Research on Hybrid Plate Girders

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This report presents the results of a series of tests on welded plate girders with 100,000-psi yield point constructional alloy steel flanges and ASTM A36 carbon steel web. Five tests were made on two specimens by subjecting them to moment and shear loading. The report discusses the deflection, buckling, and yielding characteristics of the specimens.

Results indicate that hybrid girders of the type tested can function satisfactorily under static load when properly designed. These girders are best suited for long spans in which the ratio of live load to total load is small. When this ratio is large, live load deflections may become excessive.

Strain gage data taken show that plane sections before bending remain plane even after the web has yielded considerably. These results and the load-deflection characteristics show that ordinary bending theory can be used in the design and analysis of hybrid beams and girders.

•IN CONTINUING efforts to build more economical bridge and building structures, engineers are exploring the use of new high-strength steels developed by the steel industry. Whereas some of these steels were developed the last 10 years, others have been in existence for 30 years. Many designers, however, have been relatively reluctant to use the high-strength steels. Undoubtedly, one of the reasons for this reluctance may be the lack of experimental information on the behavior of structural members built with such steel.

To meet this need a series of tests was performed in the Civil Engineering Laboratories of the University of Texas in which plate girders with 100,000-psi minimum yield strength constructional alloy steel flanges and carbon steel webs were used. The high-strength steel used for the flanges was Sheffield Super-Strength 100 and the carbon steel was ASTM A36 60 T. Such plate girders are referred to in this paper as hybrid with cross-sections made of two or more steels. This will distinguish them from the composite girders in which the variation in steel is along the length of the girder while throughout each cross-section the same steel is used.

Use of New Steels

Efforts by the steel industry to increase and improve the properties of steels used in construction have resulted in several grades of structural steels. In general, these steels can be separated into three major groups: (a) the carbon steels consisting of A7, A373, and A36; (b) the high-strength low-alloy steels consisting of A440, A441 and A242; and (c) the superstrength heat-treated steels.

In the carbon group, ASTM A36 steel has a minimum yield point of 36,000 psi and thus possesses a 9.1 percent higher yield point than ASTM A7 steel, long considered the common denominator of structural design. The yield point of the second group varies from 42,000 psi for thicknesses above 1½ in. to 50,000 for thicknesses of ¾ in. or less. Finally, the yield point of the quenched and tempered steels, that of the third group, is 100,000 psi. Included in the last group are such steels with proprietary names as USS T-1, USS Type T1-A, Jalloy S-100, N-A-XTRA 100, and SSS-100.

This report is focused on the third group of steels. In addition to their high yield strength and good ductility these new steels have excellent welding qualities which make

them particularly suitable for long-span plate girder construction. The erected cost of plate girders fabricated using these new steels is only 1½ to 2 times as high as the cost of girders with carbon steels. Because the web of a plate girder accounts for a large portion of its weight but contributes only a small part of its moment-carrying capacity, the use of cheaper low-strength steel for the web becomes intuitively apparent. This has been done by the California Division of Highways in some of its recent heavy bridges (1).

Another reason that makes it desirable to use carbon steel for the web of plate girders is that current design specifications permit the use of thinner webs for carbon steel girders than for higher strength steel girders when the minimum depth-to-thickness ratio governs in the design.

Objectives and Methods

The purpose of this investigation was to determine what effect yielding of the web would have on the ultimate static carrying capacity of the hybrid girder. Fatigue loading was not investigated.

In these tests two girders were fabricated with 100,000-psi yield strength constructional alloy steel flanges and A 36-60 T steel webs. Each specimen was first subjected to a static load test with load located at the third points, and then each specimen was modified and tested again with the loads applied at different points. Because of a failure in the lateral support system the second test was repeated for one specimen.

In this report the specimens are referred to as S1 and S2 indicating specimens 1 and 2. A further designation indicates the test number; thus, S1-T1 indicates test 1 on specimen 1.

Girder S1 had a 24- by ¼-in. web with no intermediate stiffeners. For test S1-T2 transverse stiffeners were added in one end panel and bearing stiffeners added at the new load points. Girder S2 had a 24- by 3/16-in. web with pairs of transverse stiffeners at 12 ¾-in. centers in each end panel. For the second test on S2, additional bearing stiffeners were added at the new load points which were nearer the center of the span.

In tests S1-T1 and S2-T1 the load was applied at the third points. These tests were terminated before the girders were severely damaged so that the specimens could be modified for tests S1-T2 and S2-T2. In tests S1-T2 and S2-T3 the loads were located as shown in Figure 1.

Resumé of Experimental Results

Except for test S2-T2, the results of the tests were as anticipated. S1-T1 was terminated at a load level of 203 kips. At this load it had deflected 2.833 in. The top flange had deflected about ⅝ in. laterally and a slight buckle was evident in the web in one end panel. Test S1-T2 was ended when the web of the specimen buckled in the unstiffened end panel under a load of 260 kips. In test S2-T1, the web was almost completely yielded in the end panels, when the test was ended with a load of 190 kips. Test S2-T2 ended unexpectedly when the top flange buckled laterally because of failure in one lateral support. Failure occurred with the load between 150 and 160 kips. Because it was impossible to know what effect the support failure had on the failure of the girder, this test was repeated as S2-T3 after repairs were made on the specimen. This time the support did not fail; however, the girder failed in the same manner and at the same load as in the first test.

TEST PROGRAM INSTRUMENTATION AND PROCEDURE

Two specimens were ordered for this investigation, shown in Figure 1. All girders were tested over a span length of 25 ft 6 in.

Girder S1 was fabricated without stiffeners except for bearing stiffeners at the load points (third points). After the first test was completed, the girder was modified by adding bearing stiffeners at points 2 ft nearer the ends. Intermediate stiffeners were also added in one end panel as shown.

Girder S2 was fabricated with bearing stiffeners at the third points and 2- by 3/16-in.

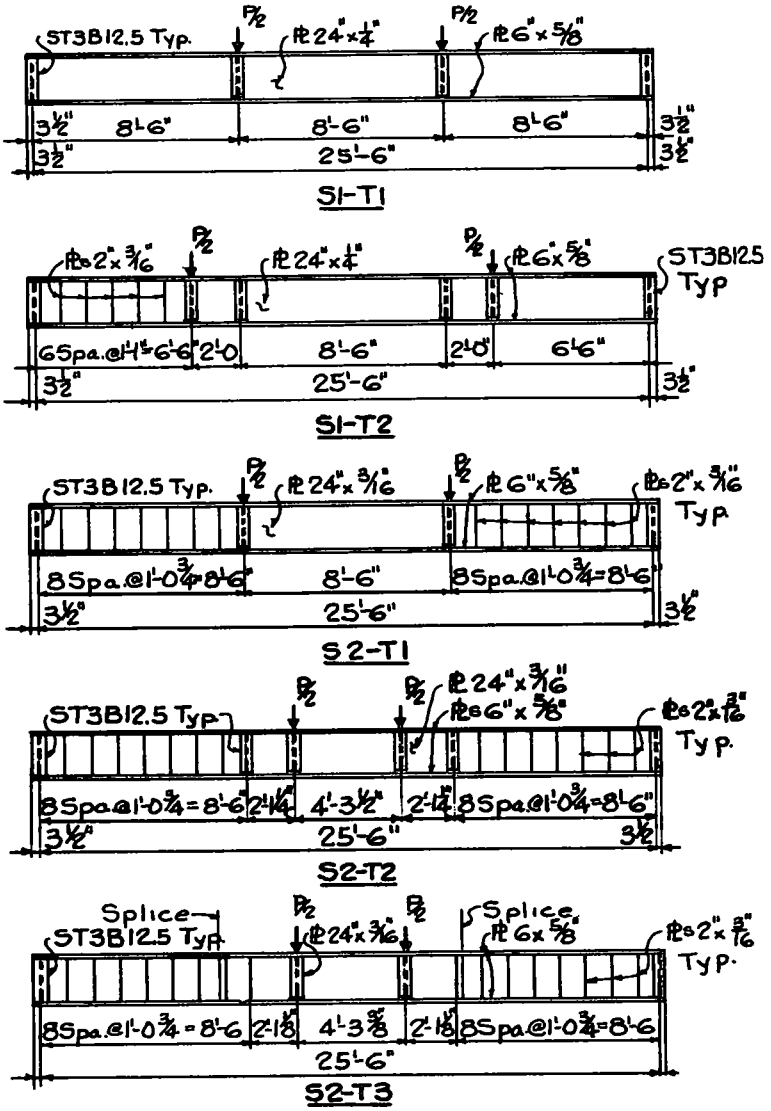


Figure 1. Test specimens.

intermediate stiffeners in the end panels. On completion of this test, new bearing stiffeners were added at points 10 ft 7 ¼ in. from the end bearings. When this specimen failed due to the failure of one of the lateral supports, it was repaired by cutting out a section 10 ft 2 in. long from the center portion and replacing it with a new piece.

