Lehigh University
Lehigh Preserve

Fritz Laboratory Reports | Civil and Environmental Engineering

1972

Fatigue strength of welded steel beam details and design considerations, Summary, January 1972 (72-5)

John W. Fisher

OFFICE

OF

RESEARCH

LEHIGH UNIVERSITY

FATIGUE STRENGTH OF WELDED
STEEL BEAM DETAILS AND
DESIGN CONSIDERATIONS

by

John W. Fisher

January 1972

Fritz Engineering Laboratory Report No. 358.34

FATIGUE STRENGTH OF WELDED STEEL BEAM DETAILS AND DESIGN CONSIDERATIONS

by

John W. Fisher

Office of Naval Research
Contract N 00014-68-A-0514

January 1972

Fritz Engineering Laboratory Report No. 358.34

FATIGUE STRENGTH OF WELDED STEEL

BEAM DETAILS AND DESIGN CONSIDERATIONS

by

John W. Fisher
Professor of Civil Engineering
Lehigh University
Bethlehem, Pennsylvania

SUMMARY

This paper summarizes some of the findings of a comprehensive study on the fatigue strength of rolled and welded built-up beams without attachments, rolled and welded beams with cover plates, and welded beams with flange splices. Altogether, 374 steel beams with two or more details were fabricated and tested.

The welded beam details discussed herein represent the upper and lower boundaries of fatigue behavior of welded beams. The lower bound is provided by beams with partial length cover plates - a severe notch producing detail. The upper bound is provided by the plain-welded beam - a minimum notch producing detail.

For purposes of design, this study has shown that the fatigue strength of the upper and lower bound details is independent of the strength of steel. A36, A441 and A514 steel beams provided the same fatigue strength for a given detail, and stress range was observed to account for nearly all the variation in cycle life.

This paper reviews briefly the major variables that influence the fatigue strength of welded details and suggests how they should be considered in design. The fracture mechanics of crack propagation is reviewed and used to focus on the major design factors. Particular attention is given to the initial flaw condition (which exists in all joints), the governing stress variables, and the influence of geometry.

1. INTRODUCTION

In the present design (1972) of steel bridges for fatigue, provisions have been specified in many instances on the basis of limited test data. Previous work had not adequately investigated the behavior of beams in terms of stress, detail and type of steel. The effects of variables such as stress, stress ratio, cover-plate geometry, details and type of steel were not clearly defined.[2,3,4] Equally important to the fatigue life of highway bridges is the significance of such factors as the loading history to which the struatures are subjected, the type of materials used, the design details, and the quality of fabrication.

Recognition of these facts lead to the bridge studies on the AASHO Road Test[1] and the development of a comprehensive study of rolled and welded built-up beams with a variety of welded details. This program was sponsored by the American Association of State Highway Officials in cooperation with the Bureau of Public Roads, U. S. Department of Transportation under the National Cooperative Highway Research Program which is administered by the Highway Research Board of the National Academy of Sciences.*

The major objective of this study was to develop suitable mathematical design relationships for between 50,000 and 10 million cycles of loading. Altogether, 374 beams with one or more details were studied. The principal design variables for this study were grouped into three categories: (i) type of weld detail, (ii) stress condition, and (iii) type of steel.

Three grades of steel were examined - ASTM A36, A441 and A514 - for the details discussed in this paper.

Four different types of beam details were examined including coverplated beams, plain-welded beams without any attachments, welded beams with groove-welded flange splices and plain-rolled beams.[5] Only two details are discussed in this report. One detail is a rolled or welded steel I beam with cover plates which provided a severe notch producing detail with crack growth from the weld toe and a lower-bound to fatigue behavior. The second detail discussed is the plain welded beam without attachment which provided a minimum notch producing detail with crack growth from an internal flow and an upper bound for the behavior of welded beams.

---

* This study was conducted under National Cooperative Highway Research Program Project 12-7. The opinions and findings expressed or implied in this paper are those of the author. They are not necessarily those of the Highway Research Board, the National Academy of Sciences, the Bureau of Public Roads, the American Association of State Highway Officials, nor of the individual states participating in the National Cooperative Highway Research Program.

Details are given in the Final Report of the study on the effect of weldments on the fatigue strength of steel beams.[5]

Figure 1 shows schematically the basic details considered in the cover-plated beam study. Cover plates were attached to both rolled and welded beams. The steel beams in the CR-CW series had 11.4 cm.(4-1/2 in.) wide cover plates which had 1.5 times the flange thickness. Other cover-plated beams were examined with thicker, wider, or multiple cover plates, but all these were fabricated from A36 steel.

The plain-welded beams were identical in cross section to the cover-plated beams and were fabricated using the same technique.

All longitudinal fillet welds were made by the automatic submerged arc process. Tack welds and the transverse end welds on the cover plates were manual welds. All beams in this study were 10 ft.-6 in. long and were tested on a 10 ft. span. The ends of the cover plate details were positioned in the shear spans 12 in. from each load point. The plain-welded beams were loaded so that a 42-in. constant moment region resulted in the center.

Minimum stress and stress range were selected as the controlled stress variables. This permitted variation in one variable while the other was maintained at a constant level. Had stress rate (ratio of minimum to maximum stress) been selected as the independent variable, both minimum stress and maximum stress would have to be changed simultaneously to maintain the ratio at a constant level.

The controlled stress levels were the nominal flexural stresses in the base metal of the tension flange at the end of the cover plate, and at points of maximum moment for the plain-welded beams.

2. INITIAL FLAW CONDITIONS

Experimental fatigue studies have nearly all confirmed that fatigue crack growth commenced at some initial 'flaw or defect'. In unwelded steel this may be at small mechanical notches, discontinuities in mill scale, surface imperfections and laminations or from gas cut edges. In welded structures small, sharp defects exist at the weld periphery or in the weldment of both fillet and groove welds, and crack growth has invariably started at the weld periphery or at an internal flaw depending on the direction of applied stress.

Signes et al.[14] have shown that fatigue cracks initiated in many details at the toes of fillet welds from small, sharp intrusions of slag that emanated from the welding flux or plate. These findings were further confirmed by Watkinson et al.[19] who showed that these defects exist in most conventional welding processes.

All experimental evidence has confirmed that crack growth does normally initiate at the toe of a weldment starting from the initial micro-flaw when the applied stress is perpendicular to it. Figure 2 shows the cracks which formed in the beam flange at the toes of both longitudinal and transverse fillet welds that connected the cover plate to the beam flange.

In some types of joints, however, fatigue cracks may initiate at points other than the weld toe. For example, in joints involving transverse load-carrying fillet welds cracking can initiate at the weld root with propagation through the weld. Provided that the welds are sufficiently large and their geometry satisfactory, joints will also experience crack growth and failure from the toe. Generally joints failing from internal defects have a relatively high fatigue strength. This was the case for the plain-welded beams. Cracks causing failure initiated at a flaw in the longitudinal fillet welds joining the web to the flanges. Typical initial flaws are porosity (gas pockets) as illustrated in Figure 3. Other sources of crack growth are at start-stop positions or weld repairs where incomplete fusion or trapped slag exists.[2,3,5] Cracks starting at porosity were initially completely inside the weld and were not visible from the surface until substantial crack growth had occurred.

In as-welded groove welds the stress concentration at the weld toe, with its associated small toe defects, is usually more severe than that caused by other minor internal flaws. However, if lack of penetration, slag inclusions and other internal flaws are comparatively large in size, crack growth can become more critical at those locations.[8] It has been common practice in bridge construction to provide non-destructive testing of groove welds so that the internal flaw can be minimized in size. Also, the weld reinforcement is often removed so that the stress concentration and toe flaw are minimized in which case, of course, internal defects become more critical.

Flaws may also be critical at the flame-cut edges of plates. Occasionally, a severe notch results from the gas torch as illustrated in Fig. 4. Reference 5 has shown that for beams with flame-cut edges with an ASA roughness of 1000 or less, failure would result from arc weld defects in the flange to web fillet weld. Other studies have shown that poor quality cutting can result in substantial reductions in fatigue strength of structural elements.[1] Obviously, occasional flaws of severity that result from blowbacks or other causes should be removed by grinding.

3. FATIGUE STRENGTH OF COVER-PLATED BEAMS

The results of tests on cover-plated beams provided data for a severe notch producing detail so that a lower bound to fatigue strength could be examined. When a transverse end weld existed, the crack initiated at the toe of the transverse weld as was illustrated in Fig. 2. During the first stage of growth, the crack grew through the flange in an elliptical shape as illustrated in Fig. 5. Thereafter, it grew toward the flange tips and into the web. Cracks initiated at the toe of the longitudinal fillet welds connecting the cover plate to the beam when no transverse end weld was present. These cracks also grew through the flange in an elliptical shape and were similar to the first stage of growth exhibited by cracks at the end with a transverse end weld.

The effects of the controlled variables of minimum stress, stress range and type of steel were analyzed using statistical methods. The dominant variable was stress range for all cover-plate geometries, end details and steels tested.

