Fatigue in Welded Beams and Girders

W. H. MUNSE and J. E. STALLMEYER, Professors of Civil Engineering,
Department of Civil Engineering, University of Illinois, Urbana

● IN RECENT YEARS there has been a marked increase in the use of welding of highway bridges, particularly for welded girder bridges. At the same time, this increase in the application of welding to bridges has resulted in a greater need for information concerning the fatigue behavior of such structures.

The current AWS design specifications (1) for welded bridges are based on a consideration of fatigue. The problem of designing for fatigue, however, is complicated by the fact that the fatigue behavior varies considerably for the component members, connections, and details of such structures. Some members or details may receive relatively few and others may receive many applications of maximum load, some members or locations may be subjected to relatively small changes in stress and others may receive reversals or large ranges of stress during a loading. These are factors of major importance and need to be taken into account in design if the structures are to resist efficiently and economically the loads to which they are subjected.

Also of importance to the behavior of structures subjected to repeated loads is the geometry of the details of the individual parts that make up the structure. The resulting stress concentrations have a marked effect on the behavior of the structure and may, in fact, be responsible for failures unless they are properly provided for in the design.

This paper summarizes the results of a number of tests made at the University of Illinois in recent years to demonstrate the effect of details on the fatigue behavior of welded flexural members. Details such as splices, stiffeners, cover plates and attachments, all of which can be expected to produce reductions in the fatigue strength of the basic member, have been included. In addition, the fatigue behavior of these members is related to that of the basic material, thereby making it possible to obtain an indication of the effective stress concentration of the various details studied.

In this evaluation of laboratory data, the test conditions are related to actual service conditions; however, because of the many different combinations of loading obtainable in the field, only selected service conditions will be related directly to the data. Nevertheless, interpolations and extrapolations from the laboratory data provide a general indication of the behavior that can be expected under various service conditions. Though very limited in number, service records from actual structures in which fatigue failures have developed provide another means of relating the laboratory data with the field behavior.

DESCRIPTION OF MATERIALS AND TESTS

The tests, except for a few of those included in a preliminary series, were conducted on specimens fabricated from ASTM A373 steel. An ASTM A7 steel was used for the preliminary tests but did not differ greatly in chemical composition or in mechanical properties from the A373 steel used in the major portion of the program. The chemical composition and physical properties of the A373 steel are given in Table 1.

Manual arc welding with E7016 electrodes was used in the fabrication of most of the specimens for the tests discussed herein. The welds were deposited with reversed polarity, in the flat position, and with 5/32-in. electrodes for the assembly of the basic section and 1/8-in. electrodes for the attachment of stiffeners. All electrodes were stored in a drying oven to prevent absorption of moisture in the electrode coating.

The beams were tested in the 200,000-lb University of Illinois fatigue testing machine shown in Figures 1 and 2. These machines, when used for the testing of welded beams, are capable of applying a maximum midspan load of approximately 112,000 lb

TABLE 1

PROPERTIES OF ASTM A373[a] STEEL PLATES FOR WELDED BEAMS

Thickness (in.)	Chemical Content (%)						Physical Properties		
	C	Mn	P	S	Si	Cu	Yield Strength (psi)	Ultimate Strength (psi)	Elongation in 8-in. (%)
3/16[b]	0.23	0.60	0.023	0.030	0.050	0.065	42,040	65,490	28.3
1/2	0.23	0.56	0.018	0.024	0.070	0.020	35,990	64,780	31.0
3/4	0.18	0.94	0.007	0.019	0.050	0.025	38,700	63,400	29.0
1[b]	0.23	0.53	0.019	0.028	0.068	0.095	37,010	65,460	28.7

[a]ASTM A373 Structural Grade Steel for Welding. [b]Average for three heats of steel.

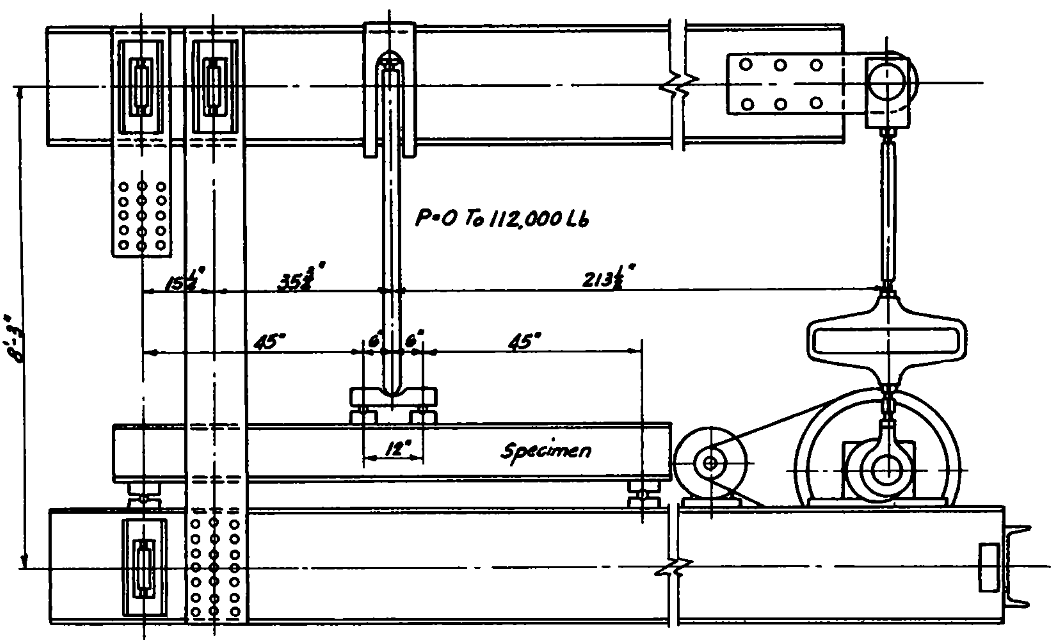


Figure 1. 200,000-lb Illinois fatigue testing machine adapted to test flexural specimens.

at a rate of 180 applications per min. Thus, approximately 250,000 cycles of loading can be applied in 1 day or 2,000,000 cycles of loading in approximately 8 days.

The capacity of the testing machines has limited to some extent the size of the members that can be tested. Nevertheless, they are large enough to permit the testing of beams with depths as great as 16 in. and spans of 8 ft 6 in. Although these members are relatively small in comparison with long-span welded bridges, they are of such a scale that the results of the tests will be directly applicable to the design of full-scale structures.

Most of the studies were conducted on a stress cycle in which the stress in the extreme fibres of the bottom flange of the member ranged from zero to tension. However, selected members have also been subjected to stress cycles of full reversal, or partial tension to a full tension in the extreme fibres. The use of these various cycles makes it possible to relate the behavior of the test members directly to the behavior that can be expected in bridges employing similar details.