The basic test setup is shown in Figure 2. Lateral support for the specimens was supplied as shown in this figure.

Three sets of data were obtained from the specimen. Strains were measured with SR-4 gages. Vertical deflections were measured with Ames dials located under the specimen. Local buckling in the web was measured with Ames dials. All specimens were whitewashed to make the observation of yield line development easier.

The steel used in the flanges of both girders was "SSS-100" produced by the Sheffield Division of Armco Steel Corporation. Tensile coupons for determining the yield point were prepared from plate samples furnished by the fabricator. Results of tests on the

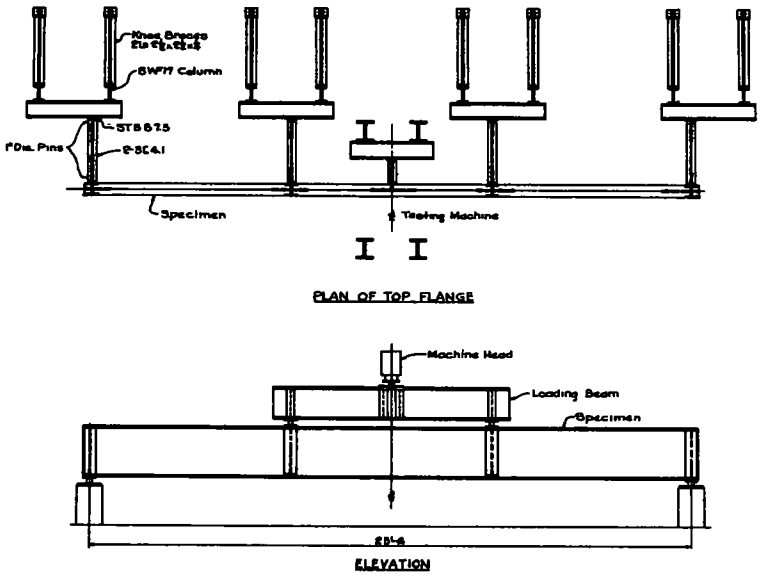


Figure 2. Test setup.

flange coupons revealed an average static yield point strength of 109,800 psi with 28 to 32 percent elongation over a 2-in. gage length. Tests of coupons cut from the A 36 steel web samples showed a static yield strength of 43,300 psi for the ¼-in. web and 36,950 psi for the 3/16-in. web. Results of all coupon tests are given in Table 1. Mill test data for the SSS-100 is also included in this table.

Reference loads and moments were calculated in accordance with the plastic design theory and the plate girder design method presented by Basler (2). Table 2 gives the reference moments, loads, and deflections.

TEST RESULTS AND OBSERVATIONS

Test results are discussed separately for each specimen and test. Results of strain gage measurements are presented at the end of this section.

Test S1-T1

In reference to the load deflection curve for this test (Fig. 3), the deflection varies only slightly from a straight line throughout the load range. No deviation is noticeable until the stress in the extreme fiber of the web, σ_{yw}, as calculated by the $\frac{Mc}{I}$ formula, is well above the yield point of the web material.

According to the theory presented by Basler (2), the web of this specimen could be expected to buckle at an ultimate shear, Vu, of 105 kips. This shear corresponds to a moment of 895 ft-kips. The plastic moment, Mp, calculated for this specimen is 1,010 ft-kips. When the test was terminated, the shear was 102.3 kips and a slight buckle had formed in one of the end panels. The compression flange had also deflected horizontally about ⅝ in. over the length of an end panel. Yield lines began to appear rather early in the test probably due to welding residual stresses; the first of these was noted at a load of 60 kips.

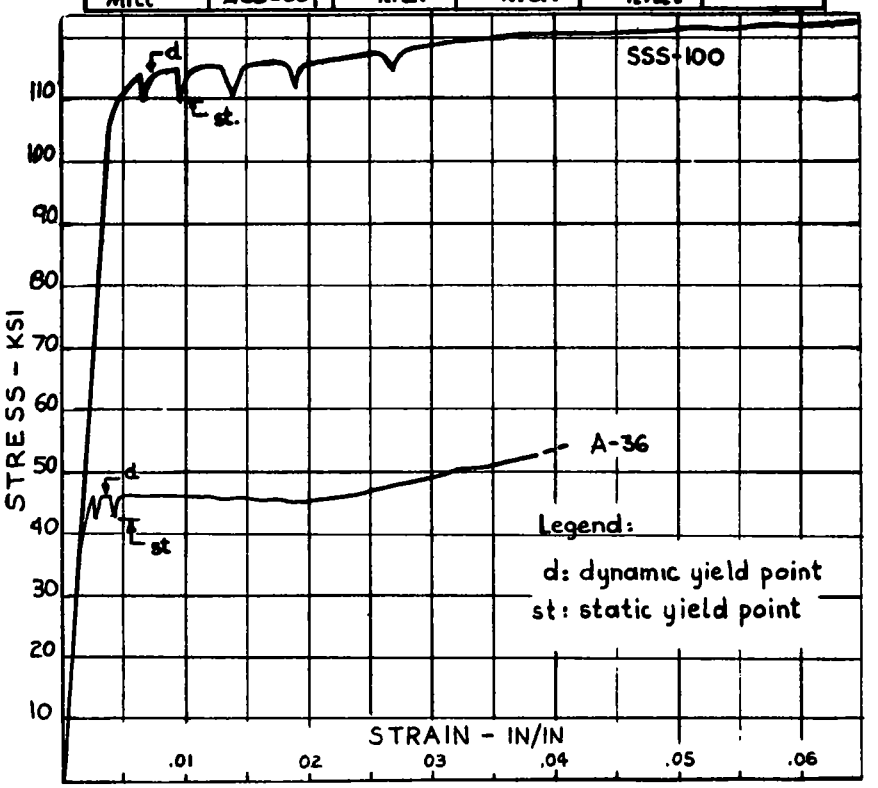
Test S1-T2

Computations indicated an ultimate shear force of 109.9 kips would cause failure in this specimen. This shear force was exceeded by about 20 kips; however, a slight buckling of the web started developing when the calculated shear force was reached.

TABLE 1

SUMMARY OF EXPERIMENTAL YIELD POINT STRESSES

SPECIMEN	TYPE STEEL	STATIC Y. P. ksi	DYNAMIC Y. P. ksi	ELONGATION %	ELONGATION G.L.
C1	SSS-100	109.6	113.3	32.0	2-in.
C2		-	113.1	29.0	
C3		-	115.8	29.0	
C4		-	114.7	31.0	
C6		-	111.5	31.0	
C7		-	114.7	30.5	
C9		109.7	115.0	28.0	
C10 1/4 in.	A36-60T	43.3	46.5	38.0	
C11 3/16		36.9	41.0	36.0	
Mill	SSS-100	110.9	-	23.5	8-in.
Mill	A36-60T	n.a.	n.a.	n.a.	-



