Tension Butt Joints with Bolts and Welds in Combination

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Combination joints, also known as load-sharing joints, are connections that use a combination of high-strength bolts and structural welds. Two different types of combination joints are used occasionally in the fabrication of steel structures: shear splices where the bolts and welds share the load on a common shear plane, and connections where the bolts and welds are used on shear planes that are not common. The former type of connection is used sometimes when repairing or modifying a connection; the welds are compact and can be easily added to an existing high-strength bolted connection to provide additional strength. The latter connection type is commonly used in steel beam-column joints.

Tests described in this paper provide information on the strength and load interaction characteristics of combination joints for a variety of weld and bolt configurations. The studies include an evaluation of methods currently available for predicting the ultimate tensile strength of combination joints. The combination joints examined in the study are limited to butt joints which use bolts and welds on a common shear plane, as shown in Fig. 1.

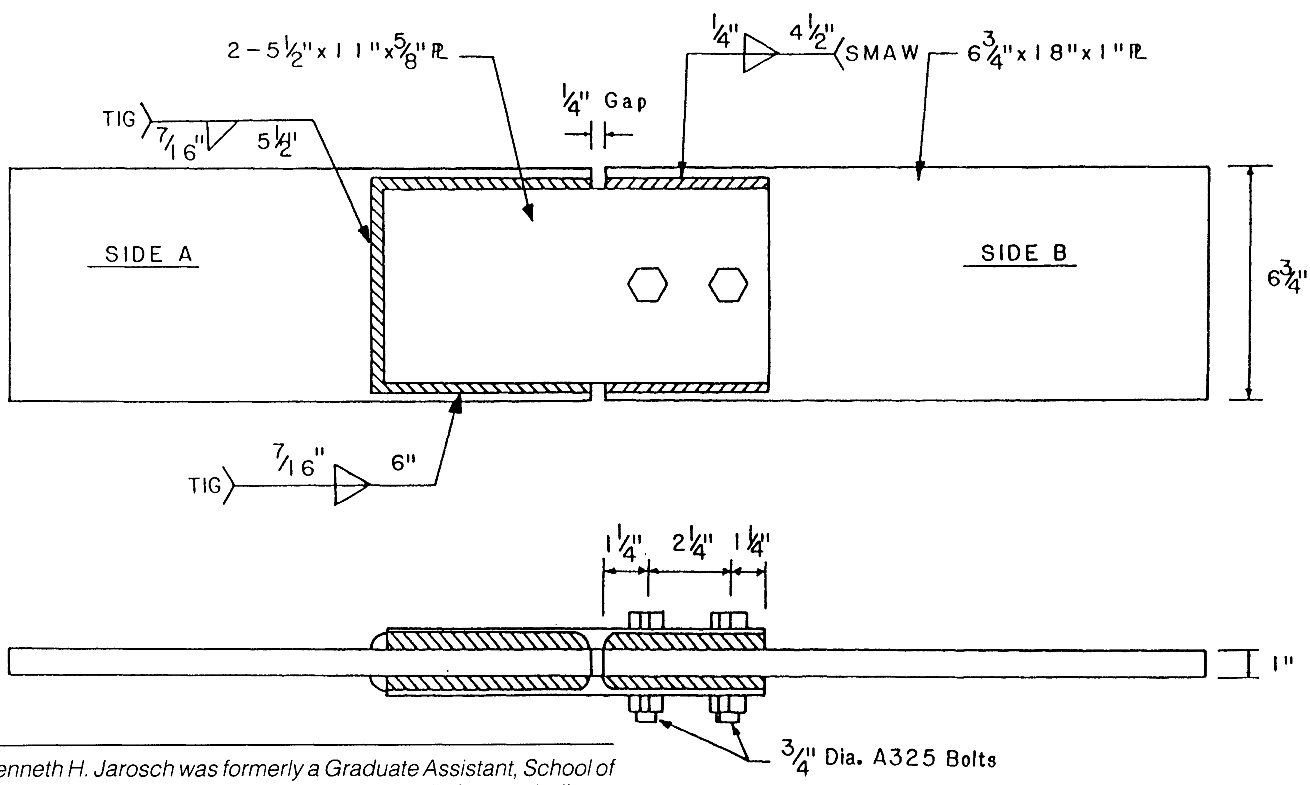

Fig. 1. Dimensions of WLB2 combination joint specimen

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BACKGROUND INFORMATION

Prior Studies

Few experimental studies have been conducted to collect data on the tensile strength of butt joints with high-strength bolts and fillet welds in combination. Fisher and Struik[1] cite the results of three studies conducted in Germany[2–4] on combination joints that included tension-type butt splices with two bolts on each side of the splice. Results of the tests conducted by Steinhardt, et al[4] indicate the capacity of a combination joint can be reasonably predicted as the sum of the slip load of the plain bolted connection and the strength of the welds.

An experimental study of combination joints was conducted also by Holtz and Kulak[5] in Nova Scotia in 1970. Holtz and Kulak's experimental work consisted of nine tension splices and six moment-resistant beam-column connections. The nine butt joint specimens were divided evenly among three groups of combination joints. Specimens from the first two groups contained longitudinal fillet welds in combination with two ¾-in. dia. ASTM A325 bolts, while specimens from the third group had transverse fillet welds combined with one ¾-in. dia. A325 bolt. Based on the test results, Holtz and Kulak[5] concluded the frictional bolt force was too unreliable to be considered in the connection analysis.

Holtz and Kulak[5] developed an analytical method for predicting the ultimate load of combination joints based upon the deformation characteristics of each fastening element. The separate load-sharing contributions for the bolts and welds are summed when the limit of one fastening element has been realized. Consequently, the load-deformation characteristics of the individual bolts and welds must be well defined.

Strength-Deformation Model

Fisher[6] demonstrated the load-deformation response of mechanical fasteners can be expressed by the following relationship:

$$R = R_{ult}(1 - e^{-\mu\Delta})^{\lambda} \tag{1}$$

where

R = force developed by a fastener at a given deformation Δ

R_{ult} = ultimate shear force attainable by the fastener

e = base of natural logarithms

μ and λ = empirical regression coefficients

The total deformation capacity Δ_{ult} for a given bolt and connected material includes bending, bearing and shearing deformations of the fastener, as well as bearing deformations of the plates.[6]

From tests conducted by Butler and Kulak[7] on ¼-in. E60 fillet welds, it was shown the strength of a weld depends on the orientation of the weld relative to the axis of the applied load. Curve fitting methods gave the following expressions for use in Eq. 1:

$$R_{ult} = \frac{10 + \theta}{0.92 + 0.0603\theta} \tag{2}$$

$$\mu = 75.0e^{0.0114\theta} \tag{3}$$

$$\lambda = 0.4e^{0.0146\theta} \tag{4}$$

where

θ = the angle, expressed in degrees, between the load and the longitudinal axis of the weld.

