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These **Digests** are issued in the interest of providing an early awareness of the research results emanating from projects in the NCHRP. By making these results known as they are developed and prior to publication of the project report in the regular NCHRP series, it is hoped that the potential users of the research findings will be encouraged toward their early implementation in operating practices. Persons wanting to pursue the project subject matter in greater depth may obtain, on a loan basis, an uncorrected draft copy of the agency's report by request to the NCHRP Program Director, Highway Research Board, 2101 Constitution Ave., N.W., Washington, D.C. 20418

Fatigue Strength of Welded Steel Beams

An NCHRP staff digest of the essential findings from the final report on Phase II of NCHRP Project 12-7, "Effects of Weldments on Fatigue Strength of Steel Beams," by John W. Fisher, Pedro A. Albrecht, Ben T. Yen, and David J. Klingerman, Fritz Engineering Laboratory, Lehigh University; and Bernard M. McNamee, Drexel University

THE PROBLEM AND ITS SOLUTION

The fatigue fractures observed in cover-plated steel-beam bridges during the AASHO Road Test, and more recently in similar structures in the field, emphasize the influence of welding and welded details on the life expectancy of highway bridges. Also of great significance in these bridges are such factors as the loading history of the structure, the types of materials used, the design details, and the quality of fabrication. Among the more important design details are cover plates, stiffeners, attachments, and splices. The study summarized here investigated the effects of these details on the fatigue strength of welded steel beams.

In the past, only approximate general design relationships have been possible on the basis of the limited existing experimental data. Lehigh University and its subcontractor, Drexel University, started work on NCHRP Project 12-7 in October 1966. The over-all objective of this study was development of mathematical design relationships that could define with statistical confidence the fatigue strength of steel beams. This objective was accomplished by a review of existing fatigue data, and development and performance of statistically valid experiments that permitted formulation of mathematical relationships between the fatigue behavior of beams and design details, applied stresses, and types of steels.

Other NCHRP research in progress in the area of fatigue includes: Project 12-12, "Welded Steel Bridge Members Under Variable-Cycle Fatigue," and Project 12-14, "Subcritical Crack Growth in Steel Bridge Members," both at United States Steel Corporation, and Project 12-15, "Detection and Repair of Fatigue Cracking in Highway Bridges," at Lehigh University.

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Project 12-7, "Effects of Weldments on Fatigue Strength of Steel Beams," was carried out in two phases. Phase I lasted 40 months and had as its specific objective the development of design relationships for rolled and welded beams, both with and without cover plates, and for welded beams with flange splices. Altogether, 374 beam specimens were tested. The results of Phase I work have been reported in *NCHRP Report 102*, "Effect of Weldments on the Fatigue Strength of Steel Beams." Recommendations contained in that report were adopted in the 1971 "AASHO Interim Specifications for Highway Bridges."

Phase II started in July 1970 and lasted 30 months. Its purpose was to extend the study to details not covered in Phase I, thereby making possible the development of comprehensive design and specification provisions. Some 157 steel beams and girders were fabricated and tested in Phase II. Most of these tests were used to define the fatigue strength of stiffeners and attachments under constant-amplitude fatigue loading.

The final report on Phase II is in the editorial and publication process and should be available during the Summer of 1973. The purpose of this Research Results Digest is to call immediate attention to the findings and recommendations.

FINDINGS

The fact that the experimental work was statistically designed into a series of factorial experiments permitted evaluation of each of the parameters of interest with statistical confidence. This approach produced a solid body of experimental data in which the results of previous related studies could be incorporated and compared.

The principal design variables included three types of steel (A36, A441, A514), various stress considerations (minimum stress, maximum stress, stress range), and various design details (in Phase I: square-ended cover plates with and without transverse end welds, cover plate width and thickness transitions, and multiple cover plates, and in Phase II: stiffeners and attachments).

Because the findings result from a carefully designed and executed experimental program and also from consideration of the results of most of the previous research related to this topic, it is believed that they are highly reliable and can be used immediately for the improvement of specifications. The current (1969) specifications of the American Association of State Highway Officials (AASHO) limit the maximum applied stress for fatigue. The fatigue stress is considered a function of minimum stress and maximum stress, the type of steel, and the slope (k_2) of a maximum fatigue stress (F_r) equation. Interim specifications were adopted in 1971 and indirectly provided a stress range design for the several details that were reported in *NCHRP Report 102*.