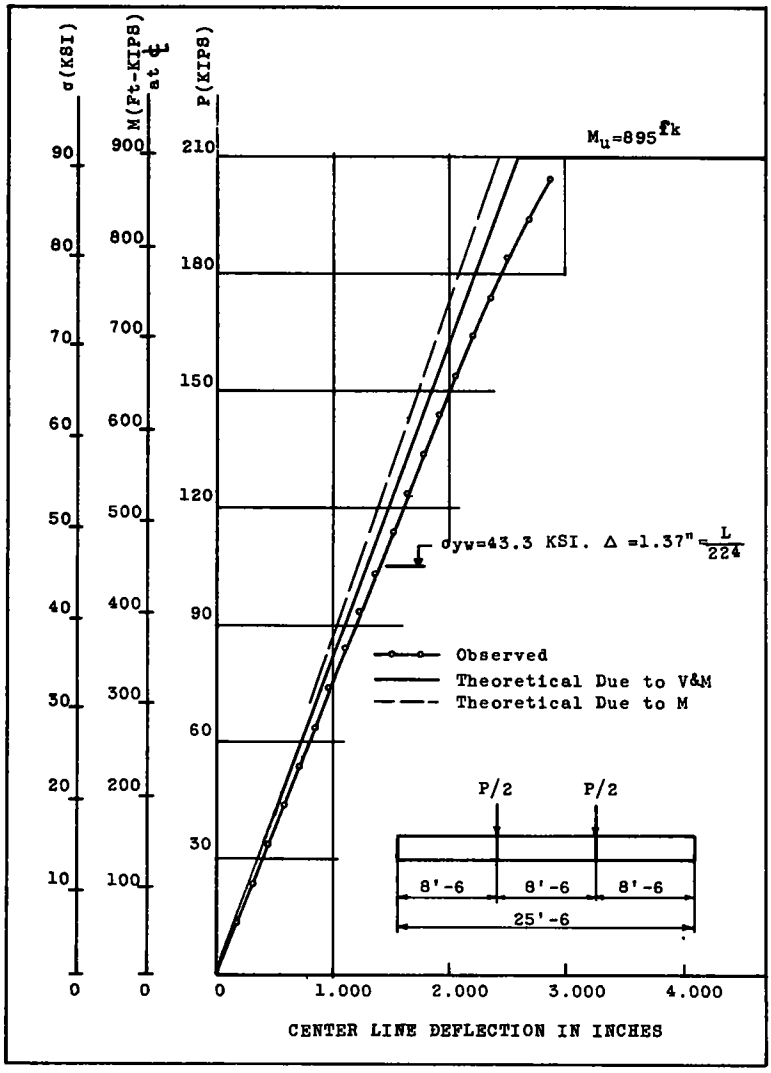


Figure 3. Load deflection curve S1 - T1.

TABLE 2

REFERENCE MOMENTS AND LOADS

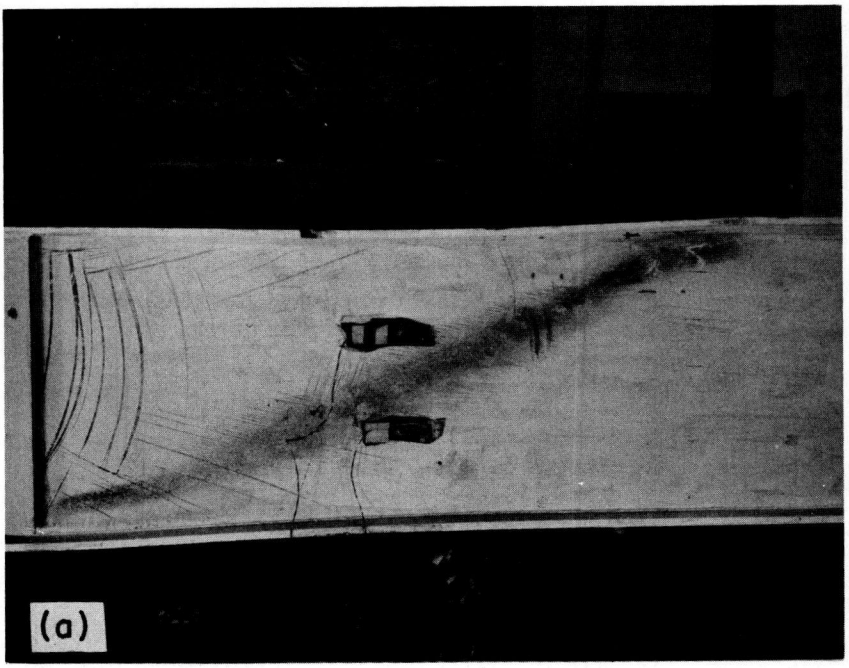
Test	M_p (ft-kips)	M_u (ft-kips)	V_p (kips)	V_u (kips)
S1-T1	1,010.2	895	148	105.1
S1-T2	1,010.2	715	148	109.9
S2-T1	963	793	95.1	93.3
S2-T2	963	885	95.1	83.5
S2-T3	963	885	95.1	83.5

This buckle continued to develop and can be seen in Figures 4a and 4b. The end of the girder shown is not the one in which a buckle occurred previously in S1-T1.

The load deflection curve (Fig. 5) for this test is similar to the curve for the first test on this specimen.

Test S2-T1

The ultimate shear force predicted for this test was 89.3 kips, resulting in a moment of 759 ft-kips. These values had been exceeded when the test was terminated with a shear of 97.3 kips.



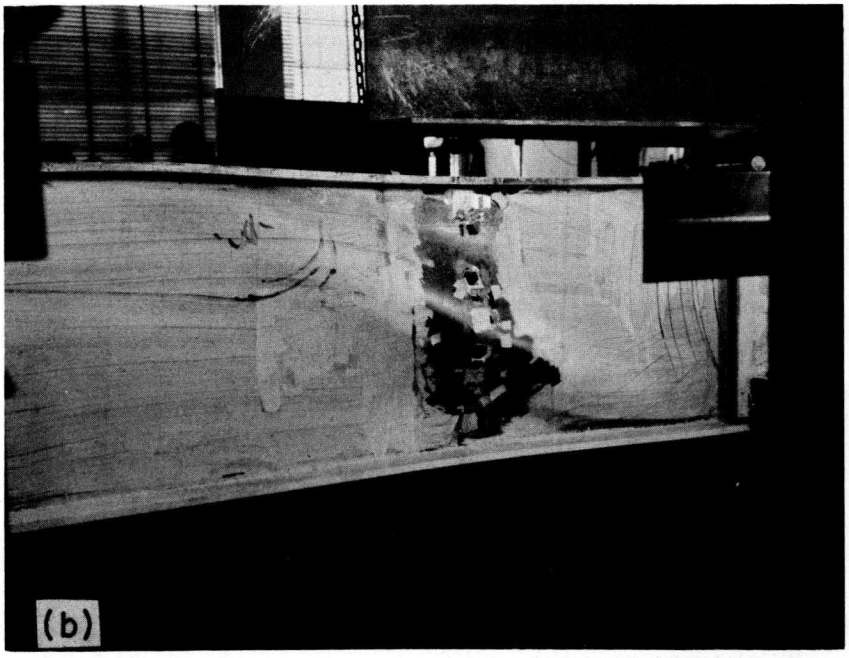


Figure 4. End panel of S1 after completion of test S1 - T2. (a) Looking from south. (b) Looking from northeast.

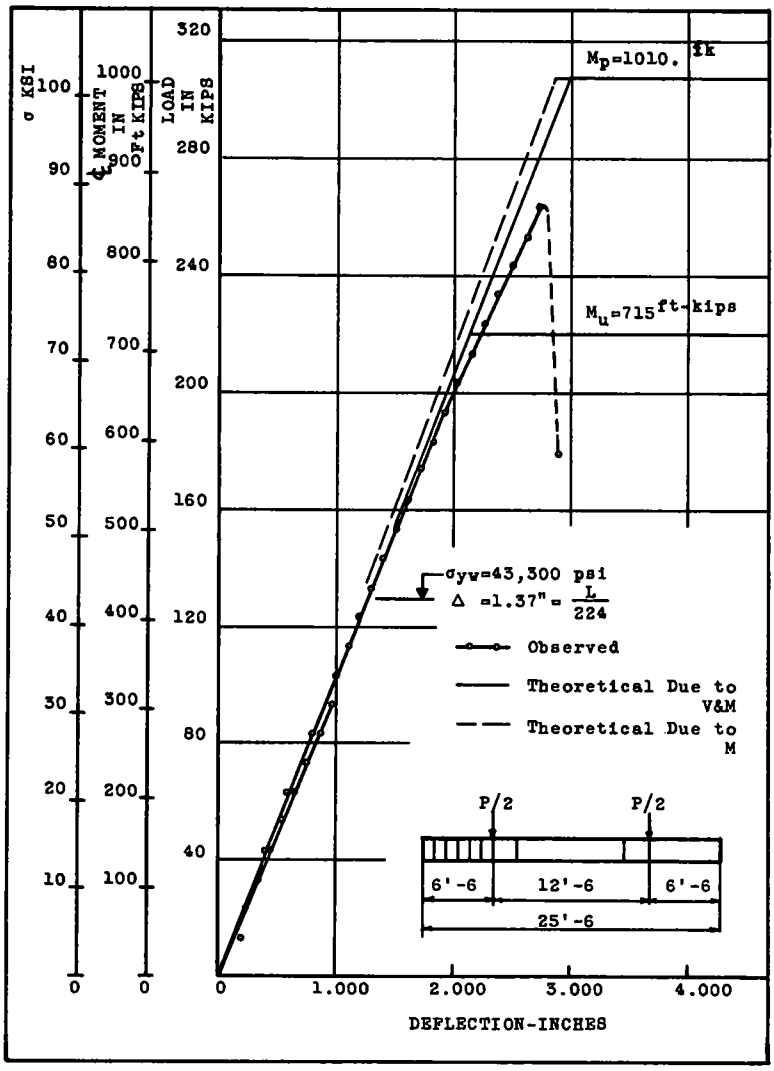


Figure 5. Load deflection curve for test S1 - T2.