The maximum values of test stress were selected to produce failures at lives rang-

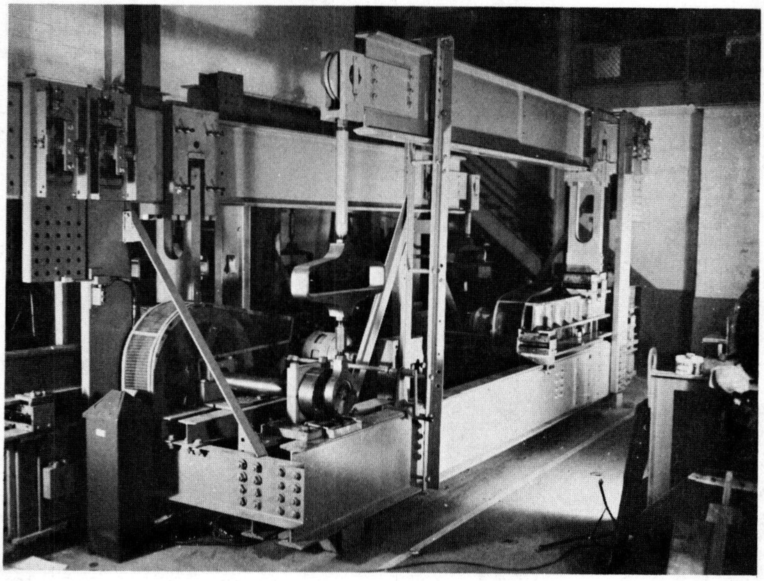


Figure 2. Flexural fatigue tests under way in 200,000-lb Illinois fatigue machines.

ranging from 100, 000 to 2, 000, 000 cycles and to provide S-N relationships which can be used to evaluate the effect of details upon the fatigue behavior of the test members. Such information also can be employed to develop design rules that will provide adequate resistance against fatigue.

After failure of the test members, the actual dimensions of the cross-section at the failures were measured and the stresses at these locations calculated using the general flexure formula,

$$s = \frac{Mc}{I}$$

Thus, although the applied loads were based on the nominal section of the test members, the failure stresses are based on the actual sections at which the failures occurred.

FATIGUE BEHAVIOR OF I-BEAMS

A limited number of tests have been conducted at the University of Illinois on plain rolled I-beams (2). Many more tests have been made on plain welded beams wherein a variety of sizes and shapes of members has been studied.

A summary of the fatigue strengths obtained for rolled beams of A7 steel, and welded beams of A7 and A373 steels are presented in Table 2. These results are only for members subjected to a stress cycle of zero to tension in the extreme fibres of the tension flange.

Comparing the fatigue resistance of the various beams with that of flat plates in the as-rolled condition, the fatigue resistance of the rolled beam appears only slightly different from that of the plain flat plate. However, the average fatigue strengths of the welded beams were about 5, 000 psi below those of the plain flat plates and those of the rolled beams. This decrease results from the details inherent in the fabricated beams—such details or factors as the weld and its geometry, the edge preparation of the various plates, the relative thickness of the components and the straightness of the member.

An examination of the test data indicates that for a stress cycle of 0 to 30, 000 psi, the fatigue life of the plain welded beams ranged from approximately 600, 000 to 2, 000, 000 cycles. This scatter, although relatively large in terms of life, is equivalent to a variation in fatigue strength of only about 7, 000 psi.

TABLE 2

FATIGUE STRENGTHS OF PLATES AND BEAMS
(Zero-to-Tension Cycle)

Type of Member	Type of Steel[a]	Fatigue Strength[b] (psi)	No. of Tests Averaged[c]
Plain plates	A7	31,700	---[d]
	A373	33,000	3
Rolled beams	A7	31,200	3
Welded beams	A7	28,200	4
	A373	26,500	16

[a]ASTM structural steels.
[b]N = 2,000,000.
[c]Extrapolation based on k = 0.135.
[d]Data from previous investigations.

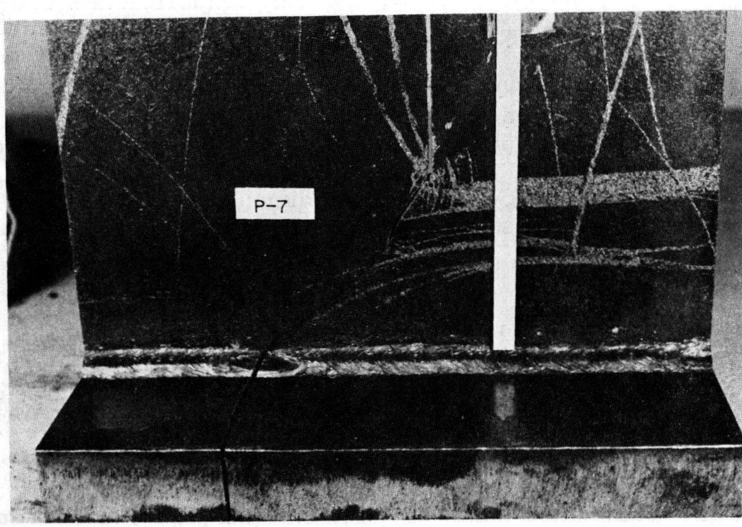


Figure 3. Fatigue crack initiating at weld crater.

One factor found to have an effect on the fatigue resistance of the welded beams is the ratio of the thickness of the flange to that of the web. In one series of tests on ASTM A7 steel beams the ratio was varied from 1.0 to 5.3. Although limited in number the tests, all conducted on the same stress cycle, gave lives ranging from 830,000 to 2,164,300 cycles. With so few tests, one would normally hesitate to draw conclusions concerning a factor such as the effect of the flange-to-web thickness ratio. However, the trend was consistent in all of the tests—the members with a thin web and heavy flanges had the greatest fatigue resistance when used in a plain welded beam.

In addition to the variation in fatigue life with the flange-to-web thickness ratio, a difference in the mode of failure was observed. The point of failure initiation was generally the same—in a weld crater of the fillet welds at the web-flange junction. However, the sequence of crack propagation differed somewhat for the various flange-to-web thickness ratios. For members with a low ratio (1 to 1), the cracks generally propagated through the flanges first and then into the web. For specimens with a high flange-to-web thickness ratio (1 to 5.3), the initial direction of propagation was into the web. In the latter case, the rate of crack propagation decreased as the crack propagated toward the neutral axis and only when the crack in the web was 2 to 3 in. long did it begin to spread into the flange. This difference in crack propagation helps to explain, at least in part, the variation in fatigue life obtained from the specimens with various ratios of flange-to-web thickness.

Most of the plain-welded beams failed in the pure moment region of the members. The cracks generally initiated at a weld crater in the web-flange junction and propagated in a direction normal to the axis of the beams. However, one of the failures initiated at a weld crater a short distance outside the pure moment area and propagated through the flange and diagonally upward into the web. This change in direction of the fatigue crack was a result of the combined state of stress existing in the web; this fracture is shown in Figure 3. The photograph also shows the manner in which the fatigue cracks initiated in the weld craters. In the other members of this type, the cracks in the web propagated vertically. However, though the weld craters may provide the points at which the failures initiate, their effect on the fatigue strength of the members is not great (see Table 2).

EFFECT OF SPLICES ON FATIGUE BEHAVIOR

Splices are often used in welded girder bridges, particularly in those that are continuous structures. The details of such splices may be of several types; they may have the welds all in a single plane or staggered, and may be fabricated with or without cope holes at the flange splices. In addition, shop splices may be made in either the web or the flange alone. All of these details, because of their inherent stress concentrations, can be expected to have an effect on the fatigue resistance of the members.

Laboratory data are available from tests on splices and splice details of the types shown in Figure 4. A comparison of the test results for the various splices on a zero-to-tension stress cycle is presented in Figure 5. On examination, the slopes of the S-N curves appear different for the two basic types of splices; the curves are steeper for the members with the splice in a single plane than for those that are staggered. Also, the cope holes reduced the fatigue strength in both instances by approximately 2,000 to 3,000 psi. Thus, both the splice and the details of the splice affect the fatigue behavior of the members.

Good quality splices in thin-web members of the type shown in Figure 4, can usually be produced without cope holes, but with a thick web it may be necessary to use the cope holes to obtain a sound flange weld. Consequently, care must be exercised in extrapolating from the laboratory tests to actual service conditions; the details that prove best in the laboratory may not always be as effective in the field but may still be necessary.