Figure 6 summarizes the test data for A36 steel beams with an end weld. Cycle life is plotted against stress range for different levels of minimum stress. Also shown are the mean regression line and the limits of dispersion as given by two standard errors of estimate. It is apparent that the variation due to minimum stress is insignificant and that stress range accounted for the variation in cycle life. The mathematical relationship between the applied stress range and cycles to failure for each series and geometry was determined using regression analysis. The analysis showed that the logarithmic transformation of stress range and cycle life provided the best fit to the data.

The effect of type of steel was also evaluated since the experiment design provided equal factorials and sample sizes of A36, A441 and A514 steel beams with cover plates. The test data for all three types of steel cover-plated beams are compared in Fig. 7. The A514 steel beams yielded only a slightly longer life. The variation in life due to type of steel is too small for consideration in the design of structures.

The results of this study have been compared with the earlier work of Wilson,[6] Lea and Whitman,[7] and Munse and Stallmeyer[8] for both end details. The limits of dispersion provided by two standard errors of estimate included almost all the data. Most of the test points falling below the lower limit of dispersion were from the early studies of Wilson.[6] This should be expected since much larger flaw conditions resulted from early welding procedures.

This study has confirmed that no great differences exist in the fatigue strength of square ended cover plates; that cover plates affect rolled and welded beams similarly; that welded cover-plated beams yield about the same fatigue strength for A36, A441 and A514 steels, and that only stress range is the critical stress variable. Greater detail is given in Ref. 5.

4. FATIGUE STRENGTH OF WELDED BEAMS WITHOUT ATTACHMENTS

The results of the tests on plain-welded beams provided a minimum notch producing detail and an upper bound to the fatigue strength of welded beams. Nearly all cracks initiated at a flaw in the fillet weld at the flange-to-web connection as illustrated in Fig. 3. The fillet weld flaw was usually a gas pocket or worm hole in the fillet weld caused by gas trapped in the weldment. Cracks were initially inside the weld as shown in Fig. 8, but eventually grew out to the fillet weld surface. They maintained a circular shape as shown in Figs. 3 and 8 until they penetrated the outside flange surface. After penetrating the outside fibers of the flange, the crack grew on two fronts toward the flange tips as well as up the web.

The effect of the design factors, minimum stress and stress range are illustrated in Fig. 9. Stress range is observed to account for the variation in cycle life. Multiple regression analysis also indicated that the logarithmic transformation of both stress range and cycle life provided the best fit to the data.

Residual stresses were measured in several of the welded shapes, and all indicated the presence of large tensile residual stresses in the vicinity of the flange-to-web fillet welds. As the strength of the steel increased, there was greater probability of the compression flange being subjected to the full tensile stress range in the vicinity of the weld since the residual stresses were about equal to the yield stress. The presence of residual tensile stresses also accounts for the behavior of the welded beams without attachments and coverplated beams and the fact that their fatigue behavior can be expressed in terms of stress range.

As was the case with cover-plated beams the experiment design provided equal sample sizes of plain-welded A36, A441 and A514 steel beams. The test data for all three types are identified in Fig. 10. There was no statistically significant difference due to type of steel. All the variation was due to stress range.

The test results were compared with the previous studies by Gurney[10] and Reemsnyder[11] on welded beams fabricated by automatic welding procedures. The test data indicating failure at accidental start-stop positions fall between the lower limit of dispersion and the mean. Beams with cracks initiating at flaws in the fillet weld fall between the mean and the upper limit of dispersion. Beams not failing at well defined weld flaws were similar to plain rolled beams and tended to provide the longest life. The A514 steel beams and T-specimens tested by Reemsnyder fall near the upper limit of dispersion as might be expected considering their careful fabrication and the fact that they were only subjected to constant stress over an 8-in. length which reduced the probability of a large flaw within the maximum stress region.

Details of the experimental work and comparisons with the earlier work are given in Refs. 5 and 12. Although not discussed in this summary, work was also done on plain-rolled beams and on welded beams with the reinforcement removed from groove-welded flange splices.

5. STRESS ANALYSIS OF CRACK PROPAGATION

In welded joints the small sharp initial defects at the weld toe and the internal flaws in the weldment (porosity) can be considered as small cracks so that the need for a crack initiation period is effectively eliminated and the life of a welded detail is taken up with crack propagation.[14,19]

For example, at the ends of the longitudinal or transverse fillet weld connecting a cover plate to a beam flange, the toe cracks propagated in a semi-elliptical shape through the flange as illustrated in Fig. 5. The internal flaws in the web-flange connection of a welded beam have taken the shape of a circle and maintained that shape until the bottom surface of the flange was completely penetrated as shown in Fig. 3.

For the prediction of crack propagation the relationship proposed by Paris[13] was used. The relationship

$$C \Delta K^n = da/dN \tag{1}$$

expresses the change in crack length per stress cycle da/dN, as a function of the range of stress intensity factor, ΔK, during that cycle and a constant of proportionality C. The K value accounts for the stress field in the vicinity of a crack tip and was introduced by Irwin.[20]

$$K = \sigma \sqrt{\pi a}\, f(a) \tag{2}$$

Since K is determined by crack size, geometry and the nominal stress and can be derived analytically, it provides a means of determining the influence of geometry and nominal stress upon the characteristics of propagation of the crack. The applicability of Eq. 1 has been illustrated for many types of materials and geometric configurations.[13,15,16,21,22]

Equation 2 can be substituted into Eq. 1 and integrated between the limits of applied cycles N_i and N_f corresponding to the values of the crack size at initiation (a_i) and failure (a_f).* This yields

$$A\, S_r^n\, \Delta N \;=\; a_i^{-\alpha} - a_f^{-\alpha} \qquad (3)$$

where S_r is the applied stress range, $\Delta\sigma = \lambda S_r$ is assumed constant, λ is the stress concentration factor, $A = C\pi^{n/2} d\lambda^n f^n(a)$, $\Delta N = N_f - N_i$, and $\alpha = n/2 - 1$. Recent studies have indicated that the assumption $\Delta\sigma = \lambda S_r$ is a reasonable approximation when evaluating the stress intensity factor at the weld toe.[22] Since the value of the final crack size, a_f, is large, the last term in Eq. 3 can be neglected. Equation 3 then can be expressed as

$$\Delta N \;=\; \frac{1}{A}\, a_i^{-\alpha}\, S_r^{-n} \qquad (4)$$

The relationship between life ΔN and the applied stress range is exponential in form which agrees with the results of the regression analysis.

The quantity of $(1/A)\,(a_i^{-\alpha})$ in Eq. 4 is a measure of the stress concentration effects and initial flaw size of each type of specimen and detail.

---

* If f(a) is a variable correction factor, numerical procedures must be employed.

$$(1/A)\ (a_i^{-\alpha}) = \frac{1}{[C\ \pi^{n/2}\ \alpha\ \lambda^n f^n(a)]\ a_i^{\alpha}} \tag{5}$$

It is therefore inversely dependent upon the initial crack size, a_i, the constant of crack growth, C, the geometrical correction, f(a), and the stress concentration factor, λ. The cover-plated beams represented the most severe condition of these parameters.

The value of the exponent, n, was observed to vary between 2.80 and 3.10 for cover plates attached to A36, A441 and A514 steel beams. Variation in yield stress from 36 ksi to 100 ksi caused a negligible change in the exponent. The variation in mean value of the exponent between the cover-plated beams and the plain-welded beams was from 2.80 to 3.33. A value of n = 3 was selected since it provided a reasonable fit to the experimental data.

Thus, Eq. 3 can be expressed as

$$\Delta N = (1/A)\ (a_i^{-1/2})\ S_r^{-3} \tag{6}$$

The crack growth in the cover-plated beams was analyzed in Ref. 5. The cracks were observed to be semi-elliptical in shape (see Fig. 5) with a ratio of a to b that remained constant and equal to 2/3. The crack size at different numbers of cycles for the unwelded end of the cover plate was determined by measuring the size at several stages. The stress intensity factor K for a semi-elliptical surface crack as developed by Irwin[17] was used along with the more accurate secant correction for finite width.[18] Equation 6 was used to evaluate the parameter A and the part containing the initial crack size, $a_i^{-1/2}$. This yielded $a_i^{-1/2} \cong 130/\sqrt{in.}$ and $A = 1.02 \times 10^{-7}/ksi^3\sqrt{in.}$ Substituting these values back into Eq. 6 gives an expression for predicting the number of cycles for the crack to propagate through the flange.

$$\Delta N = 1.28 \times 10^9 S_r^{-3} \quad (7)$$

Equation 7 is compared with the mean regression line for beams with end welded cover plates in Fig. 11. The predicted crack growth is in good agreement with the experimental results.

Crack growth in the plain-welded beams has recently been analyzed by assuming the internal flaws in the web-flange connection to grow in a penny shape.[21] This shape was maintained until the crack penetrated the outside surface of the beam flange. The average initial crack radius was taken as 0.04 in. from samples of measured pores. The correction factor f(a) in Eq. 2 for a penny-shaped crack is $2/\pi$. With the exponent $n = 3$ and the coefficient $C = 2.05 \times 10^{-10}$ (see Ref. 21) the number of cycles for the crack to propagate to the extreme fiber of the flange can be determined from Eq. 1

$$\Delta N = 2.25 \times 10^{10} S_r^{-3} \quad (8)$$

Also shown in Fig. 11 is the mean regression curve for plain-welded beams together with the model using the exponent $n = 3$. The use of $n = 3$ is seen to give good correlation for both the upper and lower bounds of the fatigue strength of welded beams.