R_{ult} in Eq. 2 is ultimate unit load in kips per lineal in. Furthermore, it was found the ultimate weld deformation in inches Δ_{max} could be expressed as:[7]

$$\Delta_{max} = 0.225\,(\theta + 5)^{-0.47} \tag{5}$$

The tests conducted by Holtz and Kulak[5] indicated the welds reach their ultimate load prior to failure of the bolts. Therefore, to evaluate the ultimate load of a combination joint, the critical weld must first be identified; i.e., the weld with the greatest angle to the applied load. Assuming all portions of the weldment deform equally, then Δ for all of the subcritical welds is set equal to the value of Δ_{max} for the critical weld. Substitution of Eqs. 2 through 4 and the value for Δ_{max} into Eq. 1 give the expression for the strength of a unit length of weld as a function of θ. The ultimate strength of the weld group R_w is given as:

$$R_w = \sum_{i=1}^{m} L_{\theta i} R_{\theta i} \tag{6}$$

where

$L_{\theta i}$ = total length of weld in the direction of θ_i

$R_{\theta i}$ = strength of a unit length of weld in the direction of θ_i

m = number of different angles between weld axes and load direction

To account for the contribution of the bolts to the ultimate connection strength, the bolt deformation is found by

$$\Delta_b = \Delta_{max} - C \tag{7}$$

where

C = clearance between the bolt and the hole edge

The strength of each bolt, R_b, can be obtained by substituting the value of Δ_b into Eq. 1 and using the appropriate values for R_{ult}, λ and μ.

The ultimate capacity of the connection is given by summing the contribution of the bolts and the welds:

$$P_{ult} = nR_b + R_w \tag{8}$$

where

n = the number of bolts

Using the above approach, Holtz and Kulak[5] obtained reasonable estimates for the ultimate strength of the specimens tested.

Specification Requirements

Design criteria for combination joints are given in Sect. 1.15.10 of the AISC *Specification for the Design, Fabrication and Erection of Structural Steel for Buildings.*[8] The Commentary indicates it is permissible to use friction-type connections in conjunction with welds in new construction. The rigidity of the connection allows the loads to be transferred across the faying surfaces. A bearing-type connection is not permitted because the welds may prevent the bolts from going into bearing.

The use of bearing-type connections is permitted when making alterations to existing structures. Presumably, the slip will have occurred and the bolts will be in bearing. Consequently, any additional loads will be taken by the welds placed after the alteration is completed. The AISC requirement for the acceptability of a bearing-type connection depends, apparently, upon the opportunity for slip of the high-strength bolts to occur prior to placement of the welds.

EXPERIMENTAL STUDY

Specimen Fabrication

Six different joint configurations were used to examine a variety of different bolt and weld combinations, as shown in Table 1. Two specimens were prepared for each of the six joint configurations, for a total of 12 tests. The joint types examined include: bolts alone; longitudinal fillet welds alone; transverse fillet welds alone; bolts with longitudinal fillet welds; bolts with transverse fillet welds; and bolts with longitudinal and transverse fillet welds. A brief description of the specimen fabrication and assembly is given below; additional information and details are found in Ref. 9.

Table 1. Specimen Descriptions

Specimen No.	Weld[a] Type	No. Bolts
WOB2-1	——	2
WOB2-2	——	2
WLB0-1	L	0
WLB0-2	L	0
WTB0-1	T	0
WTB0-2	T	0
WLB2-1	L	2
WLB2-2	L	2
WTB2-1	T	2
WTB2-2	T	2
LTB2-1	L,T	2
LTB2-2	L,T	2

[a]*L* = longitudinal weld
T = transverse weld

The specimen dimensions are shown in Fig. 1. ASTM A36 steel plates were cut to the dimensions of 18 in.x6¾ in. and 11 in.x5½ in. from singles plates of 1-in. and ⅝-in. thicknesses, respectively, to fabricate the specimens. Standard sized holes, 1/16-in. larger than the nominal bolt diameter, were drilled in the plates to accommodate specimen assembly. Three-quarter in. dia. ASTM A325 bolts were placed in the holes and tightened to a snug tight condition, as defined in the AISC Specification.[8] Washers were placed on the nut end, which was later turned for final tightening.

The Shielded Metal Arc Welding (SMAW) process was used to join the ⅝-in. splice plates to the *test* side (Side B), while the Tungsten Inert Gas (TIG) process was used on the "anchor" side (Side A)—see Fig. 1. A one-quarter in. weld leg size was placed manually on Side B using 5/32-in. E6010 SMAW stick electrodes. One pass was made for each weld, and no end returns were made around the corners. The plates were not preheated. Side A and the splice plates were joined with a 7/16-in. TIG weld. No weld material was permitted to fill the ¼-in. gap between Plates A and B.

The weld leg on Side A is significantly larger than the weld on Side B, so failure would occur on the test side rather than on the anchor side. Also, to ensure adequate strength on the anchor side of the two LTB2 Specimens, two 1-in. dia. ASTM A490 high-strength bolts were installed in addition to the welds (see Fig. 2).

After the welded plates cooled, the ¾-in. dia. bolts were re-snugged and then turned one-half turn with a long-handled socket wrench. For the specimens with both ¾-in. dia. A325 bolts and 1-in. dia. A490 bolts, the 1-in. bolts were turned ⅓ turn (or as much as possible) prior to tightening the ¾-in. bolts.

Instrumentation and Testing

The instrumentation utilized in the study consisted of mechanical dial gages and electrical resistance strain (ERS) gages. The location of all instrumentation is described in Table 2, with reference to Figs. 3 and 4.

Two dial gages were attached directly to each specimen. The anticipated amount of deformation governed the choice of dial gage precision. The gages were attached to the specimens using two small metal brackets, one of which was glued to the ⅝-in. splice plate and the other to the 1-in. main plate. The bracket attachment points were located directly opposite one another to measure the relative displacement of the two plates.