A summary of the fatigue strengths of the various types of spliced members is presented in Table 3. The fatigue strength of the spliced beams can be made to approach or equal that of a plain plate with a transverse butt weld; however, the fatigue strength of the spliced members at 2,000,000 cycles is only about two-thirds as great as the fatigue strength of the plain plate material. Even the flange splice alone or the cope holes alone (specimen types C and F) have an effect on the fatigue strength of the members and again demonstrate the effect of welding and geometrical details on the fatigue behavior of the members.

Although most of the tests of spliced members were conducted on a stress cycle of zero to tension, cycles of full reversal and half-tension to full tension have been used for the type A and D splices. The results of these latter tests, showing the effect of stress cycle, are given in Figure 6. Again it is evident that the type D splice without cope holes had a slightly higher fatigue strength than did the splice fabricated with cope holes.

As the next step of this evaluation the fatigue data may be analyzed with respect to the current specifications of the American Welding Society for the design of welded highway and railway bridges (1). The results of such a comparison are given in Figure 7. The lines identified as AWS-15 and AWS-7 represent the AWS design relationships for butt welds subjected to tension.

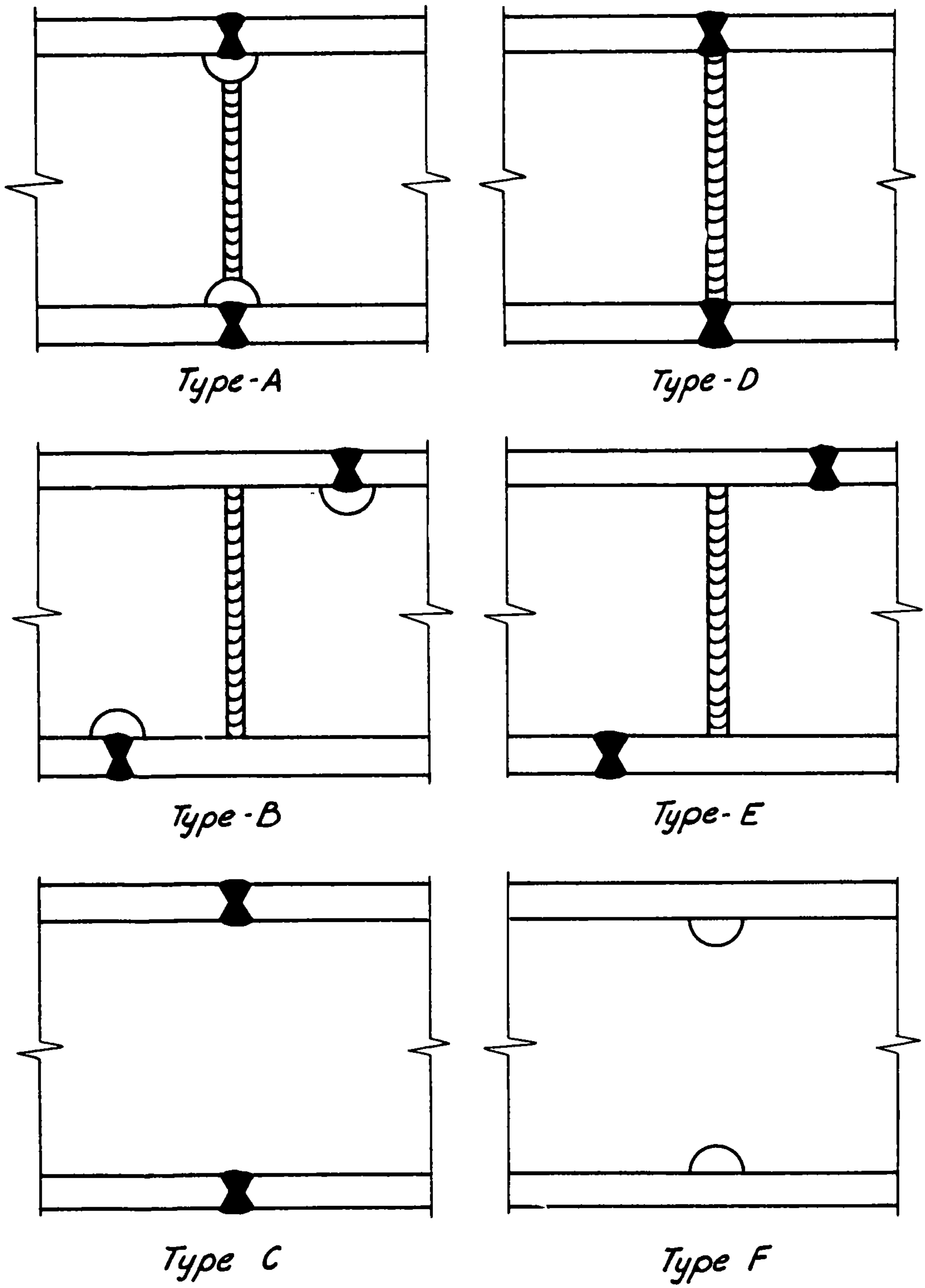


Figure 4. Details for splices in welded beams.

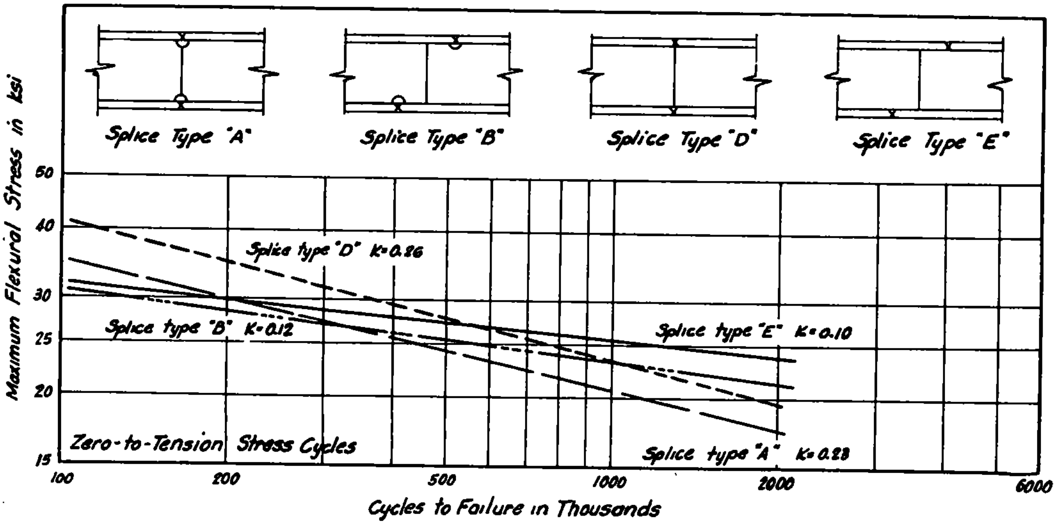


Figure 5. Comparison of test results for spliced beams.

TABLE 3

SUMMARY OF FATIGUE STRENGTHS FOR SPLICED MEMBERS (Zero-to-Tension Cycle)

Type of Member[a]	Fatigue Strength (psi)	
	n = 100,000	n = 2,000,000
Plain plate	-	33,000
Transverse butt welded joint (as welded)	-	22,500
In-line splice with cope holes, type A (Fig. 4)	33,500	17,500
Staggered splice with cope holes, type B (Fig. 4)	31,000	21,000
In-line splice, as welded, type D (Fig. 4)	40,000	19,500
Staggered splice, as welded, type E (Fig. 4)	32,000	23,000
Flange splice only, as welded, type C (Fig. 4)	-	26,000(2)[b]
Cope hole only, type F (Fig. 4)	-	23,000(3)[b]

[a]Fabricated from ASTM A373 steel.
[b]Number in parentheses indicates number of tests averaged when extrapolation based on only a small number of tests.

