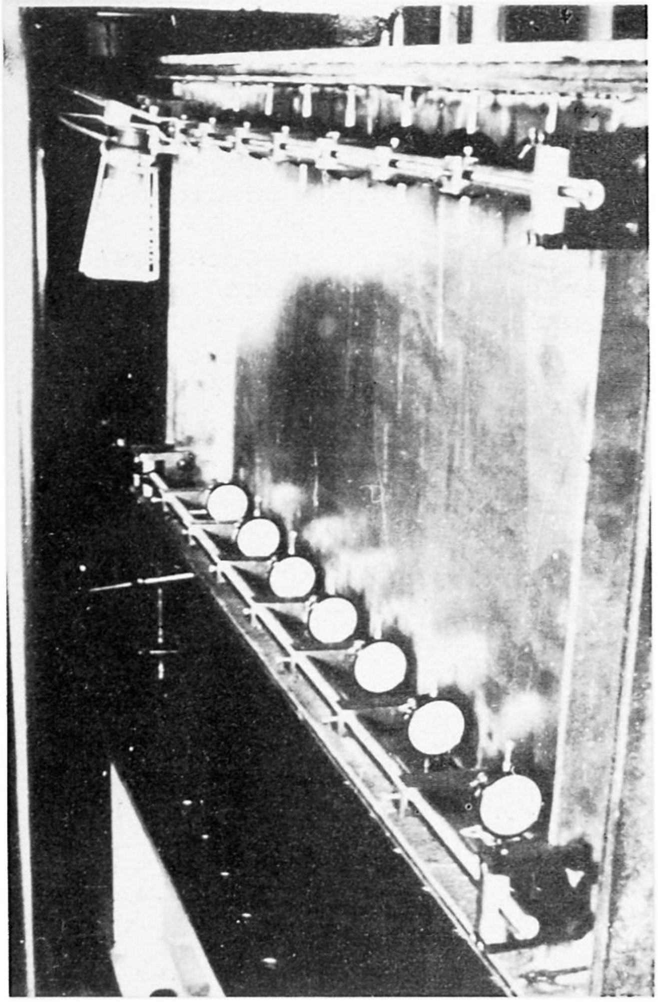


FIGURE 9

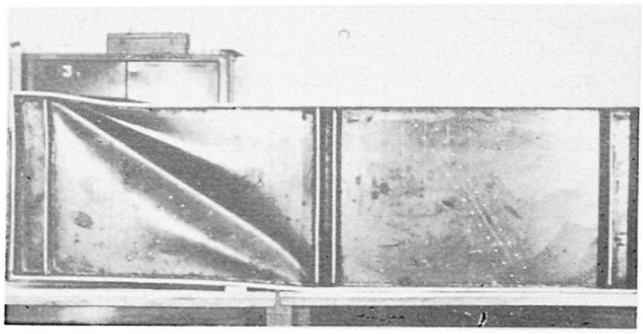


FIGURE 10 - GIRDER TG5 AFTER TEST TO FAILURE

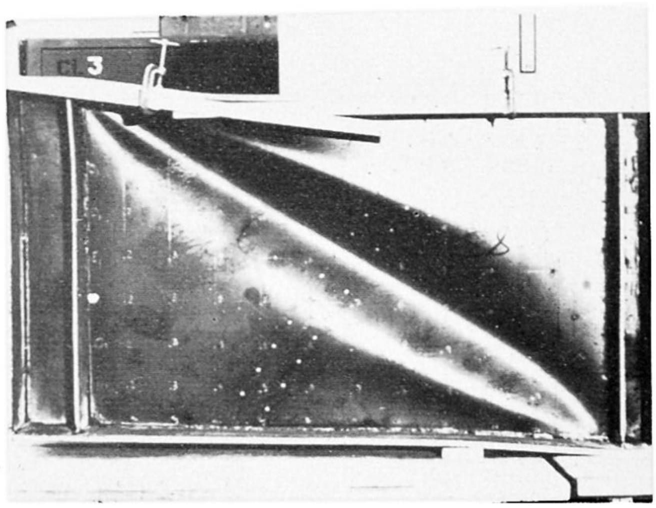


FIGURE 11 - GIRDER TG5

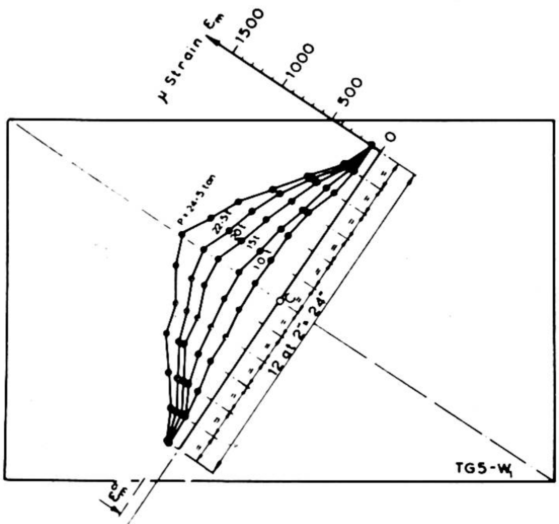


FIGURE 12 - STRAIN DISTRIBUTION IN PANEL W1 OF GIRDER TG5

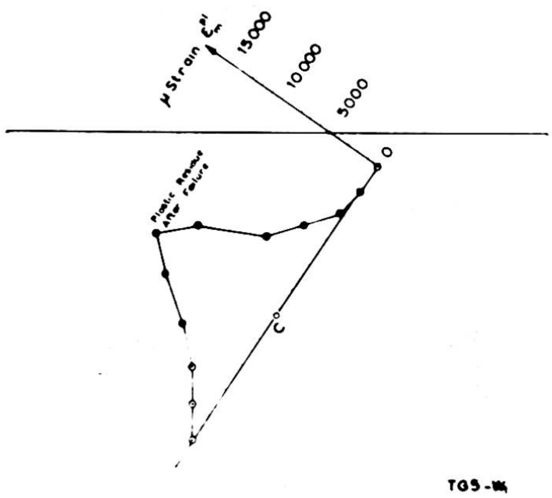


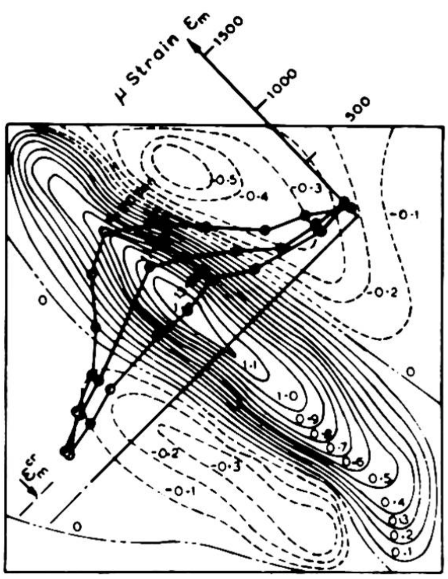
FIGURE 13 - RESIDUAL STRAINS IN PANEL W1 OF GIRDER TG5

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.



Test Girder 1–W_1.

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

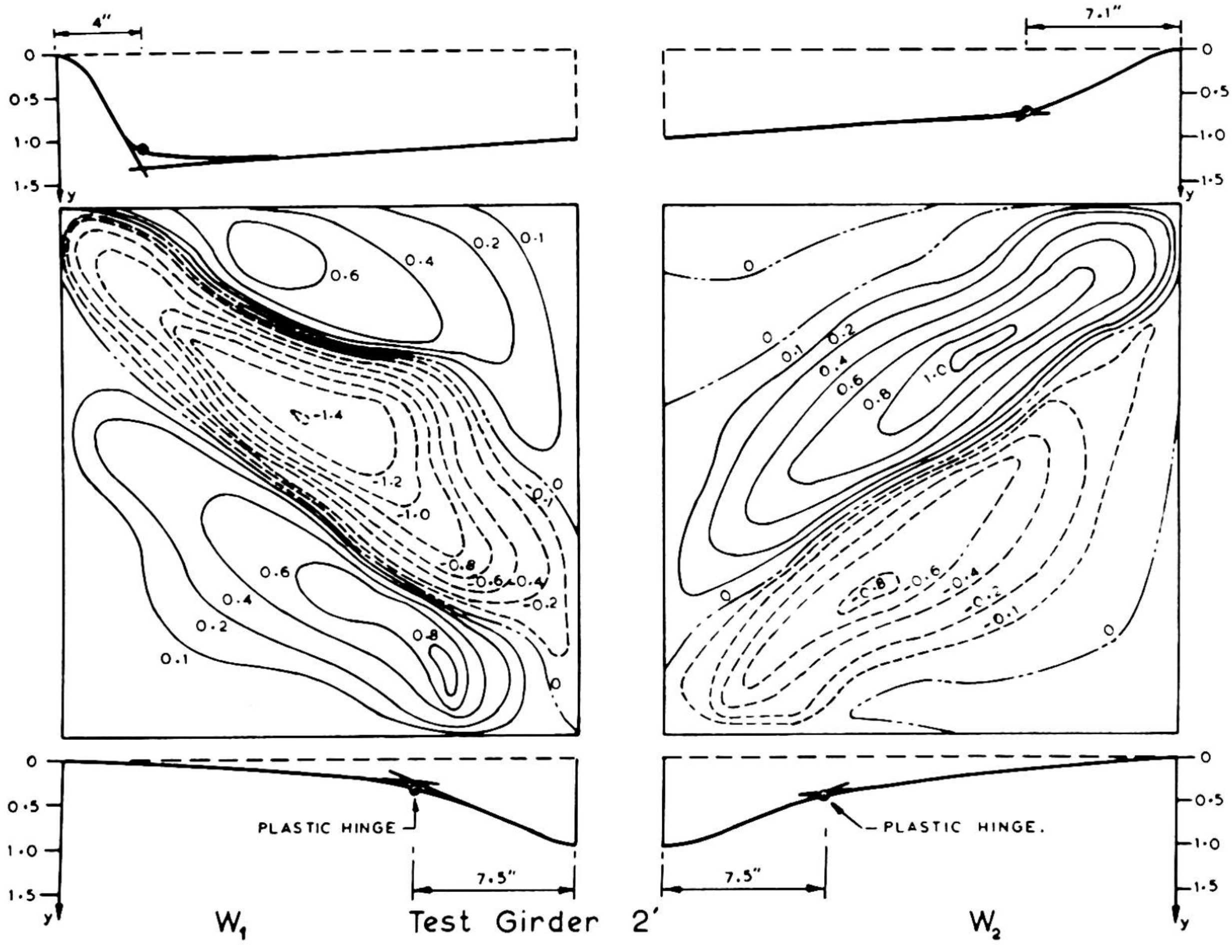


FIGURE 15 - RESIDUAL FLANGE AND WEB DEFLECTIONS (INS) FOR GIRDER TG2'

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

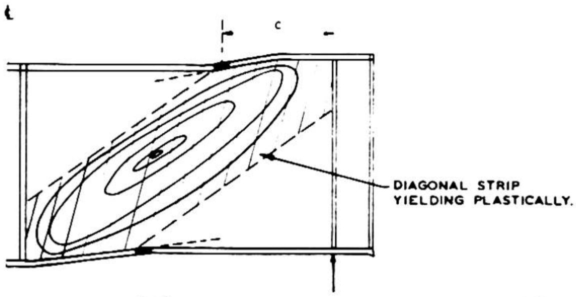


FIGURE 16 - DIAGRAMATIC ILLUSTRATION OF COLLAPSE MECHANISM

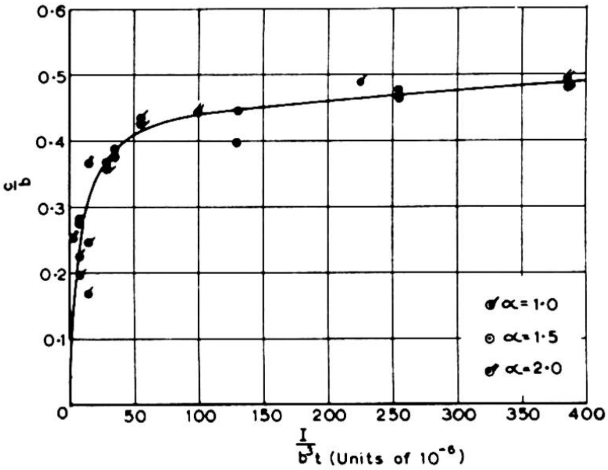


FIGURE 17 - VARIATION OF HINGE POSITION WITH I/b^3t PARAMETER

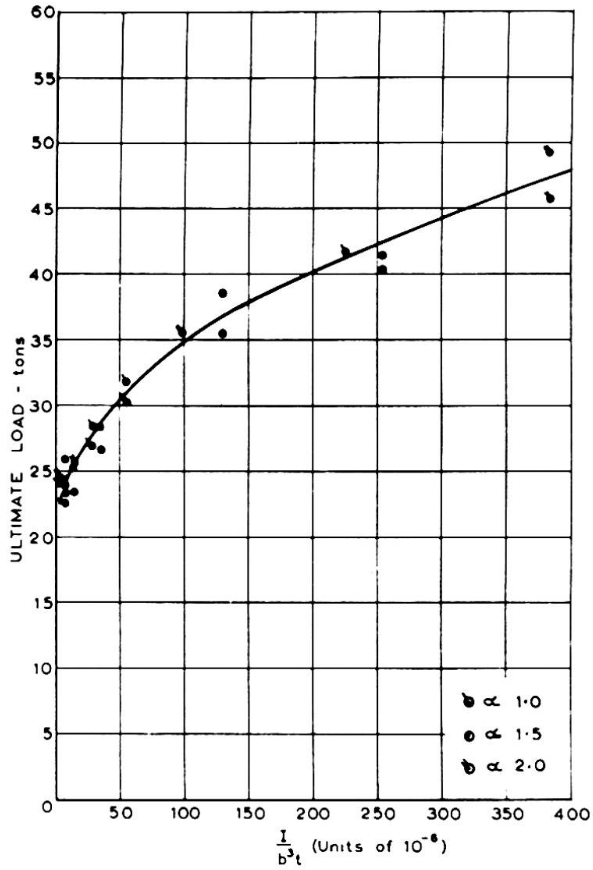


FIGURE 18 - VARIATION OF ULTIMATE LOAD WITH I/b^3t PARAMETER

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.

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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 = ($T_y dt$)

T_y Shear yield stress

T_{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.

Research on Plate Girders at Lehigh University

Recherche sur les poutres de grandes dimensions à âme mince à l'Université de Lehigh

Forschung an hohen Blechträgern an der Lehigh University

ALEXIS OSTAPENKO BEN T. YEN LYNN S. BEEDLE

1. INTRODUCTION

The purpose of this discussion is to report on the most recent work which is currently in progress at Lehigh University and on some of the results obtained. Particular areas which will be discussed and which are in addition to and will supplement the presentation by Mr. Massonnet[1] are:

1. Edge loading - girders with loads applied to the compression flange between transverse stiffeners (Item 11 of the research problems proposed by M. Massonnet).
2. Ultimate strength of unsymmetrical plate girders.
3. Fatigue strength of plate girders (Item 2).