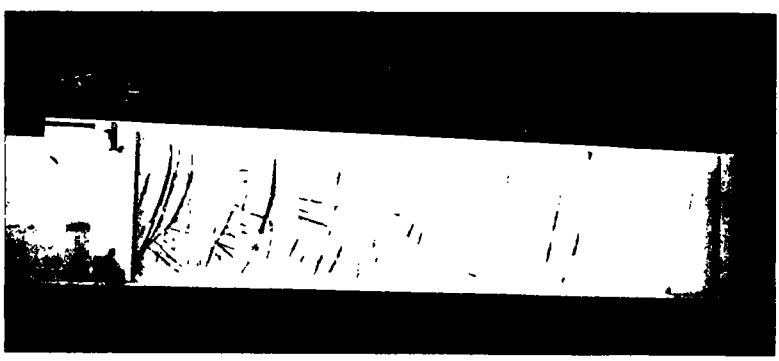


Figure 6. End panel of specimen after completion of test S2 - T1 (west end of girder, 8 panels from reaction, north side of web).

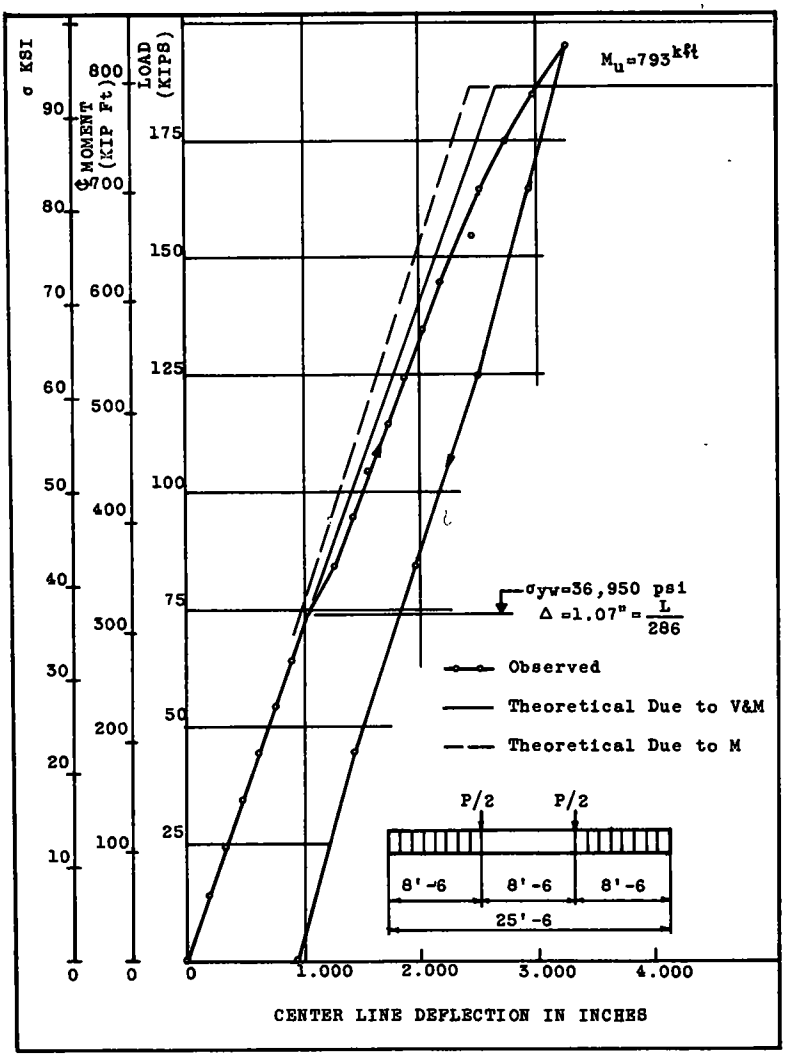


Figure 7. Load deflection curve for test S2 - T1.

The end web panels of this specimen were almost completely yielded when the test was stopped. The extent of this yielding can be seen in Figure 6. Yield lines were first evident when the load reached the 20-kip level and they were apparently due to residual stresses.

The deflection curve for this specimen (Fig. 7) shows a sudden increase in deflection as the yield point of the web material is reached. After this sudden increase a straight line relationship between load and deflection is again apparent until the load reaches the 145-kip level.

Test S2-T2

The ultimate shear force for this test was calculated to be 81.8 kips. This would correspond to an ultimate moment of 868 ft-kips. The test was completed before this value was attained. Failure occurred when the shear force was 76.5 kips and was due to the failure of a lateral brace.

The first yield lines appeared when the load was a mere 10 kips. These lines

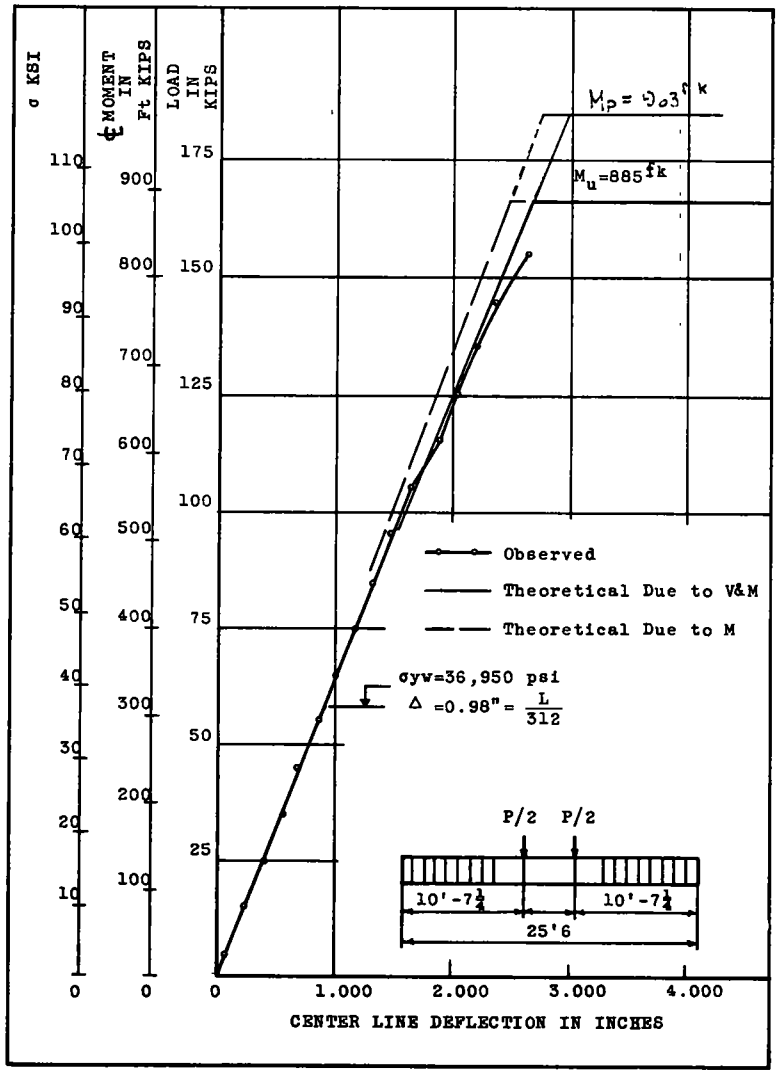


Figure 8. Load deflection curve for test S2 - T2.

appeared at the bottom of the new bearing stiffeners and were due to residual stresses from welding. A general pattern of yield lines had developed by the time the load reached 30 kips.

The load deflection curve for this test (Fig. 8) indicates elastic behavior far above the yield point stress in the web. This was due to strain hardening of the web in test S2-T1.

Test S2-T3

Test S2-T3 was a repetition of test S2-T2. However, behavior of the specimen could be expected to be somewhat different from that in S2-T2, inasmuch as part of the center portion of the web had not been stressed before, and the end portion had been yielded in test S2-T1 and S2-T2. Another factor that influenced the results of this test was that the web had some initial curvature in the panels where the splices were made. This initial curvature caused some concern when the early development of yield lines adjacent to the splice indicated that buckling might occur; however, buckling failed to

Figure 9. Buckle in panel adjacent to load in test S2 - T3.

develop at this location. Figure 9 shows these yield lines and the buckling condition in the panel adjacent to the load point. Concurrently with the buckling of this panel the compression flange of the girder buckled horizontally into an S-curve under the load of 150 kips. The permanent lateral deflection due to the lateral buckling of the flange can be seen in Figure 10.