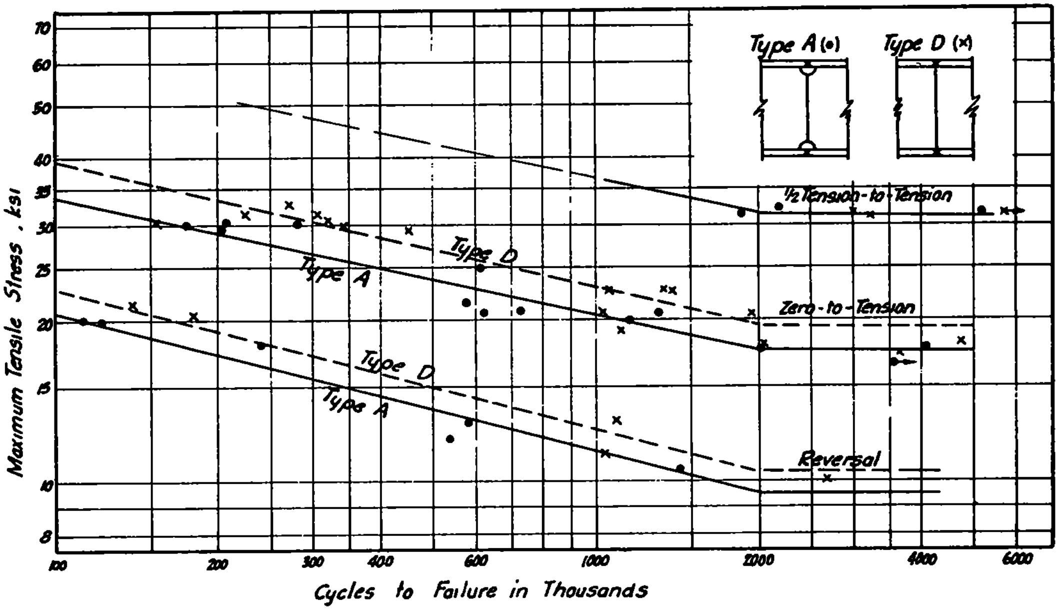


Figure 6. Results of fatigue tests on welded beams with splices.

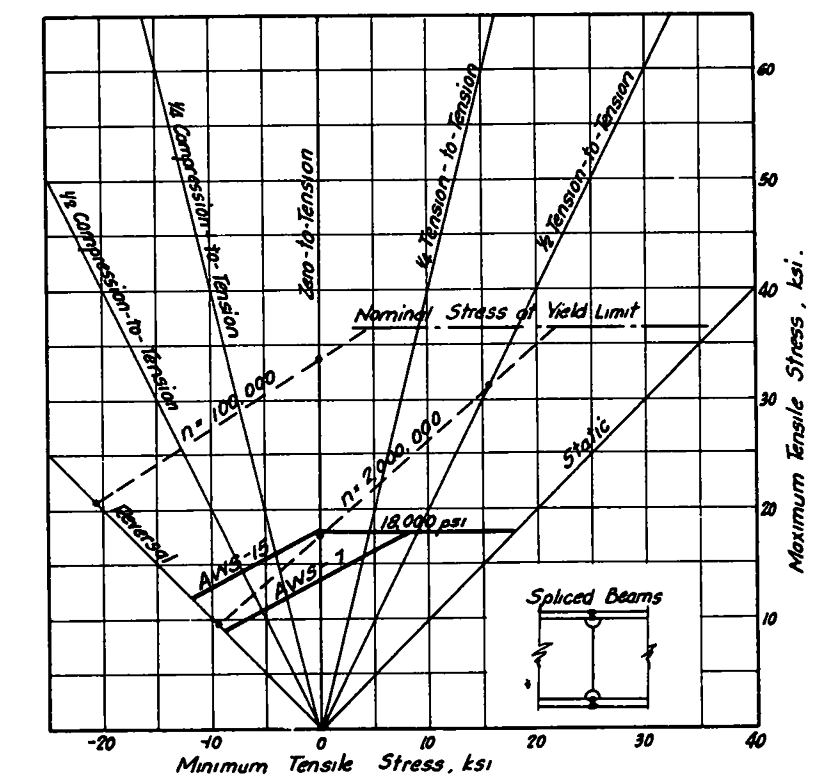


Figure 7. Relationship between test results for spliced beams and design specifications.

AWS Formula 7 (butt weld at 2, 000, 000 cycles):

$$A = \frac{\text{Max.} - 1/2\ \text{Min.}}{13,500} \quad \text{or} \quad s = \frac{13,500}{1 - 1/2\ k}$$

$$\text{but} \quad \geq \frac{\text{Max.}}{18,000} \qquad \text{but} \quad \leq 18,000$$

$$\text{where} \quad k = \left(\frac{\text{Min.}}{\text{Max.}}\right)$$

AWS Formula 15 (butt weld at 100, 000 cycles):

$$A = \frac{\text{Max.} - 1/2\ \text{Min.}}{18,000} \quad \text{or} \quad s = \frac{18,000}{1 - 1/2\ k}$$

$$\text{but} \geq 18,000 \qquad \text{but} \leq 18,000$$

$$\text{where } k = \left(\frac{\text{Min.}}{\text{Max.}}\right)$$

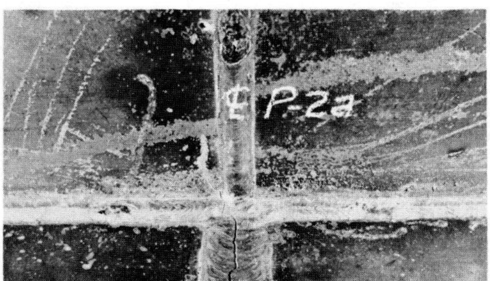


Figure 8. Flange-weld fatigue cracks in beams with butt-welded splices: (a) in-line-splice, and (b) staggered splice.

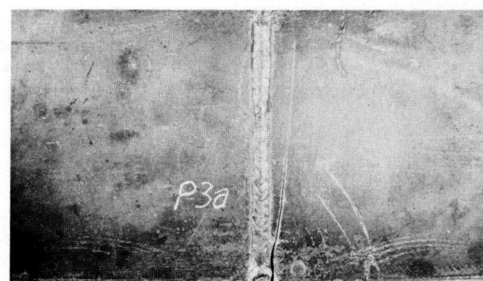


Figure 9. Fatigue cracks at edge of flange welds in beams with butt-welded splices: (a) in-line splice, and (b) staggered splice.

Line AWS-15 is generally thought to represent a condition comparable to 100, 000 cycles of loading, whereas line AWS-7 corresponds to a condition representing 2, 000, 000 cycles of loading. In the diagram it is evident that when the stress cycle varies from one of partial tension to full tension, the design factor of safety (relationship of fatigue strength to allowable design stress) may become relatively large. However, under a cycle of full reversal, a spliced member that is expected to receive 2, 000, 000 cycles of maximum loading during its lifetime will have a relatively small factor of safety.

A further indication of the fatigue behavior of the spliced members can be obtained from an examination of the fractures of these members. Although the number of tests was not great, a variety of failures was obtained. Some failures occurred in the weld (Fig. 8), some initiated at the toe of the butt weld (Fig. 9), and others initiated at the toe of the fillet welds associated with the cope holes (Fig. 10). In spite of this variety of failures the fatigue resistance of the various members did not differ greatly.