2. EDGE LOADING

Quite often the question arises in design as to whether a bearing stiffener is needed when the loads are relatively small. This problem becomes important not only for service loads, but also for erection loads, when for example the girder must be rolled out as a cantilever. Figure 1 shows a typical girder with a web panel under stresses due to bending moment M, shear V, and the edge loading q. Theoretical analysis of the buckling strength of a plate girder web subjected to an edge loading has been performed recently by Wilkesmann,[2] Klöppel and Wagemann,[3] Warkenthin,[4] and Kawano and Yamakoshi.[5] In all these solutions the

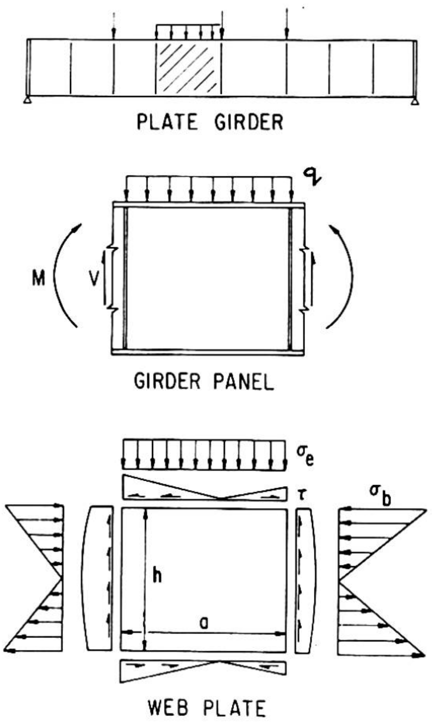

Figure 1

edges of the plate were assumed to be simply supported. Basler proposed a simplified buckling analysis considering the top edge of the web to be simply supported or fixed.[6] However, neither the effect of bending nor of shear was included. Basler's approach was later incorporated in the Specification of the American Institute of Steel Construction.[7]

Normally the ultimate strength of plates is substantially greater than the buckling strength; but, in general, there is no direct relationship between these two quantities. Since all previous work on edge loaded web has been concerned only with the elastic buckling strength, a series of tests was conducted at Lehigh University to investigate the ultimate strength and to arrive at a method of evaluating it.[8]

The theoretical buckling stress was computed by means of the finite difference method. The web plate was assumed to be fixed at the flanges and simply supported at the stiffeners. This assumption is based on the observations of the web behavior during tests on plate girders of ordinary proportions. Figure 2 gives the buckling coefficient K as a function of the aspect ratio, $\alpha = a/h$, and the bending stress ratio σ_b/σ_e. The bending stress is seen to have a pronounced effect on the value of the buckling coefficient and thus on the critical edge stress.

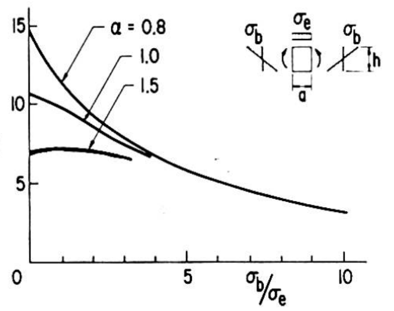

Figure 2

Ten tests were conducted on three plate girder specimens. The principal parameters were:

1. Aspect ratio a/h which varied from 0.8 to 1.6.

2. The ratio of the bending stress to the edge stress, σ_b/σ_e,

which varied from 0 to 5.0.

The edge loading consisted of two to four concentrated loads spread over the compression flange through 1.5 inch thick plates to simulate uniform distribution.

The ultimate strength from the tests is plotted non-dimensionally in Fig. 3. It is remarkable that the plotted points lie in a relatively narrow band seemingly independent of the bending stress ratio σ_b/σ_e. A horizontal line passing through the ordinate equal to 3.0 is seen to give a conservative estimate of the ultimate strength. It follows then that the ultimate strength can be approximated fairly well from the buckling strength. The resulting equation for the allowable intensity of the edge stress σ_a, with a factor of safety against the ultimate strength equal to 2.0, is shown in Fig. 3 by the light horizontal line.

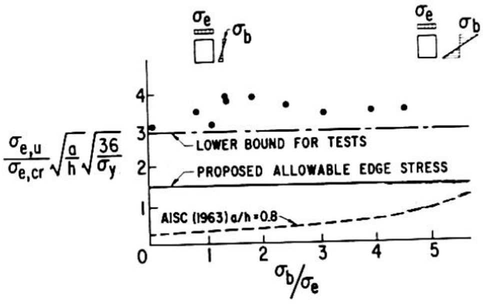

Figure 3

$$\sigma_a = \frac{\sigma_{e,u}}{\text{F.S.} = 2.0} = \frac{4000}{(d/t)^2\sqrt{a/h}} \sqrt{\frac{36}{\sigma_y}}\ K \quad \text{(ksi)}$$

where

h = girder depth
t = web plate thickness
a = distance between transverse stiffeners
σ_y = yield stress of the web plate in ksi
K = buckling coefficient

Pending results of an analytical study and additional tests, this formula is subjected to the following limitations: 34 ksi $< \sigma_y <$ 45 ksi; $240 < h/t < 300$; $\sigma_b/\sigma_e < 5$; $0.8 < a/h < 2.0$.

For comparison, the dashed curve in Fig. 3 gives the allowable edge stress according to the American Institute of Steel Construction Specification (a/h = 0.8, loaded edge fixed). Although quite conservative, this curve leads to a reduction of the margin of safety with an increasing bending stress.

3. UNSYMMETRICAL GIRDERS

Most prior research on the ultimate strength of plate girders has been concerned with symmetrical members, that is, girders with the centroidal axis at the mid-depth, such as the cross section in Fig. 4(a).[9,10,11,12] However, many plate girders are unsymmetrical in the sense that they have unequal flanges and their centroidal axis is close to the larger flange, for example, composite and orthotropic deck girders shown in Figs. 4(b) and 4(c). Designing such girders as symmetrical members would lead to results that are either overconservative or unconservative depending on whether the larger portion of the web is in tension or in compression. Currently, a study is being conducted at Lehigh University on the ultimate strength of unsymmetrical plate girders subjected to bending, shear, and a combination of bending and shear.

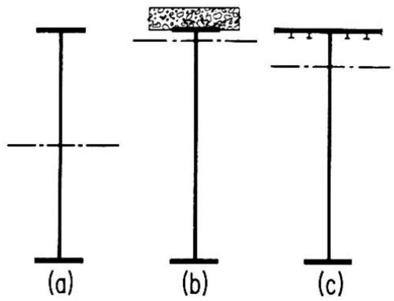

Figure 4

The experimental phase of the study consisted of fourteen tests conducted on four plate girders with transverse stiffeners.[13,14] One flange had approximately double the area of the other flange.

In the new method of analysis developed as part of the theoretical work, the ultimate strength of a plate girder panel is assumed to be controlled principally by the following three contributions shown schematically in Fig. 5:

a) <u>Beam strength</u> up to the point at which web buckling would theoretically occur,

b) <u>Post-buckling</u> strength of the web, and,

c) <u>Frame action</u> which is based on the formation of a panel mechanism with plastic hinges in the flanges.

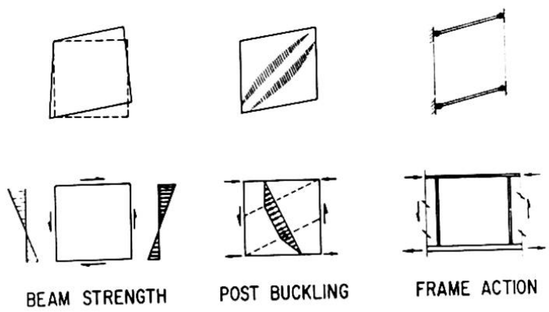

Figure 5

An interaction relationship between moment and shear for a typical plate girder panel is shown in Fig. 6. The non-dimensionalizing values are the ultimate capacity in shear V_u and the ultimate bending capacity M_u, a moment which acts to produce compression in the larger portion of the web. To the right is the interaction plot for a loading condition when a larger portion of the web is in compression and to the left is the interaction plot for a loading condition when a larger portion of the web is in tension. The ultimate strength is seen to be different for these two cases.

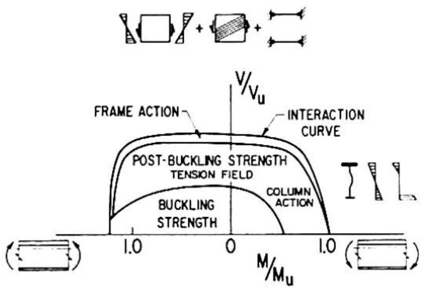

Figure 6

The buckling strength (called beam strength in Fig. 5) is obtained for a combination of shearing and bending stresses assuming the web plate to be fixed at the flanges and simply supported at the stiffeners. It represents the upper limit of ordinary beam action.

The post-buckling strength of the web is developed differently in shear and in bending. In shear, it means a tension field assumed to be of the pattern shown in Fig. 5. In bending it means redistribution of the additional web stresses toward the compression flange.

The frame action contributes to the shear capacity by developing a panel mechanism with plastic hinges in the flanges. A portion of the web equal to 20 times the thickness is considered to be part of the flange and the axial flange force is included in the evaluation of the plastic moments.

The ultimate strength in shear is given as a sum of all three contributions as shown in the top sketch of Fig. 6.

The ultimate strength on pure bending is limited by the buckling capacity of the compression flange or the yielding of the tension flange with a portion of the web. Shear reduces the bending strength by contributing to the flange forces through the development of a full or partial tension field and frame mechanism.

The effect of the location of the centroidal axis on the mode of failure of a girder panel under pure bending is depicted in Fig. 7. Fixed values for this plot are the yield stress, the bracing spacing for the compression flange, and the ratio of the compression flange area to the web area. Compression flange failure is to be expected in the region

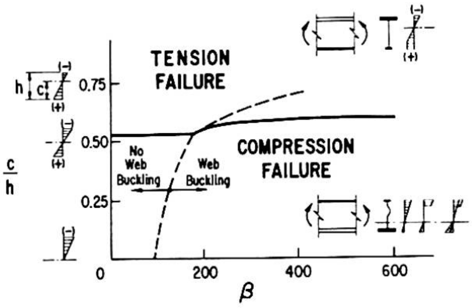

Figure 7

below the heavy solid line whereas tension flange yielding would be expected above that line. The dashed line separates the regions in which the web will buckle or will not buckle before the ultimate panel strength is reached.

The method is compared with the available test results in Fig. 8. Symmetrical and unsymmetrical girders are grouped separately for each of the loading cases: shear, combined bending and shear, and bending. The average agreement is within 5 percent with the maximum deviation being approximately 12 percent.

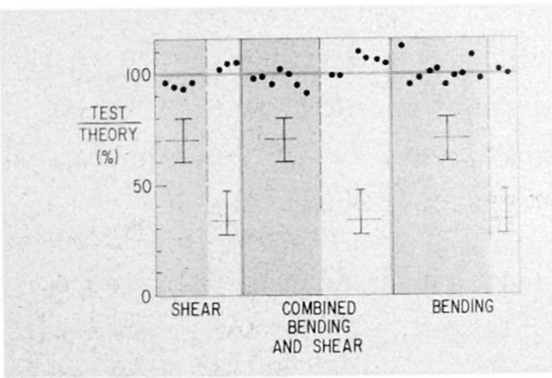

Figure 8

The method has also been extended to girders with one longitudinal stiffener and the correlation with test results is within the same range.

4. FATIGUE

A special problem arises in "thin-walled" plate girders, because of web deflection. Figure 9 shows exaggerated deformation in a panel as accentuated by painted black lines. Under repeated loading, a web plate moves back and forth, and fatigue cracks occur at the boundaries.[15,16,17]

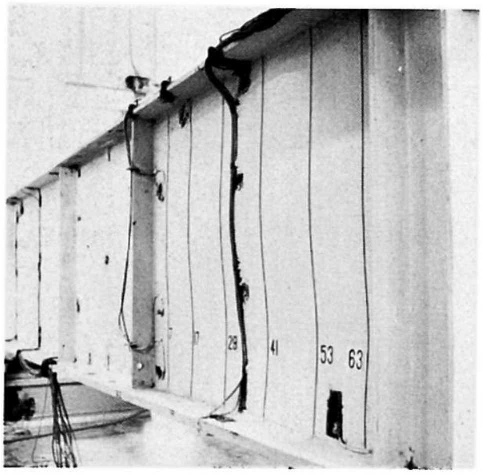

Figure 9

The most probable locations of the fatigue cracks are compared with the contours of the web deflection in Fig. 10. Crowding of controus indicates high curvature, hence high plate bending stress and thus the possibility of a fatigue crack.

A preliminary analysis has been made, and Fig. 11 gives an example of the results.[18] The web panel of a test girder

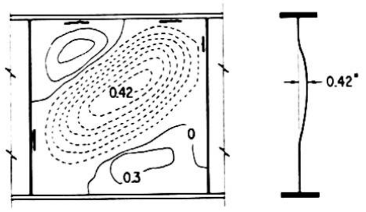

Figure 10

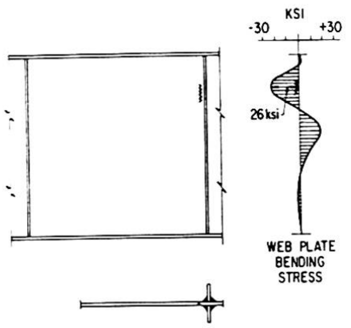

Figure 11

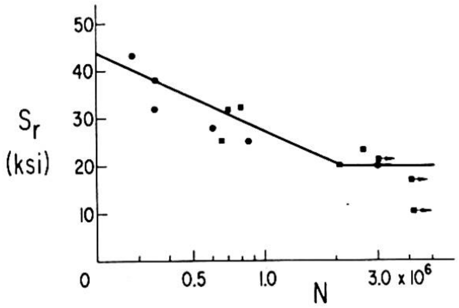

Figure 12

is shown with a fatigue crack at the toe of the web fillet weld. The web plate bending stresses have been estimated from web deflections measured under load, and Fig. 11 shows that the crack occurred at the point of highest stress.

The results of all fatigue tests for which deflection data are available are shown in Fig. 12. The relationship is a typical S-N curve between the range of variation of web plate bending stress and the number of load applications that were needed to develop a crack.

Generally speaking, large web deflections generate large stresses and thus lead to early cracks. If the web deflections are small, the corresponding stresses are small; and there is a run-out.

Based on these results, limits of web plate bending stresses and web deflections are being established. Two "proof" girders, designed according to preliminary rules, have endured at least 2,000,000 cycles without cracks.

5. CONCLUDING REMARKS

It is the eventual objective of the Lehigh research to develop design recommendations for symmetrical and unsymmetrical girders, both transversely and longitudinally stiffened, taking advantage of the most recent findings both in the United States and from abroad.

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SUMMARY

Current topics of research on plate girders and some of the findings are discussed. A formula is proposed for the ultimate capacity of a girder panel subjected to edge loading and bending moment; it is based on an elastic buckling analysis and ultimate strength tests. Ultimate strength of unsymmetrical plate girders is given by the sum of the beam action, post-buckling strength, and frame action; the theory is in good agreement with test results. Occurrence of fatigue cracks in plate girder web is dependent on the lateral web deflections and can be controlled by limiting the web slenderness ratio.

RÉSUMÉ

Cet article présente les recherches en cours ainsi que quelques résultats obtenus sur les poutres de grandes dimensions à âme mince. Une formule est proposée pour la charge de ruine d'un panneau d'âme de poutre situé entre deux raidisseurs assujetti à une charge répartie agissant sur la semelle supérieure. Cette formule est basée sur l'analyse élastique du voilement, et sur la charge de ruine obtenue lors des essais. La charge de ruine des grandes poutres à âmes minces est donnée comme superposition d'une action de flexion, de l'effet du voilement postcritique, et d'un mécanisme de panneau; la théorie concorde bien avec les résultats d'essais. L'apparition de fissures dans l'âme des poutres dues à la fatigue dépend de la flexion latérale de l'âme et peut être anticipée en limitant l'élancement de l'âme.

ZUSAMMENFASSUNG

Die Themen und einige Ergebnisse der gegenwärtigen Forschung auf dem Gebiet der Blechträger werden erörtert. Eine Formel für die Traglast von Stegblechfeldern unter Längsrandbelastung und Biegung ist vorgeschlagen. Diese Formel basiert auf der elastischen Beultheorie sowie auf Ergebnissen von Traglastversuchen. Die Traglast von unsymmetrischen Blechträgern wird aus der Balkenwirkung, der überkritischen Tragfähigkeit und der Rahmenwirkung zusammengesetzt angenommen; diese Theorie stimmt mit Versuchsergebnissen gut überein. Das Auftreten von Ermüdungsrissen im Steg ist von der Stegdurchbiegung abhängig und kann durch eine Begrenzung der Stegschlankheit verhindert werden.