Fracture mechanics concepts of stablé crack growth confirm the experimentally observed behavior of the welded beam details. It has verified that stress range is the major stress variable influencing the fatigue strength. Other factors of major importance are the stress concentration effects and the initial flaw size.

6. DESIGN OF WELDED DETAILS

Available experimental data on a variety of structural details and studies of the mechanics of stable crack growth have all indicated that the major factors governing the fatigue strength are the applied stress range, the stress concentration of the detail, and the initial flaw condition. These factors were observed to be critical for the two details discussed in this paper. Obviously, the greatest strength is achieved when the severity of these three conditions are minimized.

All welding processes introduce small flaws in or near the weldments. Good welding practice will minimize the number and size of these flaws but cannot eliminate them. They exist and good design practice must reflect this fact.

As far as the designer is concerned, the easiest, and most important parameter to control is the severity of the stress concentration introduced at the detail design stage. Thus, in structures which are susceptible to fatigue damage, an attempt should be made to avoid the use of fillet welded joints which involve weld toes and weld ends being situated in regions of significant cyclic stress, and if such details cannot be avoided the stress range must be reduced to accommodate them.

The cover-plated beam is a typical structural member which contains a low strength joint. Micro cracks occur at the ends of continuous longitudinal welds or at the toe of the transverse end welds, and this flaw condition is coupled with a severe change in geometry. Thus, all the factors necessary for reduced fatigue strength are present in such details. Unfortunately, many other commonly used details incorporate similar conditions as illustrated in Fig. 12. Many attachments to the tension flange exhibit fatigue strengths that are the same as the coverplated beam. Research in progress has indicated that attachments with a length of 6 to 8 inches provide the same fatigue strength as coverplated beams. Micro cracks exist at the weld toes and these are always coupled with fairly large stress concentrations because of the connection geometry. Hence, designers must provide for such details by considering the design life required and the permissible stress range associated with it.

As a general rule, structural details which involve failure from internal defects, such as porosity and slag inclusions, will have a relatively high fatigue strength. This is primarily due to the fact that such defects by themselves produce less severe stress concentrations than those formed by weld toes with their associated small, sharp defects and geometric stress concentrations. Consequently, internal defects only tend to become critical when low strength joints have been eliminated from the design except, of course, when the internal defects are particularly severe.

The 1969 AISC Specification is one of the few that recognizes that stress range is the major design factor influencing fatigue strength. This concept has recently been incorporated in several categories of the AASHO Interim Specifications of 1971. It is also known that the new British fatigue design rules, which are in preparation, will also specify the fatigue strength of welded joints in terms of stress range.

7. SUMMARY AND CONCLUSIONS

This report summarizes the results of a study on the fatigue strength of welded beams. The study has determined the significance of several design factors in a rational manner for the first time. The conclusions are based on the analysis and evaluation of the experimental data, on a study and correlation with earlier work, and on theoretical studies based on the application of continuum mechanics to crack propagation. Details of the study are given in Refs. 5 and 12.

(1) Stress range was the dominant stress variable for all welded details and beams tested.

(2) Other stress variables were not significant for design purposes.

(3) Structural steels with 36 ksi to 100 ksi did not exhibit

significantly different fatigue strength for a given welded detail.

(4) A theoretical stress analysis based on fracture mechanics for micro crack propagation substantiated the experimental model that provided the best fit to the test data.

(5) Design of welded details should consider the initial flaw condition and source of fatigue crack growth as well as the stress range and joint geometry.

ACKNOWLEDGEMENTS

The research summarized in this paper was performed under NCHRP Project 12-7 by the Fritz Engineering Laboratory, Department of Civil Engineering, Lehigh University, Bethlehem, Pennsylvania and the Department of Civil Engineering, Drexel University, Philadelphia, Pennsylvania. Lehigh University was the contractor for this study which commenced in 1966. Partial support was also provided by the Office of Naval Research, Department of Defense, under Contract N00014-68-A-514; NR064-509.

The author is indebted to Dr. Karl H. Frank and Dr. Manfred A. Hirt, former Research Assistants, and Dr. Ben T. Yen, Associate Professor at Fritz Engineering Laboratory, for their assistance and work throughout this study. Thanks are also due Dr. Bernard M. McNamee, Professor of Civil Engineering at Drexel University.

REFERENCES

1. FISHER, J. W. and VIEST, I. M.
   Fatigue Life of Bridge Beams Subjected to Controlled Truck Traffic, Preliminary Publications, 7th Congress, IABSE, pp. 497-510, 1964

2. GURNEY, T. R.
   Fatigue of Welded Structures, Cambridge University Press, 1968

3. MUNSE, W. H. and GROVER, L. M.
   Fatigue of Welded Steel Structures, Welding Research Council, New York, N. Y., 1964

4. ASCE TASK COMMITTEE ON FLEXURAL MEMBERS
   Commentary on Welded Cover-Plated Beams, Journal of the Structural Division, ASCE, Vol. 93, No. ST4, August 1967

5. FISHER, J. W., FRANK, K. H., HIRT, M. A. and McNAMEE, B. M.
   Effect of Weldments on the Fatigue Strength of Steel Beams, Final Report, NCHRP Report 102, Highway Research Board, 1970

6. WILSON, W. M.
   Flexural Fatigue Strength of Steel Beams, Bulletin No. 377, Univ. of Illinois, Urbana, Ill., 1943

7. LEA, F. C. and WHITMAN, J. G.
   The Failure of Girders Under Repeated Stresses, Welding Journal Vol. 18, January 1939

8. MUNSE, W. H. and STALLMEYER, J. E.
   Fatigue in Welded Beams and Girders, Bulletin No. 315, Highway Research Board, 1962

9. HARRISON, J. D.
   The Analysis of Fatigue Test Results for Butt Welds with Lack of Penetration Defects Using a Fracture Mechanics Approach, Welding in the World, Vol. 8, No. 3, 1970, pp. 168-181

10. GURNEY, T. R.
    Investigation into the Fatigue Strength of Welded Beams, Part II: High Tensile Steel Beams Without Stiffeners, British Welding Journal, Vol. 7, 1962

11. REEMSNYDER, H. S.
    Fatigue Strength of Longitudinal Fillet Weldments in Constructional Alloy Steel, Welding Journal, Vol. 44, October, 1965

12. HIRT, M. A., YEN, B. T. and FISHER, J. W.
Fatigue Strength of Rolled and Welded Steel Beams, Journal of the Structural Division, ASCE, Vol. 97, No. ST7, July 1971

13. PARIS, P. C.
The Fracture Mechanics Approach to Fatigue, Proc. 10th Sagamore Conf., Syracuse University Press, p. 107, 1965

14. SIGNES, E. G., BAKER, R. G., HARRISON, J. D., and BURDEKIN, F. M.
Factors Affecting the Fatigue Strength of Welded High Strength Steels, British Welding Journal, Vol. 14, March 1967

15. JOHNSON, H. H. and PARIS, P. C.
Subcritical Flaw Growth, Engineering Fracture Mechanics, Vol. 1, No. 1, June 1968

16. GURNEY, T. R.
The Effect of Mean Stress and Material Yield Stress on Fatigue Crack Propagation in Steels, Metal Construction and British Welding Journal, Vol. 1, No. 2, February 1969

17. PARIS, P. C. and SIH, G. C.
Stress Analysis of Cracks, STP No. 381, ASTM, 1965

18. IRWIN, G. R., LIEBOWITZ, H. and PARIS, P. C.
A Mystery of Fracture Mechanics, Engineering Fracture Mechanics, Vol. 1, No. 1, June 1968

19. WATKINSON, F., BODGER, P. H., and HARRISON, J. D.
The Fatigue Strength of Welded Joints and Methods for its Improvement, Proceedings, Fatigue of Welded Structures Conference, The Welding Institute, Brighton, England, Paper No. 7, July 1970, pp. 97-113

20. IRWIN, G. R.
Analysis of Stresses and Strains Near the End of a Crack Traversing a Plate, Transactions, ASME, Series E, Vol. 24, No. 3, Sept. 1957, pp. 361-364

21. HIRT, M. A.
Fatigue Behavior of Rolled and Welded Beams, Thesis presented to Lehigh University at Bethlehem, Pa. in 1971, in partial fulfillment of the requirement for the degree of Doctor of Philosophy

22. FRANK, K. H.
The Fatigue Strength of Fillet Welded Connections, Thesis presented to Lehigh University at Bethlehem, Pa. in 1971, in partial fulfillment of the requirement for the degree of Doctor of Philosophy