EFFECT OF STIFFENERS ON FATIGUE BEHAVIOR

The American Welding Society's specification for welded highway and railway bridges (1) states:

> Ends of stiffeners and other attachments may be welded to flanges only at points where the flanges carry compressive stress or where the tensile stress does not exceed 75 percent of the maximum allowable stress permitted by the applicable general specification.

Thus, the specifications are concerned only with the effect the stiffeners have on the behavior of the flanges and not with their effect on the behavior of the web. Nevertheless, the results of recent tests suggest that both the flanges and webs may need to be considered in designing for fatigue.

In tests recently conducted at the University of Illinois, the effects of various stiffeners on the fatigue behavior of welded flexural members have been studied. Figure 11 shows the variations in the details studied, including members with stiffeners on one or both sides of the web, members with and without the stiffener attached to the tension and compression flanges, members with the stiffeners attached to the web with intermittent fillet welds, and members with stiffeners attached only over a part of the web. Although these are but a few of the many details that could be used, they provide a general picture of the effect of stiffeners on the flexural behavior of the members.

All fatigue tests conducted on beams with stiffeners were made with a stress cycle of zero to tension in the extreme fibres of the bottom flange. The results of these tests, for five types of stiffeners, are shown by the S-N diagram in Figure 12. It should be noted, however, that these data are presented on the basis of the maximum flexural stress in the extreme fibres of the test member and not

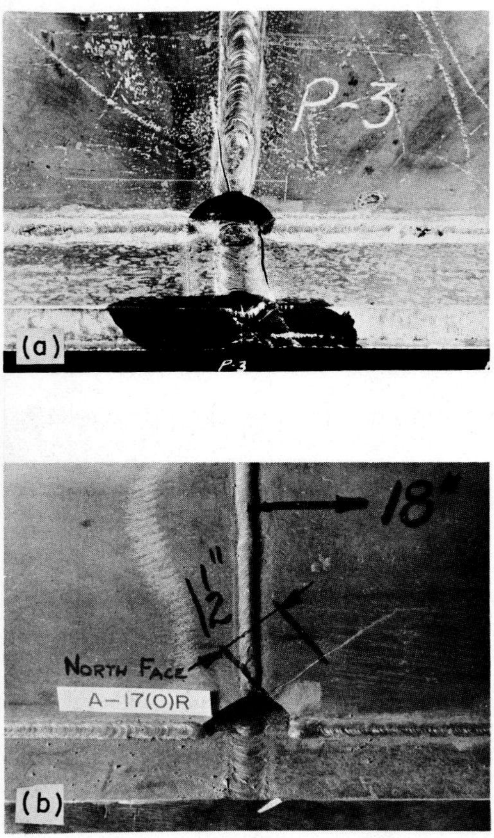


Figure 10. Fatigue cracks at cope-holes: (a) flange failure at toe of fillet, and (b) failure in web at cope hole.

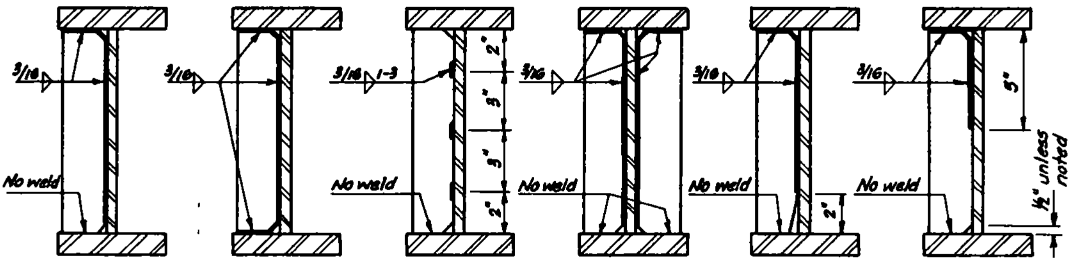


Figure 11. Details of various stiffener types.

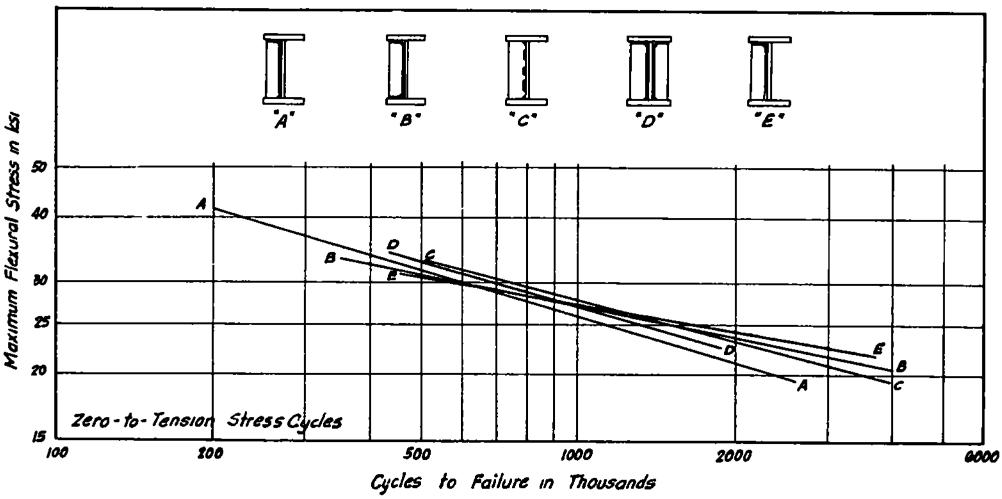
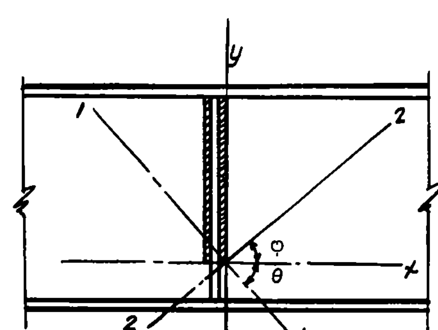


Figure 12. S-N diagram for stiffened beams based on maximum flexural stress.

necessarily on the basis of the stress at the point of failure. Nevertheless, excellent consistency was obtained for all of the stiffener details included in the study. Obviously the members all have about the same total flexural resistance; however, because the failures occurred at various points along the length of the span, the members did not necessarily have the same flexural fatigue strength at the points of failure.