The deflection curve for this test (Fig. 11) did not agree as closely with the theoretical curve as that of test S2-T2. A slight change in slope is again evident at the point where the extreme fiber stress of the web reached the yield point of 36,950 psi. When the load was removed and then reapplied, this point moved up the curve to approximately the level at which a change in slope occurred in test S2-T2.

Strain Gage Measurements

SR-4 electrical strain gages were used to measure strains at various load levels and compared to theoretical strains derived using the ordinary flexure formula $\sigma = My/I$ and a modulus of elasticity of $E = 30,000,000$ psi. Uniaxial gages were applied at the midspan of each girder to measure strains for pure moment condition and at one moment-shear panel, 3 ft 9 ½ in. from the end reaction for both specimens.

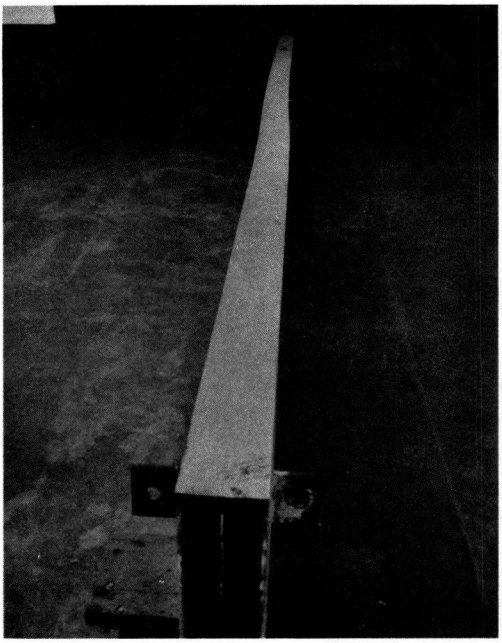


Figure 10. Girder at completion of test showing permanent set in flange due to lateral buckling.

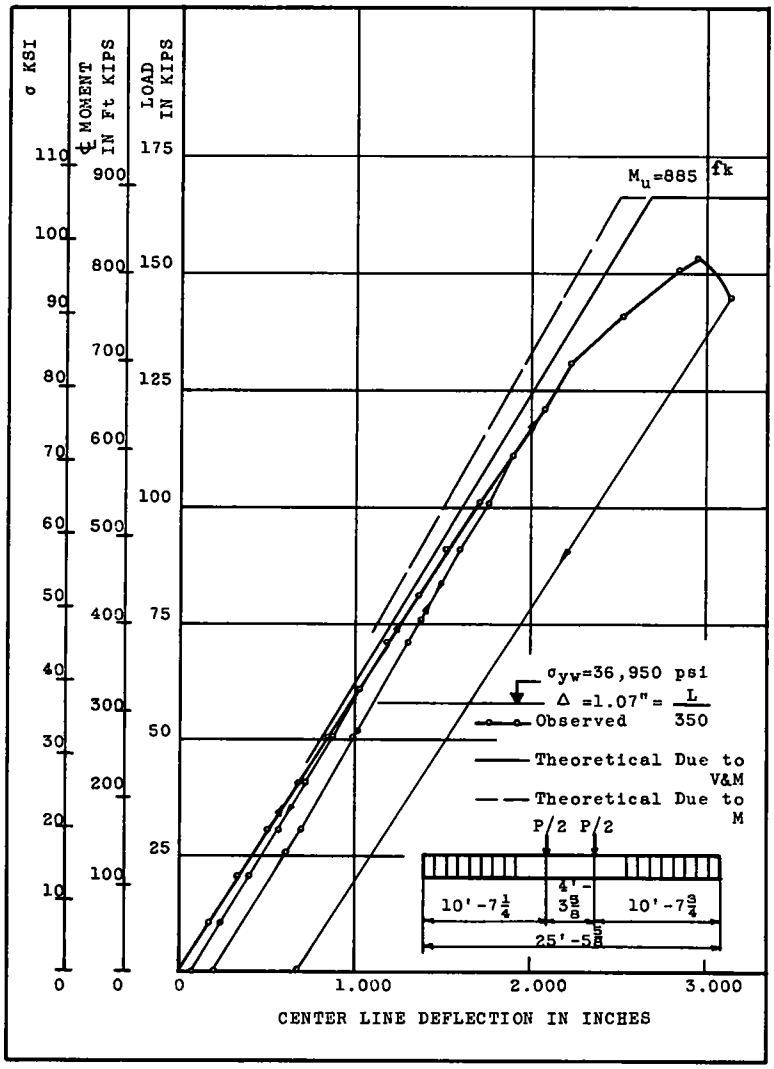


Figure 11. Load deflection curve for test S2 - T3.

Figures 12a and 12b show the measured strains for specimen S1-T1. Figures 13a and 13b show the same for S1-T2. Measured and theoretical strains for specimens S2-T1, and S2-T2 are shown in Figures 14a and 14b and 15a and 15b, respectively.

These results indicate that strains are proportional to stresses, under pure moment condition, even though the web has yielded throughout. Therefore, the assumption of plane sections before bending remaining plane after bending holds true for hybrid girders. The strain gage data indicates that the flanges of the specimens never reached their yield point strain $\Sigma_y = 109.6/30,000 = 0.00365$ in. per in. This was also verified by visual inspection for yield lines in the flanges, as it has been discussed elsewhere.

SUMMARY OF TEST RESULTS

Yield lines were evident in the web at an early load level in all tests. However, no yield lines caused by bending stress were observed in the flanges during any of the tests. The first yield lines always appeared adjacent to the load points, where residual stresses from welding of the bearing stiffeners may be expected to be concentrated.

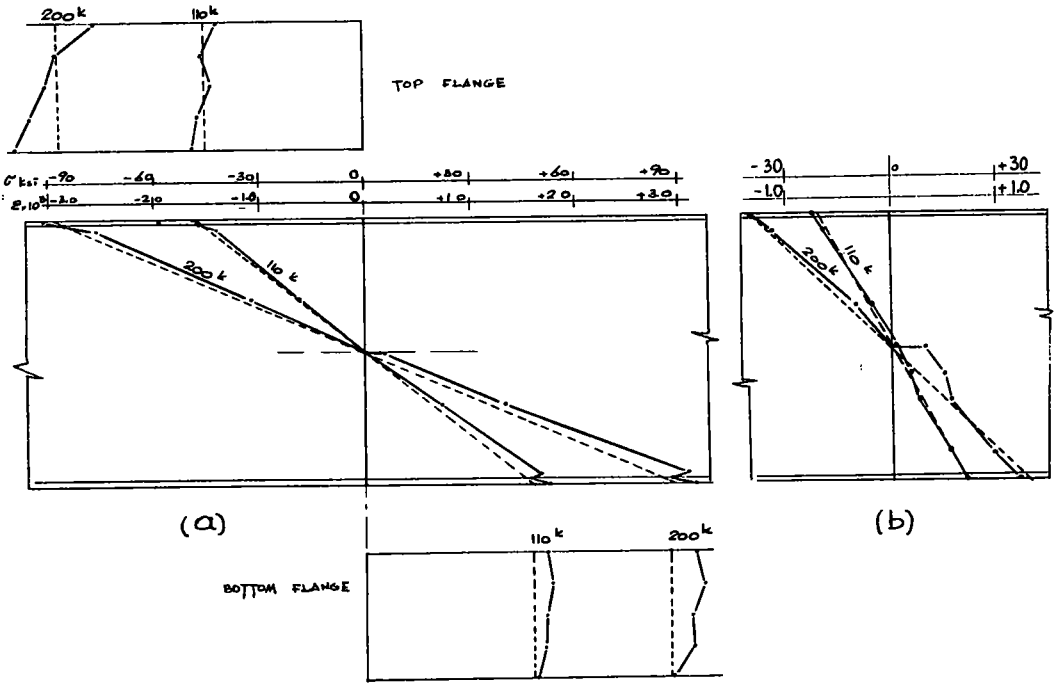


Figure 12. Comparison of measured and calculated strains (specimen S1 - T1). (a) Pure moment section. (b) Moment - shear section.