Phase II has continued to confirm that stress range alone is the only significant factor for designing a given detail against fatigue. Minimum stress and type of steel did not significantly affect the fatigue strength of beams with transverse stiffeners or attachments.

Sixty-nine welded beams and girders with transverse stiffeners were tested during this study and provided data on 118 stiffener details. The stiffeners were either welded to the web alone or were welded to both the web and the flanges. As shown in Figure 1, stiffeners attached to the web alone were classified as Type 1 or Type 2, depending on whether they were in the moment-gradient or constant-moment regions. These stiffeners were not fitted between flanges and the weld terminated on the beam web 1/2 in. to 4 in. above the tension flange surface. The second kind of stiffener, classified as Type 3, was welded to the beam flanges as well as the web.

The test data for stiffener details are plotted in Figure 2. The following conclusions on the effects of stiffener details are based on this study:

- The beam bending stress range at the weld toe termination dominated the fatigue strength of full-depth stiffener details welded to the web alone. The bending stress range at the stiffener-to-flange weld was dominant for stiffeners welded to the web and flanges.

- Minimum stress was not a significant design factor for any stiffener detail. The presence of residual tensile stresses at the toe of transverse stiffener welds made the full stress range effective for stress reversals.

- The principal stress and its direction are not significant for purposes of design in stiffened bridge members, even though principal stress provides the best theoretical correlation.

- The type of steel was not a significant design factor for any stiffener detail.

- The crack causing failure at all stiffeners welded to the web alone initiated at the end of the stiffener-to-web weld. When stiffeners were welded to the web and flange the crack initiated at the toe of the stiffener-to-flange weld.

- Failure occurred after the crack had destroyed most of the tension flange of the beam. Cracks originating in the web propagated into the flange and eventually terminated the test.

- The same stress range-life relationship is equally applicable to stiffeners welded to the web alone or stiffeners welded to the web and flange when the stress is determined in terms of the beam bending stress range at the weld toe termination where cracks first occur.

- Attaching diagonal bracing to the stiffeners of beams and girders had no effect on their fatigue strength. Within the range of estimated lateral forces and displacements on highway bridges, the out-of-plane deflection at the stiffeners had no influence on crack growth.

- No fatigue (endurance) limit was observed for the bending stress ranges (13.7 to 28.7 ksi) examined for the stiffener details.

- A fracture mechanics analysis of crack growth indicated that the behavior of stiffener details was not significantly affected by the thickness of the flange or web plate.

Fifty-six welded beams were tested with attachment plates welded to the tension and compression flanges. Each beam provided data on crack growth at two cross-sections. The attachments were all welded to the outer surfaces of the flange as shown in Figure 3.

The variables in the study of fatigue life of the welded attachment details were: length of attachment, stress, and type of weld detail. The material in all cases was A441 steel. There were four attachment lengths: 1/4, 2, 4, and 8 in. There were two weld details for the 4-in. attachments: longitudinal welds only and welds all around. The weld was placed all around the 1/4-, 2-, and 8-in. attachments.

The effects of the primary variables were evaluated using statistical analysis and theoretical considerations. The results of the analysis indicated that the dominant stress variable was stress range for all the attachment configurations tested. Figure 3 illustrates this fact for the 4-in. attachment with welds all around. It is apparent that minimum stress was insignificant at all levels of stress range. Stress range alone accounted for the variation in cycle life.

The following conclusions on the effects of welded flange attachments are based on this study:

- The failure lives of the different attachment details differed from each other. The life decreased as the attachment length increased.

- The crack causing failure of all attachment details originated at the most highly stressed weld toes of the welds connecting the attachment to the beam flange. At ends of transversely welded attachments the crack originated at several sites along the weld toe, but was more severe near the center of the weld. At attachments without transverse welds the cracks initiated at the weld toe at the termination of the longitudinal fillet weld.

- End-welded short attachments gave shorter lives than those with unwelded ends. This difference was apparent, but not statistically significant.

- Failure occurred in the tension flange of all beams with flange attachments. Many cracks were also observed in the compression flange at the weld toe terminations. When the flange was subjected to nominal compression stress these cracks were arrested after they grew out of the local residual tensile stress zone. When subjected to stress reversal some additional crack growth was apparent.

- Minimum stress was not a significant variable for any of the attachments examined. The presence of residual tensile stresses at weld toes made the full stress range effective.