In evaluating the data from the stiffener tests, several factors must be considered. The most important of these are the failure location and the general occurrence of fractures at stiffeners that were not in the region of pure moment. In fact, most of the fractures occurred at stiffeners located where the flexural stresses were considerably lower than the maximum flexural stresses. After a thorough examination of the data, the best correlation was obtained when the data were analyzed on the basis of the maximum principal tensile stress (including the effect of the shear) at the point of failure. The following relationships were used to determine the maximum principal tensile stress at the point where failure initiated.



$$\sigma_p = \frac{\sigma_x + \sigma_y}{2} \pm \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau^2} \qquad (1)$$

$$\tau_p = \frac{\sigma_1 - \sigma_2}{2} = \sqrt{\frac{\left(\sigma_x - \sigma_y\right)^2}{2} + \tau^2} \qquad (2)$$

$$\tan 2\theta = -\frac{2\tau}{\sigma_x - \sigma_y} \qquad (3)$$

$$\varphi = 90^\circ - \theta \qquad (4)$$

where

$$\tau = \frac{VQ}{It}, \quad \sigma_y = 0, \quad \sigma_x = \frac{My}{I}$$

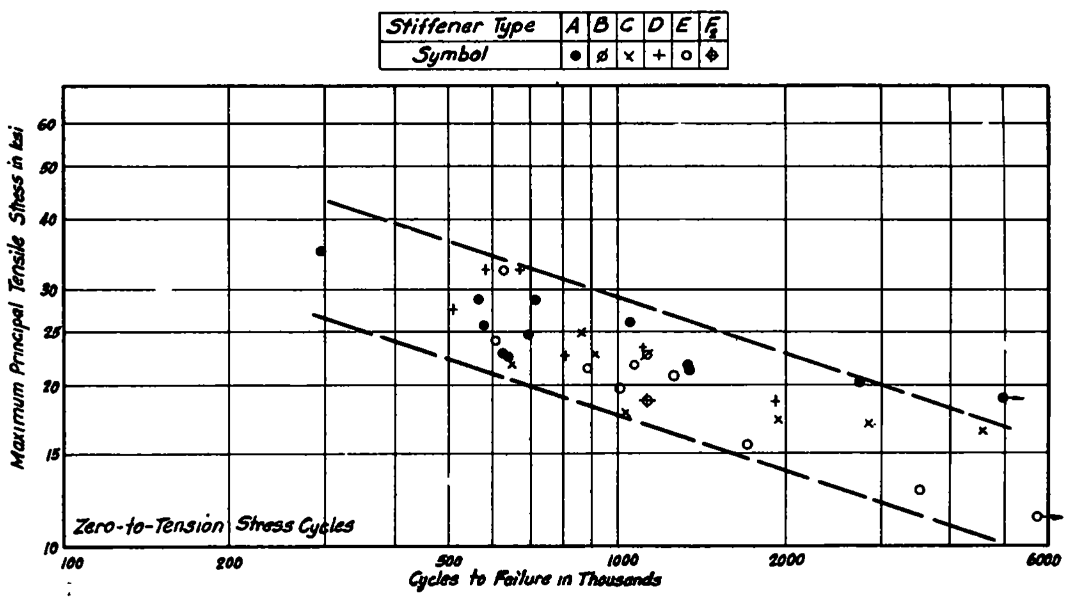


Figure 13. S-N diagram for maximum principal tensile stress at failure section.

The comparison obtained on the basis of a principal stress analysis is shown in Figure 13. Although there is still a scatter in the test results, the band is well defined for the entire range of lives and stresses used in tests.

A further indication of the fatigue behavior of the members with stiffeners can be obtained from an examination of the fatigue failures. As might be expected, a variety of failures was obtained. Members with types A, D, or E stiffeners and members with continuous welds connecting the stiffeners to the web had failures that initiated at the bottom of the fillet welds connecting the stiffeners to the web. Upon initiation, these cracks propagated diagonally upward into the web in a direction approximately normal to the principal tensile stress.

Another indication of the importance of the principal stress is the fact that the fractures were just as likely to initiate at a section near the supports as they were to initiate near the center of the beam.

Figure 14 shows a typical failure at a stiffener. In this instance, the crack initiated in the web at a point 25 $^1/_2$ in. from the beam support and propagated at an angle of approximately 50 deg. The computed angle for the maximum principal stress at this location was only slightly greater than 50 deg. The maximum flexural stress in this same member occurred at approximately 45 in. from the supports.

Figure 14. Web crack at type A stiffener (continuous web weld).

The fatigue cracks in members with type C stiffeners initiated at either the the top or bottom of the intermittent fillet welds connecting the stiffeners to the web. These cracks were then found to propagate diagonally upward into the web and downward to the flange. An indication of one such failure is shown in Figure 15. This failure initiated in the web of the member at a distance of approximately 20 in. from the support.

Figure 15. Web crack at type C stiffener (intermittent web weld).

The members with type B stiffeners were the only ones for which the stiffeners were welded to the tension flange. These members generally failed at the toe of the fillet weld connecting the stiffener to the flange of the beam, in the region near the maximum moment, and propagated vertically through the flange and up into the web. Nevertheless, the total fatigue resistance of the members with the type B stiffeners (those welded all around) was as great or greater than that of the members fabricated with the other stiffener details.

In view of the fatigue behavior that has been observed, it may be desirable to re-examine the restrictions of current specifications for the attachment of stiffeners. It would appear that when stiffeners are not welded to the tension flange of a flexural member that is subjected to repeated loads, the shear in the web of the member should be considered. This consideration need not include the effect of shear as a separate factor but only as it affects the principal tensile stress in the web of the member.

EFFECT OF COVER PLATES ON FATIGUE BEHAVIOR

Because attachments to the tension flange of a beam change its geometry and produce stress concentrations, it can be expected that such attachments will also change the fatigue behavior of the member at the sections where such attachments are affixed. A partial-length cover plate on a beam, for example, can be expected to have a marked effect on the fatigue behavior of the beam.

Recently, studies were made at the University of Illinois on a variety of cover and flange plate details. The end details used for the cover plates are shown in Figure 16 and flange transitions in Figure 17. Varying the width or thickness of the flange plate in the manner shown in Figure 17 provides a more uniform transfer of stress than does the cover plate and can be expected to have a greater fatigue resistance than that of the member with a partial-length cover plate. However, the partial-length cover plate is a simple, economical, and effective means of providing an increase in section modulus.

Only a limited number of tests have been conducted on members with various cover plate details. Nevertheless, the data from these tests have been sufficient to provide S-N curves for each of the details and a loading cycle of zero to tension. A summary of these data (Table 4) shows that there is a relatively large variation in the fatigue resistance of the members with various cover plate or flange plate details. At 2,000,000 cycles the fatigue strengths were found to range from 11,300 to 14,500 psi for partial-length cover plates. However, the flange transitions provided a fatigue strength of approximately 19,000 psi.

A study of Table 4 indicates that any means used to make the change in cross-section more gradual is effective in increasing the fatigue strength of the beam under repeated loads. At the higher loads, the beams that exhibited the best behavior were those with the type F cover plate; this type of end detail combined the beneficial effects of tapering the end of the cover plate and of eliminating trans-