Two lines of ERS gages, with five gages per line, were placed on four of the specimens with both bolts and welds to monitor changes in the strain distribution during loading.

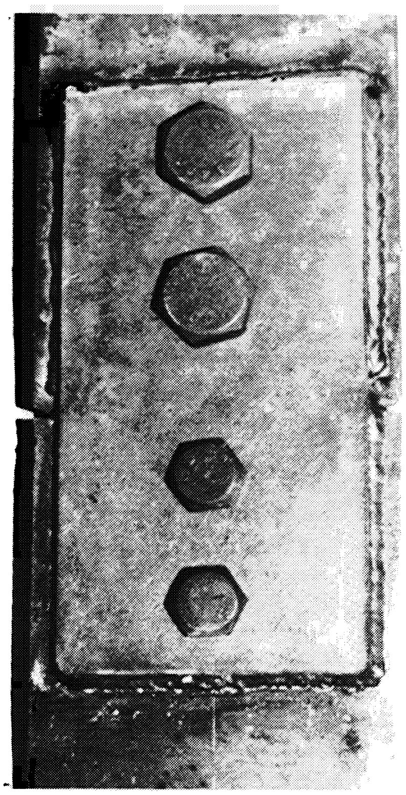

Fig. 2. Connection detail for Specimen LTB2-2

Table 2. Strain Gage and Dial Gage Locations

Specimen No.	ERS Gages Location[a]	Quantity	Type	Dial Gages Location[b]	Precision
WOB2-1	H, E1, E2	1 each	250BB	X1, X2	0.001″
	I	1 each	240LZ		
WOB2-2	E1, E2	1 each	240LZ	X1, X2	0.001″
WLB0-1	E1, E2	1 each	240LZ	Y1, Y2	0.0001″
WLB0-2	E1, E2	1 each	240LZ	Y1, Y2	0.0001″
WTB0-1	E1, E2	1 each	240LZ	Y2	0.001″
				Z1	0.0001″
WTB0-2	E1, E2	1 each	240LZ	Y2	0.001″
				Z1	0.0001″
WLB2-1	line 1	5	240LZ	X1, X2	0.0001″
	line 2	5	250BB		
WLB2-2	—	—	—	X1	0.0001″
				X2	0.001″
WTB2-1	line 1	5	240LZ	X2	0.001″
	line 2	5	250BB	Z2	0.0001″
WTB2-2	—	—	—	X2	0.001″
				Z2	0.0001″
LTB2-1	line 1	5	240LZ		
	line 2	5	250BB	X1	0.0001″
	F, G	1 each	250BB	Z2	0.0001″
	E1, E2	1 each	240LZ		
LTB2-2	line 1	5	240LZ		
	line 2	5	250BB	X1	0.0001″
	F, G	1 each	250BB	Z2	0.0001″
	E1, E2	1 each	240LZ		

[a] Refer to Fig. 3
[b] Refer to Fig. 4

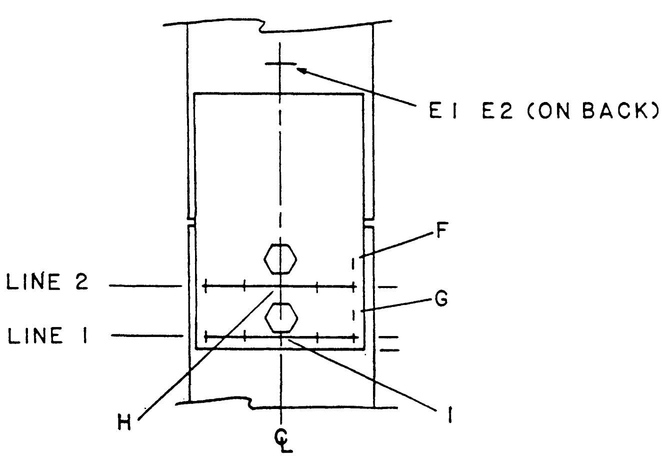

Fig. 3. ERS gage locations

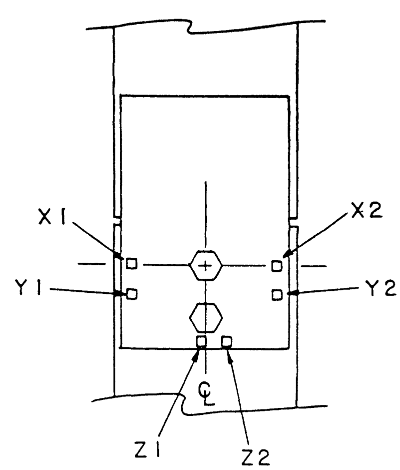

Fig. 4. Dial gage locations

Two types of gages were used: Micro Measurements EA-06-240LZ-120 and EA-06-250BB-120. The larger 250BB gages would not fit between the bolt and the plate edge, so the smaller 240LZ gages were required. The ERS gages were connected through a quarter-bridge circuit within the switch and balance units. A Vishay/Ellis strain indicator was used to read the strains. Figure 5 illustrates the dial gage and ERS gage instrumentation for one of the specimens prior to testing.

The specimens were loaded to tensile failure in the 600,000-lb. Baldwin Universal testing machine in the Structural Engineering Laboratory at Purdue University. The loading was gradually applied in 10 to 30 kip intervals, depending on the anticipated specimen strength. When significant deformation changes were observed, then the load increments were reduced to 5 kips or less until fracture occurred. At each increment, the load was held constant until all of the dial gage and strain indicator readings were recorded.

TEST RESULTS

Load was applied to each specimen in the test program until complete specimen fracture or significant unloading of the testing machine occurred. All of the specimens failed on the *test* side through the connecting bolts and welds. The tensile capacity of each specimen and the average ultimate load for each pair of specimens are given in Table 3.

Load-deformation curves for one specimen of each joint type are shown in Figs. 6–11. Two curves are shown for each specimen, corresponding to the two dial gages monitored during each test to evaluate deformation of the splice plate relative to the main plate. For specimens without transverse welds, both of the dial gages were positioned near the middle of the longitudinal welds. For specimens with transverse welds, one gage was placed near the middle of the transverse weld and one near the middle of the splice plate overlap.