- Although slight differences in the slope of the mean regression lines existed among the various attachments, these differences were not significant. The difference reflected the small number of test specimens evaluated for each attachment length. Over-all, the slopes for all attachments were about the same as the plain welded and cover-plated beams.

- A fracture mechanics analysis of crack growth accounted for the fatigue behavior of the flange attachments. The primary cause of the differences in fatigue life among various attachment lengths was found to be the differences in the stress concentration at the terminating weld toes.

- No fatigue limit was observed for the 4-in. and 8-in. attachments for the stress ranges that were examined (8 to 24 ksi). The 9/32-in. and 2-in. attachments were tested under stress ranges of 12 and 28 ksi. No fatigue limit was observed for the 2-in. attachment. The 9/32-in. attachments exhibited no failures at the 12-ksi stress range level.

The 95 percent confidence limits for 95 percent survival are shown in Figure 4 for the stiffener details and attachments tested in this study, as well as for the cover-plated and plain welded beams reported in *NCHRP Report 102*. It is apparent that the stress range-cycle life relationships for these details are provided by a family of lines that are approximately parallel.

It is recommended that allowable stress range values for fatigue design be selected from the lower confidence limits. This provides a rational means of selecting design stress values and takes into consideration the variability of the test data and the size of the sample tested.

For purposes of design, Phase II confirmed the applicability of the design stress values developed for A36 and A441 steel rolled beams to A514 steel rolled beams.

Current (1969) AASHO specifications do not provide sufficient latitude for base metal adjacent to or connected by fillet welds. Only one category (F) is provided and was derived from the cover-plated beam. A second category (K) is applicable to base metal adjacent to transverse stiffeners in A514/A517 steel. The AASHO specification provision for base metal adjacent to transverse stiffeners in A514/A517 steel should be applied to all steels. Only the stress range should control the design, because the strength of the detail is independent of the type of steel.

Provisions should also be added to reflect the higher fatigue strength of very short length attachments as compared to the fatigue strength of cover plates on beams. The cover-plated beam is a "lower bound" detail and provides the least fatigue strength. At least one category should be added intermediate to the cases of the cover-plated beam and the beam with transverse stiffeners. In addition, recognition of the adverse effect of a groove weld termination at an abrupt change in geometry is needed. The same fatigue strength and crack growth behavior result when a groove weld or a fillet weld toe terminates in a region of stress concentration with a microcrack in a plane perpendicular to the applied stresses.

None of the details or beams examined developed a fatigue limit at 2 million cycles. Only the 1/4-in. attachments and the A514 rolled steel beams experienced no visible crack growth up to 10 million or more cycles at the lowest stress range level. It appears desirable to specify a loading condition on highway bridges for "over 2 million cycles" so as to reduce the possibility of crack growth under extreme cyclic applications.

APPLICATIONS

The findings from this study should be of value to structural engineers involved in the design of welded steel beams, researchers working in the subject area, and, perhaps most of all, members of specification writing bodies. The suggested revisions to the "AASHO Specifications for Highway Bridges" included here warrant consideration. Further, the suggested revisions can also be applied to other specifications, such as those of the American Welding Society and the American Railway Engineering Association. The findings result from a meticulously designed and executed experimental effort verified by analyses of crack propagation and fracture mechanics and appear to warrant serious consideration for immediate inclusion in design specifications.

The study can provide the basis for a comprehensive change in the fatigue provisions of the AASHO specifications. The use of stress range for each design detail and loading condition will greatly simplify design computations. At the same time it will reflect the available experimental and theoretical findings on the significant design variables.

Although all design details have not been evaluated during this study, the basic framework has been developed and the critical design parameters defined. Hence, a review of published fatigue data with these findings in mind will permit more rational interpretation of the results and development of design values that are more rational than existing specification provisions.

Table 1 gives suggested allowable ranges of stress for a number of categories that are defined in detail in Table 2. The framework is directly comparable to the provisions given in the AISC specification. Similar provisions are now being prepared for the new British fatigue design rules. Four categories of life are included, rather than the three used in the current AASHO specifications. Because the fatigue strength at 2 million cycles does not correspond to the fatigue limit, a fourth category was added to account for extreme cyclic fatigue conditions.