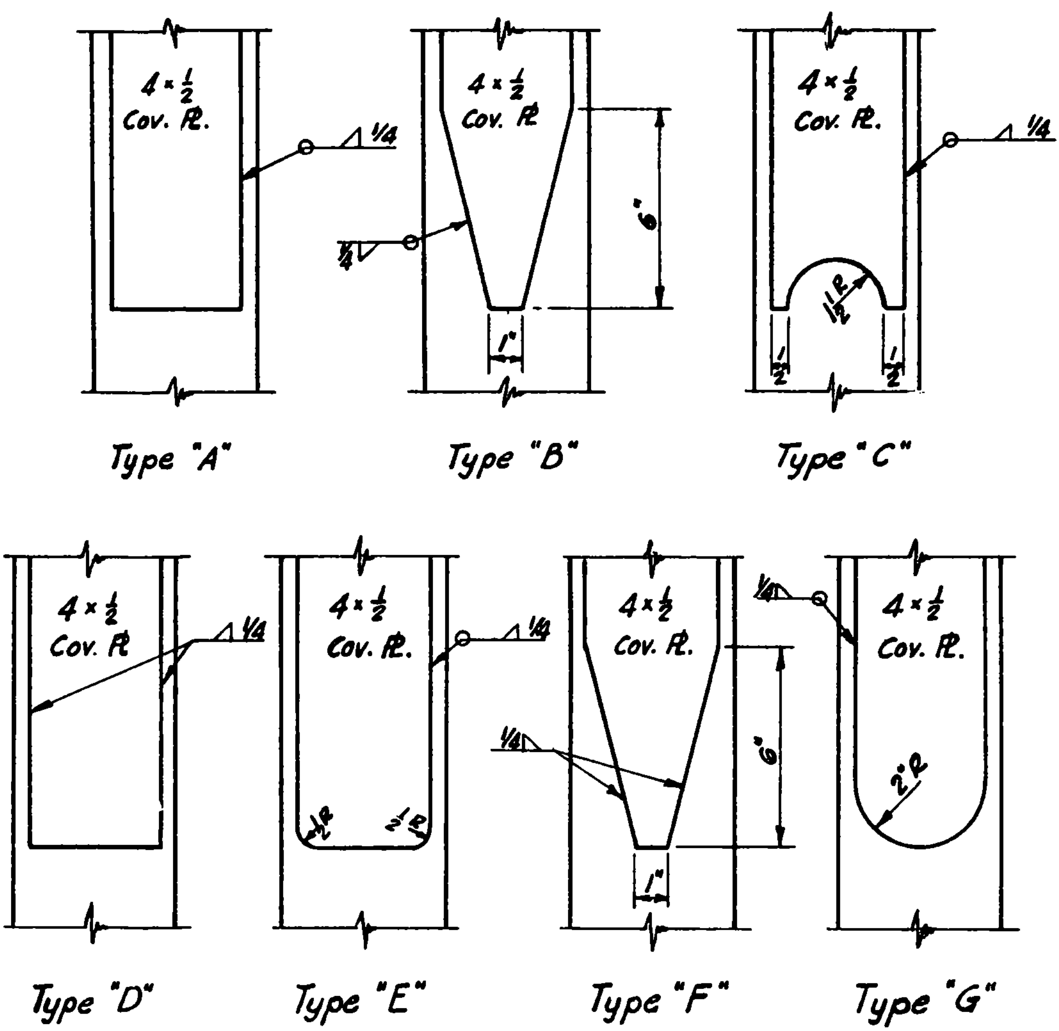


Figure 16. Details of various types of cover plates.

TABLE 4

SUMMARY OF FATIGUE STRENGTHS OF BEAMS WITH VARIOUS CHANGES IN FLANGE AREA
(Zero-to-Tension Cycle)

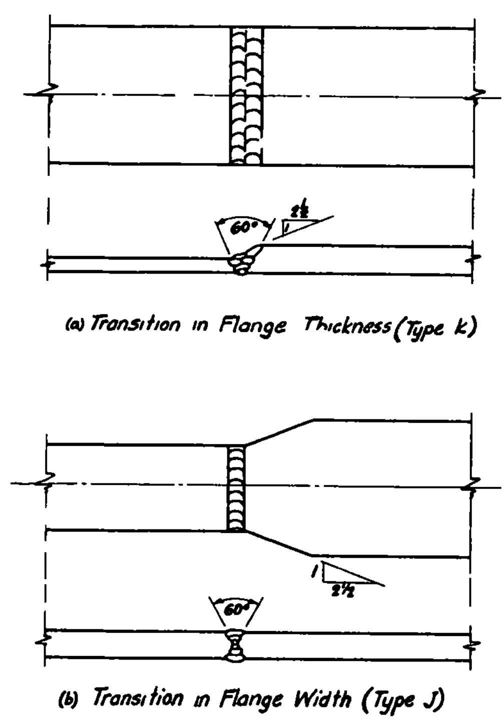
Description	$F_{100,000}$ psi	$F_{2,000,000}$ psi
Partial-length cover plates, square ends with continuous weld all around (Type A)	26,500	11,300
Partial-length cover plates, tapered ends with continuous weld all around (Type B)	34,000	11,400
Partial-length cover plates, concave profile with continuous weld all around (Type C)	30,700	14,500
Partial-length cover plates, square ends with continuous welds along edges only (Type D)	34,700	12,100
Partial-length cover plates, tapered ends with continuous weld along edges only (Type F)	37,800	13,400
Partial-length cover plates, convex profile with continuous weld all around (Type G)	29,400	11,600
Butt-welded flange transition, tapered in width (Type J)	34,900	19,500
Butt-welded flange transition, tapered in thickness (Type K)	34,600	18,500

verse welds. At the lower loads (longer life), beams with the type C end detail gave the best results. However, the difference in fatigue behavior of members with these various details was not great.

The omission of the transverse welds at the ends of the cover plates appears to provide an increase in the fatigue resistance of the members. Nevertheless, the most effective means of increasing the fatigue resistance of the beams of variable section was to vary the flange in width or thickness, in the manner shown in Figure 17. In this way, a fatigue resistance almost as great as that of a spliced beam was obtained, again demonstrating the advantage of reducing the stress concentration to a minimum when fatigue is involved.

RELATIONSHIP BETWEEN FATIGUE BEHAVIOR AND DESIGN SPECIFICATIONS

A variety of welded details and their effect on the fatigue behavior of a welded beam have been discussed. Although the test data considered herein provide a general indication of the fatigue behavior of welded beams and girders, it is evident



(a) Transition in Flange Thickness (Type K)

(b) Transition in Flange Width (Type J)

Figure 17. Weld details for flanges of beams with flange-plate transitions.

that the results in individual S-N diagrams or from a single series of tests can not provide sufficient information to relate directly the behavior of the members of all field or service conditions. However, effective extrapolations can be made to service conditions.