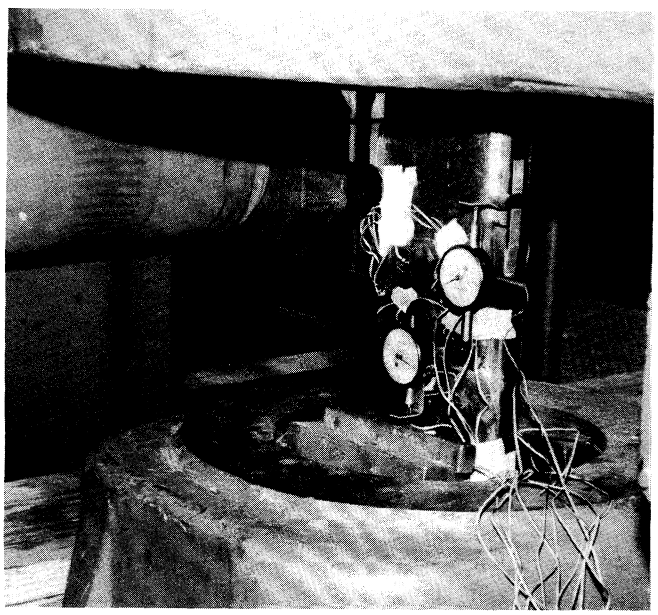

Fig. 5. Instrumentation for combination joint prior to testing

Table 3. Ultimate Loads for Test Specimens

Specimen No.	Weld[a] Type	No. of Bolts	Failure Load kips	Average Load kips
WOB2-1	—	2	152.5	152.8
WOB2-2	—	2	153.0	
WLB0-1	L	0	177.5	184.1
WLB0-2	L	0	190.8	
WTB0-1	T	0	142.0	166.8
WTB0-2	T	0	191.5	
WLB2-1	L	2	261.0	253.5
WLB2-2	L	2	246.0	
WTB2-1	T	2	175.0	178.8
WTB2-2	T	2	182.5	
LTB2-1	L, T	2	350.5	337.8
LTB2-2	L, T	2	325.0	

[a]L = longitudinal weld
T = transverse weld

Bolts Alone

Both bolts failed in direct shear in the same plane at the end of each test. The largest measured relative plate movement prior to failure was 0.17-in. The slip load was approximately 35 kips and 25 kips for Specimens WOB2-1 and WOB2-2, respectively. A sudden audible slip occurred on WOB2-1, but the slip on WOB2-2 was gradual. There was a notable elongation of both 13/16-in. holes in the 1-in. thick plate, with more deformation occurring at the hole closest to the end of the plate. Less hole deformation was observed in the 5/8-in. splice plates.

The ultimate fastener load computed from R_{ult} of Eq. 1 is compared to the ultimate test load in Table 4. A value of R_{ult} = 74 kips per bolt was used for the 3/4-in. dia. ASTM A325 bolts.[10] Excellent agreement was obtained between the calculated and actual ultimate bolt shear strengths.

Longitudinal Welds Alone

The two specimens with longitudinal welds alone sheared through the throat of the weld for nearly the entire weld length. The fracture occurred suddenly after significant plastic deformation of the weld. The largest deformation measured before failure was 0.033-in. for Specimen WLBO-2, as shown in Fig. 7. Deformation readings for

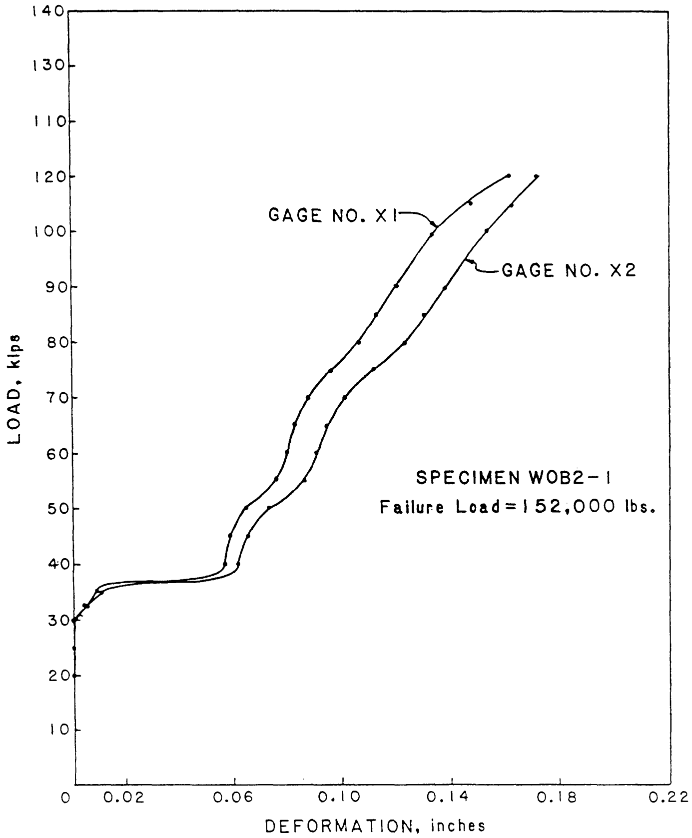

Fig. 6. Load-deformation curve for Specimen WOB2-1

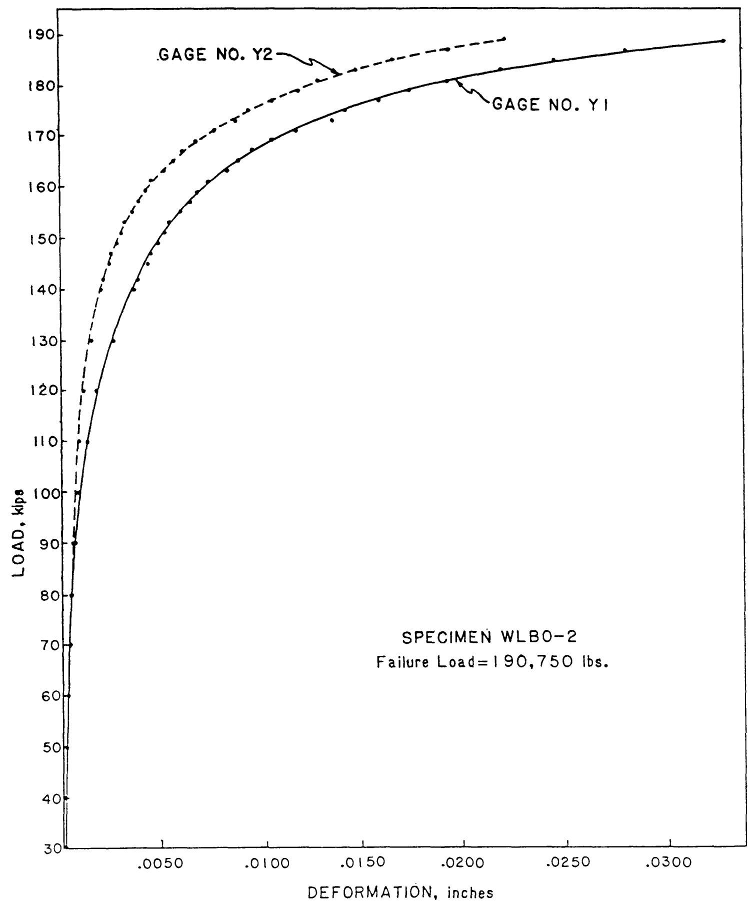

Fig. 7. Load-deformation curve for Specimen WLB0-2

Specimen WLBO-1 were not taken in the last 27,000-lbs. before fracture.

