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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.

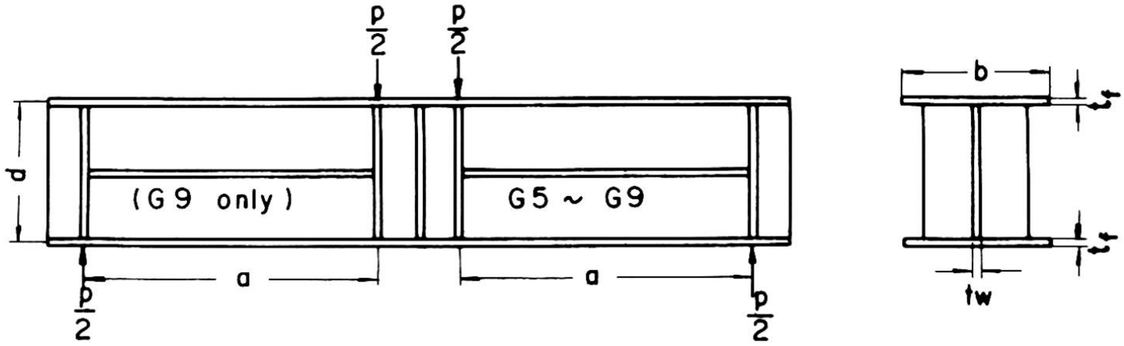
The aspect ratios, width to depth ratios of the web panels, were 2.6 for all test girders. The aspect ratio was determined from two considerations; a large ratio is prefered in order to determine the lowest possible shearing strength of a web panel, while it is prefered to have a shorter so-called shear 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



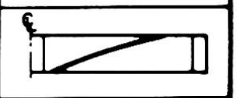
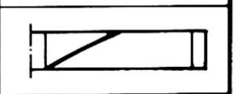
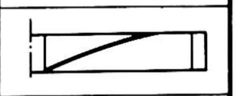
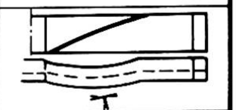
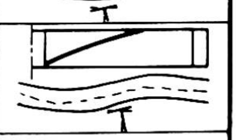
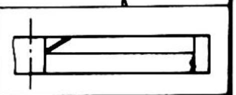
Girder	tw	d/tw	a/d	Flange mm x mm	Web mm	a	Vertical Stiffs.	Horizontal Stiffs.	Failure Mode
G1	9.1	59.6	2.69	301 x 22.4	543	1450	90.0x9.20		
G2	9.1	59.6	2.69	220x 22.4	543	1450	90.0x9.20		
G3	9.4	76.8	2.64	302x 22.2	722	1900	90.0x9.07		
G4	9.2	78.3	2.64	243x 22.1	720	1900	90.0x9.13		
G5	9.0	99.9	2.62	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.