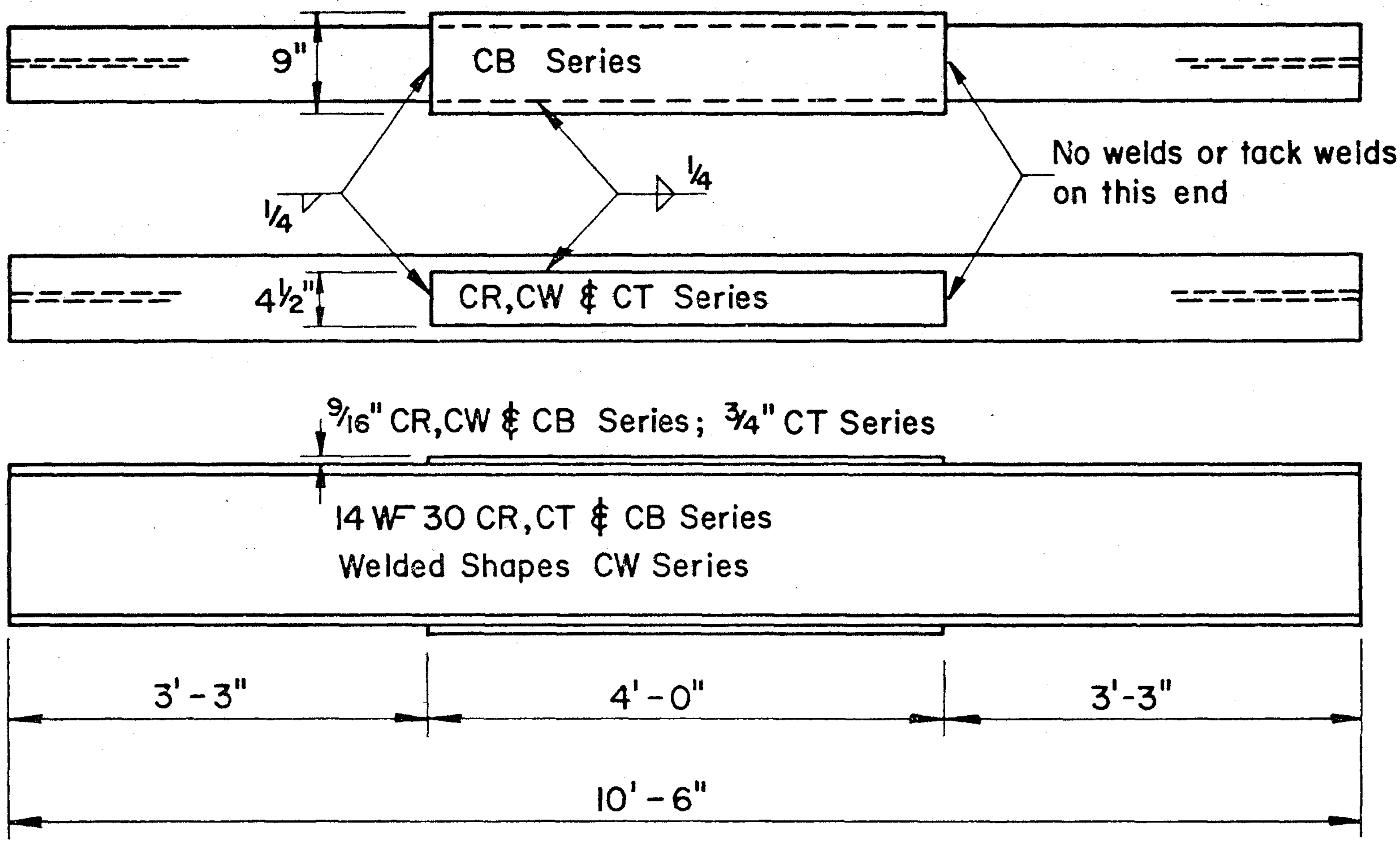

Fig. 1 Details of coverplated beams

Fig. 2a Crack formation at toe of longitudinal fillet weld

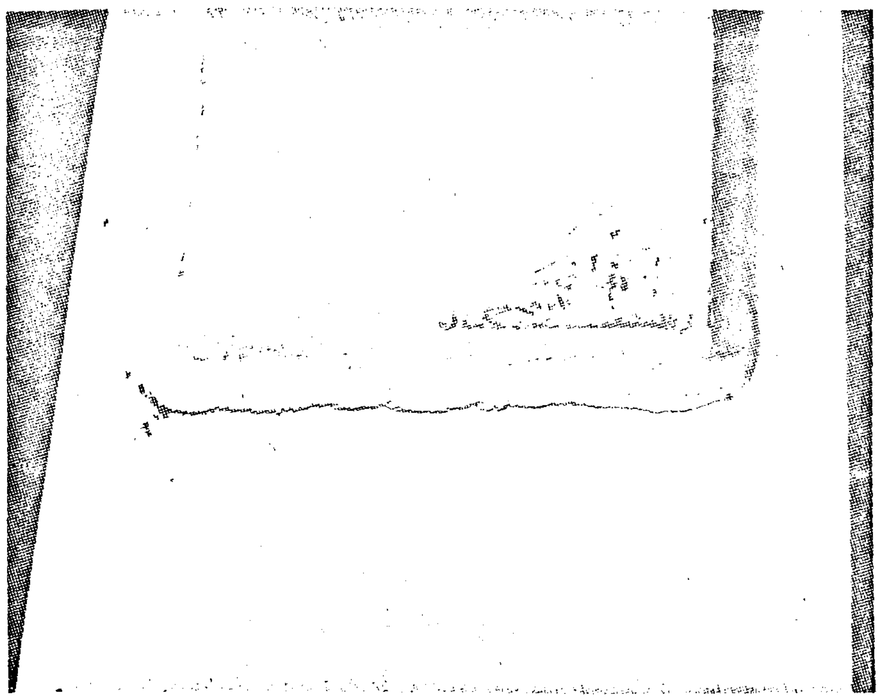

Fig. 2b Crack formation at toe of transverse fillet weld

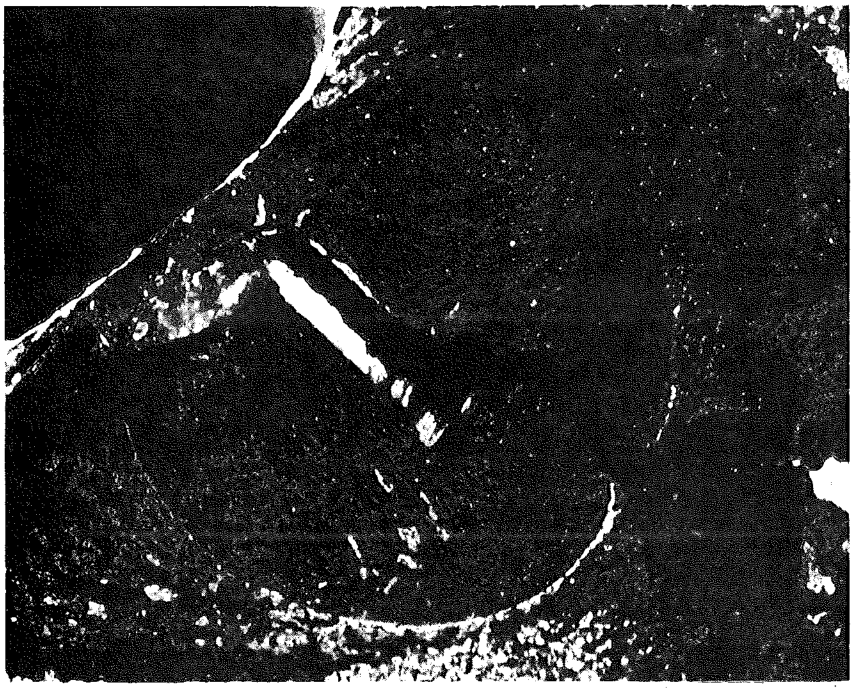

Fig. 3a Small crack with penetration to the fillet-weld surface (~x11)

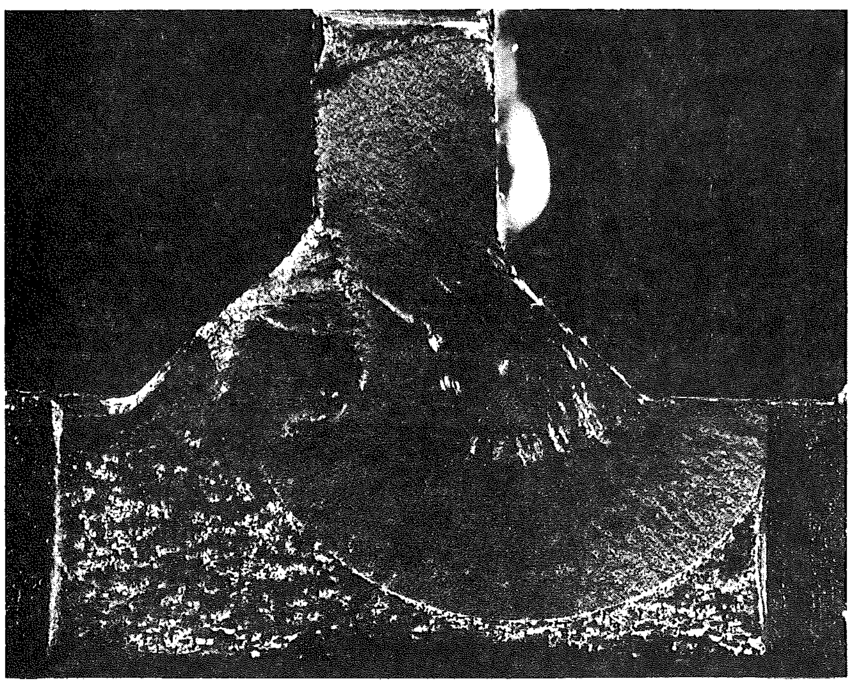

Fig. 3b Crack in flange-to-web junction approaching the extreme fibre of the tension flange (x3.6)