The ultimate load for the welded specimens was predicted by multiplying the ultimate weld load per unit length from Eq. 2 times the measured fillet weld lengths. The expression for the ultimate weld shear strength per unit length was developed for welds with ¼-in. leg sizes.[7] Consequently, the value of R_{ult} given by Eq. 2 was modified by the ratio of the actual weld throat dimension to that of a ¼-in. weld. The predicted ultimate weld loads for Specimens WLBO-1 and WLBO-2 are given in Table 4. As noted, excellent agreement was observed between the predicted and test ultimate weld loads.

Transverse Welds Alone

Specimens with transverse fillet welds alone failed in shear along the interface between the 1-in. plate and the weld for nearly the entire length of weld. The welds sheared all at once, with maximum deformations near failure measured

Table 4. Comparison of Computed and Observed Ultimate Loads

Specimen No.	Test Load kips	Computed Load (kips) Weld	Bolt	Total	Ratio of Computed Load to Test Load
WOB2-1	152.5	0	148.0	148.0	0.971
WOB2-2	153.0	0	148.0	148.0	0.967
WLB0-1	177.5	170.5	0	170.5	0.961
WLB0-2	190.8	181.7	0	181.7	0.952
WTB0-1	142.0	133.3	0	133.3	0.939
WTB0-2	191.5	171.7	0	171.7	0.897
WLB2-1	261.0	208.1	83.1	291.2	1.116
WLB2-2	246.0	203.0	83.1	286.1	1.163
WTB2-1	175.0	181.9	0	181.9	1.039
WTB2-2	182.5	180.5	0	180.5	0.989
LTB2-1	350.5	378.7	0	378.7	1.080
LTB2-2	325.0	370.0	0	370.0	1.138

at 0.010-in. and 0.021-in. for Specimens WTBO-1 and WTBO-2, respectively.

The ultimate weld load was predicted as before, using Eq. 2 and the actual throat dimension. However, the ultimate weld strength per unit length is greater for the transverse welds than for the longitudinal welds because the weld axis is perpendicular to the applied load. The comparison between the calculated and test ultimate load, as shown in Table 4, indicates the predicted strength is 6 to 10% low.

Longitudinal Welds with Two Bolts

Both specimens with longitudinal welds and bolts (WLB2-1 and WLB2-2) exhibited similar behavior and modes of failure. For Specimen WLB2-1, the last measured relative plate movement before the load peaked was 0.072-in. at a load of 250 kips. After reaching the peak at 261 kips, the load dropped off, while the plate continued to deform. At a load of approximately 150 kips, the entire connection failed at once. Note, the final failure load is roughly equal to the shear strength of two bolts.