Experimental Investigation of Strength of Plate Girders in Shear

Recherches expérimentales sur l'effort admissible de poutres à âme pleine soumises au cisaillement

Experimentelle Untersuchung über die Festigkeit in Blechträgern unter Schub

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1. INTRODUCTION

With progress of welding technique and repletion of facilities for fabrication of steel structural members, there is a tendency that plate girder bridges of a relatively short span are shop-produced in large quantities. From economic point of view in fabrication with employment of automatic welding in mind, a design with no intermediate stiffner or a design with only horizontal stiffners is prefered to that equipped with intermediate transverse stiffners. In designing a plate girder, intermediate transverse stiffners are usually placed sufficiently close to prevent shear failure of the web panels and as a consequent the failure of the girders are mostly due to bending moment. The importance of carrying capacity of web panels to shear force is more pronounced in a plate girder with no intermediate transverse stiffners except at the points where heavy loads are concentrated.

The report present the results of shear tests on a series of full size plate girders with transverse stiffneres only at the supports and at the center of the test specimens where loading was applied. The main purpose of the tests is to determine safety coefficients with respect to buckling for girders devoid of intermediate transverse stiffners.

In most of the current specifications, the design criteria of a web panel are buckling of plates simply supported at the four edges. The existence of post-buckling strength is utilized with the adoption of lower safety coefficients against web buckling, however the safety coefficients in current specifications are in wide variety. Japanese specifications for highway bridges limit the depth-thickness ratio of web panels to 60 regardless of working stress for this type of plate girders made of structural carbon steel with minimum yield stress of 24 kg/mm^2 and to a smaller value for girders of higher strength steels. A similar specification is adopted in British standards (BS153) with a limiting value of 85 for a steel of the same group. The American specifications (AASHO) use pure shear buckling as a design criterion with a possible safety factor of around 2.6 for the buckling. German specifications (DIN 4114) are more complete in the sense that they are based on bucking. Safety factor of 1.35 is specified regardless of the state of stress.

2. TEST PROGRAM

A series of nine full-size girders of a higher strength steel were tested in this experimental study. The depth to thickness ratios of the web plates ranged from 60 to 120 by an interval of 20. The thickness of the web plates was 9 mm for all of the test girders and thus the depth of the web panels varied from 540 mm to 1080 mm depending on the depth-thickness ratios. Two girders were prepared for each depth-thickness ratio with variation in flange plates so that the effect of difference in flange area to carrying capacity of the girders could be detected experimentally. Horizontal stiffners were placed at mid-depth on one of the two web panels of the girders with depth-thickness ratios of 100 and 120 so that failure would take place on the other web panel. In addition, a girder with horizontal stiffners placed on both web panels at mid-depth was tested for the overall depth-thickness ratio of 120. The stiffness of the stiffners was less than one-tenth of the so-called strictry rigid stiffner.

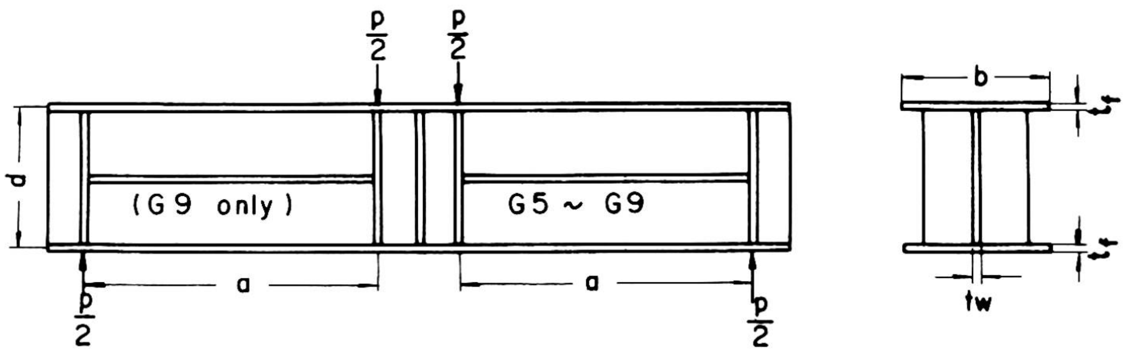
The aspect ratios, width to depth ratios of the web panels, were 2.6 for all test girders. The aspect ratio was determined from two considerations; a large ratio is prefered in order to determine the lowest possible shearing strength of a web panel, while it is prefered to have a shorter so-called shear span in order to avoid the bending failure of the test girders with practical cross sectional dimensions. The buckling strength of a simply supported rectangular plate of the width-depth ratio of 2.6 is only 10% more than that for an infinitively long plate under pure shear.

Table 1 summarizes the dimensions of all the test girders. The dimensions are the averages of true measurements on the specimens. The girders were tested with simply supported condition at the ends and under an essentially concentrated load at mid-span as shown in the figure in Table 1, producing constant shear force and linearly varying moments. Two types of flanges, smaller flanges and heavier flanges, were selected in designing the girders for each depth-thickness ratio so that with this test set-up both flange failure due to bending moment and web failure primarily due to shear force would occure.

All girders were welded, where no particular attention was paid to minimize distortion, instead it was intended to test girders of practical use. The flange to web connection was made by 7 mm fillet welds, while the stiffners were connected by 6 mm fillet welds. The sub-merged arc welding was used for the welds between flanges and webs, while manual welding was used elsewhere.

The steel used for the test girders is identified by the trade name 'KH-36', a Nb-Cr type high strength steel with minimum yield strength of 36 kg/mm^2. Both the 22 mm thick plates used for flanges and the 9 mm thick plates used for webs came from one heat. The chemical composition of the heat is listed in Table 2. Among the mechanical properties, the yield stress is the most important. The standard tensile coupons of 200 mm gage length were tested to determine yield stress σ_Y, ultimate tensile strength σ_u and percentage elongation. Attention was paid during the test so as to be able to obtain the so-called static yield stress. The results of the coupon tests are given in Table 3.

Table 1 Detail of Test Girders



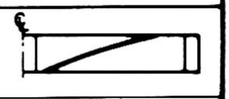
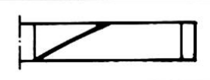
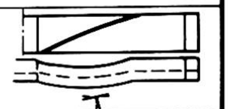
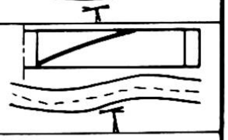
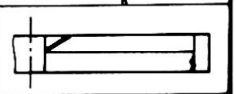
Girder	tw	d/tw	a/d	Flange mm x mm	Web mm	a	Vertical Stiffs.	Horizontal Stiffs.	Failure Mode
G1	9.1	59.6	2.69	301 x 22.4	543	1450	90.0x9.20		
G2	9.1	59.6	2.69	220x 22.4	543	1450	90.0x9.20		
G3	9.4	76.8	2.64	302x 22.2	722	1900	90.0x9.07		
G4	9.2	78.3	2.64	243x 22.1	720	1900	90.0x9.13		
G5	9.0	99.9	2.62	291x 22.3	899	2360	90.0x8.95	75.0x9.08	
G6	8.9	101.2	2.62	212x 22.3	900	2360	90.0x9.18	75.0x9.02	
G7	9.1	118.9	2.64	282x 22.4	1080	2850	90.0x9.07	75.0x9.05	
G8	8.9	121.3	2.64	221 x 22.2	1080	2850	90.0x8.94	75.0x8.98	
G9	9.1	118.7	**2.64**	282 x 22.4 (C.300x22.0)	1080	2850	90.0x9.00	50.0x9.01	

Table 2 Chemical Composition of Girder Material (%)

C	Si	Mn	P	S	Cr	Nb
0.15	0.04	1.31	0.16	0.20	0.20	0.20

Table 3 Coupon Test Results

Component	Thickness (mm)	σ_Y (kg/mm^2)	σ_u (kg/mm^2)	Elong. (%) GL. = 200 mm
Flange	22	44	57	28
Web	9	38	57	20

3. TEST SET-UP AND PROCEDURE

Loading to the girders was applied with a 2000 ton hydraulic universal testing machine. Rollers were used both at the end supports and at the loading point so as to minimize the effect of friction for carrying capacity of the girders. To prevent a lateral failure of compression flanges, bracing was provided by frames at both ends and at two intermediate points; with this set-up no premature lateral failure was observed during the tests. Roller bearings were inserted between all the possible points where free vertical movements of the girders might be restricted.

Instrumentation consisted of dial gages and electrical wire strain gages. Dial gages were used to measure deflection at mid-span relative to end supports and to measure lateral movements of web panels at a number of points where large deflections were expected due to web buckling. The dial gages for measurements of web deflection were fixed on steel rods, of which one end was welded to the test girders at the connection of the tension flange and the web plate. Since the rotation of tension flange will affect the readings with this dial gage set-up, the absolute values of the readings may have no definite meaning, however the readings would indicate sensitivity of the web deflection to the loading and thus the characteristic of web buckling.

Surface strains at a number of points on flanges and web plates were measured using electric wire strain gages. Two rosettes were attached on one point on web plates, one at each side of the web so as to be able to separate the strains due to bending and due to membrane. Figure 1 shows the over-all test set-up of a girder.

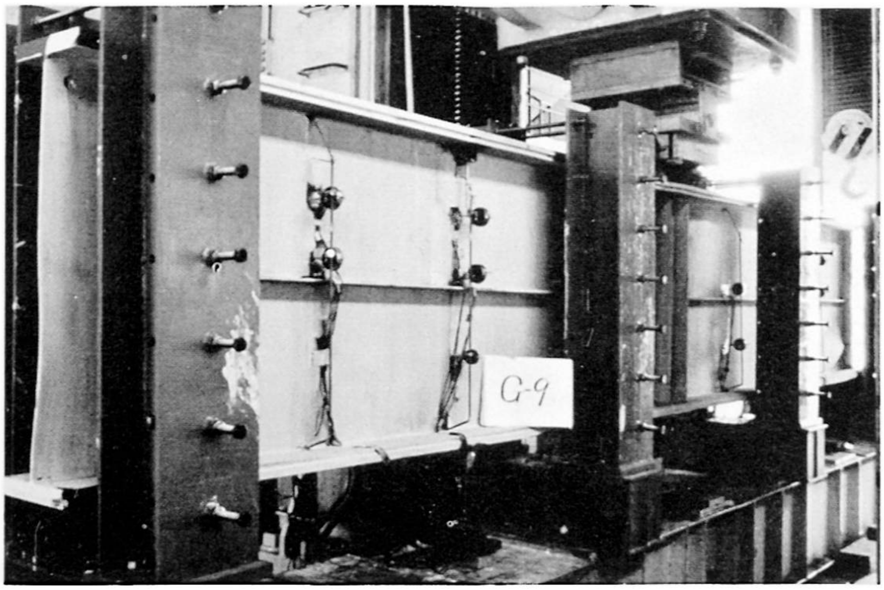


Fig. 1 Test Set-up

Two cycles of loading procedures were used for most of the test specimens. For the first cycle, loading was applied gradually from the initial load of 20 tons with suitable increments until deviation from the straight line was observed on the load responese relationship of the deflection at mid-span, then the load was reduced to the initial load. All of the necessary measurements were made at each loading level when the load stabilized so as to avoid the dynamic effect. Starting from the initial load again, a similar loading procedure was followed as the first cycle, until, this time, the ultimate load was reached and unloading was observed with increase of mid-span deflection. The loading increments were controlled by both increments of mid-span deflection determined from plots of the gauge readings and increments of particular gauge readings. Once the load-deflection relationships indicated deviation from the straight line, the increments of loading were kept comparably small, such that the critical and the ultimate loads would be noted on the load-deflection curve.

4. TEST RESULTS AND DISCUSSIONS

The load-deflection relationships of the test girders are shown in Fig. 2, in which over-all behavior of the girders are delineated. Also shown in the figure are the shear buckling loads, P_{cr}, for the web panels calculated under the assumptions that the panel is simply supported at all four edges and is subjected to pure shear. The girders with larger depth-thickness ratios remained practically elastic to a load exceeding the buckling load. The linear load-deflection relationships start to deviate with increase of loads, and then the gradient of the curves slowly decreases reaching the ultimate loads. Solid lines in Fig. 2 show the load deflection relationships up to the ultimate loads, while the