The suggested values of stress range (S_r) are based on the 95 percent confidence limits for 95 percent survival. They provide a uniform estimate of survival and account for the variation in the available test observations. Only stress ranges that cause tensile or reversal stresses are included. As in Phase I, this study showed that although cracks may form in the residual tension zones of details subjected to compression stresses, these cracks arrested when they grew out of the residual tensile zone. When subjected to stress reversal, the tension component of the stress cycle is enough to continue driving the crack after it grows out of the residual tensile zone. The slight difference in life at this stage of fatigue crack growth is not significant for design purposes.

Several details described in Table 2 were not studied in Project 12-7. However, the stress category assigned to these details in Table 2 was based on data available in the literature. Current specification provisions are based on these same sources. These details include: (1) built-up plates or shapes connected by continuous full-penetration groove welds parallel to the line of stress; (2) base metal and weld metal in or adjacent to full-penetration groove-welded splices at transitions in thickness, with the welds ground to provide slopes no steeper than 1 to 2-1/2, with grinding in the direction of applied stress, and the weld soundness established by nondestructive inspection; (3) base metal and weld metal in or adjacent to full-penetration groove-welded splices, with or without transitions having slopes no greater than 1 to 2-1/2 when the reinforcement is not removed and weld soundness is established by nondestructive inspection; and (4) base metal at details attached by groove welds subject to transverse or longitudinal loading.

TABLE 1

Fatigue Stresses

Category See Table 2	Allowable Range of Stress, F_{sr} (ksi)			
	For 100,000 Cycles	For 500,000 Cycles	For 2,000,000 Cycles	Over 2,000,000 Cycles
A	60	36	24	24
B	45	27.5	18	16
C	32	19	13	10
D	27	16	10	7
E	21	12.5	8	5