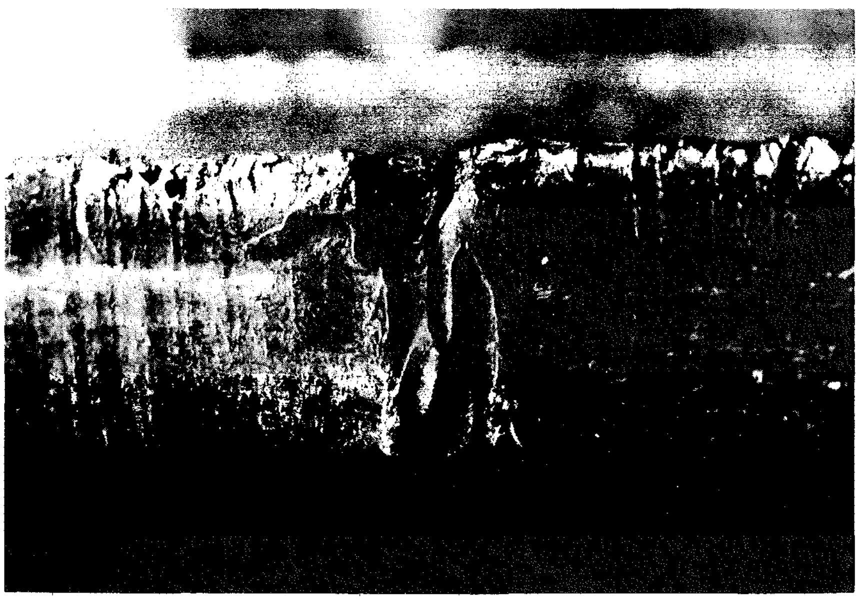

Fig. 4 Severe notch at the flame-cut flange-tip of a welded beam

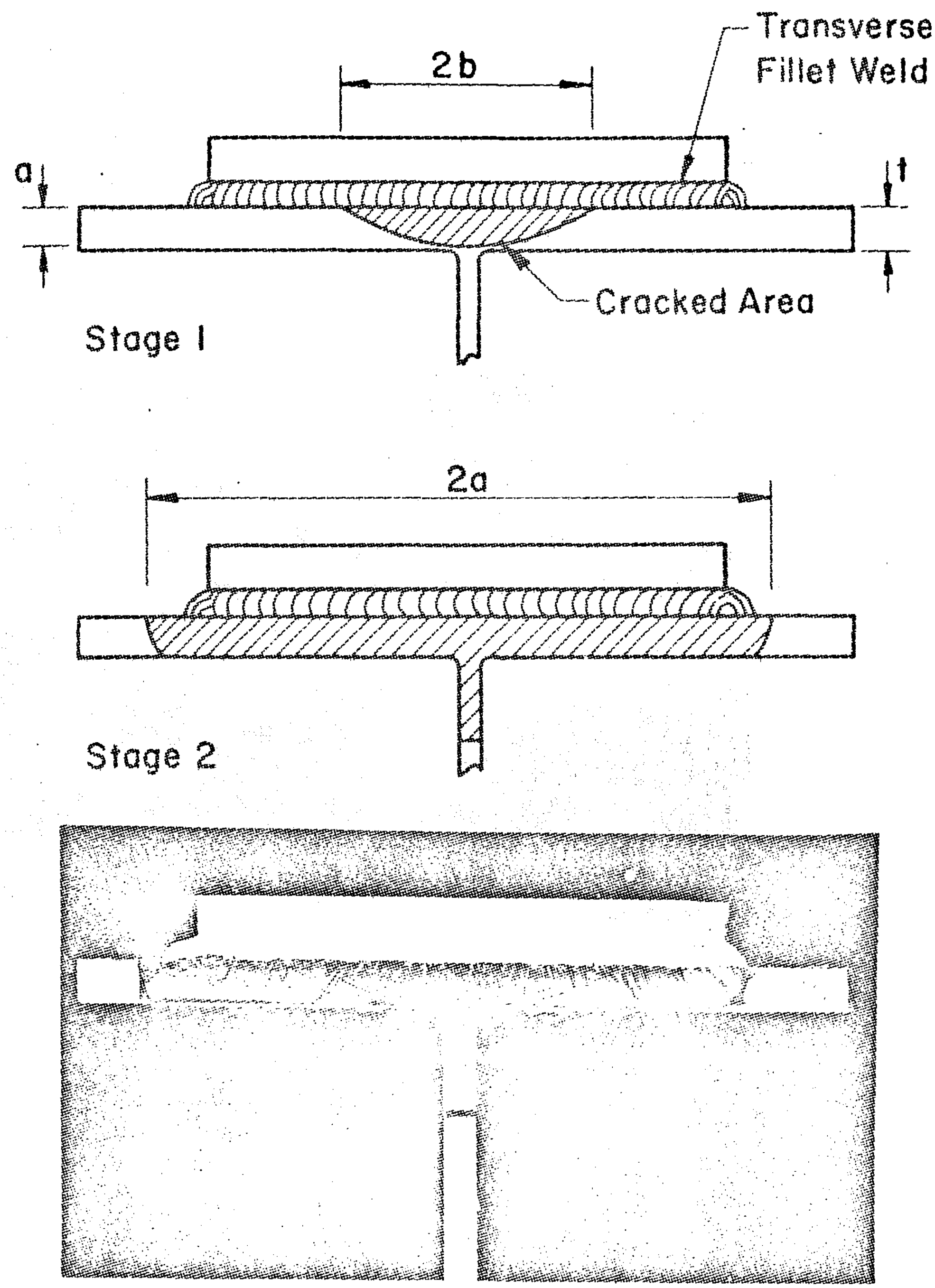

Fig. 5 Crack growth at the transversely welded end of cover-plated beams

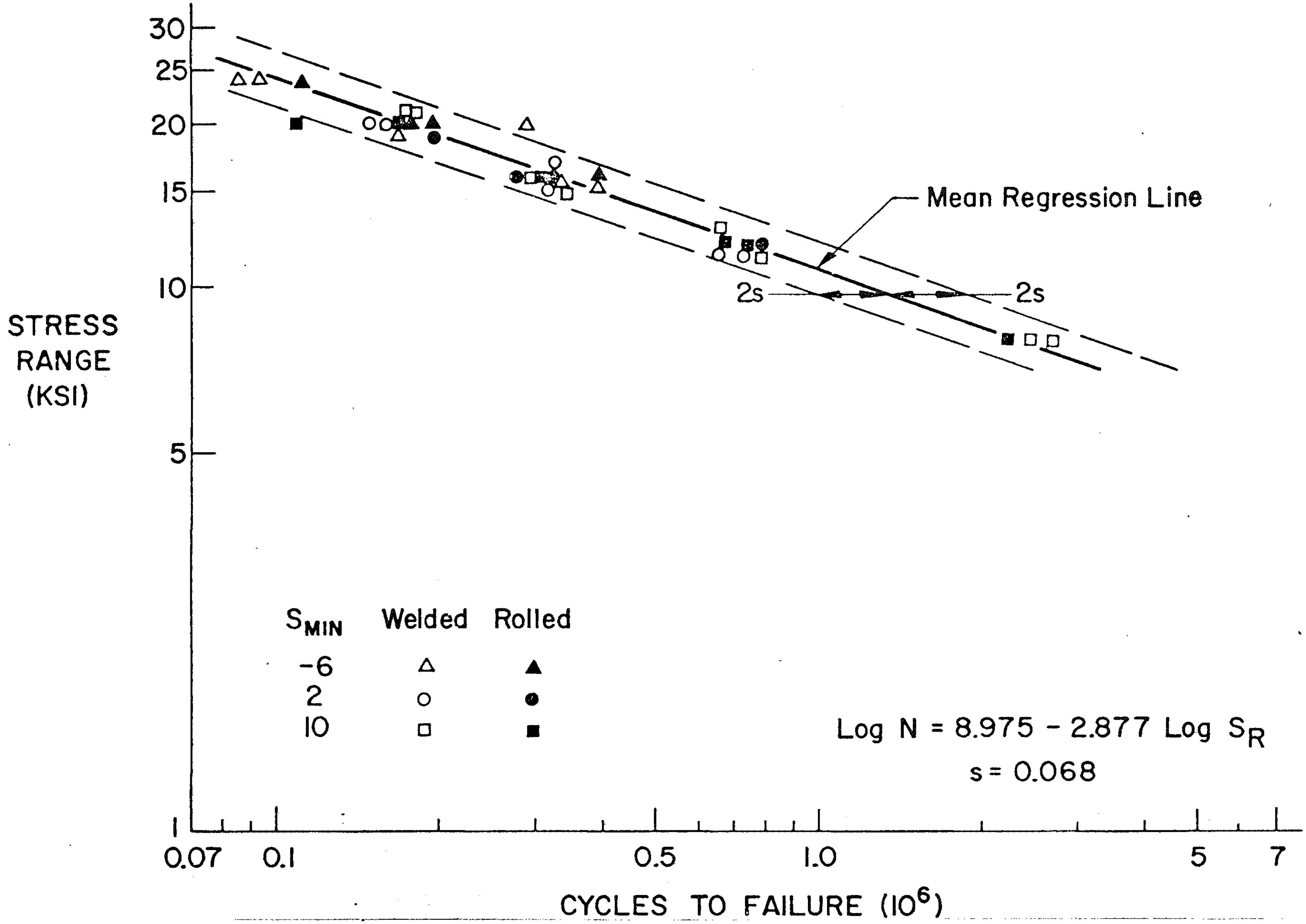

Fig. 6 Effect of stress range and minimum stress on the cycle life for the welded end of coverplated beams