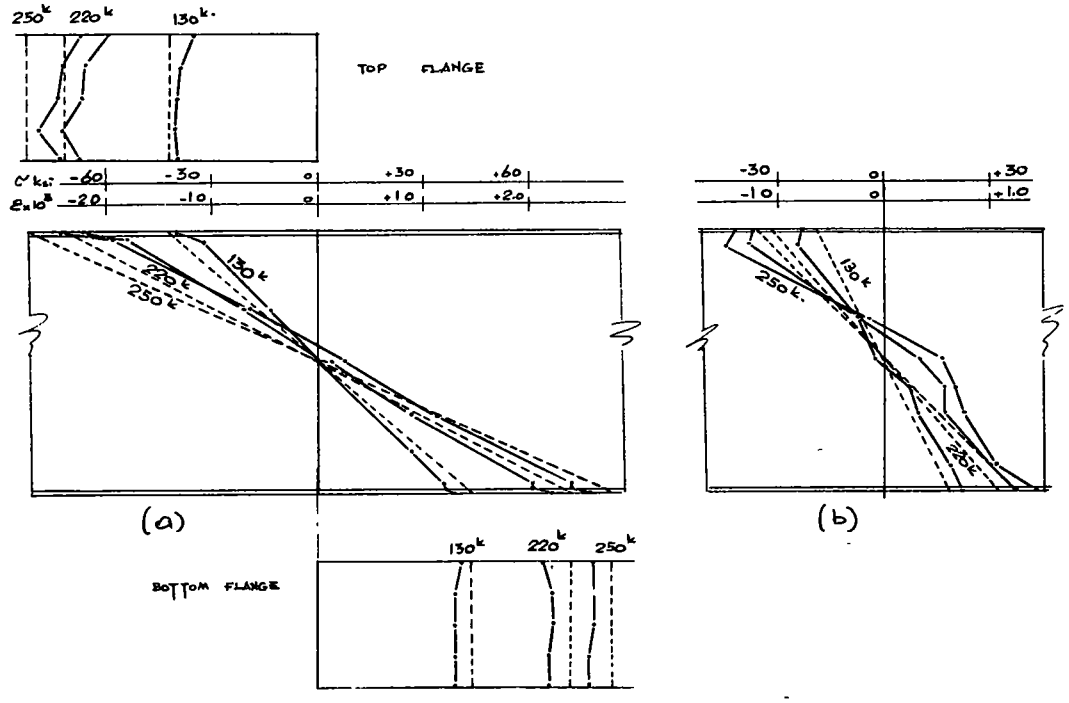


Figure 13. Comparison of measured and calculated strains (specimen S1 - T2). (a) Pure moment section. (b) Moment - shear section.

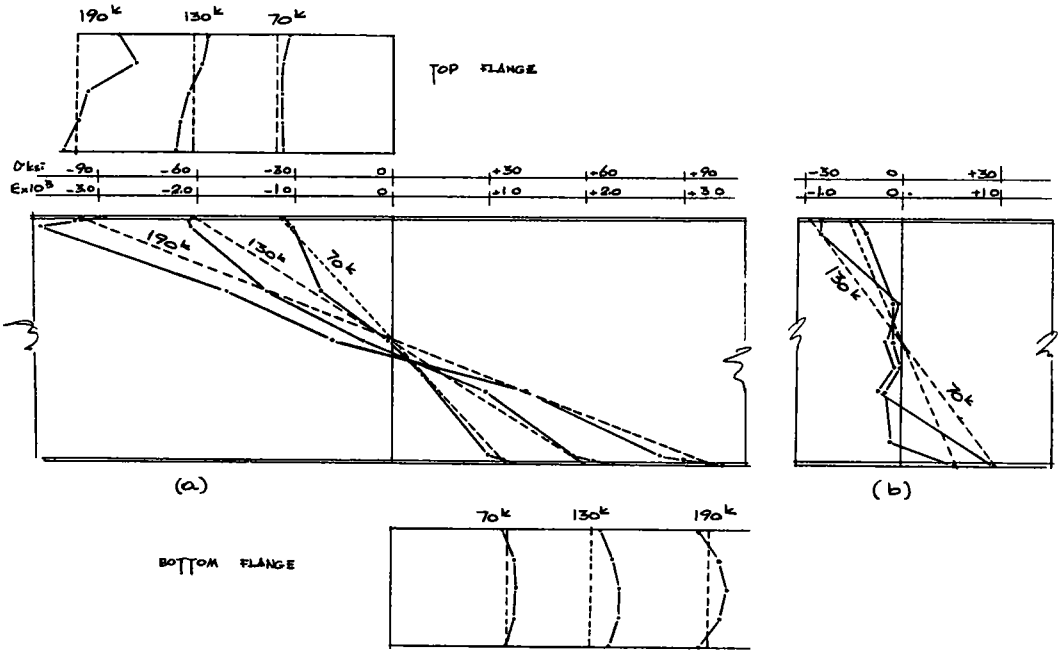


Figure 14. Comparison of measured and calculated strains (specimen S2 - T1). (a) Pure moment section. (b) Moment - shear section.

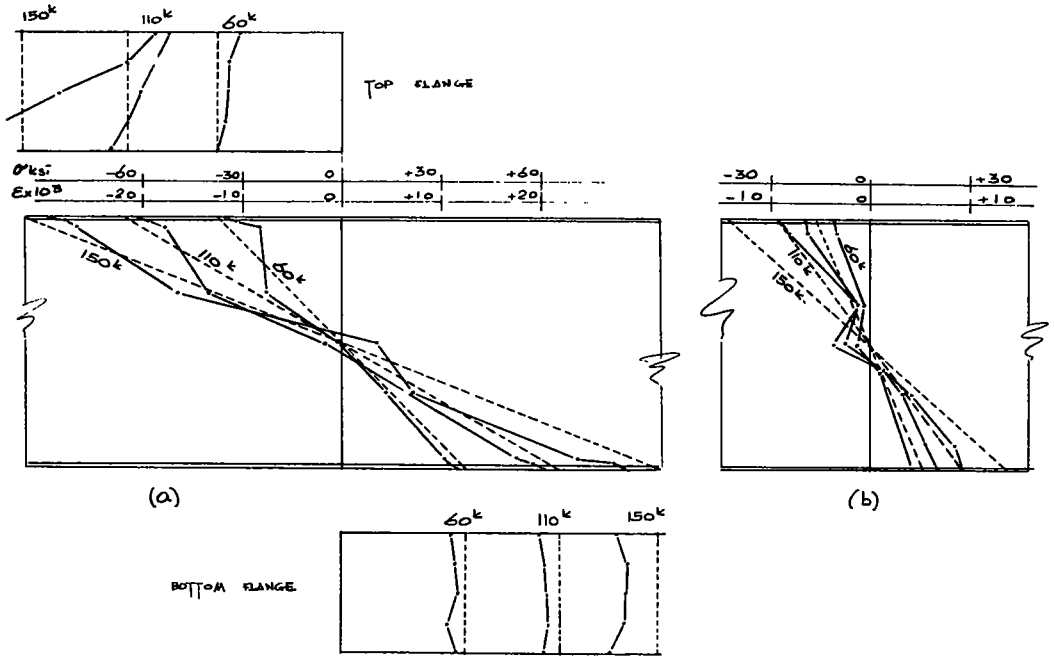


Figure 15. Comparison of measured and calculated strains (specimen S2 - T2). (a) Pure moment section. (b) Moment - shear section.

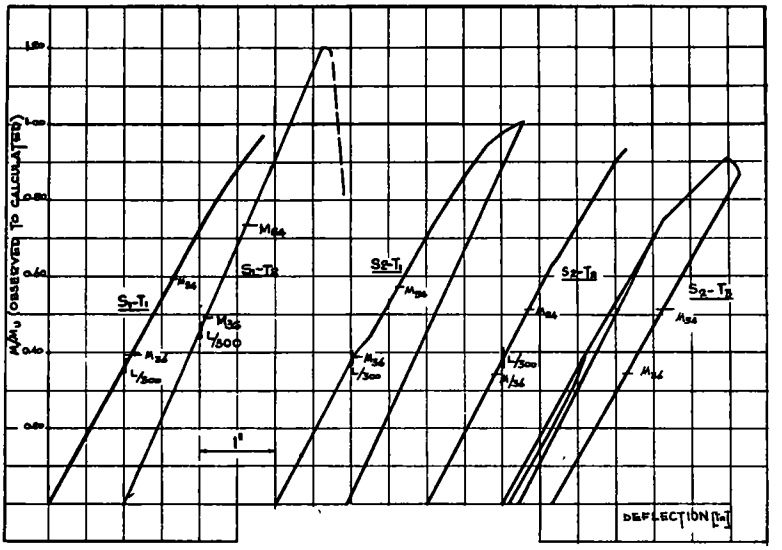


Figure 16. M - Δ curves.