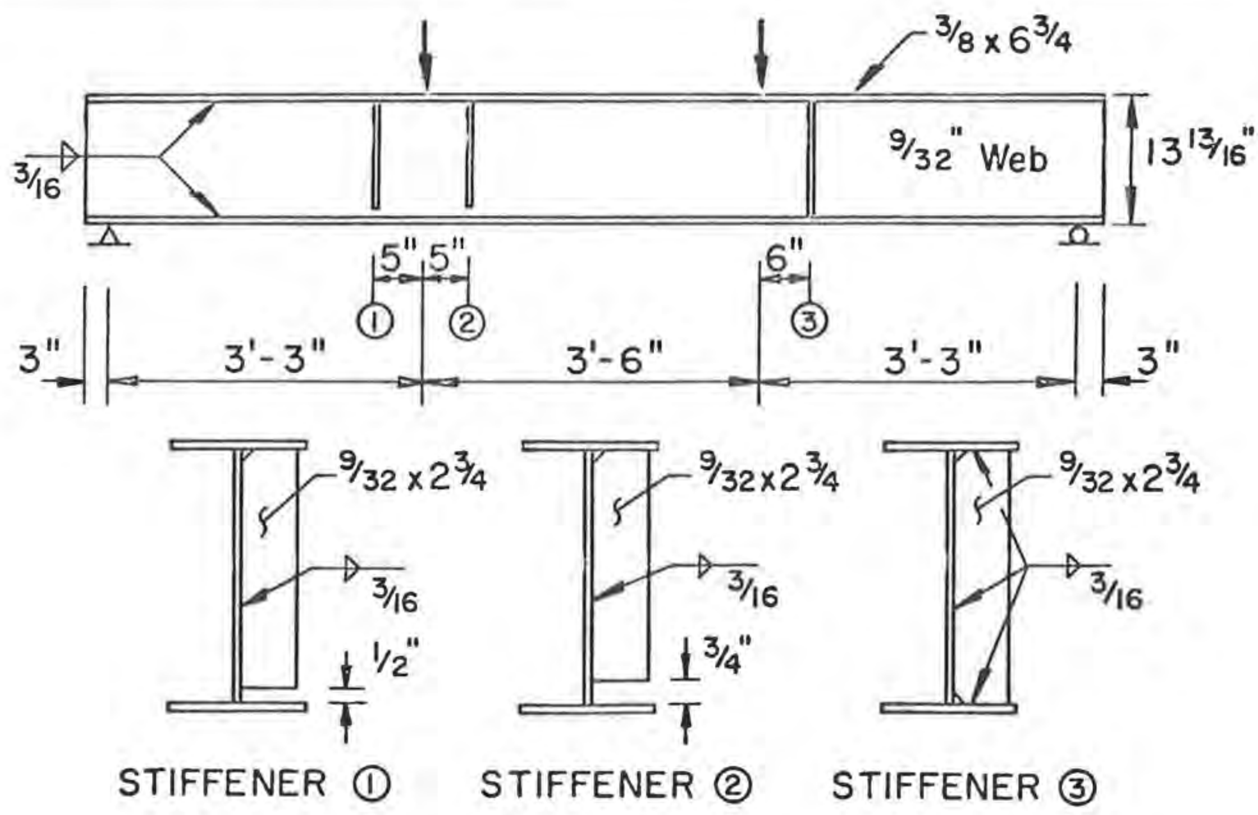

Figure 1. Details of SC beams.

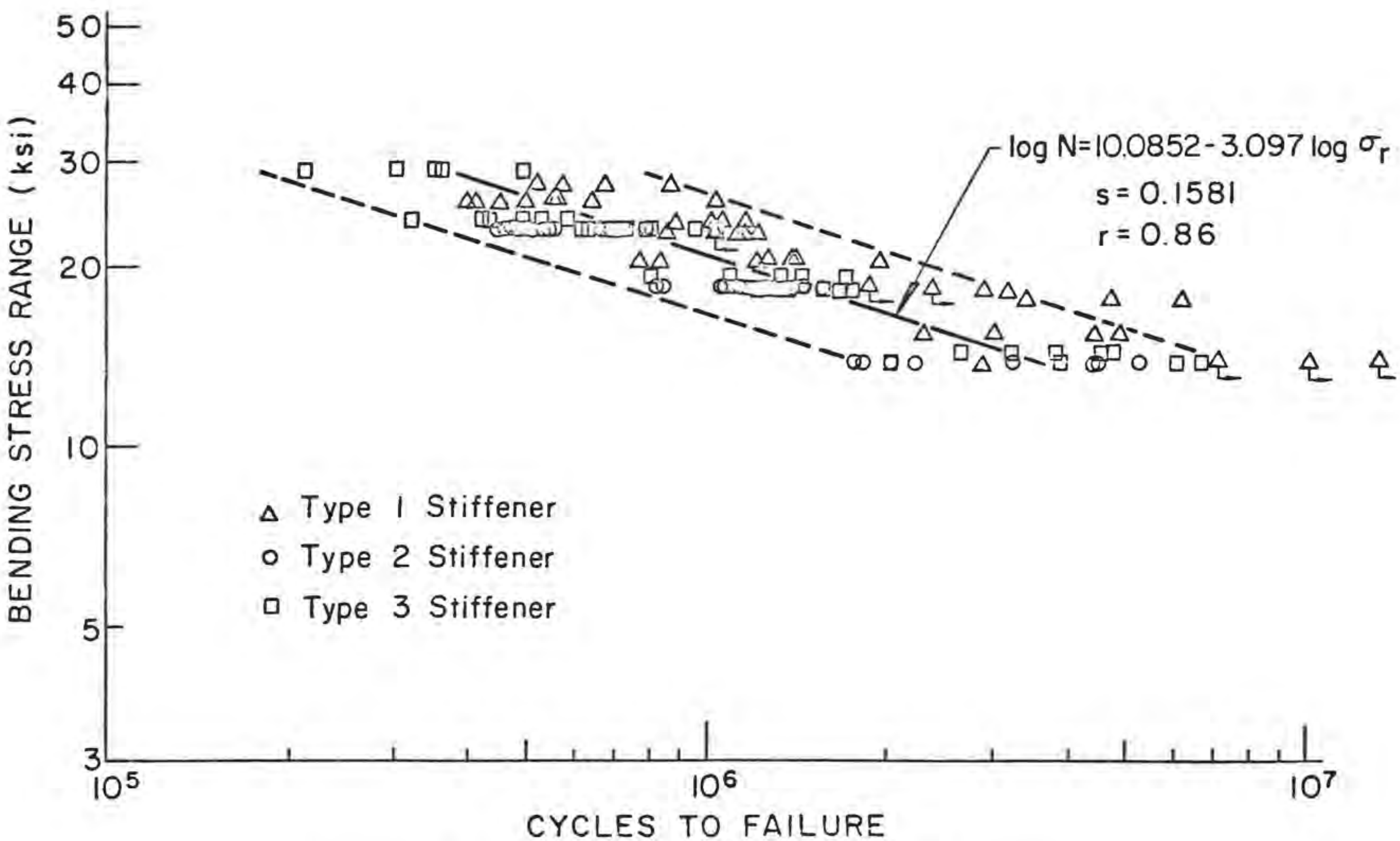

Figure 2. Summary S-N plot for Type 1, 2, and 3 stiffeners.