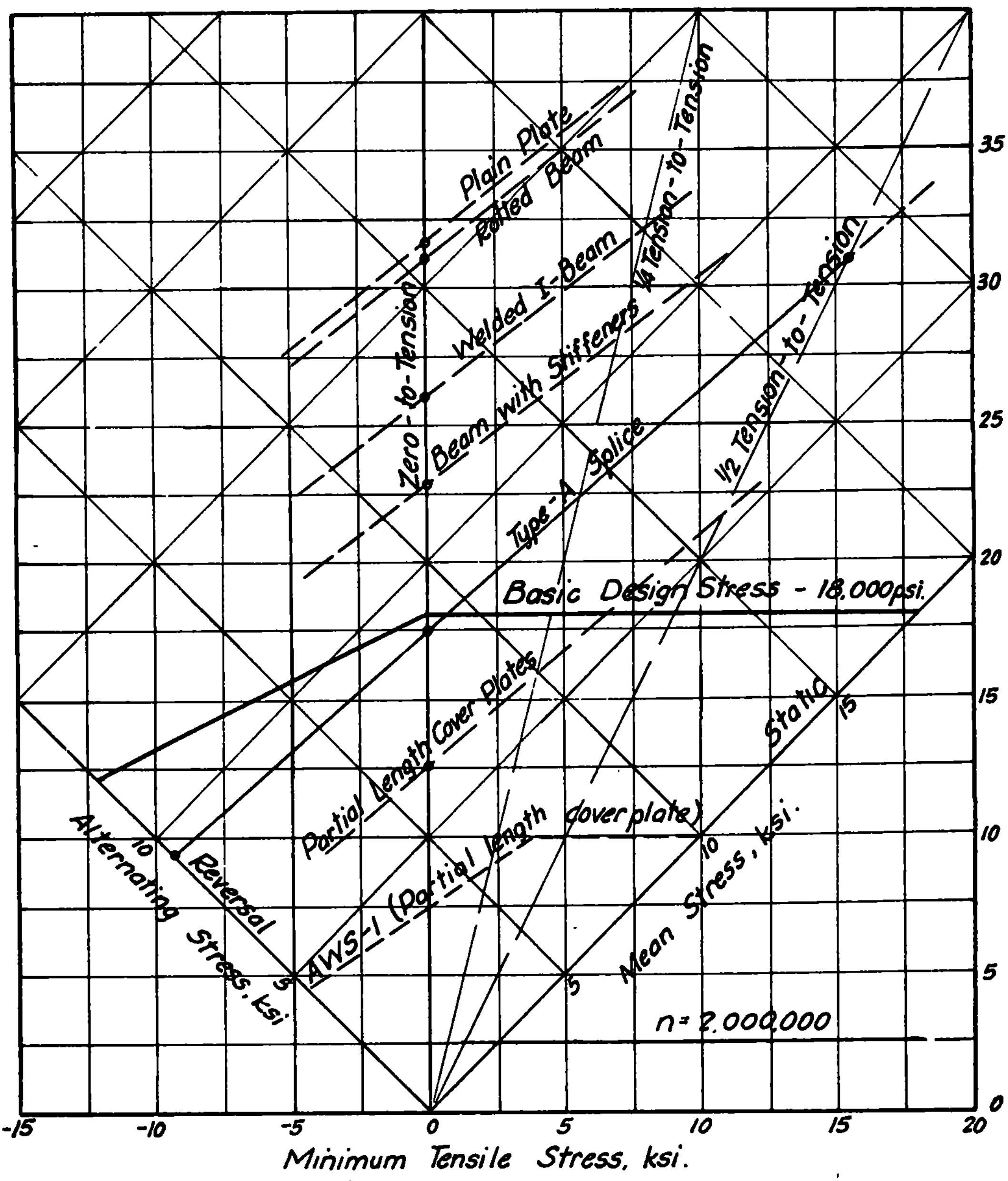


Figure 18. Relationship between fatigue strength of welded beams with basic design stresses.

All of the weld details are known to reduce the fatigue resistance of a welded beam or girder below that of the basic material. However, the degree of this reduction depends on the magnitude of the stress concentrations resulting from the details. A summary of the approximate fatigue strengths of welded beams with various details is presented in Table 5. Here it may be seen that most of the details, although decreasing the fatigue strength of the basic material, provide members with a relatively high fatigue resistance; only the partial-length cover plate produces a major reduction in the fatigue strength of the material.

The data of Table 5 provide an indication only of the fatigue strength of the members at 2,000,000 cycles and for a zero-to-tension loading cycle. To understand more fully the general fatigue behavior of the members, it is necessary to consider what might be expected under other loading conditions. Data are not available to describe directly the behavior of the members under various loading conditions but, by means of relationships such as shown in Figure 18, it is possible to relate approximately the behavior of welded flexural members to the maximum stresses employed in design.

In Figure 18, the basic tensile design stresses for bridges are compared with the results of the fatigue tests discussed herein. The data demonstrate that plain welded beams and those with stiffeners should not fail in fatigue under any conditions of loading, so long as the basic design stresses are not exceeded. However, when splices or partial-length cover plates are used, further reductions in design stress may be necessary for members subjected to repeated loads; the magnitude of the reduction depending on the expected life and the loading conditions.

TABLE 5

SUMMARY OF APPROXIMATE FATIGUE STRENGTHS OF WELDED BEAMS WITH VARIOUS DETAILS
(Zero-to-Tension Cycle)

Member or Section	$F_{2,000,000}$ psi	Ratio to Plain Plate
Plain plate	31,700	1.0
Rolled I-beam	31,200	0.98
Welded I-beam	26,500	0.83
Welded beam with stiffeners	23,000	0.73
Welded beam with splice	20,000	0.63
Welded beam with butt-welded flange transitions	19,000	0.60
Welded beam with partial-length cover plate	12,500	0.39

In general, welded girder highway bridges are subjected to loads that produce cycles of maximum stress ranging from $\frac{1}{4}$ or $\frac{1}{2}$ tension to tension. Under such loading conditions, all but members with partial-length cover plates and possibly members with splices would appear to have adequate fatigue capacity at the basic design stresses for 2,000,000 cycles of loading. However, the magnitude of the factor of safety for the various types of members varies considerably. In the case of members with partial-length cover plates or those with splices, lower design stresses are necessary to provide a suitable factor of safety.

The current provisions of the AWS specifications for welded highway and railway bridges provide the following reduced allowable unit stresses for members with partial-length cover plates and subjected to 2,000,000 cycles or more of loading.
AWS Formula 1 (tension in member adjacent to fillet welds):

$$A = \frac{\text{Max.} - 2/3\ \text{Min.}}{7,500} \quad \text{or} \quad s = \frac{7,500}{1 - 2/3\,k}$$

$$\text{but} \geq \frac{\text{Max.}}{10,000} \qquad \text{but} \leq 10,000$$

$$\text{where } k = \left(\frac{\text{Min.}}{\text{Max.}}\right)$$

This relationship, shown in Figure 18 as AWS-1, provides safety against failure in members with partial-length cover plates for all types of stress cycles. The relationships shown in Figure 7 provide for members with splices.

On the basis of the data discussed herein it can be seen that when the current specifications are correctly applied, welded beams and girders of highway bridges, if properly fabricated, should have adequate fatigue capacity. However, in view of the web failures observed in the laboratory tests, further consideration should be given to the effect of shear (principal tensile stresses in the webs) on the fatigue behavior of members with thin webs.

ACKNOWLEDGMENTS

The fatigue tests described herein were conducted in the structural laboratories of the Civil Engineering Department at the University of Illinois under the sponsorship of the Bureau of Public Roads, Department of Commerce. The tests were conducted by B. J. Goodall, W. E. Fisher, N. G. Kouba and L. R. Hall, research assistants in Civil Engineering at the University of Illinois.

The Fatigue Committee of the Welding Research Council acted in an advisory capacity in the planning of the program. However, the views expressed are those of the authors and do not necessarily reflect the views of the sponsor or the Fatigue Committee.

REFERENCES

1. "Standard Specifications for Welded Highway and Railway Bridges." American Welding Society (1956).
2. Wilson, W. M., "Flexural Fatigue Strength of Steel Beams." Univ. of Ill. Engineering Experiment Station, Bull. 377 (1948).
3. Wilson, W. M., and Munse, W. H., "The Fatigue Strength of Various Details Used for the Repair of Bridge Members." Univ. of Ill. Engineering Experiment Station, Bull. 382 (1949).
4. Wilson, W. M., Munse, W. H., and Snyder, I. S., "Fatigue Strength of Various Types of Butt Welds Connecting Steel Plates." Univ. of Ill. Engineering Experiment Station, Bull. 384 (1950)
5. Stallmeyer, J. E., Munse, W. H., and Goodal, B. J., "Behavior of Welded Built-Up Beams Under Repeated Loads." Welding Jour. 36: No. 1 (Jan. 1957).
6. Conyers, A. L., and Ozell, A. M., "Transfer of Stresses in Welded Cover Plates." Department of Civil Engineering, Univ. of Florida, Preliminary Report (Feb. 1959)
7. Hall, L. R., and Stallmeyer, J. E., "The Fatigue Strength of Flexural Members." Department of Civil Engineering, Univ. of Ill., Preliminary Report (April 1959)
8. Fisher, W. E., and Stallmeyer, J. E., "Behavior of Welded Built-Up Beams Under Repeated Loads." Department of Civil Engineering, Univ. of Ill., Preliminary Report (March 1958).
9. Kouba, N. G., and Stallmeyer, J. E., "The Behavior of Stiffener Beams Under Repeated Loads." Department of Civil Engineering, Univ. of Ill., Preliminary Report (April 1959).