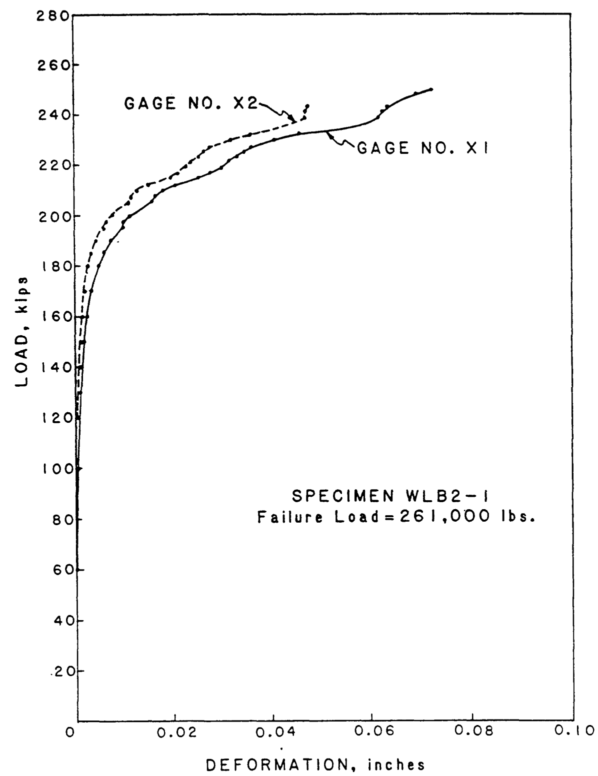

Fig. 9. Load-deformation curve for Specimen WLB2-1

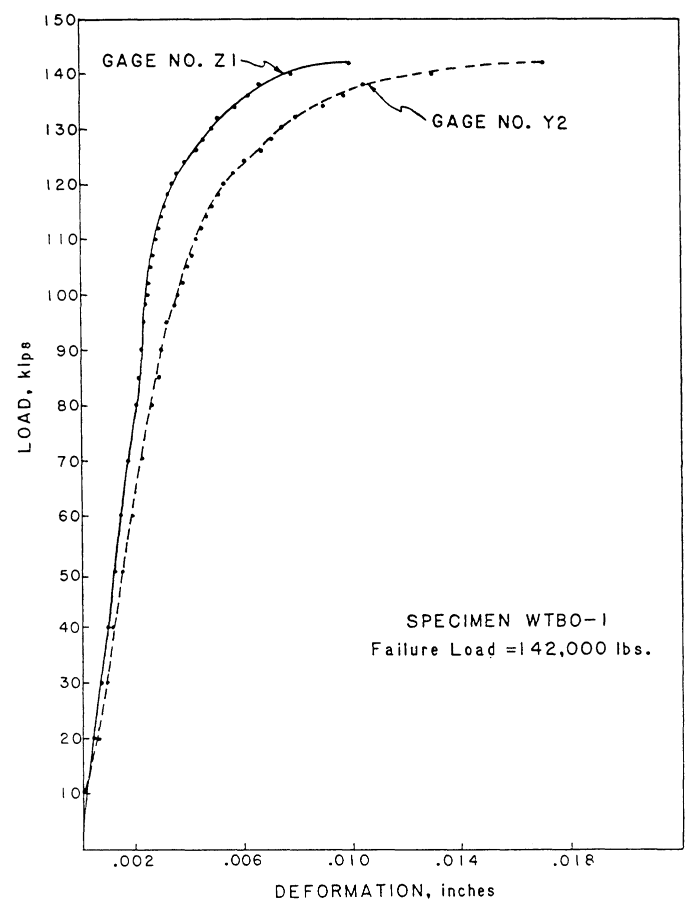

Fig. 8. Load-deformation curve for Specimen WTB0-1

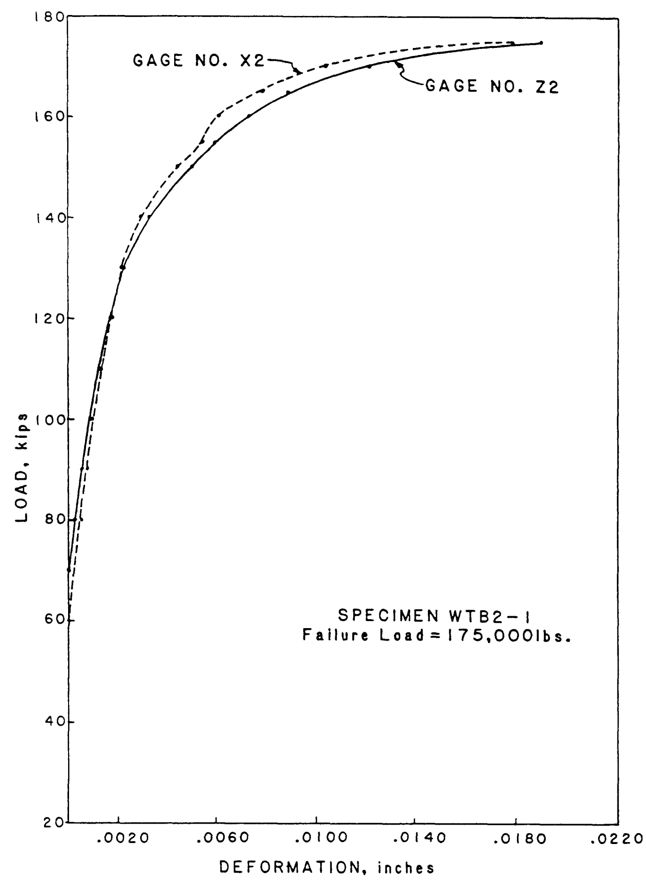

Fig. 10. Load-deformation curve for Specimen WTB2-1

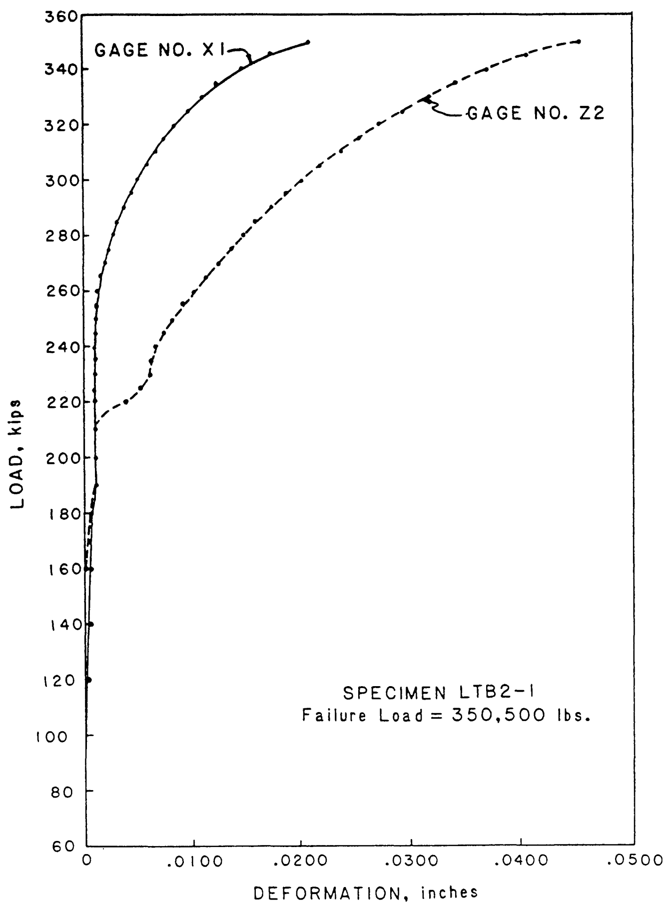

Fig. 11. Load-deformation curve for Specimen LTB2-1

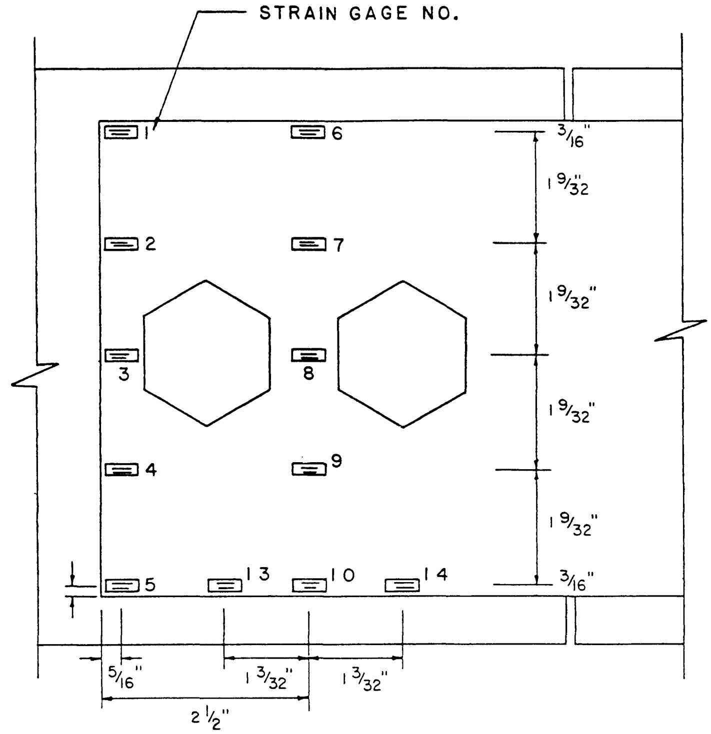

Fig. 12. Strain gage location numbers

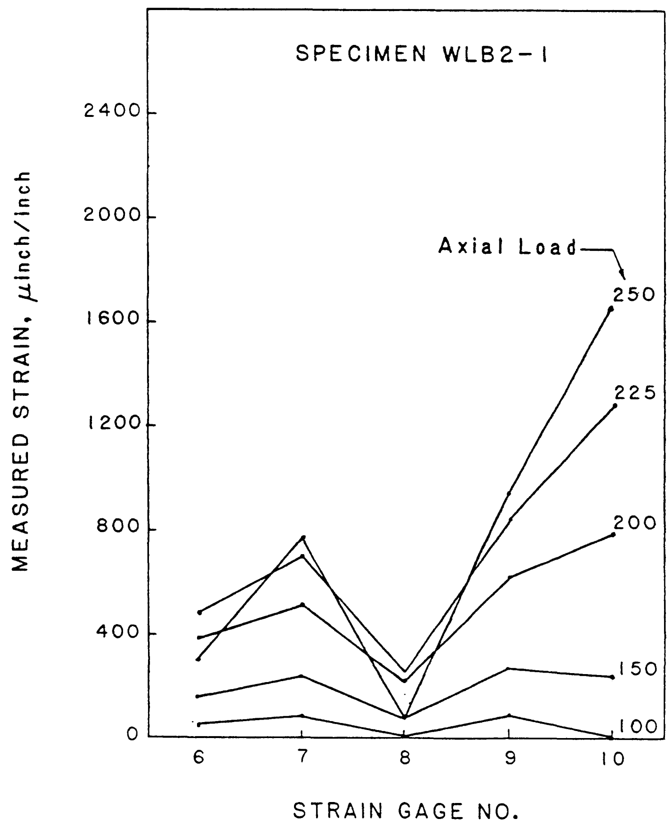

Fig. 13. Strains measured in Gage Nos. 6-10 for Specimen WLB2-1

Specimen WLB2-1 was one of the four specimens instrumented to monitor the strain in two lines across the plate width. Figure 12 shows all of the ERS gage location numbers for the four specimens; Gage Nos. 13 and 14 were not placed on Specimen WLB2-1. The strains measured in Specimen WLB2-1 at Gage Nos. 6-10, the gage line between the two bolts, are shown in Fig. 13 for a number of tensile load levels. A nearly uniform strain distribution was observed for loads less than 100 kips. However, as the load increased above 100 kips, the strain measured between the bolts (Gage No. 8) was notably less than the strain elsewhere. Furthermore, the strain between the bolts decreased for loads above 225 kips. The strain decrease suggests that the bolts go into bearing between 225 kips and 250 kips. The change in slope of the load-deformation curves (Fig. 9) at 240 kips tends to support this observation.

The predicted ultimate load is given in Table 4. The strength-deformation method overestimates the strength of the specimens by 12-16%. In computing the ultimate load, the critical deformation is the ultimate weld deformation given by Eq. 5. The bolt strength was computed using Eq. 1, with m = 10 and l = 0.55 as given by Crawford and

Table 5. **Ultimate Load for Combination Joint Based Upon Weld Strength Plus Slip Load**

Specimen No.	Test Load kips	Calculated Weld Load kips	Slip Load kips	Calculated Total Load kips	Ratio of Calculated Total Load to Test Load
WLB2-1	261.0	208.1	30.0	238.1	0.912
WLB2-2	246.0	203.0	30.0	233.0	0.947

Kulak,[10] and a fastener deformation equal to the ultimate weld deformation minus the bolt clearance of 1/16-in.

Another technique to obtain the strength of the combination joint is to add the ultimate weld load to the load required to cause slip of the bolts.[1,4] The average slip load from the two tests with bolts alone is 30 kips. Table 5 shows a more favorable comparison between the computed and test ultimate loads when the slip load is added to the ultimate weld load.

Transverse Welds with Two Bolts