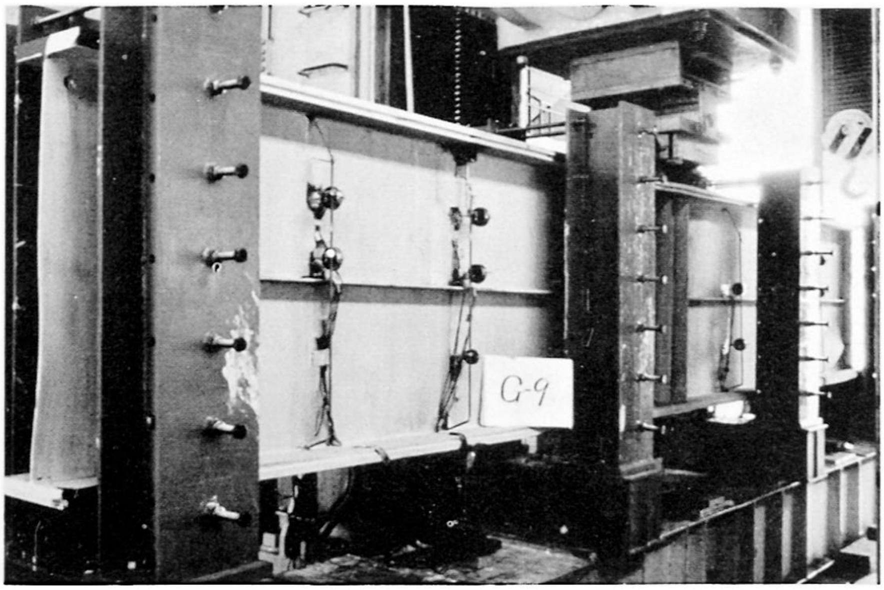


Fig. 1 Test Set-up

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

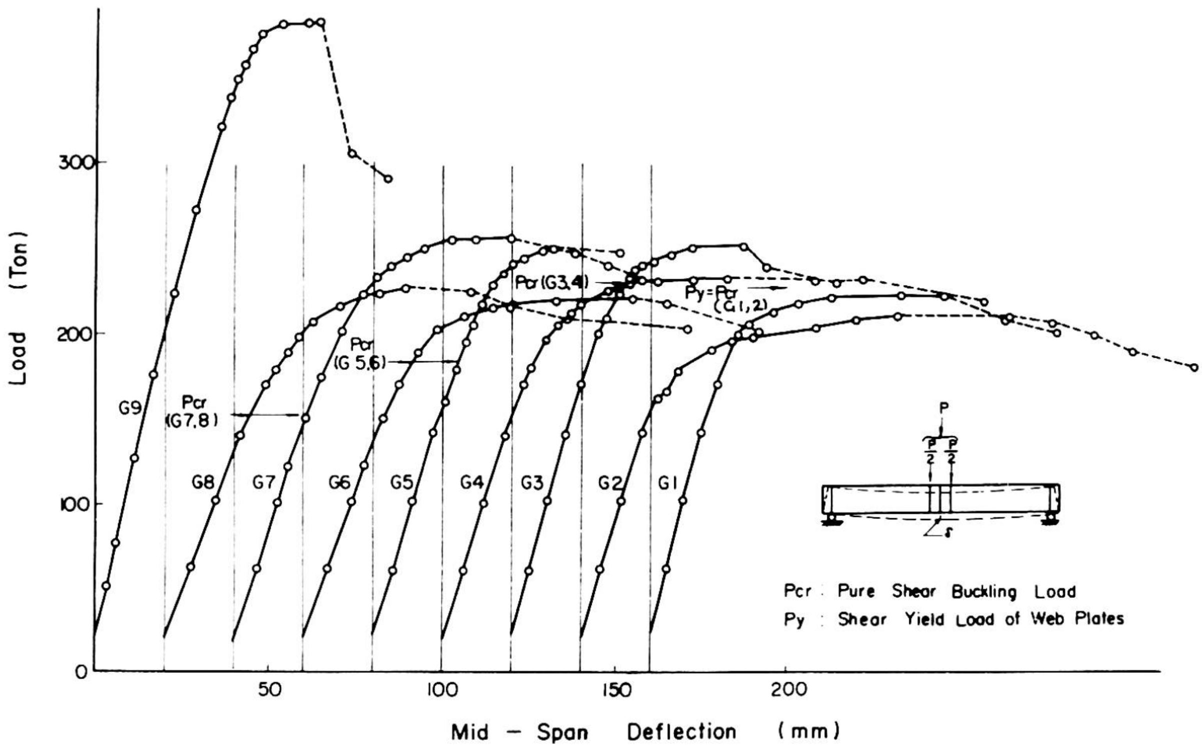


Fig. 2 Load-Deflection Relationship

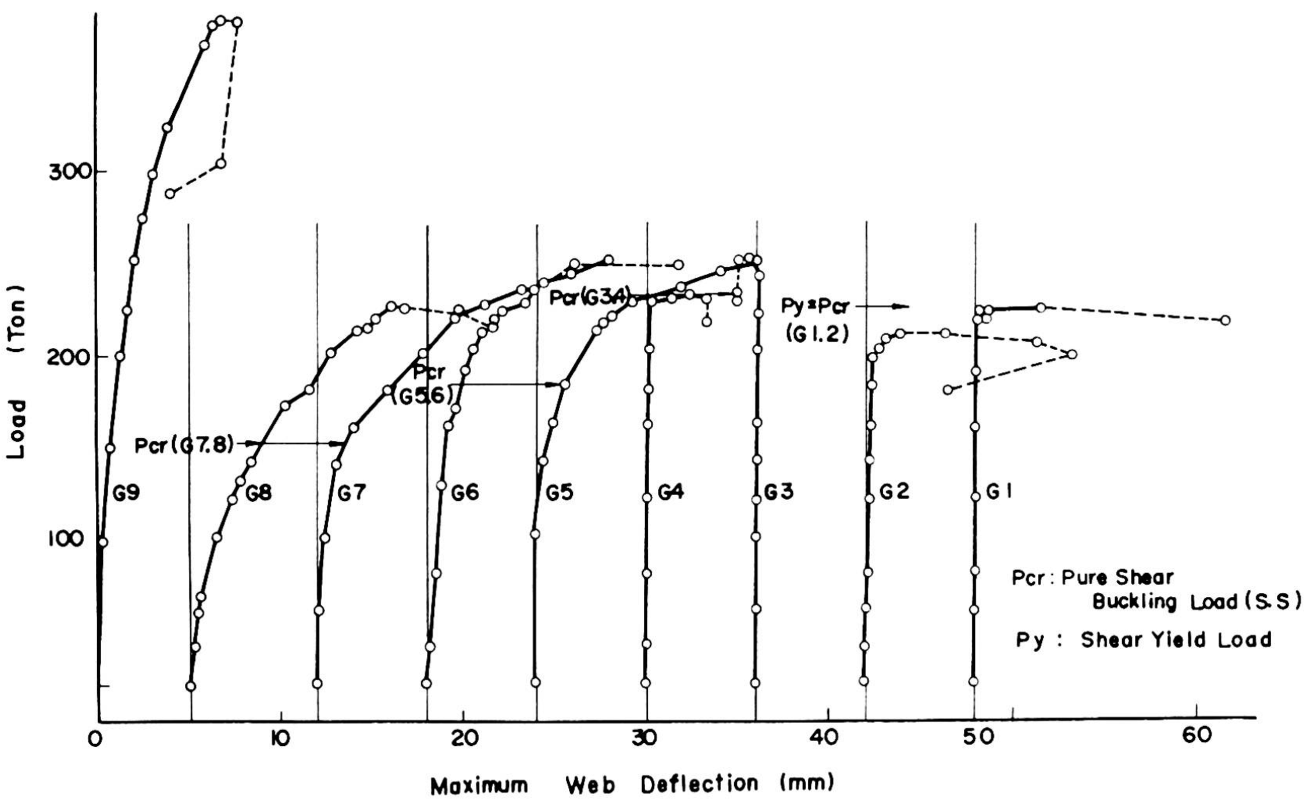


Fig. 3 Load-Web Deflection Relationship

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.