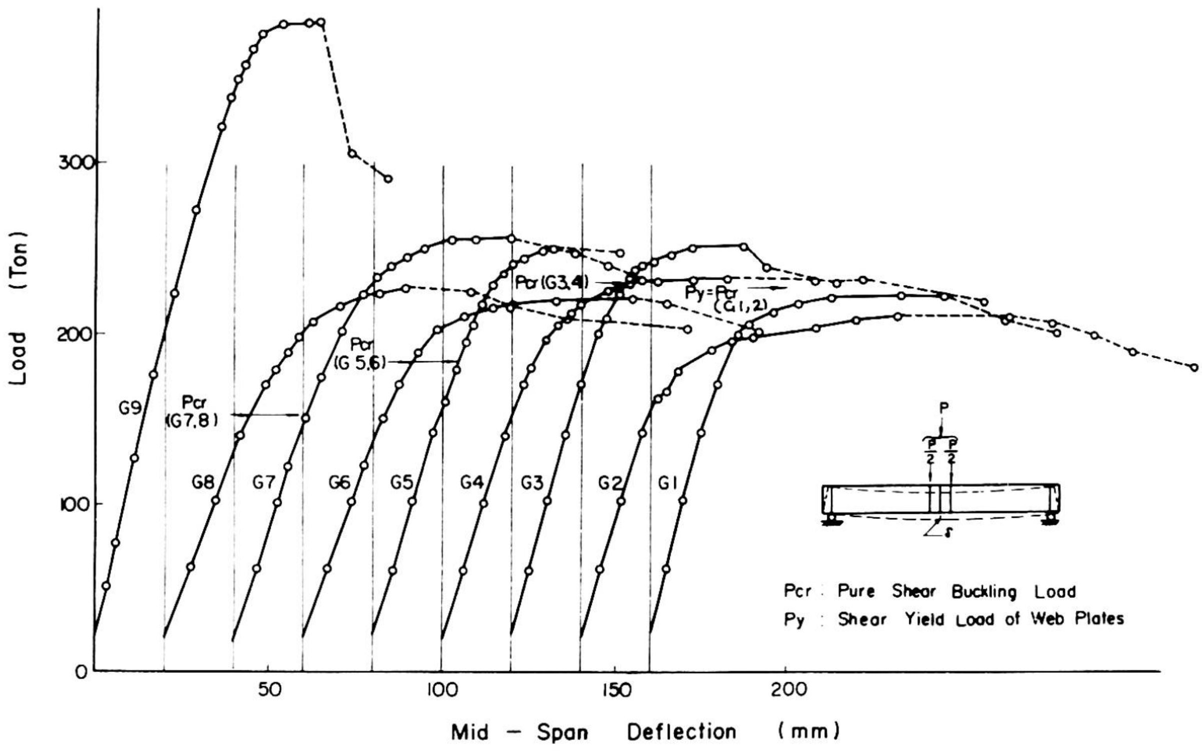


Fig. 2 Load-Deflection Relationship

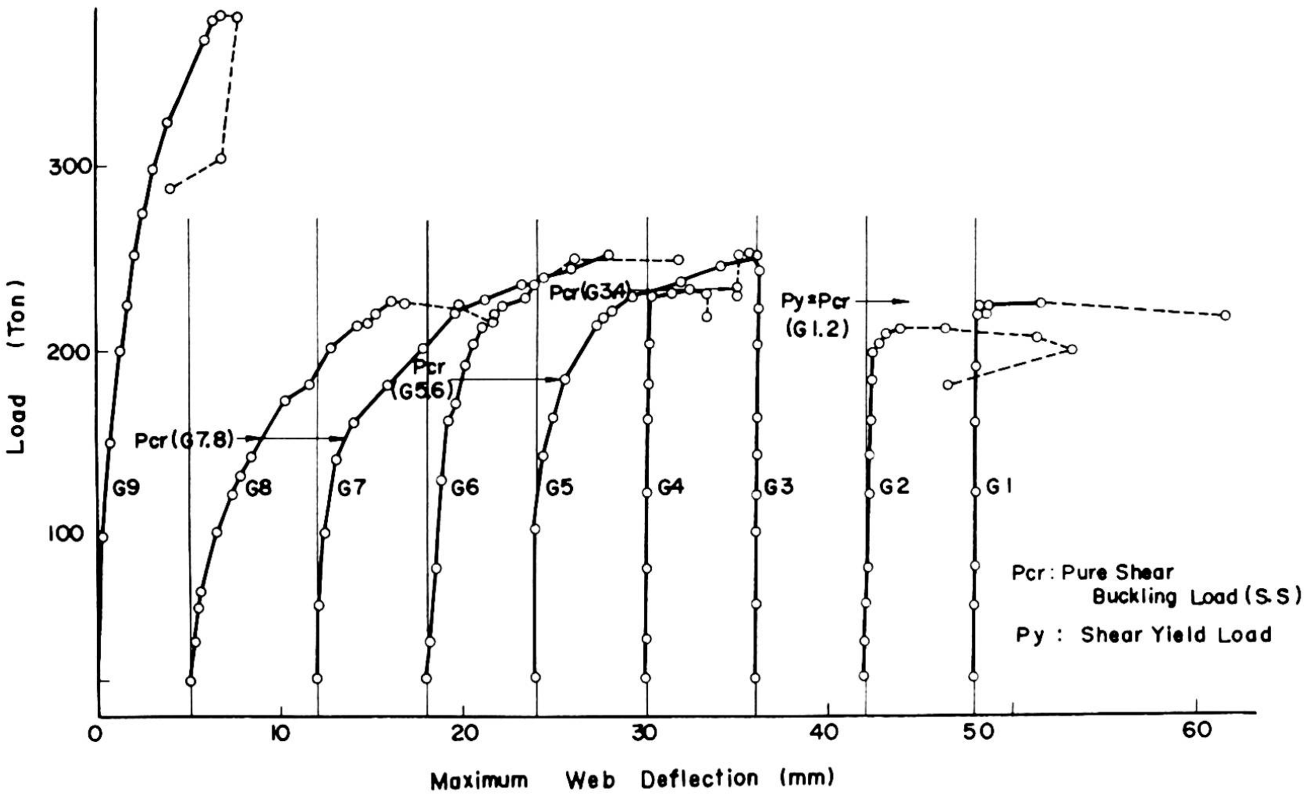


Fig. 3 Load-Web Deflection Relationship

broken lines were drawn for the relationships after the ultimate loads were passed. It is of interest that the deflections at which the ultimate loads were reached are of the order of 40 mm regardless of the span length; expressing differently, a larger deflection at the ultimate load was recorded for unit span length in girders with smaller depth-thickness ratios. The same is true for deflection capacity after the ultimate load was passed. For the same depth-thickness ratio, the girder with heavier flanges showed less deflection capacity.

After the ultimate loads were passed a sharp decrease of loading was observed for girders G1, G3, G5, G7 and G9, girders with heavier flanges among each pair. The difference seems to be due to the cause of the failure of each test girder. Girders G1, G2 and G5 failed due to excessive shear deformation of the web panels. Three types of deformations, shear buckling of the web, lateral displacement and twisting of the compression flange, were observed when girders G7 and G8 were subjected to ultimate loads. G9 failed due to the buckling of the bearing stiffner on one end support. Although the failure of G2 was visually identified as shear failure, it was also true that the average normal stress in flanges was exceeding the yield stress of the material. In the failure of the rest of the girders G4, G6 and G8, no marked shear buckling was observed; the failures were due to instability of the compression flanges. Sketches of the failure modes are shown in Table 1.

Lateral deflection of web plates was measured at a number of points for each panel, among which the load-deflection relationship of the point where the maximum reading was recorded under the ultimate load is shown for each girder in Fig. 3. As can be seen in the figure, no marked web deflection was recorded for the test of G1 up to the load of 215 tons which was more than 95 percent of the ultimate load of 221 tons, and then the deflection was becoming large all of a sudden. The sharp knee of the load deflection curve together with little post buckling strength indicates that the instability of the web panel took place after the penetration of yielding over a large portion of the panel. A similar relationship was observed for G2. No noticeable difference can be seen in the relationships of girders G3 and G4 compared with those of G1 and G2 except somewhat round knees. Large web deflection and thus failure of the web panels took place only on one side of the girder for all of the tests G1 through G4. The web panels of the girders with larger d/t_w ratios showed a tendency to deflect at a comparably smaller load and to increase in magnitude at an accerelated rate with increase of the loads. The rate of the increase of web deflection is more pronounced in a deeper girder. The maximum deflection observed on panels with horizontal stiffners was far small even at the ultimate load for G5 through G8 compared with that on the other side with no stiffner. The fact indicates that the stiffner effectively stabilized the web panel for shear buckling at least up to the maximum loads. The load-deflection curve of girder G9, however is slightly different compared with those obtained for G1 and G2. The web deflection of girder G9 started to increase gradually, although small in magnitude, with increase of the load, whereas the web panels of G1 and G2 remained practically at their original position until a load close to the ultimate loads was reached. The stiffness of the horizontal stiffners was sufficient to raise the shear carrying capacity of the web panel close to its yield load, but it was not sufficient enough to work as a so-called rigid stiffner, with which the panel would have behaved like panels with d/t_w of 60. This was also confirmed in the deflected configuration of the panel in the vertical direction; the shape was not a full wave but was a half wave with clear lateral movement at mid-depth.

No particular change can be seen in all of the load-deflection relationships of Fig. 3 at and around the shear buckling loads indicated by P_{cr}. The curves show that the instability of the web panel is a progressive phenomenon and agree with reports in literature (1) (2) (3) on tests of deep girders.

In order to compare the experimentally obtained ultimate loads with theories available, buckling loads as well as collapse loads based on the plastic analysis and on the theory* proposed by Basler (4) were computed and the results are listed in Table 4.

Table 4 Test Results and Comparison with Prediction

Girder	$\frac{d}{t_w}$	P_{max}^{ex}	P_y	P_{cr}	f	P_{cr}	P_{mps}	P_u	$\frac{P_{max}^{ex}}{P_{cr}}$	Safety Coeff.
G 1	60	221	213	213	0.74	161	235	213	1.08	2.01
G 2	60	208	213	213	1.00	126	188	-	0.98	1.96
G 3	80	249	284	227	0.98	186	253	248	1.10	3.11
G 4	80	229	284	227	1.23	165	218	-	1.01	2.86
G 5	100	247	355	182	1.27	162	260	244	1.35	3.82
G 6	100	218	355	182	1.75	164	217	-	1.20	3.38
G 7	120	254	427	151	1.58	141	268	250	1.68	4.70
G 8	120	224	427	151	2.01	135	226	-	1.48	4.14
G 9	120	381	427	288	-	-	458	-	-	-

P_{mps} in the table indicates the plastic collapse load with consideration to the effect of the presence of heavy shear stress in the webs. P_u indicates the resistance of the web panels to shear collapse proposed by Basler. The buckling loads were computed for boundary conditions of simply supported at four edges and fixed at both top and bottom edges. The presence of pure shear and in addition the presence of both shear and normal stresses as shown in Fig. 4 are considered in the computation. The results for simply supported panel at all four edges are listed in Table 4 with heading of P_{cr} for pure shear buckling and P_{cr}^* for buckling under the presence of shear and normal stresses.

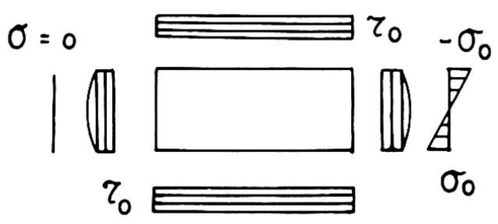


Fig. 4 Stress for Buckling Analysis

The ultimate loads obtained for girders G1 and G2, $d/t_w = 60$, were close to the full shear

* A question has been raised on the theory with a proposal of possible modification in Ref. 5.

yield load of the web panel, with a slightly lower load for G2. Although shear failure seemed to be dominant in the failure of G2, the smaller ultimate load indicates that the failure was due to the penetration of full yielding of the flanges from the mid-portion of the girder resulting in loss of framing rigidity which in turn changed the supporting condition of the panel and led to web buckling. The strain readings of the tension flange at mid-span clearly showed the difference between G1 and G2; the yield strain was just reached at the ultimate load for G1, whereas the strain was far exceeding the yield strain for G2. The maximum load of G3 was exceeding by 9 percent to its counterpart, G4, both of which had d/t_w of 80 with difference in flange sizes. The maximum load which G4 carried was close to the shear buckling strength of the web panel as can be seen in Fig. 3, while that observed in G3 was exceeding the buckling load by 10 percent. Concerning the girders with d/t_w of 100, G5 and G6, both the ultimate loads exceeded the shear buckling load by 36 percent and 20 percent, respectively. The largest reserves of strength above the shear buckling load were observed, as expected, for girders G7 and G8 with d/t_w of 120, 68 percent and 48 percent respectively; the depth-thickness ratio of 120 was the largest among the girders tested. Girder G9, over-all d/t_w of 120 but with horizontal stiffners at mid-depth failed, when one of the bearing stiffners on the supports buckled. The stiffness of the horizontal stiffners was approximately one-tenth of the minimum stiffness required to divide the buckling wave into two. With the stiffner, the buckling load of the panel due to pure shear was raised almost twice from 151 tons without stiffner to 288 tons. The experimentary obtained maximum load of 381 tons is, however, very close to the full yielding load of the web panel of 426 tons and is far above the shear buckling load for the web without the horizontal stiffners. Although the maximum load which the girder would have carried if no instability took place on the bearing stiffner remained to be unknown, it may be concluded that the stiffner worked effectively to prevent the shear buckling of the first mode with steels of one-tenth of the weight of the web panel. The effectiveness of the horizontal stiffners placed at half the depth suggests a feasibility of an economical design of horizontally stiffened deep plate girders for pre-fabricated bridges, which are not permitted in some of the current design specifications. The observed ultimate loads divided by the buckling loads for pure shear loading are listed in Table 4. The observed collapse loads divided by working loads give safety coefficients. Table 4 also includes in the last column the safety coefficients using the working loads based on the maximum permissible shear stress of

$$\tau_{all} = 394000 \left(\frac{t_w}{d} \right)^2 \qquad \text{kg/mm}^2$$

specified in AASHO design specifications. Among the test girders, premature failure took place due to bending for girders G4, G6, G7 and G8 and consequently the safety coefficients for the girders give the values below the lowest limit which can be expected for a girder with the same d/t_w ratio, when failure of the girder is due to web buckling. The load factors thus obtained turned out to be very conservative.

The experimental results are compared with buckling curves of average shear stress of the web panels versus depth-thickness ratio in Fig. 5. The solid lines are for plates simply supported at all edges, while the broken lines are for plates fixed at top and bottom edges and simply supported at other two edges. The elastic buckling curves in Fig. 5 are cut by horizontal yield lines satisfying the yield condition of von Mises'. Also shown in the figure are the limitations for the design of web plates with no intermediate vertical stiffner specified in AASHO and Japanese design specifications. A large reserve of strength above the buckling curves for simply supported plates are clearly seen with more reserve

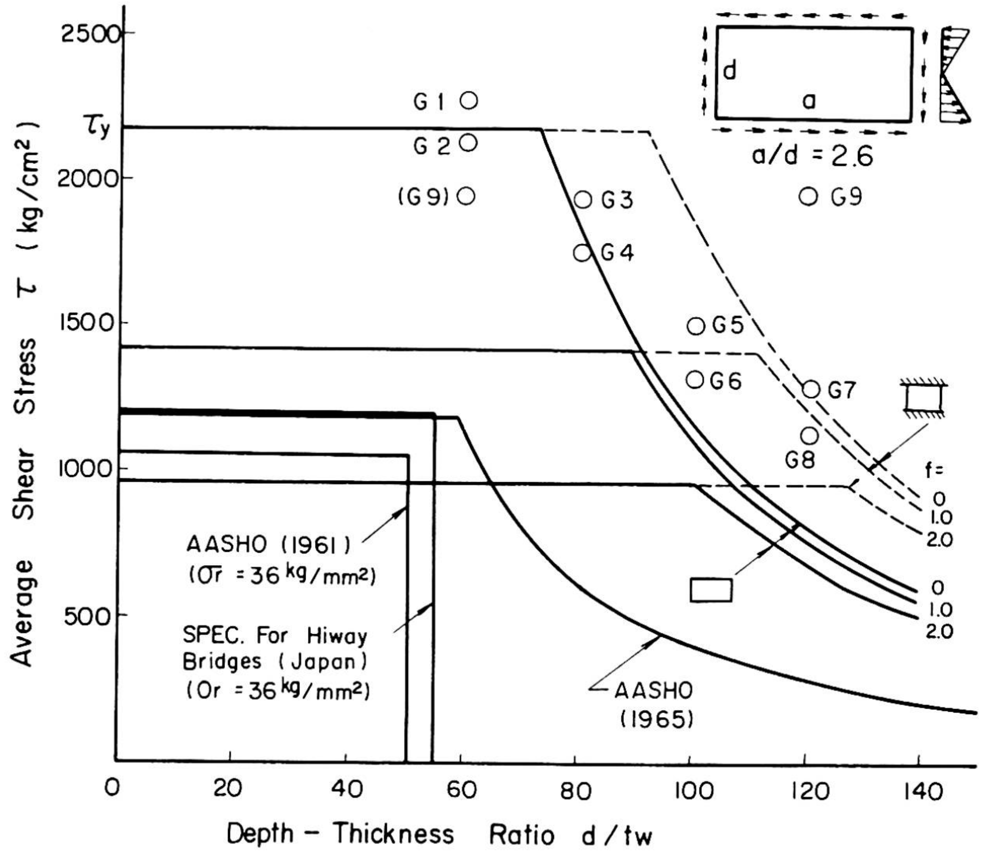


Fig. 5 Test Results and Buckling Curves

for plates with large d/t_w ratios. The factor f on which the buckling curves depend is a function of cross sectional dimension and can be defined by

$$f = \frac{d^2 t_w}{(d + t_f)\, b t_f}$$

With the factor f, the ratio of the normal to shear stresses working on the web panel as shown in Fig. 4 can be determined

$$\frac{\sigma_o}{\tau_o} = f\, \frac{b}{a}$$

The values of f for the test girders are shown in Table 4. The presence of the normal stress distribution lowers the pure shear buckling strength (f = O), however, the effect is more pronounced for the initiation of yielding as can be seen by the horizontal lines in the figure. The buckling curves for fixed plates at both the top and bottom edges locate above the experimental points.

Despite the prediction being made at the time of planning, the plastic collapse loads agreed remarkably well with the experimentally obtained ultimate loads even for the girders with d/t_w ratio of as large as 120, which corresponds to d/t_w of 145 for a girder of structural carbon steel (σ_Y = 24 kg/mm^2). [1] The differences were less than 5 percent for all girders except G2 and G9 with theoretical loads exceeding the experimental ultimate loads. Among each pair, better agreement exists for the girder with smaller flanges, while a slightly larger difference is present in the girder with heavier flanges, for whose failure shear force played a more predominant role. The results of this study are opposed to the report (6) that plastic analysis was no longer applicable for a structural steel girder with d/t_w = 120. The buckling behavior of a structural steel girder with d/t_w = 120 may resemble to a high strength steel girder of this tests with d/t_w = 100, if the difference of yield strength is taken into account. The discrepancy may be due to the difference in strength of steel, shear span length, cross sectional properties and some others, however since detailed information is not given in Ref. 6, no detailed consideration into the causes of the discrepancy has been made. The largest disagreement was observed for girder G1 with the ultimate load exceeding the prediction by 10 percent. As has been pointed out, the immediate cause of the failure was the penetration of yielding into the flanges from the center of the span. Since a large moment gradient exists with this test set-up, the penetration of full yielding into the cross section at only the limited length did not necessary force the girder to failure, instead failure took place after a large portion of flanges had yielded with the increase of the load. The relatively large reserve of strength over the plastic collapse load observed for G1 suggests a somewhat different situation for girders with other loading condition. The good agreement between the collapse loads and the ultimate loads obtained for the rest of the girders may be due to the large moment gradient. Therefore the conclusion of this study that the plastic collapse load represents the carrying capacity of a plate girder with d/t_w ratio of as large as 120 should have to be understood together with the loading condition and the cross sectional properties.