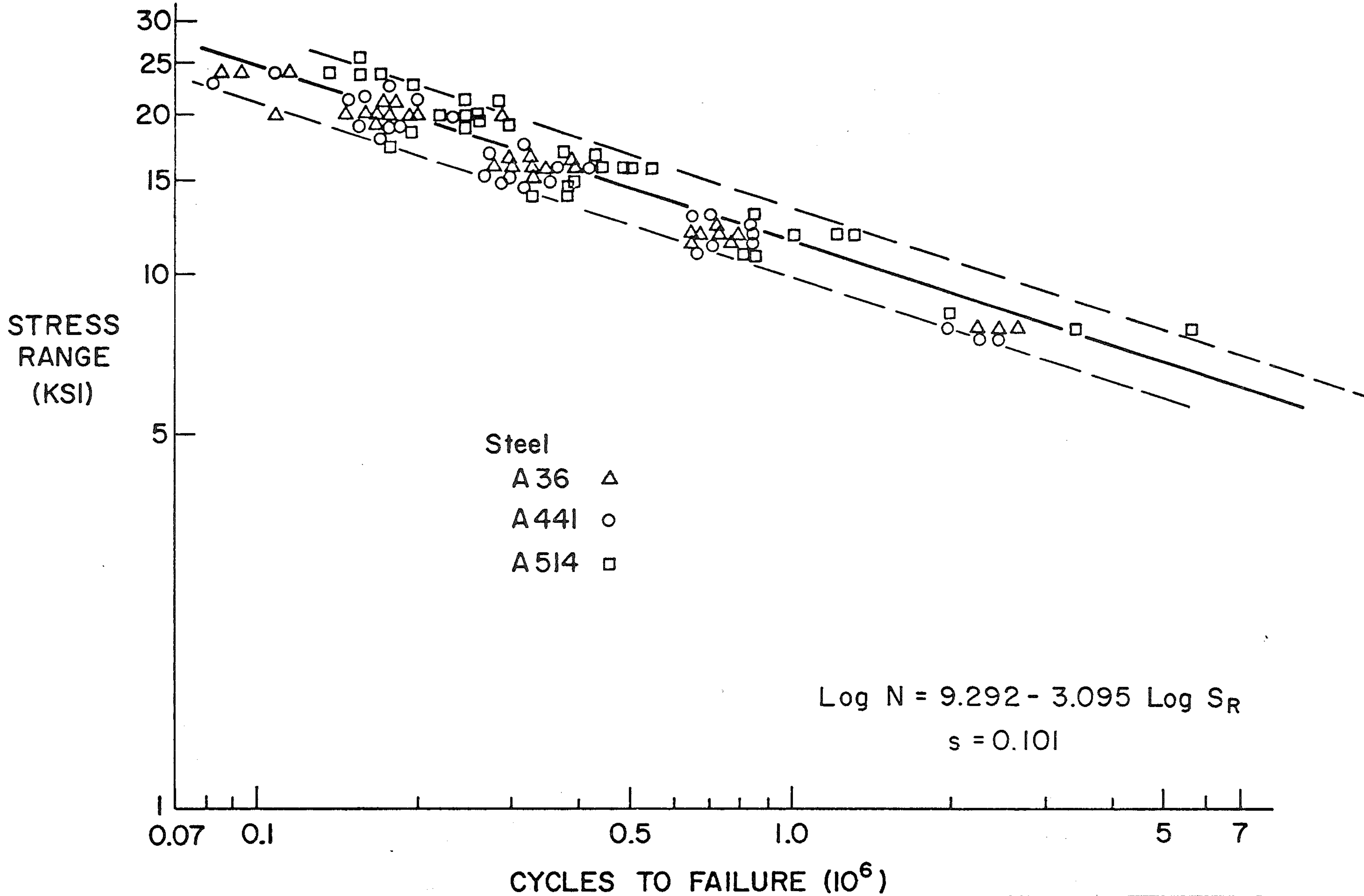

Fig. 7 Effect of grade of steel on the fatigue strength of beams with transversely end-welded cover plates

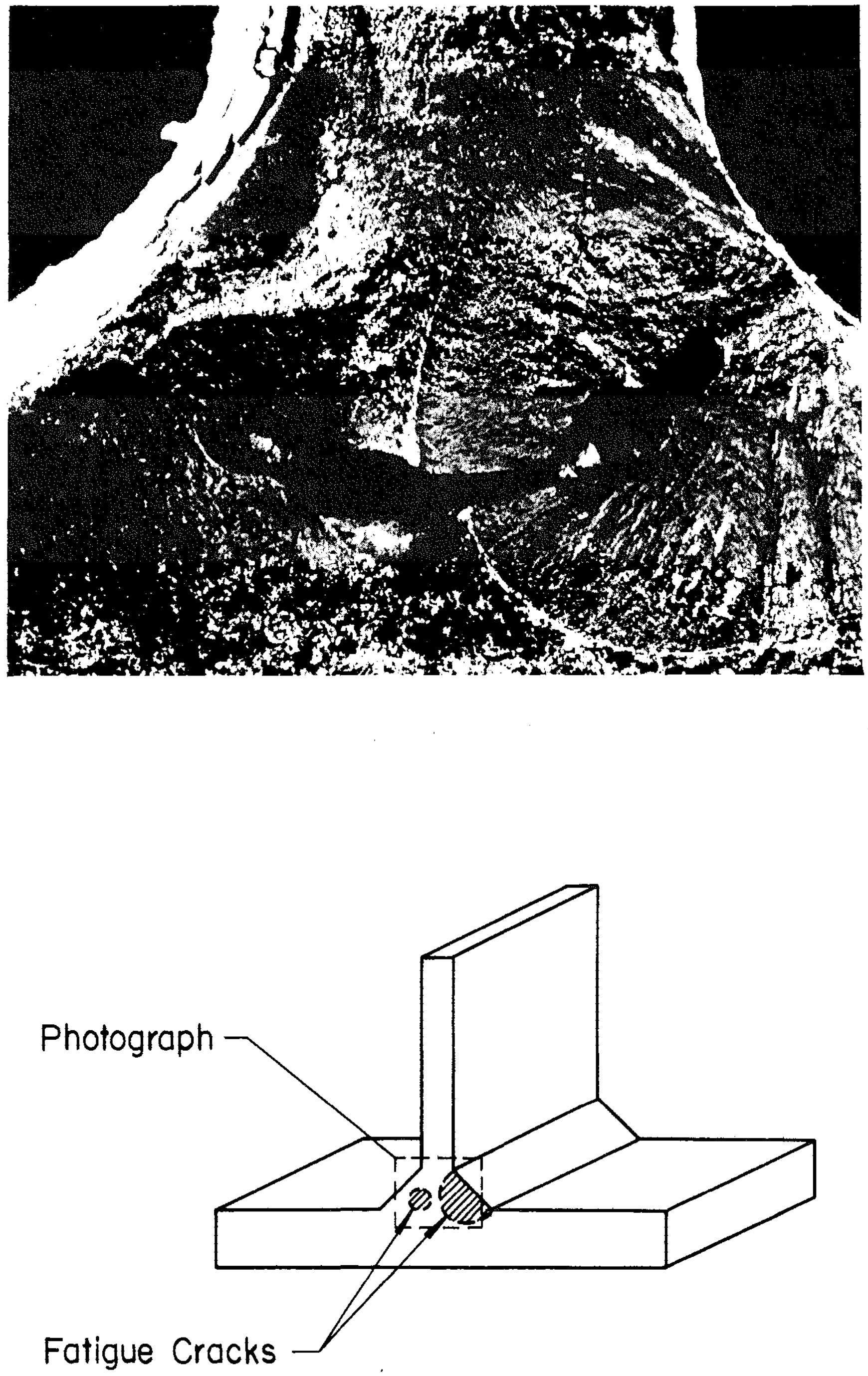

Fig. 8 Two small fatigue cracks that initiated from pores in the longitudinal fillet-weld and grew perpendicular to the axis of the weld (~x8.5)