Failure of the combination joint specimens with transverse welds and two bolts occurred by shearing of the welds at the interface of the 1-in. plate and weld. The weld deformation prior to fracture was approximately 0.020-in. The bolts did not fail in either test, due to the sudden unloading of the testing machine when the welds fractured.

The transverse weld strength was predicted by multiplying the actual weld lengths times the weld ultimate shear strength, Eq. 2. Prediction of the load-sharing contribution of the high-strength bolts requires consideration of the plate deformation. The bolts will not go into bearing if the relative plate deformation that occurs is less than the bolt hole clearance. The critical weld deformation given by Eq. 5 is 0.026-in., which is less than the 1/16-in. bolt hole clearance used for standard holes. Consequently, the predicted ultimate strength of the connection did not include the bearing strength of the bolts.

The predicted ultimate load of the combination joint is given in Table 4. Excellent agreement is observed between the calculated and observed ultimate load values. The average ratio of computed load to test load, which is based on weld strength only, is 1.01. The strength-deformation method accurately predicts the lack of load-sharing attributed to the bolts.

All-around Weld with Two Bolts

Failure of the specimens with two bolts in addition to longitudinal and transverse fillet welds occurred in two stages: welds breaking first, followed shortly afterward, but not immediately, by shearing of the bolts. The measured deformations and strains suggest that the transverse welds failed first, followed by tearing of the longitudinal weld from the free end back towards the transverse weld. Shearing of the bolts occurred in the final stage, as the remaining ligaments of the longitudinal weld fractured.

Both specimens were instrumented as shown in Fig. 12. Additional strain gages (Gage Nos. 13 and 14) were placed on the specimen to monitor the strain parallel to the longitudinal weld. The measured strains in Specimen LTB2-1 along the gage line between the bolts are shown in Fig. 14. As before, the strain between the two bolts (Gage 8) prior to failure is less than the strain elsewhere.

The distribution of measured strain parallel to one of the longitudinal welds (Fig. 15) illustrates very little strain (or deformation) occurs at the intersection of the longitudinal and transverse welds. Moreover, the strain increases significantly near the middle of the longitudinal weld as the applied load nears the connection capacity.