It is reported [(2) (3)] that the shear resistance to collapse proposed by Basler which is the basis of the AISC specifications agrees well with the experimental results of deep plate girders, however no information is reported whether the prediction represents the strength of a girder with a relatively thick web plate as tested in this study. The shear collapse loads were computed as one of the reference loads and are listed in Table 4 under the heading of P_u. In the computation, no interaction with bending moment was considered so that the prediction may better be compared with girders with heavier flanges among each pair. It so happened that the numerical values of this collapse loads were close to the plastic collapse loads and thus a good correlation existed between the predictions and the test results, however no definite conclusion may be drawn from the comparison and the applicability of the theory to girders with relatively thick web plates remained to be unanswered. The plastic collapse loads increase with increase of flange area for the same web plate, whereas the theory by Basler predicts the same collapse loads; therefor testing of girders with still heavier flanges may reveal the difference and may lead to a clear conclusion.

5. SUMMARY AND CONCLUSIONS

Tests were performed on a series of nine full size plate girders with transverse vertical stiffners only at supports and at the center. The specimens were made of a high strength steel with yield stress of around 40 kg/mm^2. The depth to thickness ratio of the web plates ranged from 60 to 120.

The girders with larger depth-thickness ratios remained practically elastic to a load exceeding the buckling loads and no particular behavior was observed around the buckling loads.

The experimentally observed maximum loads exceeded the web buckling loads for all of the test girders, with exceptions of the girders with small depth-thickness ratios and failed by flange instability. Despite the comparatively small rigidity of the frameworks due to large aspect ratio of web panels of around 2.6 and in addition due to relatively smaller depth-thickness ratios of 120 at most, a large reserve of strengths above the buckling load was observed among the deeper girders, even for the girders failed by flange instability.

Although some of the design specifications restrict the use of web panels without intermediate vertical stiffners except panels with thick plates, the tests revealed no particular ground for the restriction under static loading condition. The permissible shear stress specified in AASHO design specifications, one of the specifications which permit the use of this type of web panels, turned out to be very conservative for the test girders. The safety coefficients to be specified for this type of plate girders in a design based on the buckling load may be reduced for webs with depth-thickness ratios exceeding 80. A horizontal stiffner placed at half the depth worked effectively to prevent premature buckling of the web due to shear force, suggesting a feasibility of an economical design of horizontally stiffened plate girders for shop-produced bridges in large quantities, in which no intermediate vertical stiffners may be placed except at the points where heavy loads are concentrated.

The shear collapse loads with consideration to tension field action agreed well with the ultimate loads of the test girders, which failed primarily by excessive shear, however with the test program of this study no definite conclusion can be drawn for the applicability of the predictions to girders with relatively thick plates as tested.

The plastic collapse loads correlated well with the test results and represented best the experimentally obtained ultimate loads for all girders tested including the girders with d/t_w ratio of as large as 120.

6. AKNOWLEDGEMENTS

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Special thanks are due to the assistance of Kawasaki Steel Corporation for donating the steel and fabrication of the test girders.

7. LIST OF REFERENCES

(1) Massonnet, C.E., STABILITY CONSIDERATIONS IN THE DESIGN OF STEEL PLATE GIRDERS, Proc. ASCE, ST1, Jan. 1960 (2) Basler, K., et al., WEB BUCKLING TESTS ON WELDED PLATE GIRDERS, WRC Bulletin, No.64, U.S.A., Sept. 1960 (3) Cooper, P.B., et al., WELDED CONSTRUCTIONAI ALLOY STEEL PLATE GIRDERS, Proc. ASCE, ST7, Feb. 1964 (4) Basler, K., STRENGTH OF PLATE GIRDERS IN SHEAR, Proc. ASCE, ST7, Oct. 1961 (5) Fujii, K., MINIMUM WEIGHT DESIGN OF STRUCTURES BASED ON BUCKLING STRENGTH AND PLASTIC COLLAPSE, J. ZOSEN Kyokai, No. 122, 1967 (6) Longbottom, E., et al., EXPERIMENTAL VERIFICATION OF THE STRENGTH OF PLATE GIRDERS DESIGNED IN ACCORDANCE WITH THE REVISED BRITISH STANDARD 153, J. ICE., 1965

SUMMARY

Tests were performed on a series of nine full size plate girders with transverse vertical stiffeners only at supports and at the center. The depth to thickness ratio ranged from 60 to 120. The observed maximum loads exceeded the web buckling loads for all of the test girders, except those with small depth to thickness ratio failing by flange instability. The shear collapse loads and the plastic collapse loads correlated well with the test results.

RÉSUMÉ

Une série de neuf poutres à âme pleine, grandeur nature, munies de raidisseurs verticaux au milieu et sur appuis seulement a été testée. Le rapport hauteur-épaisseur variait de 60 à 120. Les charges de rupture dépassaient dans tous les cas les charges de voilement, excepté pour les poutres à rapport hauteur-épaisseur inférieur, qui se cassaient par instabilité des membrures. Les charges de ruine de cisaillement et les moments plastiques de rupture coincidaient pour tous les cas avec les valeurs calculées.

ZUSAMMENFASSUNG

Eine Serie von neun Vollwandträgern in natürlicher Grösse, mit Vertikalaussteifungen nur in der Mitte und über den Auflagern, wurde untersucht. Das Verhältnis Höhe-Stegdicke reicht von 60 bis 120. Die beobachteten Bruchlasten überschritten in allen Fällen die Beullasten, ausgenommen für die Träger mit kleinem Höhe-Dicke-Verhältnis, die infolge Flanschinstabilität zusammenstürzten. Sowohl die Schubbruchlasten als auch die plastischen Bruchmomente stimmten für alle Testbalken gut mit den Rechenwerten überein.

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Thin-Walled Deep Plate Girders under Static Loads

Poutres à âmes pleines minces et hautes sous charge statique

Hohe Vollwandträger mit dünnen Stegen unter ruhender Last

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Introduction

During the last years there has been an increasing interest in using welded steel plate girders with exceptionally slender webs. Usually such girders have web stiffeners as these increase the bearing capacity considerably.

The cost of the stiffeners are however out of proportion as the manual work disturbs the normal flow of automatic production. They increase the cost of painting and future maintenance too. Also they make it more difficult to use the girder as a standardized structural member as they may restrict the positions of the forces. Thus it has often been discussed to limit the number of web stiffeners or wholly to avoid them.

In USA the 1963 AISC specifications allow omitting of stiffeners up to a web slenderness ratio h/d of 260. In Sweden there are temporary specifications allowing h/d up to 320 with stiffeners at the supports only. According to these Swedish specifications important roof structures have been built with plate girders having clear spans up to 50 m.

Formerly it was a common thought that the upper bound of possible stress in the web was the theoretical buckling stress based on the assumptions of small deflections and no initial deflections. It is now, however, since long well known that this theoretical load by no means corresponds to that shear force, which can be carried by a girder with a slender web. Is is evident that in such a girder redistributions of the inner forces are possible, which lead to much higher failure loads.

If the girder has web stiffeners it is suitably calculated following the theory of tension fields. For a girder without stiffeners it has been pointed out [1] that for a long slender plate, under certain boundary conditions, a shearing force up to 3 times the just mentioned idealized theoretical buckling load ought to be carried.

Extensive tests on plate girders have been reported upon for instance by Massonnet [2] and prior to the working out of the 1963 AISC specifications by Basler and Thürlimann [3]. During recent years Corbit and Marsh, Schilling, Cooper and others have completed our knowledge.

Testing programme

The following tests have been performed in order to get an experimental evaluation of the influence of web stiffeners. The intention was both to limit the number of web stiffeners and to develop new methods of fastening them.

The main view points of the testing programme were concentrated on the following problems.

a. Information on the all-over behaviour of girders with exceptionally slender webs

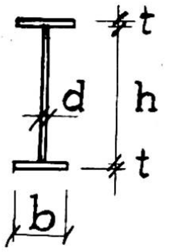
b. General influence of web stiffeners on the bearing capacity of the slender plate girders

c. Web stiffening - with special regards to the method of application, height of stiffeners and other special questions concerning stiffener details.

d. Web crippling under concentrated loads

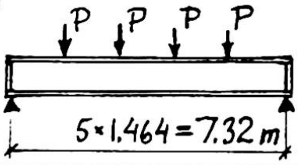
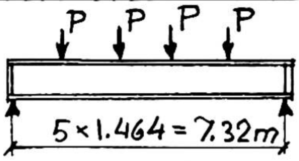
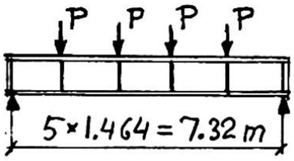
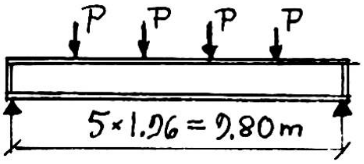
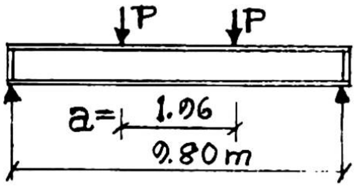
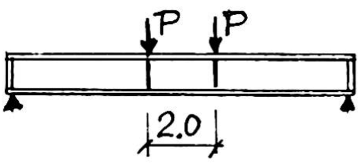
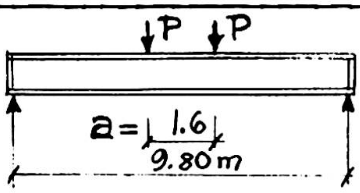
A great many tests have been conducted on test specimens with dimensions as shown in Table 1. The span widths and loading cases are shown in Table 2.

TABLE 1. DIMENSIONS OF TEST GIRDERS

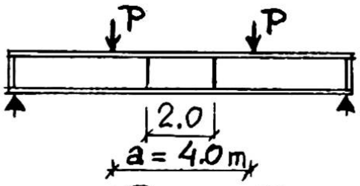
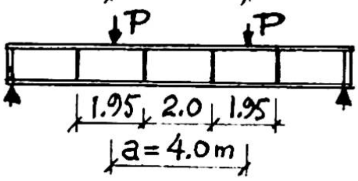
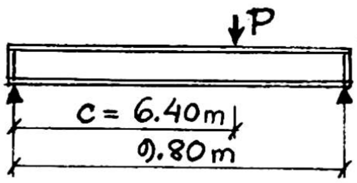
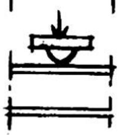
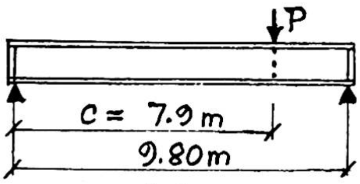
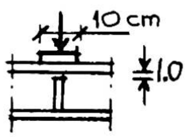
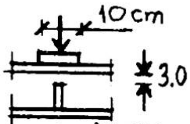
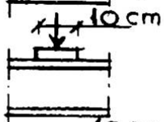
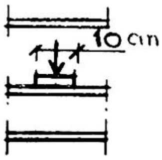


Girder No.	h cm	d cm	b cm	t cm	h/d	$\sigma_{yield, flange}$ $\sigma_{yield, web}$ kp/cm²
1	60	0,2	17,5	0,6	300	2900 3400
2	60	0,2	17,5	0,6	300	St 37
3 - 4	59	0,3	20	0,8	197	St 44
5	59	0,3	20	0,8	197	3100 2800
6	59	0,3	20	0,8	197	2750 3300
7	59	0,3	20	0,8	197	2950 3270
8 - 9	30	0,2	10	0,6	150	St 37
10 - 11	40	0,2	10	0,8	200	St 37
12 - 13	50	0,2	10	1,0	250	St 37
14 - 15	60	0,2	10	1,2	300	St 37
16 - 17	70	0,2	10	1,5	350	St 37

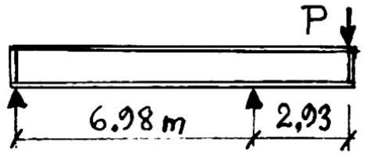
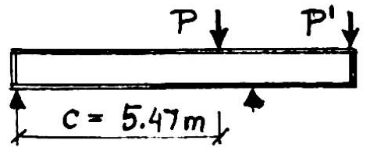
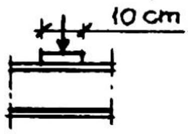
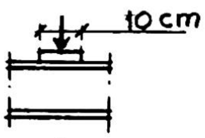
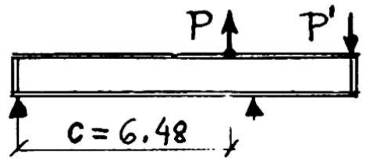
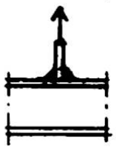
TABLE 2. LOADING CASES, BENDING AND SHEARING STRESSES

Girder No. / Test No.		Loading cases and span widths meters	Maximum observed load, P Collapse loads P_c Megapond (1000 kilopond)	Maximum stress σ_{max}, τ_{max} kp/cm^2	Notations $\frac{\tau_{max}}{\tau_{cr}}$ etc.
1	1		P_c = 2,8 Total compression flange instability	σ = 1640 τ = 467	4,1 See fig. 1
2	1		P = 2,0 Two loading cycles. No rupture		
	2		P_c = 3,8 Local flange buckling under the second point load	σ = 2220 τ = 635	
3	1		P = 5,2 Exc. lat. defl.	σ = 2750 τ = 587	2,25
	2		P = 6,85 Exc. lat. defl.	σ = 2400 τ = 387	1,48
	3	a = 4,0	P_c = 9,2 Web crippling	σ = 2400 τ = 520	1,98
	4	a = 5,0	P_c = 10,2 Web crippling	σ = 2200 τ = 576	2,20
	5	a = 6,0	P_c = 10,95 Web crippling	σ = 1865 τ = 620	2,37
	6	a = 8,0	P_c = 11,7 Web crippling	σ = 940 τ = 650	2,50
	7		P_c = 10,7 Local flange buckling under one P load	$\sigma > \sigma_{yield}$ τ = 605	
4	1		P_c = 3,1 Excessive flange rotation under one point load.	σ = 1140	The jack was hinged to the flange

TAB.2.cont.

Girder	Test	Loading cases	Maximum load	Max. stress	Notations
4	2	a = 2,0	P_c = 5,5 Lateral deflection and local yielding	σ = 1930	
	3	a = 3,0	P_c = 5,62 Excessive lateral deflection	σ = 1740	
	4	a = 4,0	P_c = 5,9 Excessive lateral deflection	σ = 1550	
	5		P_c = 5,48 Excessive lateral deflection	σ = 1430	
	6		P_c = 6,0	σ = 1560	For a second loading cycle P_c = 5,5
	7		P_c = 8,45 Web crippling		
	8	c = 2,38	P_c = 9,48 Web crippling	σ = 1530 τ = 405	
	9	c = 7,4	P_c = 9,8 Web crippling	σ = 1600 τ = 418	
	10	c = 2,4	P_c = 8,8 Web crippling		
	11		P_c = 13,7 Web crippling	σ = 1870 τ = 625	
	12	c = 6,9	P_c = 12,4 Web crippling	σ = 2270 τ = 490	
	13	c = 7,4	P_c = 9,9 Web crippling	σ = 1610 τ = 670	
	14	c = 6,4	P_c = 8,7 Web crippling	σ = 1730 τ = 320	

TAB.2.cont.