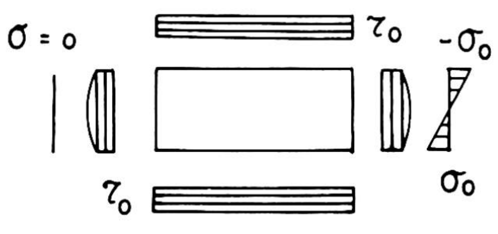


Fig. 4 Stress for Buckling Analysis

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

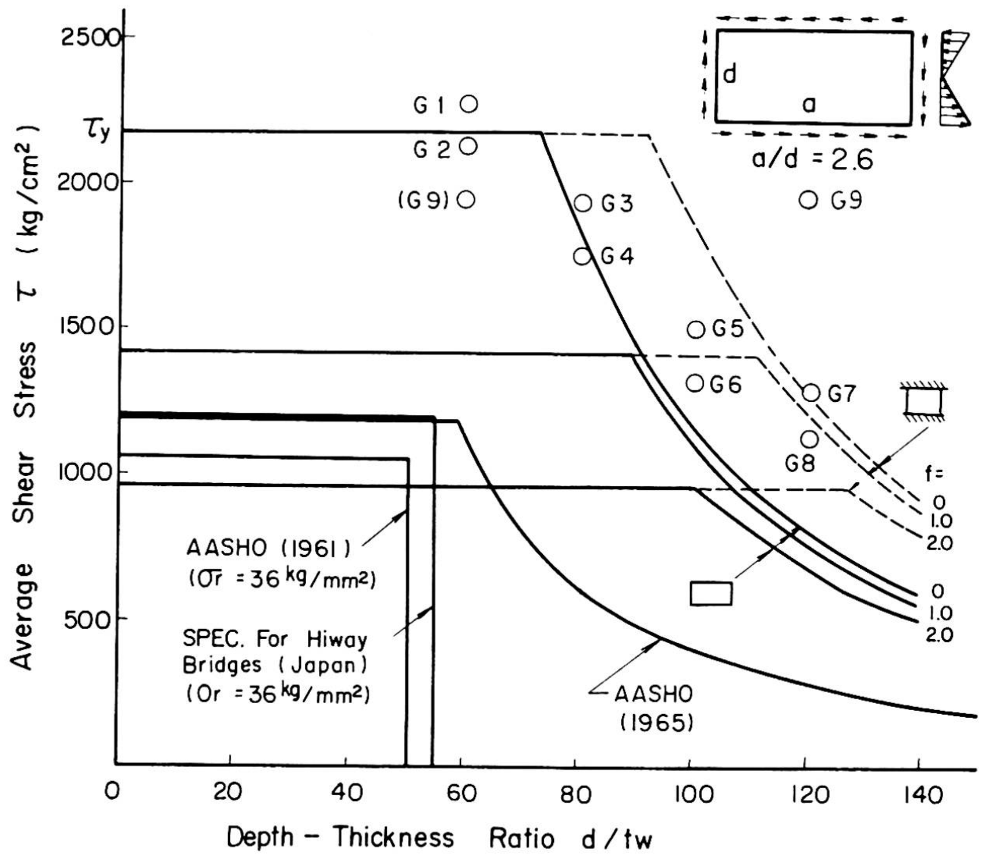


Fig. 5 Test Results and Buckling Curves

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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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.