Load-displacement data for Specimen LTB2-1 are shown in Fig. 11. The curve labeled Gage X1 refers to the measured deformation near the middle of the longitudinal weld, while Gage Z2 refers to the deformation at the middle of the transverse weld. The deformation of the transverse weld prior to failure was nearly twice that of the longitudinal weld in both tests. The transverse weld deforms elastically up to a load of roughly 220 kips. However, after a small plastic deformation, the load increases sharply above the 220-kip level, possibly indicating the transverse

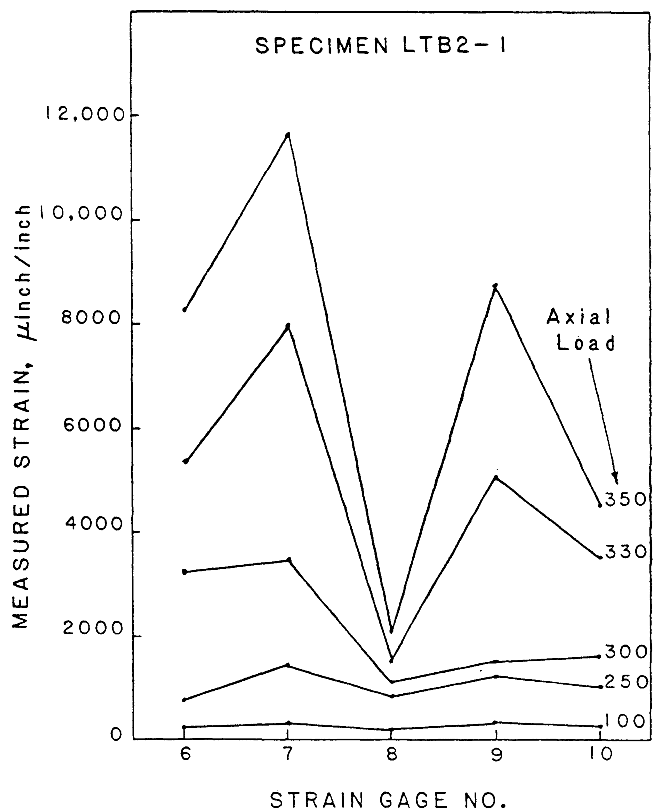

Fig. 14. Strains measured in Gage Nos. 6-10 for Specimen LTB2-1

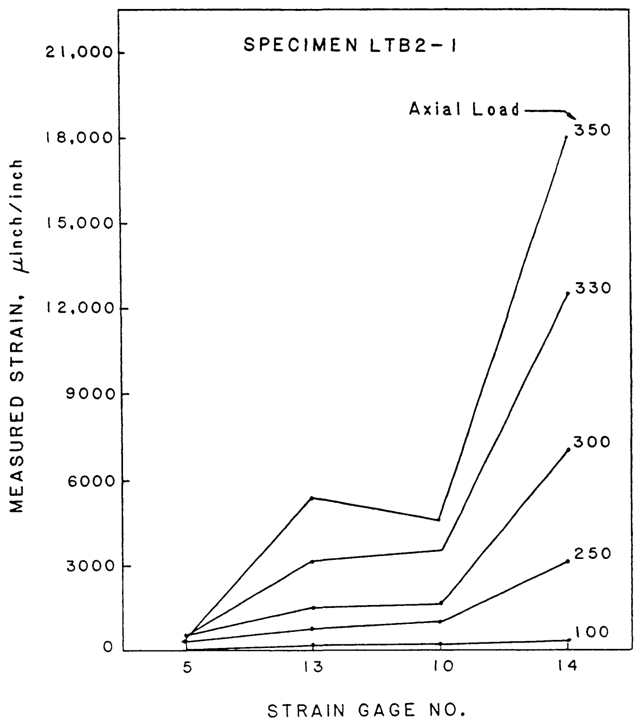

Fig. 15. Strains measured parallel to longitudinal weld of Specimen LTB2-1

weld dial gage is picking up a mobilization of load sharing with the high-strength bolts.

A comparison of the predicted and test ultimate loads is given in Table 4. The bolt shear strength is ignored because the critical deformation of the transverse weld at failure is less than the amount needed to close the bolt hole clearance and bring the bolts into bearing. The strength-deformation method overestimates the ultimate load, even though the bolt strengths are neglected.

The cause of the excessive ultimate weld load is believed to be an overestimation of the longitudinal weld strength. For the strength-deformation method, it is assumed all welds are subject to a deformation corresponding to the critical (transverse) weld deformation. However, the deformations measured near the middle of the longitudinal welds are less than those of the transverse welds, and the strain parallel to the longitudinal weld decreases to nearly zero near the intersection of the transverse and longitudinal welds. These measurements indicate the ultimate weld strength per unit length will not be equal to the full value, given by the critical weld deformation, for the entire length of the longitudinal weld.

SUMMARY AND CONCLUSIONS

Based on the test results of the experimental program reported herein, the following observations and conclusions can be noted:

1. Specimens with two bolts and fillet welds parallel to the direction of loading sustained a deformation adequate to permit the bolts to go into bearing. The ultimate load was most reliably predicted by summing the weld shear strength and the load required to cause bolt slippage.
2. Specimens with two bolts and fillet welds perpendicular to the direction of loading failed by shearing of the welds only. The transverse weld strength was exceeded before the bolts could slip and participate in bearing. The ultimate load was accurately predicted by the strength-deformation model.
3. Specimens with two bolts and fillet welds parallel and perpendicular to the direction of loading failed initially at the transverse welds, followed by tearing of the longitudinal welds and shearing of the bolts. The prediction for the combined weld strength alone exceeded the ultimate test loads. It is believed the geometry of the weld group is responsible for unequal local weld deformations that led to overestimating the longitudinal weld strength by the strength-deformation method.
4. Test results indicate transverse welds reach their failure load before the bolts, placed in standard holes, have an opportunity to go into bearing. Care should be taken when using high-strength bolts and transverse welds together.

The foregoing observations are based on a very limited number of experimental tests. Further study is needed to develop reliable strength relationships for joints with combined welds and bolts.

ACKNOWLEDGMENTS

The investigation discussed herein was supported in part by the School of Civil Engineering at Purdue University. All of the high-strength bolts were provided by International Steel Corporation. The advice and support provided by John F. W. Koch is gratefully acknowledged. Appreciation is extended also to William A. Cook, who assisted in testing of the specimens.

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