Girder	Test	Loading cases	Maximum load	Max. stress	Notations
5	1		P = 10,53		Three loadi cycles
5	2		P′ = 8,0 P_c = 10,63 Web crippling		P′ remaine constand du ring loading
	3	c = 6,48	P′ = 9,0 P_c = 9,9 Web crippling		
	4		P′ = 9 P_c = 13,7 Yielding in ten-sion flange		P′ remaine constant
6	1 - 46				See fig. 4
7	1 - 27				See fig. 4
8 - 17					See fig. 3
8S - 17S					Stub tests on girders 8 - 1

All test girders were made of steel qualities with the designations SIS 1311 or SIS 1411 corresponding to the Swedish Steel Specifications. This means among others that the yield and failure stresses ought to be at least 2200 and 3700 respectively 2600 and 4400 kp/cm^2 (St 37 resp. St 44). Generally the real yield stresses were higher.

Beside of the usual measurements by mechanical dial gauges for vertical and lateral deflections the strain distributions over the beam cross sections have been measured by electrical strain gauges. The compression flange was braced against both lateral deflection and rotation, except for test 1:1 and 8S - 17S.

In some cases special measurements have been carried out. Some of the beams rested at one end on a two-point support in order to measure the over-turning moment acting on the beam. The two parts of the support consisted of a loading cell each. Knowing the distance between the cells it was possible to get an estimate of the overturning moment and the degree of instability.

All-over behaviour of girder and influence of web stiffeners

Here it is chosen to describe test 1:1 from Table 1 and 2 in order to illus-trate the all-over behaviour. A comparision with test 2:2, which is the same girder but now with stiffeners on it, gives the influence of web stiffeners.

Test 1:1 is concentrated in fig. 1, where the mid-span deflection is shown

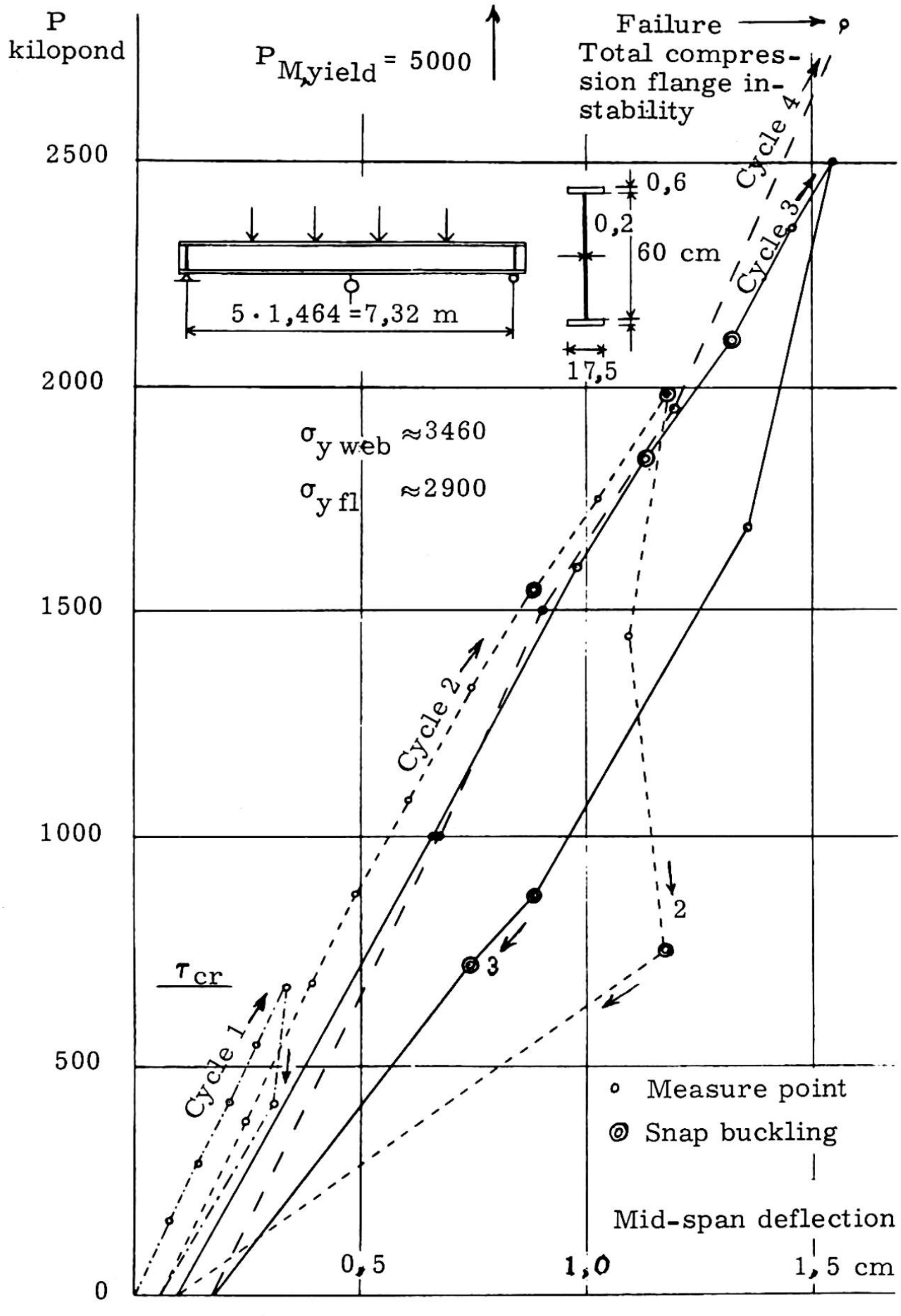


Fig. 1 Ratio between jack loads and mid-span deflection. Test girder 1.

versus the load of each loading jack. The theoretical span of the girder is 7,32 m, the depth 60 cm and the web thickness 0,2 cm. The slenderness ratio is thus h/d = 300. The loading were four point loads and the compressed flange may rotate, but not deflect laterally (cp. p. 5).

In the diagram the jack force giving τ_{cr} for the traditionally calculated buckling load is marked and also the force giving yield stresses σ_y following Navier's theory of bending. The girder was first loaded to τ_{cr}. Then unloaded and reloaded to 3 τ_{cr}. After another loading cycle to near under a supposed failure load it was finally loaded to collapse.

When the load passes τ_{cr}, nothing in particular happened and even passing 3 τ_{cr} had no influence even on this girder with stiffeners only at the supports at a distance of 7,32 m apart. Soon there-after it collapsed, however. This happened before reaching the compression yield stress and was a stability failure. Suddenly the steered compression flange rotated in its whole length and the transverse bending stiffness of the web was so little that a plastic zone developed in the web reaching from support to support.

A quite different behaviour was observed in the case of tests 3:1, 3:2, 4:3 and 4:4, where even in the case of a braced compression flange the web deflected laterally and caused the tension flange to warp and tilt.

Notwithstanding there was no indication of the importance of τ_{cr} during the test, it possibly had some influence when passing it at unloading. As marked in the diagram of fig. 1 one observed web snap buckling in three cases just at the level of τ_{cr}. Comparing this with fig. 2 which is taken from p. 657 of ref. [3] it may perhaps be said that it illustrates the branching conditions near the formal buckling load. Basler and Thürlimann state that if the plate at this stress level could be brought to a position on the dashed branches in quadrant 3, which are unstable equilibrium positions, it could then snap to either side of the reference plane and stabilize there. As indicated in fig. 1 snap buckling was observed also in several cases at over double the just mentioned stress level (but in other girder tests also at several other levels).

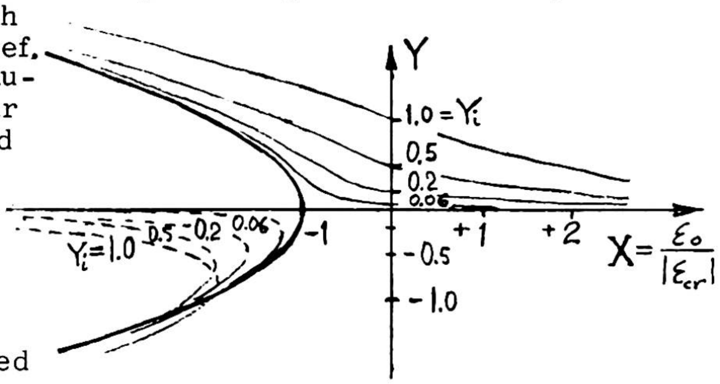


Fig. 2 Plate deflection Y versus applied edge strain X, ref [3].

The snap buckling at the unloading part of load cycle 2 has strongly influenced the deflection of the girder. This is often the case in snap buckling. The snaps are strongly dependant on the rate of loading and the energy releases increase with increased rates of loading. It was impossible to state if there was a simple and direct relation between the intensity of load and the snap.

As to calculation of the load causing flange yielding in bending the following comments may be of interest. The technical beam theory is based upon the assumptions of Hooke and Bernoulli. But Bernoulli s assumption of plane sections remaining plane after deformation is applicable only to a certain degree and the usual assumption that the cross section remains rigid during deformation does not hold. The consistency of these assumptions is a function of the slenderness and of the degree of stiffening applied to the web (web stiffeners) and to the flanges (bracing).

The different measurements of strain distributions over the cross sections showed that great transversal bending stresses existed in the web and the middle part of it was ineffective in taking up in-plane bending. Fig. 3 illustrates typical

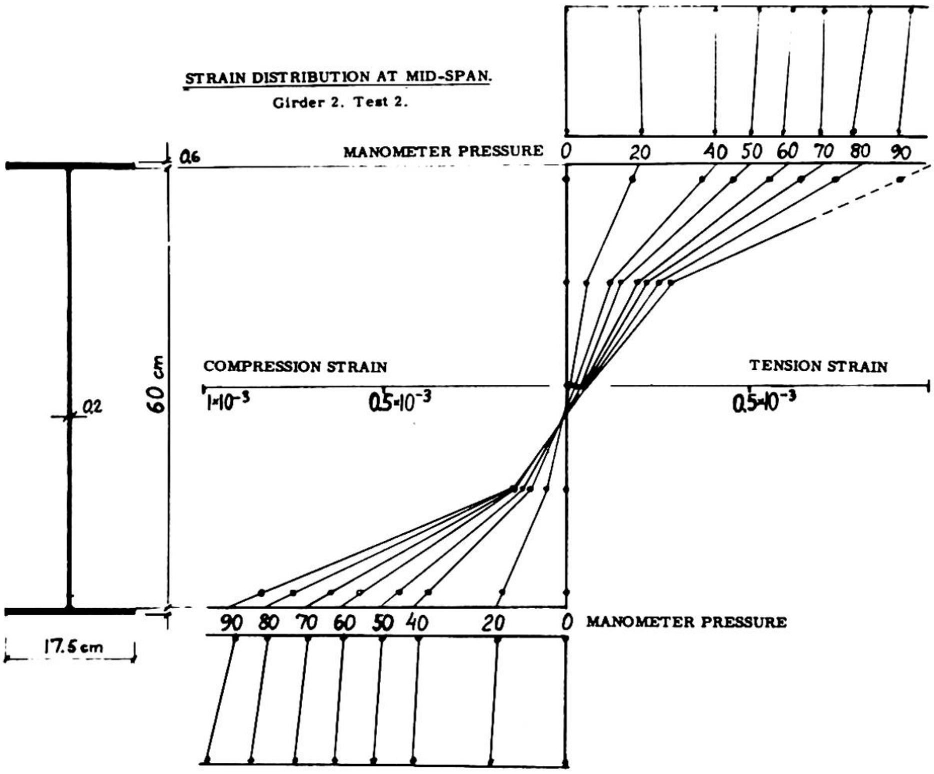


Fig. 3 Strain distribution at mid-span.

test-results. The test points marked on the figure show that the average strain in the web is much less than compared to a linear stress distribution. This is to be expected because of the initial and additional lateral deflections of the web. The relative weakness is especially typical for the compression side, but it has to be observed that a deviation from the linear theory also occurs at the tension side. This do not confirm the results of Basler and Thürlimann [3], who stated that only the compressed part of the web was not taking up bending stresses in full.

Test 2:2 compared to test 1:1 shows the influence of web stiffeners. The failure occured here not until the load was about 35 % higher and nearer to that giving yield stresses in the flanges. The failure reason was local buckling of the compression flange. The web stiffeners hold the tension flange so that there was no folding of the web (cp. test 3:1, 3:2, 4:3, 4:4 and even 4:5).

Web crippling

As shown the web stiffeners are useful to uphold the invariability of the cross section of the girder. They increase the bearing capacity by making tension fields possible and prevent excessive web buckling. Another reason, which often makes them necessary, is to prevent local web crippling.

At the supports they are practically inevitable. Between supports intermittent stiffeners could often be avoided, however.

Tests 8-17 are put together in fig. 4. They constitute a test series intended to show the influence of a point load on the web behaviour. The web thickness

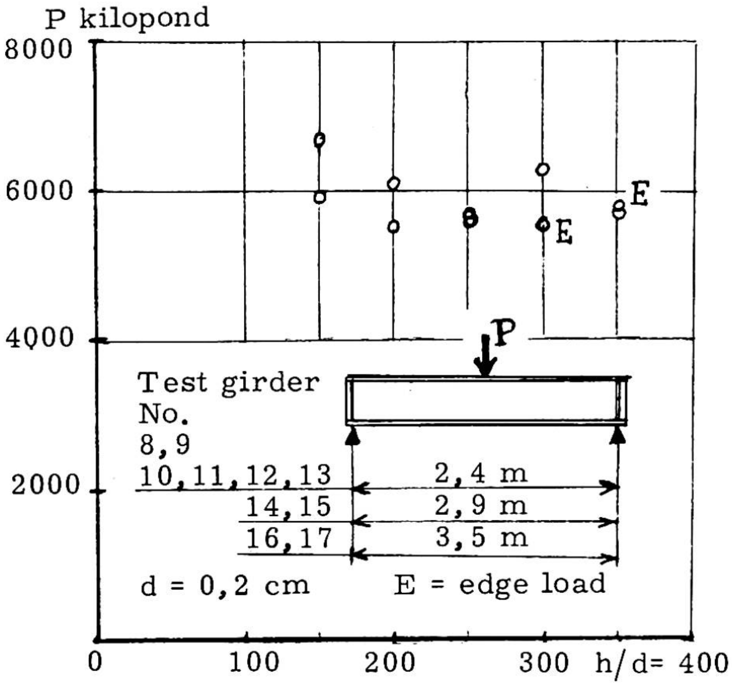


Fig. 4 Crippling loads for different web ratios.

was 0,2 cm. The depth varied from 30 to 70 cm and the flange areas from 0,6 x 10 to 1,5 x 10 cm^2. It may be seen in fig. 4 that the cripple load only varied between approximately 5500 and 6500 kp. The failure loads seem thus almost independent both of web depth and flange dimensions. Other tests show that they depend almost only on the web thickness. Even the span of the girder, and with it the bending stresses, seem to have a very small influence. Not until the bending compression stress in the web approaches the yield stress the influence seems to be more pronounced. From some other tests, no 3:3, 3-6, 4:7-9 and 5:2-3 it is seen that in these cases, however, the cripple load diminishes with increasing bending stress. Even the influence of the type of load transfer seems very small - either it was characterized by transmitting the point load through a half round or a short rectangular bar or through a 18 cm long plate.

Of course the results were quite different for tests on stub girders without any web stiffeners. If they were pressed directly over the supports web buckling took place according to the corresponding case of simple column buckling.

Previous tests have been made [4] and the cripple load was then given as $P_c \approx 0,9 \cdot 10^5 \cdot d^2$ with d in cm and P_c in kp, or $P_c \approx \sigma_s \cdot d^2/0,03$. The current tests also point on the fact that P_c is mainly depending on d, but perhaps the exponent should be smaller than the one used in this formulea.