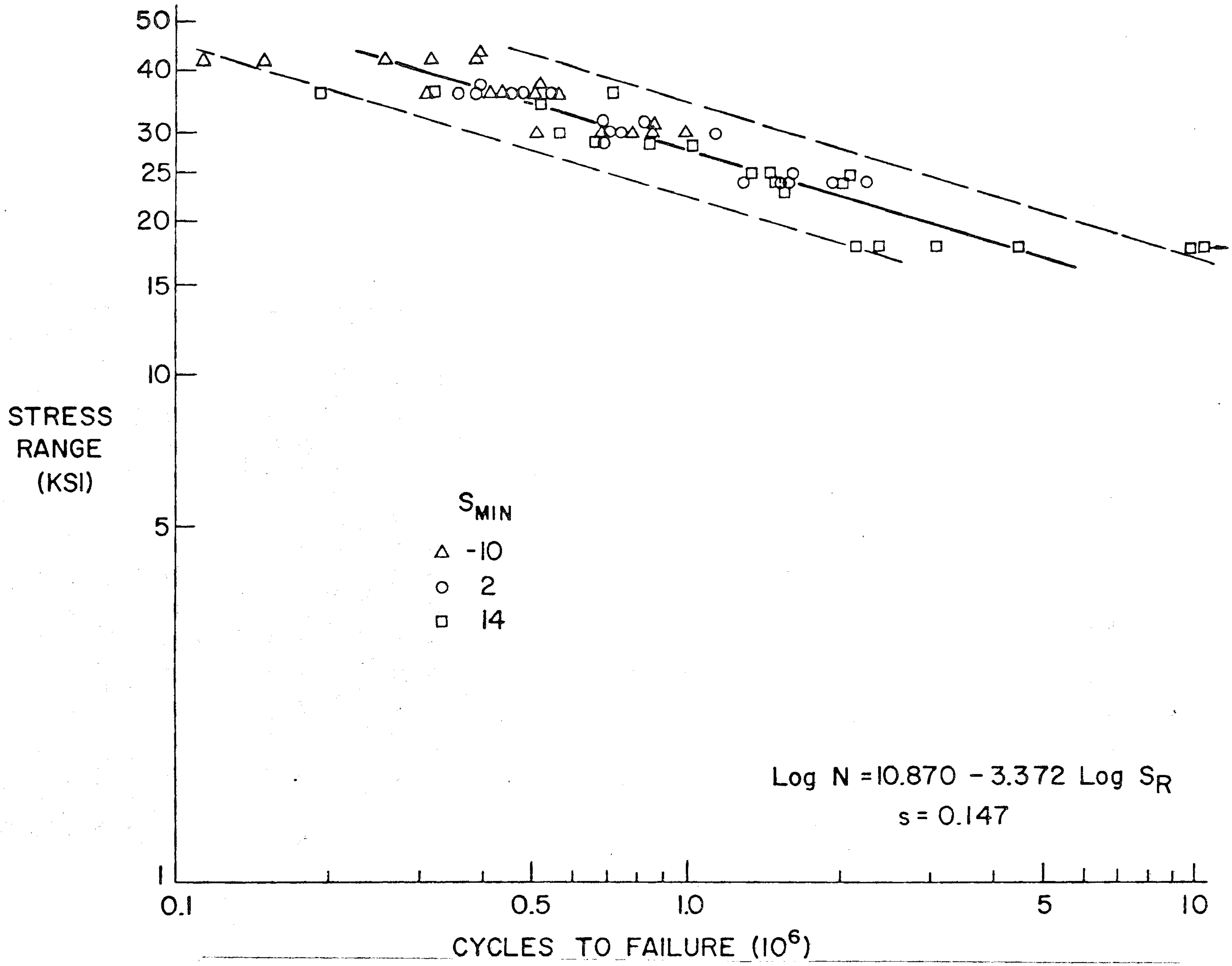

Fig. 9 Effect of stress range and minimum stress on the fatigue strength of welded beams