This yielding did not seem to influence the behavior of the specimens because no changes in deflection are evident in any load-deflection curves until after the yield point of the web material is exceeded.

Figure 16 summarizes all the moment-deflection curves and Table 3 gives load factors based on the M_u observed M_{max} and the moments at 36 ksi and 54 ksi extreme fiber stress. The load factors indicate that 36 ksi is a more feasible allowable stress at the present. Examination of the load-deflection curve for test S2-T1 shows an abrupt change as the yield point of the web is reached (see also Fig. 7). This test was also marked by extensive yielding in the end panels.

The load deflection curves for all tests show that behavior of the specimens was very nearly elastic throughout the entire load range. Perhaps the best demonstration of the behavior of a girder of this type can be seen in the curves for tests S2-T1S2 T2S2-T3. These curves show that as yielding takes place in the web a redistribution of stress occurs that permits the member to continue to deform elastically. When the load is removed, residual stresses are induced in the web as the flanges attempt to return to their original shape. When load is reapplied to the member, adjustment of the deflection curve occurs until the residual stresses are overcome and the yield stress of the web is again reached. Such readjustment of the stresses in a member is not detrimental to its behavior; in fact, it is necessary in numerous structural members.

The important fact that was brought out by these tests was that the member continued to act elastically even after a considerable portion of the web had been yielded. A sudden readjustment in deflection may take place as the yield point of the web is reached. This occurred during the testing of specimen S2-T1 and S2-T2, but after that the beam continued upward in a straight line. This would seem to indicate continued elastic behavior in spite of the extensive yielding in the web and shows the dominating effect of the flanges on over-all structural behavior.

TABLE 3

LOAD FACTORS

Test	$\frac{M_u}{M_{36}}$	$\frac{M_u}{M_{54}}$	$\frac{M_{max}}{M_{36}}$	$\frac{M_{max}}{M_{54}}$
S1-T1	2.52	1.65	2.45	1.65
S1-T2	2.04	1.35	2.34	1.56
S2-T1	2.61	1.74	2.70	1.80
S2-T2	2.91	1.94	2.70	1.80
S2-T3	2.91	1.94	2.65	1.76

Deflection of a flexural member varies inversely with the moment of inertia. In an I-shaped member the flanges account for most of the moment of inertia because they are farthest from the neutral axis. When high-strength steel is used for the flanges, the designer can reduce their area to such an extent that the moment of inertia of the section is reduced to a value that is inadequate for deflection purposes. Where the dead load comprises a major portion of the total load, this reduction in cross-section can be overcome by cambering the member an amount equal to the dead load deflection; however, this cambering is not a solution in overcoming excessive live load deflection. The designer must either increase the depth or resort to composite construction.

When stresses as high as 54,000 psi are considered, all specimens in these tests had deflections much larger than the allowable live load deflection of L/800 as specified for bridges by the American Association of State Highway Officials and L/360 as specified for buildings by the American Institute of Steel Construction. This fact cannot necessarily be used as a measure of the usefulness of this type of member because almost all of the load in these tests could be considered as live load. This fact would indicate that the proper place to use this type of member is on long heavy spans where the dead-to-total-load ratio is high. Moreover, when the specimens were designed, deflection considerations were not taken into account. The level of moment at which the extreme fiber stress is 36 ksi and the deflection 1/300 of the span is indicated on all the curves in Figure 16. For the specimens tested, these two points almost coincide for all tests. However, this discussion does not imply that in actual practice this would be the case. In an actual structure designed with 36 ksi, the live load stresses will be only a fraction of this allowable stress and the deflections will be proportionately smaller.

In some of these tests the specimens tended to buckle laterally. Any saving in weight by reducing the size of the flanges can be erased by the need for additional support; however, this would be true only in the negative moment region of continuous spans if the top flange is supported by a slab.

Both the lateral buckling problem and the deflection problem encountered through the use of high-strength steel flanges can be resolved by using composite construction. The use of this method of construction not only furnishes lateral support for the compression flange but also increases the moment of inertia of the section so that it will not deflect as much. Because the stress in the compression flange of a composite section is usually quite low, this flange would not need to be composed of high-strength steel. The use of composite construction seems to offer the most possibilities for the efficient use of high-strength steel and carbon steel combinations.

CONCLUSIONS AND RECOMMENDATIONS

The results of these tests indicate the following:

1. When a girder with high strength steel flanges and carbon steel web is loaded so that the yield point of the web is exceeded, a redistribution of stress will occur and allow the girder to continue to deform elastically. When, after having the load removed, the girder is reloaded, complete elastic behavior extends over a load range almost twice the first range.
2. Girders of this type are particularly adaptable to long heavy spans where the ratio of dead-to-total-load is high and live load deflection is not critical. Also, this type of girder would be adaptable to composite construction in which the slab can be used in positive moment regions to reduce deflections.
3. Deflection and lateral buckling become more important when this type of girder is used because of the reduction in flange cross-section.
4. The fact that the web is stressed beyond its yield strength has little adverse effect on the ultimate carrying capacity of the member.
5. Stresses can be calculated to a high degree of accuracy using ordinary beam theory.
6. Deflections can be calculated by virtual work or any other similar method. When shear deflections are included theoretical deflections check the experimental results to a very good accuracy especially at low loads.

Research to determine the fatigue properties of girders of this type needs to be undertaken. Composite girders with a carbon steel compression flange and web and high-yield strength steel tension flange should be tested. These should be tested with shear connectors over the full length and with shear connectors over that portion of the length where composite action is required for bending stresses.

Hybrid girders are adaptable to construction where static loads are involved and live load deflection is of minor importance. An allowable stress of 36 ksi should insure adequate safety and satisfactory performance. To insure elastic behavior throughout the working stress range, the member could be preloaded during fabrication to redistribute the stresses.

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Appendix

COMPARATIVE STUDY OF A CARBON STEEL AND HYBRID PLATE GIRDER FOR STATIC LOADS

A design of a hybrid plate girder is presented using constructional alloy steel flanges and low carbon steel (ASTM A 36) web. The design is based on the results of the investigation presented in this report. For comparative purposes, an example of a building plate girder has been worked out using standard AISC specifications for buildings, and ASTM A 7 steel throughout. Figure 17 shows the loads and the dimensions of the simple span plate girder chosen to be designed by both methods.

Design A—Carbon Steel Girder

$V_{max} = 272^k$ and $M_{max} = 5,590$ ft K. Calculations for this girder are not presented because design procedures for this are well known and standardized. Figure 18 gives the results of this design.

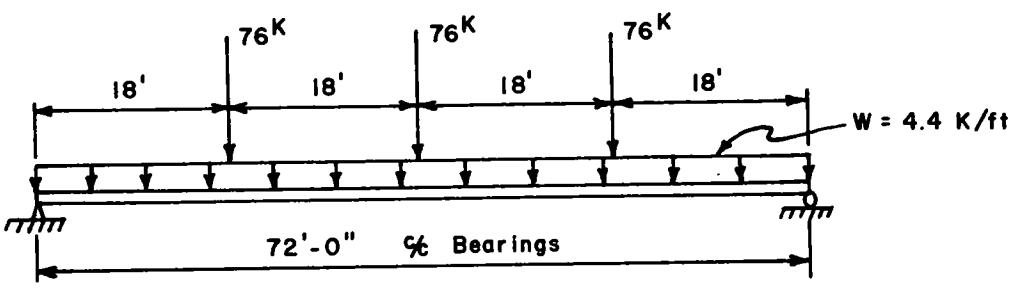
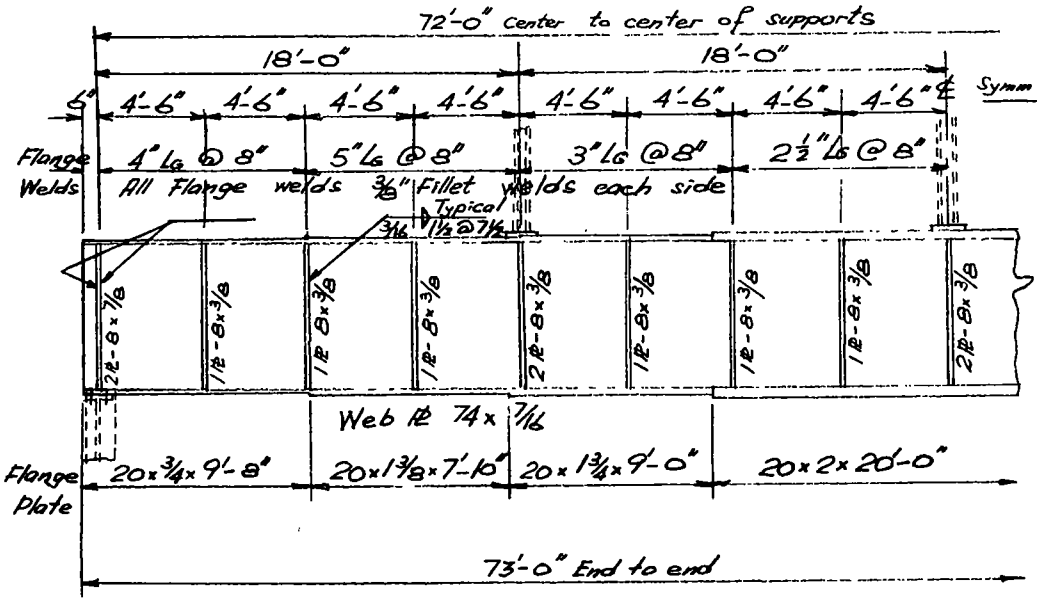