When working out a theoretical estimate one usually starts from the theory of beams on elastic foundations. The flange is considered as the beam and the web as the elastic foundation. Considering the flange as a beam one reaches the formulae of Klöppel and Lie [5]. The beam thus reaches a certain "elastic length". In the elastic range the flange ought to function as a T-beam together with the nearest part of the web. Already with only a small part of the web co-operating, the web will have a dominating influence on the inertia of the "beam" and thus d is an important factor. This is the reason perhaps, why the load P_c greatly depends on d and not so much on the flange dimensions.

For the slender girders tested some uncertainities were unevitable. Owing to fabrication methods the flange was not centric over the web and the flanges were not plane. Sometimes the flange was so distorted that a proper contact between flange and jack was difficult to establish without filling in plates. Such excentricities of course initiated early buckling and crippling.

Web stiffeners and their heights.

Often the loads taken by the web with stiffeners at the supports only, are fairly large. Sometimes the point loads on the span are still larger or the total load is so large that using stiffeners is the correct solution.

Test 6 and 7, the results of which are concentrated in fig. 5, is a series treating on one hand the interaction of combined bending moment and shear forces as well as cripple loads, and on the other hand the length of any vertical web stiffeners.

It is seen that a lower bound for the measured points is near under the horizontal line $P_c/P_o = 1$, where $P_o \approx 9000$ kp, which ought to be cripple load for the unstiffened web (length of stiffeners = 0). The fact that this line is practically horizontal, confirms what just has been said about the cripple load being almost independent of the bending stresses. A bound to the right in fig. 5 is a vertical line corresponding to the yield moment.

It may also be seen that for stiffeners of the height h/3 to h/2 the cripple load will be higher, but in the case of large bending stresses only a bit higher than for no stiffeners. With stiffeners over the total web height the level of failure load is further increased. Usually the bound does here not any longer depend on the cripple load but on the shear force. The bearing capacity of the girder

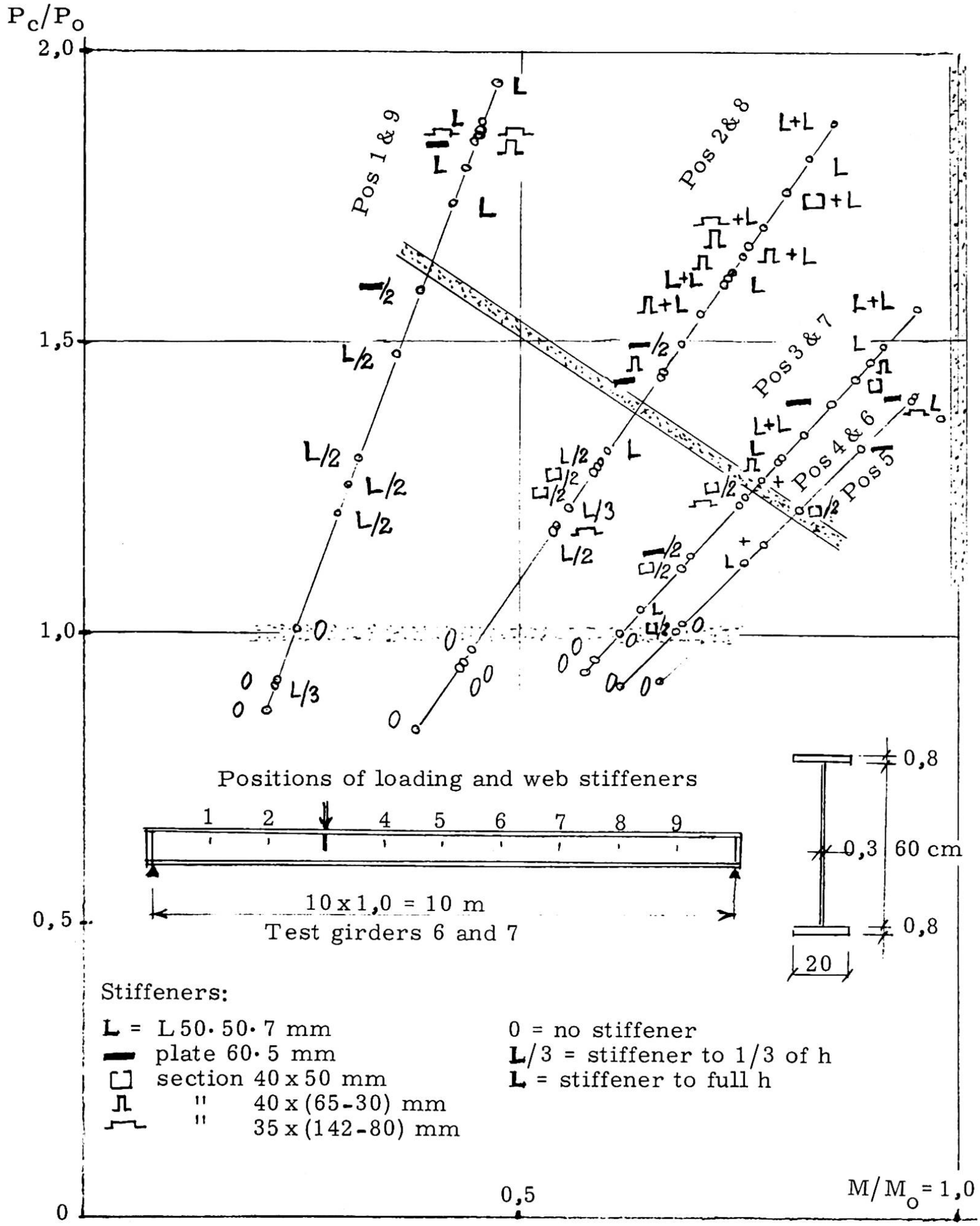


Fig. 5 Failure Loads P_c/P_o as function of M/M_o for different type of stiffeners.

is the sum of the shear force in the web and the load in the tension field between the stiffeners. There is an agreement with the calculations according to ref. [3].

The compression flange

The post-critical bearing capacity of the web is rather large. This holds true, however, only if the girder as a whole is able to support the load. Yielding of the flanges is of course a limit. Another is often the lateral or local buckling of the compression flange. Several tests on these phenomena have been made and reported. For the actual girders with web slenderness h/d = 150 to 400, tests were made to verify how to brace the compression flange and how to find the necessary bracing forces.

In connection with these problems it was necessary to penetrate which calculating model is the correct one for the stability behaviour of the compression flange. The question has been raised if the compression flange forces shall be considered as direction-invariant or if they point in the tangent direction to the center line of the flange.

Comment on the girder tests

Owing to the extremely thin dimensions and special methods of fabrication new factors stress to be of importance compared to those met with in conventional construction with rolled beams and heavier sections.

The following factors tend to be of interest and may have great influence on girder behaviour:

The inhomogenity of the material
The diviations of the real from the nominell dimensions
The presence of internal stresses due to fabrication methods
Owing to the fabrication methods initial deflections and lack of straightness are induced in the finished girder. The lack of straightness may be considerable.

The factors influence the test results and cause a wide divergence. In this way even girders of the same dimensions may show a difference in behaviour during the loading tests. Anyhow the increased instability risks call for additional bracing compared to girders with thicker dimensions.

Comparison between specifications

It is known that for slender webs the post-critical reserve in bearing capacity as characterized by the critical stresses according to the linear small deflection theory is greater than for thicker ones. This fact has resulted in a certain audacity among specification writes. As an example the beforementioned. temporary Swedish specifications allow stresses that for a slenderness ratio larger than 81 are 1,05 to 2,92 times those of the 1963 AISC specifications.

h/d	$\tau_{temp\ Sw.}/\tau_{AISC}$
81 to 221	$0,756+0,442 \cdot (h/d)^2 \cdot 10^{-4} = 1,05$ to $2,92$
>221	2,92

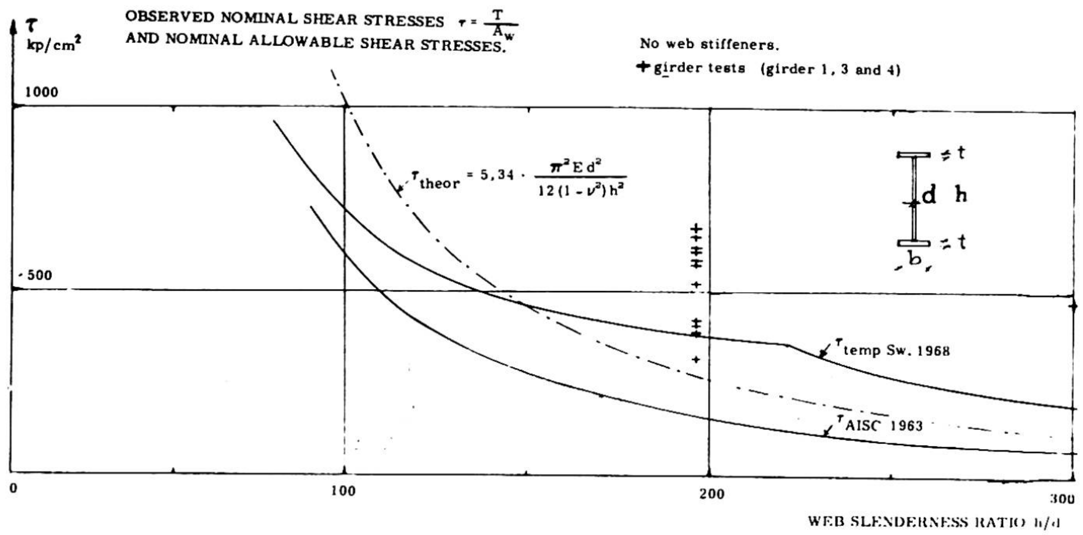


Fig. 6 Comparasion between theoretical, allowed and measured shear stresses.

Fig. 6 illustrates the comparasion. It has to be observed that the curves in the diagram apply to girders with unstiffened webs, e. g. a/h = ∞. Just at the web slenderness ratio h/d = 221 there is a minor anomaly in the Swedish temporary specifications, which depends on approximations in the simplifying curves used. On the diagram the nominal shear stress at failure for the beams 1, 3 and 4 without web stiffeners are marked. Some of the points lie rather low, but it has to be observed that in no case there occurred a shear failure. The maximum loads were limited be excessive lateral deflections, flange rotations or local web crippling under concentrated loads.

Despite this audacity as regards the allowable stresses the specifications have been and are still used for many important structures with hitherto good results as mentioned in the introduction.

*

[1] S.Bergman: Behaviour of buckled rectangular plates under the action of shearing forces (p. 119). Diss, Stockholm 1948.

[1 b] G.Wästlund and S.Bergman: Buckling of webs in deep steel I girders. Stockholm 1947.

[2] Ch.Massonnet: Essais de voilement sur poutres à âme raidie. AIPC Mém. 14, p. 125, 1954.

[3] K.Basler and B.Thürlimann: Welded plate girders. Trans. ASCE, Vol. 128 II, p. 655, 1963.

[4] C-A.Granholm: Lättbalkar (Light girders). Teknisk Tidskrift, Stockholm 1961, p. 455.

[5] Klöppel und Lie: Beulung des rechteckigen, allseitig belasteten und einspannungsfrei gelagerten Bleches. VDI-Zeitschrift, Bd 86, H 5/6, 1942.

SUMMARY

The paper summarizes results from several tests on slender, welded steel plate girders with special considerations to web behaviour and web stiffening.

RÉSUMÉ

L'article traite brèvement les résultats d'essais sur poutres soudées avec l'âme mince et en considération spéciale du comportement des âmes et leur raidissage.

ZUSAMMENFASSUNG

Dieser Aufsatz beschreibt knapp die Ergebnisse der Versuche mit geschweissten, hohen Blechträgern mit besonderer Berücksichtigung des Stehblechverhaltens und dessen Aussteifungen.

IIc

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.

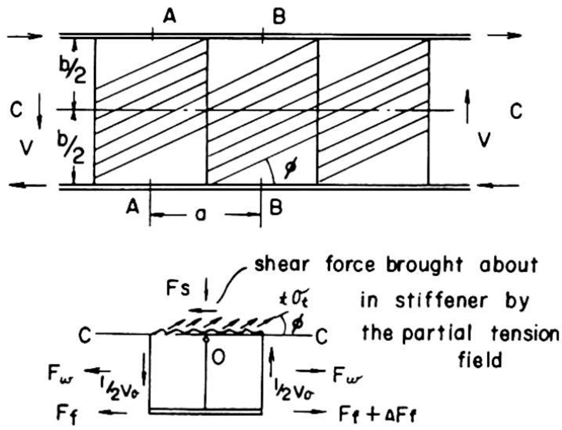


Fig. 1

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,

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

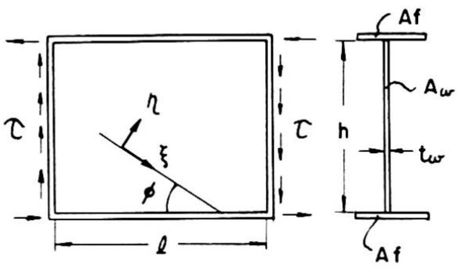


Fig. 2

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

$$\begin{aligned} \sigma_{\xi cr} &= \tau_{cr} \sin 2\phi, \\ \sigma_{\eta cr} &= -\tau_{cr} \sin 2\phi, \\ \tau_{\xi\eta cr} &= \tau_{cr} \cos 2\phi. \end{aligned} \tag{2}$$

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.

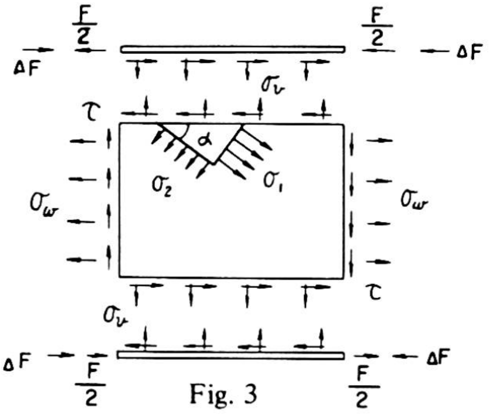


Fig. 3

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,

$$\sigma_1 \sin^2\alpha + \sigma_2 \cos^2\alpha = \sigma_v , \tag{3}$$

$$(\sigma_1 - \sigma_2) \sin\alpha \cos\alpha = \tau , \tag{4}$$

Shearing force is,

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

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,

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

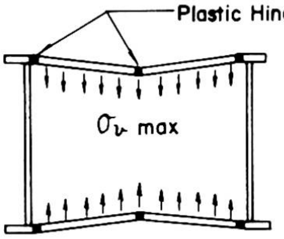


Fig. 4

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.

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),

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

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

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

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

$$\nu_{u0} = 2\nu_{cr}/(1+\nu_{cr}^2), \quad \tan\alpha_0 = \nu_{cr} \tag{13'}$$

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,

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

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)

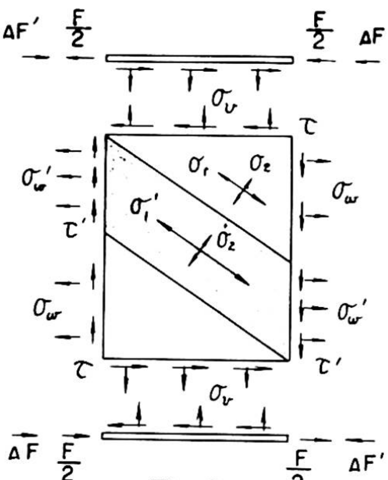


Fig. 5

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 $\partial v / \partial \alpha = 0$, as follows,

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

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

$$v_u = \sqrt{\lambda^2 + (1 + \lambda v_{cr})^2} - (1 - \varepsilon)\lambda \tag{20}$$

Particularly, if $\varepsilon = 0$,

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

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

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

or

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

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

Values of v_{cr} are calculated by the following equations which are modified Johnson's column formula.