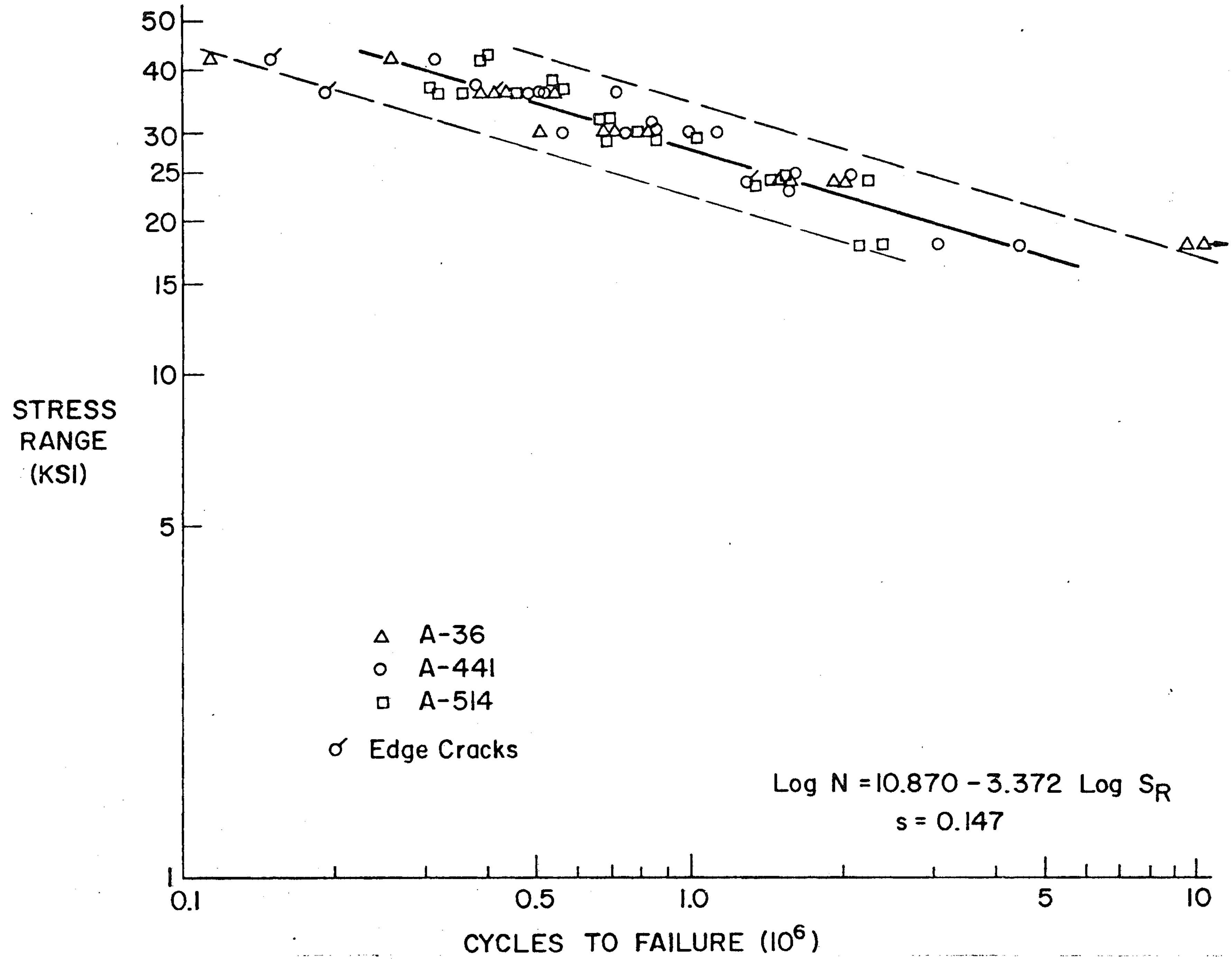

Fig. 10 Effect of grade of steel on fatigue strength of welded beams

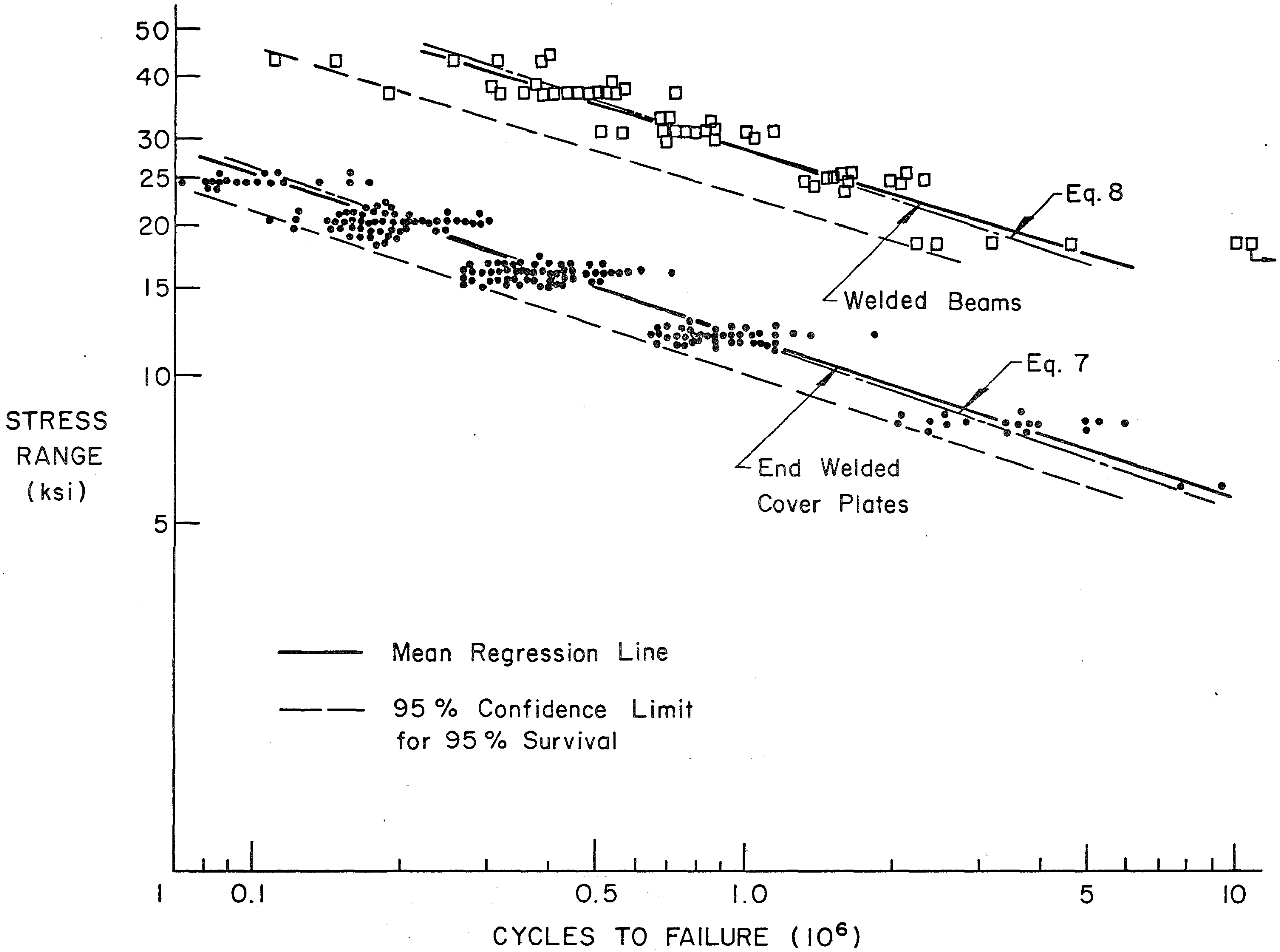

Fig. 11 Mean fatigue strength and 95% confidence limits for welded and coverplated beams and their correlation with predicted strength

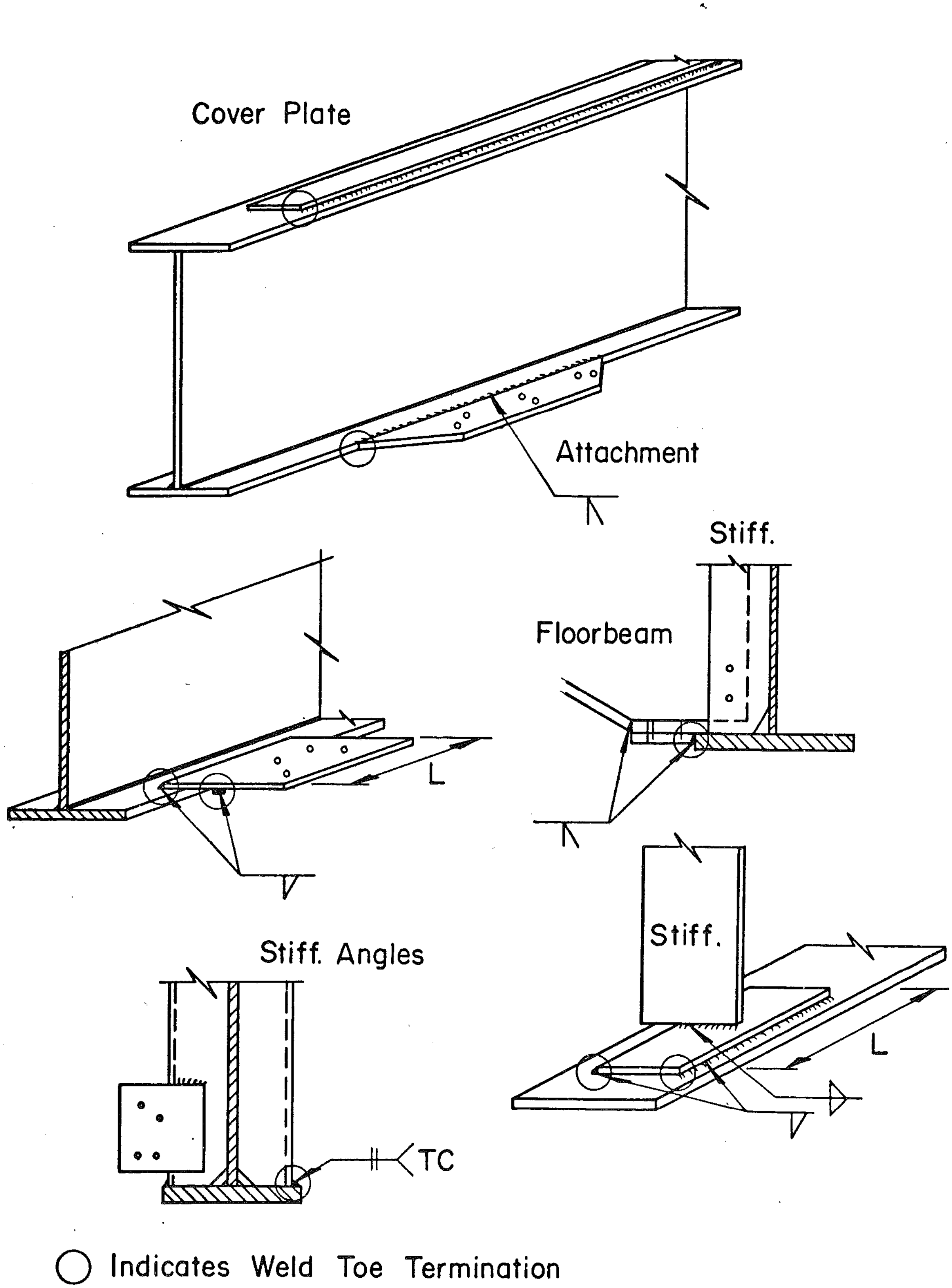

Fig. 12 Typical details that are comparable to coverplated beams

Security Classification

DOCUMENT CONTROL DATA - R & D

(Security classification of title, body of abstract and indexing annotation must be entered when the overall report is classified)

1. ORIGINATING ACTIVITY (Corporate author) Lehigh University	2a. REPORT SECURITY CLASSIFICATION Unclassified 2b. GROUP

3. REPORT TITLE

FATIGUE STRENGTH OF WELDED STEEL BEAM DETAILS AND DESIGN CONSIDERATIONS

4. DESCRIPTIVE NOTES (Type of report and inclusive dates)

5. AUTHOR(S) (First name, middle initial, last name)

J. W. Fisher

6. REPORT DATE January 1972	7a. TOTAL NO. OF PAGES 34	7b. NO. OF REFS 22

8a. CONTRACT OR GRANT NO. N00014-68-A-0514 b. PROJECT NO. 358 c. d.	9a. ORIGINATOR'S REPORT NUMBER(S) Fritz Eng. Lab. Rep. No. 358.34 9b. OTHER REPORT NO(S) (Any other numbers that may be assigned this report)

10. DISTRIBUTION STATEMENT

"Approved for public release; Distribution unlimited".

11. SUPPLEMENTARY NOTES	12. SPONSORING MILITARY ACTIVITY Office of Naval Research

13. ABSTRACT

This paper summarizes some of the findings of a comprehensive study on the fatigue strength of rolled and welded built-up beams without attachments, rolled and welded beams with cover plates, and welded beams with flange splices. Altogether, 374 steel beams with two or more details were fabricated and tested.

The welded beam details discussed herein represent the upper and lower boundaries of fatigue behavior of welded beams. The lower bound is provided by beams with partial length cover plates - a severe notch producing detail.

For purposes of design, this study has shown that the fatigue strength of the upper and lower bound details is independent of the strength of steel. A36, A441 and A514 steel beams provided the same fatigue strength for a given detail, and stress range was observed to account for nearly all the variation in cycle life.

This paper reviews briefly the major variables that influence the fatigue strength of welded details and suggests how they should be considered in design. The fracture mechanics of crack propagation is reviewed and used to focus on the major design factors. Particular attention is given to the initial flaw condition (which exists in all joints), the governing stress variables, and the influence of geometry.

Security Classification

14. KEY WORDS	LINK A		LINK B		LINK C	
	ROLE	WT	ROLE	WT	ROLE	WT

DD FORM 1 NOV 65 1473 (BACK)
(PAGE 2)

Security Classification