Figure 17. Girder dimensions and working loads.



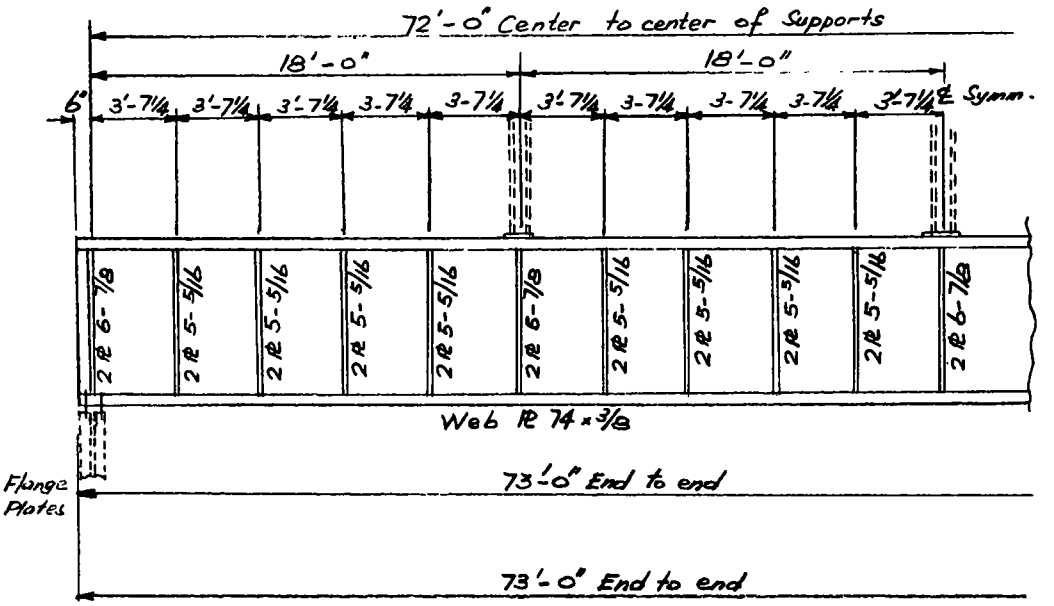
LIST OF MATERIALS		
Item	Plate Size	Weight
Flange	2 - 20" x 2" x 20'-0 4 - 20 x 1 3/4" x 9-0 4 - 20" - 1 3/8 x 7'-10 4 - 20" x 3/4 x 9'-8	5,450 4,290 2,930 1,970
Web	1 - 74" x 7/16 x 73-0	8,020
Stiff.	18 - 8 x 3/8 x 6'-2 4 - 8 x 7/8 x 6'-2	1,132 586
	Total =	24,378 #

Figure 18. Dimensions, and weights for carbon steel girder. (Design A)

TABLE 4

RELATIVE COST ANALYSIS

Design	Weight (lb)			Cost in Place Index		Relative Cost (unit)		
	Flange	Web	Stiff.	Flange	Web and Stiff.	Flange	Web and Stiff.	Total
A	14,640	8,020	1,718	1.0	1.0			24,378
B	7,450	6,900	2,185	1.6	1.0	11,900	9,085	20,985



LIST OF MATERIALS		
Item	Plate Size	Weight
Flange	2- 15" x 1" x 73'-0	11,900
Web	1- 74" x 3/8 x 73-0	6,900
Stiff.	32- 5 x 5/16 x 6'-2 10- 6 x 7/8 x 6'-2	1,065 1,120
	Total =	20,985#

Figure 19. Dimensions, and weights for hybrid girder. (Design B)

Design B—Carbon Steel Web and Constructional Alloy Steel Flanges (Limit Design Method)

Using the same dead loads and live loads just mentioned and a load factor of 1.85 according to AISC plastic design specifications,

$$\begin{array}{l} \text{max. } M = 5{,}590 \times 1.85 = 10{,}350 \text{ ft k} \\ \text{max. } V = \phantom{5{,}}272 \times 1.85 = \phantom{10{,}}503 \text{ k} \end{array} \qquad \sigma_\gamma \begin{cases} \text{flange} = 100 \text{ ksi} \\ \text{web} = 36 \text{ ksi} \end{cases}$$

Web Design (2)

Try 74 by 3/8 web (A = 27.8 sq in.)

$$\tau_\gamma = \frac{\sigma_\gamma}{\sqrt{3}} = 20.8 \text{ ksi}; \; V_p = 27.8 \times 20.8 = 579 \text{ k}$$

$$Z_w = 515 \text{ in.}^3 \text{ and } M_w = (36 \times 515)/12 = 1{,}545 \text{ ft k}$$

Intermediate Stiffeners

Try 74-in. spacing: $a = 74/74 = 1$; $K = 5.34 + 4 \times 1^2 = 9.34$

$$\tau_{cr} = \frac{k\pi^2 E t^2}{12(1-\gamma^2)b^2} = \frac{9.34 \times 9.87 \times 30 \times 10^3 \times 0.375 \times 0.375}{10.92 \times 74 \times 74} = 6.55 \text{ ksi} < \frac{272}{27.8} = 9.8 \text{ NG}$$

Try 54-in. spacing: $a = 0.73$; $k = 7.47$; $\tau_{cr} = 9.86$ NG

Try 43¼-in. spacing: $a = 0.585$; $k = 6.71$; $\tau_{cr} = 13.8$ ksi > 9.8 OK

Size of stiffeners: $A = 0.0005d^2 = 0.0005 \times 74^2 = 2.74$ sq in. Use two 5- by 5/16-in. stiffeners.

Check ultimate available shear capacity:

$$\frac{V_u}{V_p} = \frac{\tau_{cr}}{\tau_y} + \frac{\sqrt{3}}{2}\,\frac{1 - \tau_{cr}/\tau_y}{\sqrt{1 + a^2}}$$

$\tau_{cr}/\tau_y = 13.8/20.8 = 0.664$ and $V_u/V_p = 0.915$. Ultimate shear force $V_u = 0.915 \times 579 = 535$ k.

Design of Flange

Try 1-in. thick flange throughout

$$M_f = \text{max. } M - M_w = 10{,}350 - 1{,}545 = 8{,}805 \text{ ft k}$$

$$A_f = \frac{8{,}805 \times 12}{100 \times 75} = 14.1 \text{ sq in.}$$

Use 15- by 1-in. plate of constructional alloy steel.

Summary

$V_p = 579$ k, $V_u = 535$ k; $v_{cr} = 27.8 \times 13.8 = 386$ k; and $V_{des.} = 272$ k. Figure 19 gives the dimensions and materials list for design B.

Factors of Safety

Against elastic buckling = 386/272 = 1.42
Against ultimate shear = 535/272 = 1.96
Against ultimate bending = 1.85+

Cost Comparison

Table 4 gives an index of cost for the two plate girders. A relative cost is thus found by assigning carbon steel fabricated and in place the index of 1.00, whereas for constructional alloy steel the index is 1.60. Table 4 shows that use of hybrid girders results in savings. In Design B, girder costs are 86 percent of those in A. The cost reduction could have been larger than 14 percent if thinner plates were used at low moments instead of using 1-in. thick flange throughout.