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

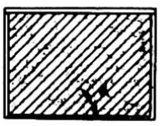
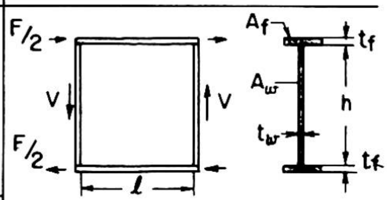
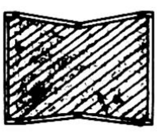
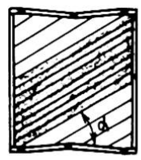
$$v_{cr} = 1 - \frac{3(1-\nu^2)}{k_s \pi^2} \left(\frac{\tau_{wY}}{E}\right) \left(\frac{h}{t_w}\right)^2 , \quad v_{cr} > 0.5 \tag{22}$$

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
	$(1 - v_{cr}) < \varepsilon$	$v_u = 1$ $\alpha = 45°$	
	$1 - \frac{\lambda + v_{cr}(1+\lambda v_{cr})}{\sqrt{\lambda^2 + (1+\lambda v_{cr})^2}}$ $< \varepsilon \leq$ $(1 - v_{cr})$	$v_u = \frac{(1-\varepsilon) v_{cr} + \sqrt{1 + v_{cr}^2 - (1-\varepsilon)^2}}{1 + v_{cr}^2}$ $\tan\alpha = \frac{v_{cr} + \sqrt{1 + v_{cr}^2 - (1-\varepsilon)^2}}{2 - \varepsilon}$	$\lambda = l/h$: aspect ratio α : inclination of tension field σ_{wY}: yield stress of web σ_{fY}: " " flange τ_{cr}: web buckling stress
	$\varepsilon \leq 1 - \frac{\lambda + v_{cr}(1+\lambda v_{cr})}{\sqrt{\lambda^2 + (1+\lambda v_{cr})^2}}$	$v_u = \sqrt{\lambda^2 + (1+\lambda v_{cr})^2} - (1-\varepsilon)\lambda$ $\tan 2\alpha = \frac{1 + \lambda v_{cr}}{\lambda}$	$\varepsilon = \frac{8}{\lambda^2} \frac{t_f}{h} \frac{A_f \sigma_{fY}}{A_w \sigma_{wY}}$ $v_{cr} = \tau_{cr} / \tau_{wY}$ $v_u = V_u / A_w \tau_{wY}$

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 V_{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.

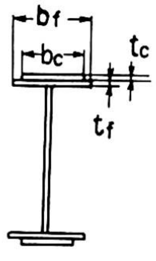


Fig. 6
Notations for cover plates

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 (1)	Experimental values: Web $h \times t_w$ (2)	Web σ_{wY} (3)	Flange $b_f \times t_f$ $b_c \times t_c$ (4)	Flange σ_{fY} σ_{cY} (5)	λ (6)	$\frac{h}{t_w}$ (7)	V_u^{ex} (8)	Theoretical values: Basler V_u^{th} (9)	Basler $\frac{V_u^{ex}}{V_u^{th}}$ (10)	Author V_p (11)	Author v_{cr} (12)	Author v_{u0} (13)	Author v_u (14)	Author $\frac{V_u^{ex}}{v_u V_p}$ (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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A Note on the Buckling of a Plate Girder Web due to Partial Edge Loadings

Bemerkung über das Ausbeulen hoher Blechträger unter Streckenlast

Remarques relative au voilement de poutres à âmes minces dues à des charges partielles agressants sur le bord

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1. INTRODUCTION

Professor Massonnet (1), in the excellent paper he has presented in the 'Preliminary Publication', has drawn attention to the need for further research into the buckling of a web under the action of a concentrated load applied to the compression flange. Subsequently, at the Conference, Beedle and his colleagues (2), have reported on experimental work they have conducted on this problem. This note briefly reports on a theoretical study which the present authors have made and which is reported in full in Reference (3).

2. THEORETICAL RESULTS

Relatively little research has been conducted into the behaviour of the buckling of the webs of plate girders when subjected to in-plane concentrated loads applied to an edge, the notable exceptions being the research of Zetlin (3) and White and Cottingham (4). However, both of these studies only involved the buckling of an isolated plate, i.e., the interaction between flange and web was not considered.

The writers have employed a finite element method of solution which is ideally suited to deal with such problems. Present space does not permit a presentation of the theoretical solutions which are given in Reports (5, 6).

Figure 1 gives details of the problem considered. The applied load, which is symmetrically distributed about the central line of the panel, is supported along the vertical edges of the panel by uniform shear stresses as shown. The vertical edges are assumed to be simply supported, that is, there is no out-of-plane deflection along their lengths. It was assumed, however, that these vertical edges can rotate about the neutral axis of the section in a manner similar to that which occurs in an actual plate girder.

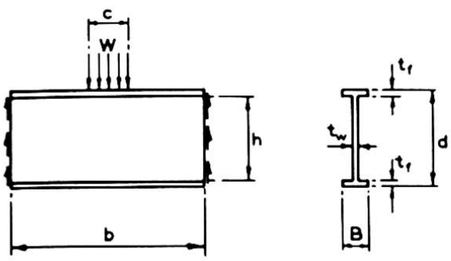

Figure 1.

The theoretical study has shown that the relationship between the applied load P_{cr}, which will cause the plate to buckle and the physical and material properties of the plate, is given by Equation (1), in which K' is a non-dimensional coefficient.

$$P_{cr} / bt = K' \pi^2 D / d^2 t \quad \ldots \ldots \quad (1)$$

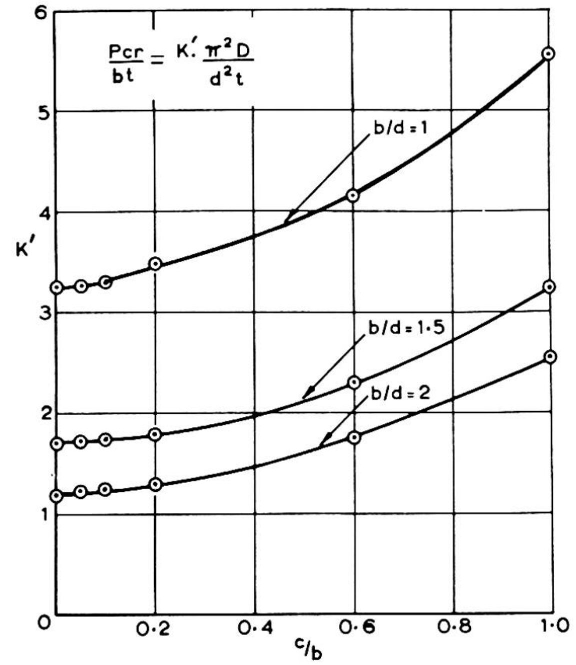

FIGURE 2
VARIATION OF BUCKLING COEFFICIENT K' WITH c/b RATIO FOR A PLATE

Where b = width of plate
d = depth of plate
t = thickness of web plate
$D = Et^3/12(1-u^2)$
E = Young's Modulus of plate
μ = Poisson's Ratio

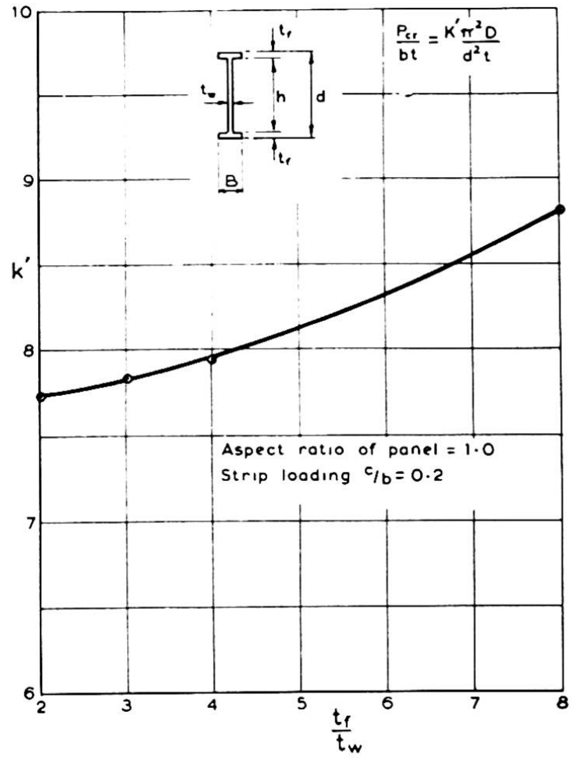

FIGURE 3
RELATIONSHIP BETWEEN K' AND RATIO OF FLANGE THICKNESS TO WEB THICKNESS

Figure 2 shows how, for a plate which is assumed to be supported against deflection along all edges, K' varies with the loading parameter c/b.

From this diagram it will be seen that as the length of application of the load is increased so a larger load is needed to buckle the web plate. In a practical girder the flange members will assist in distributing the load and will also provide additional restraint to the web plate. Figure 3 shows how for a square panel, K' varies with the ratio of flange thickness/web plate thickness. It will be noted that with a flange plate of only twice the web thickness, the value of K' is 2.3 times as great as the corresponding value given in Figure 2 for an isolated plate. The influence of the flanges upon the buckling stress is therefore seen to be most significant. In forthcoming reports the authors will be providing curves giving the relationships between buckling coefficient K', the ratios c/b and b/d where the web plate is subjected to the combined action of bending, shear and a concentrated load applied to the upper flange.

3. CONCLUSION

This note deals with the buckling of the web plate of a plate girder when it is subjected to a concentrated vertical load applied to the flanges and shows how this critical load varies with the physical properties of the flange members.

4. ACKNOWLEDGEMENT

This paper is based on research work conducted for the Fabricated Products Division, British Steel Corporation, Gorseinon, Swansea, to whom the authors wish to make grateful acknowledgement.

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IIc

Remarques de l'auteur du rapport introductif
Bemerkungen des Verfassers des Einführungsberichtes
Comments by the author of the introductory report

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Les sept mémoires présentés pour la Discussion Préparée ont certainement contribué à une meilleure compréhension du problème. Deux d'entre eux, celui de Bergfelt et Hövik et celui de Okumura et Nishino, soulignent la forte tendance à éviter les raidisseurs transversaux, au moins pour les poutres soudées de taille moyenne employées dans les bâtiments. Le but de cette suppression est de faciliter la production en série par soudage automatique et ainsi de réduire le prix unitaire de l'acier mis en oeuvre. Les deux mémoires cités montrent des possibilités intéressantes à ce point de vue, mais on n'oserait affirmer qu'ils fournissent une méthode complète de dimensionnement pour ce type de poutre.

Les deux mémoires signés par Rockey - Skaloud et par Owen, Rockey et Skaloud apportent des informations de valeur concernant l'effet de la rigidité du cadre entourant un panneau d'âme dans le cas de la sollicitation par cisaillement, ainsi que concernant l'effet de la rigidité d'un ou de deux raidisseurs longitudinaux dans le cas de la flexion pure. Ils contribuent à la démonstration du fait que la théorie linéaire du voilement est une très médiocre représentation de la réalité.
Cependant, nous sommes encore loin d'un ensemble complet de règles de dimensionnement pour les raidisseurs et, en particulier, nous ne connaissons toujours pas la philosophie de base conduisant au choix des raidisseurs réalisant le comportement optimal de la poutre complète.

J'ai déjà eu l'occasion, dans mon rapport introductif, de souligner les caractéristiques intéressantes de la méthode de l'état limite développée par Basler et Thürlimann, mais aussi de noter les limitations de cette approche, dont la principale est de ne pas s'appliquer aux poutres munies de raidisseurs longitudinaux. Un perfectionnement intéressant de cette méthode a été présenté par Fujii. Rockey et Skaloud, de leur côté, ont montré que la rigidité de la semelle exerce une profonde influence sur la forme du champ incomplet de tension diagonale qui se développe dans l'âme dans le stade de ruine et par conséquent sur la résistance ultime de la poutre. Je considère leur mémoire comme une importante contribution à une meilleure compréhension de la mécanique des panneaux d'âme.

Au nom de MM. Ostapenko, Yen et Beedle, le professeur Beedle nous a présenté au Congrès une vue d'ensemble des importantes recherches expérimentales exécutées par son groupe de l'Université Lehigh dans les domaines du chargement près d'un bord, des poutres dissymétriques et de la fatigue des poutres à parois minces. Il nous a donné quelques indications générales sur l'extension, à laquelle travaille son groupe de chercheurs, de l'approche Basler-Thürlimann au cas des poutres munies de raidisseurs longitudinaux.
Nous devons attendre une publication détaillée sur ce sujet avant de pouvoir nous faire une opinion.

Finalement, nous attendons aussi avec un vif intérêt la publication du mémoire des professeurs Rockey et Owen intitulé
An ultimate method of design for plate girders,
qui est mentionnée dans la bibliographie du mémoire de Owen, Skaloud et Rockey comme étant en préparation.

Dans l'intervalle, je suis obligé de reconnaître que nous n'avons pas encore une réponse générale satisfaisante au thème "Poutres de grandes dimensions à âme mince".

Au risque d'élargir un peu le problème, je voudrais remarquer que ce que le praticien désire, c'est avoir des règles de dimensionnement applicables, non seulement aux ponts classiques à poutres en double té, mais aussi aux ponts en caisson. Je ne connais rien de satisfaisant à propos de la résistance postcritique et du dimensionnement des parois horizontales raidies de ces caissons, qui sont soumises à compression, excepté que leur réserve de résistance postcritique est sûrement beaucoup moindre que celle des âmes soumises à cisaillement ou/et à flexion. De plus, deux mémoires récents [1 , 2] présentés par Nölke, soulignent le fait que, dans une poutre à âme pleine, la réserve de résistance postcritique de la semelle comprimée est très faible, parce que ses bords latéraux sont libres, de sorte qu'elle peut voiler en une surface approximativement développable.

Plus généralement, je voudrais souligner que ce dont notre profession a besoin, c'est d'une théorie unifiée, expliquant non seulement le comportement des poutres en double té ou en caisson, mais encore celui des panneaux de tôle pliée ainsi que celui des plaques soumises simultanément à une compression dans leur plan et à une pression transversale (portes d'écluse, vannes de barrage, navires, etc...). En effet, alors que le comportement postcritique des parois comprimées d'un panneau en tôle pliée appuyé sur ses deux bords latéraux est correctement régi par la formule de largeur effective du professeur Winter, nous n'avons aucune information pour le cas où l'un de ces bords est libre ou soutenu pas un raidisseur, ni pour le cas des panneaux sollicités dans leur plan à flexion pure ou composée.

Toutes ces remarques m'amènent à exprimer la conviction que les efforts de la recherche future devraient être avant tout axés vers une compréhension plus fondamentale du comportement des plaques dans le domaine postcritique.

Récemment, deux publications indépendantes ayant cette direction ont apparu à peu près simultanément. L'une est due à Capurso [3] et l'autre au soussigné [4]. Les développements mathématiques sont pratiquement les mêmes et ils donnent le moyen d'analyser le problème en généralisant au domaine élasto-plastique les équations bien connues de von Kármán pour les grandes déformations élastiques des plaques. Quelques résultats numériques intéressants ont déjà été obtenus à Naples [5] en intégrant ces nouvelles équations sur ordinateur par la méthode des différences finies. Mais on n'a envisagé jusqu'à présent que des plaques carrées isolées soumises à une pression transversale uniforme. Or, il est insuffisant de considérer un panneau isolé parce que la rigidité du cadre qui l'entoure (composé des semelles, des deux raidisseurs transversaux) et celle des raidisseurs longitudinaux éventuels ont une influence capitale; c'est pourquoi nous essayons en ce moment de développer à Liège le programme correspondant pour ordinateur, basé sur la technique des différences finies. Il serait intéressant d'exécuter des calculs semblables en utilisant la technique des éléments finis.

Quoi qu'il en soit, à l'aide de ce programme, nous voulons simuler un grand nombre de cas, en faisant varier la forme du panneau, la position des raidisseurs, et leur rigidité ainsi que celle du cadre entourant le panneau, de façon à déboucher, nous l'espérons, sur des règles pratiques de dimensionnement aussi simples que possible.

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