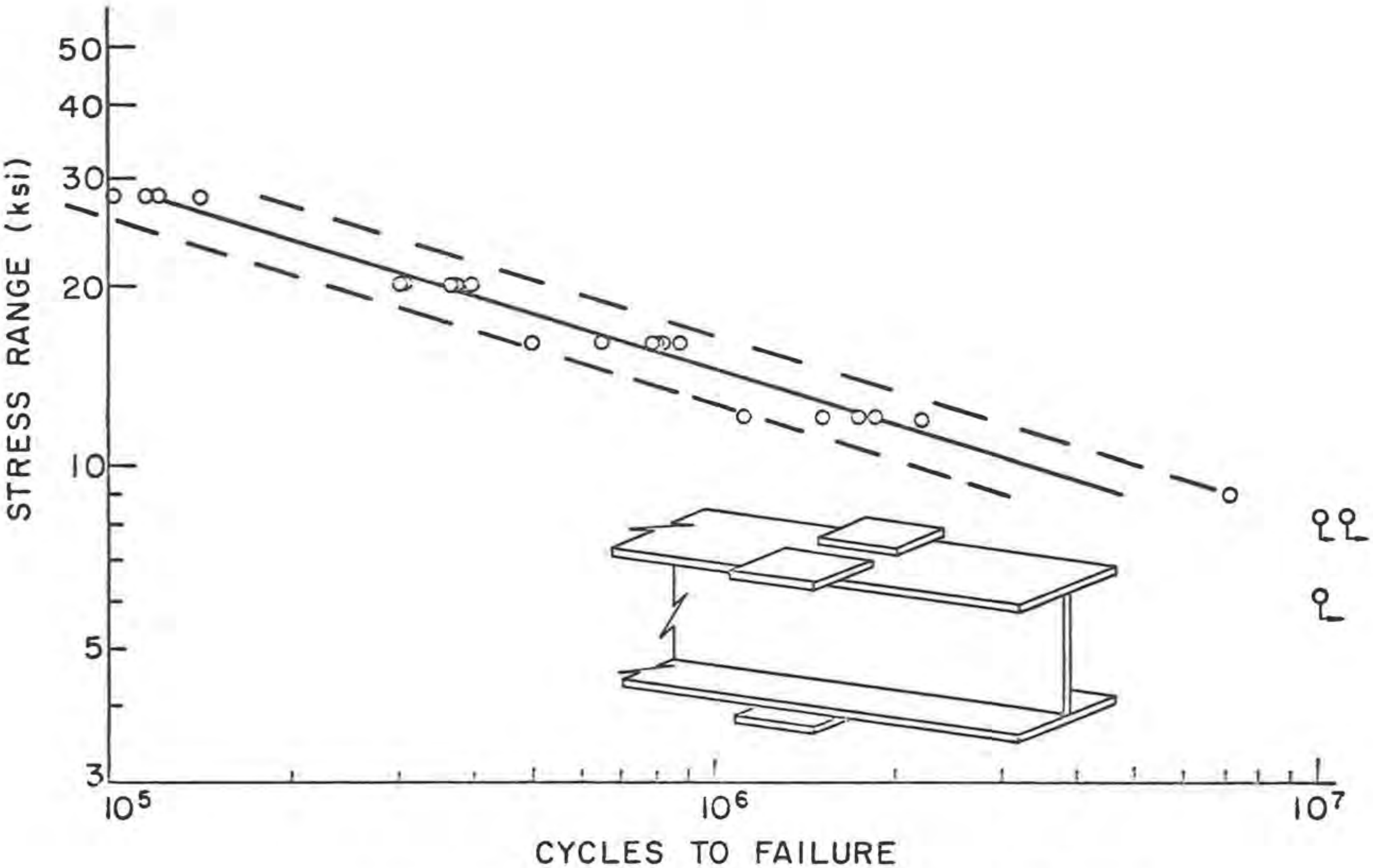

Figure 3. S-N plot for 4-in. attachments welded all around.

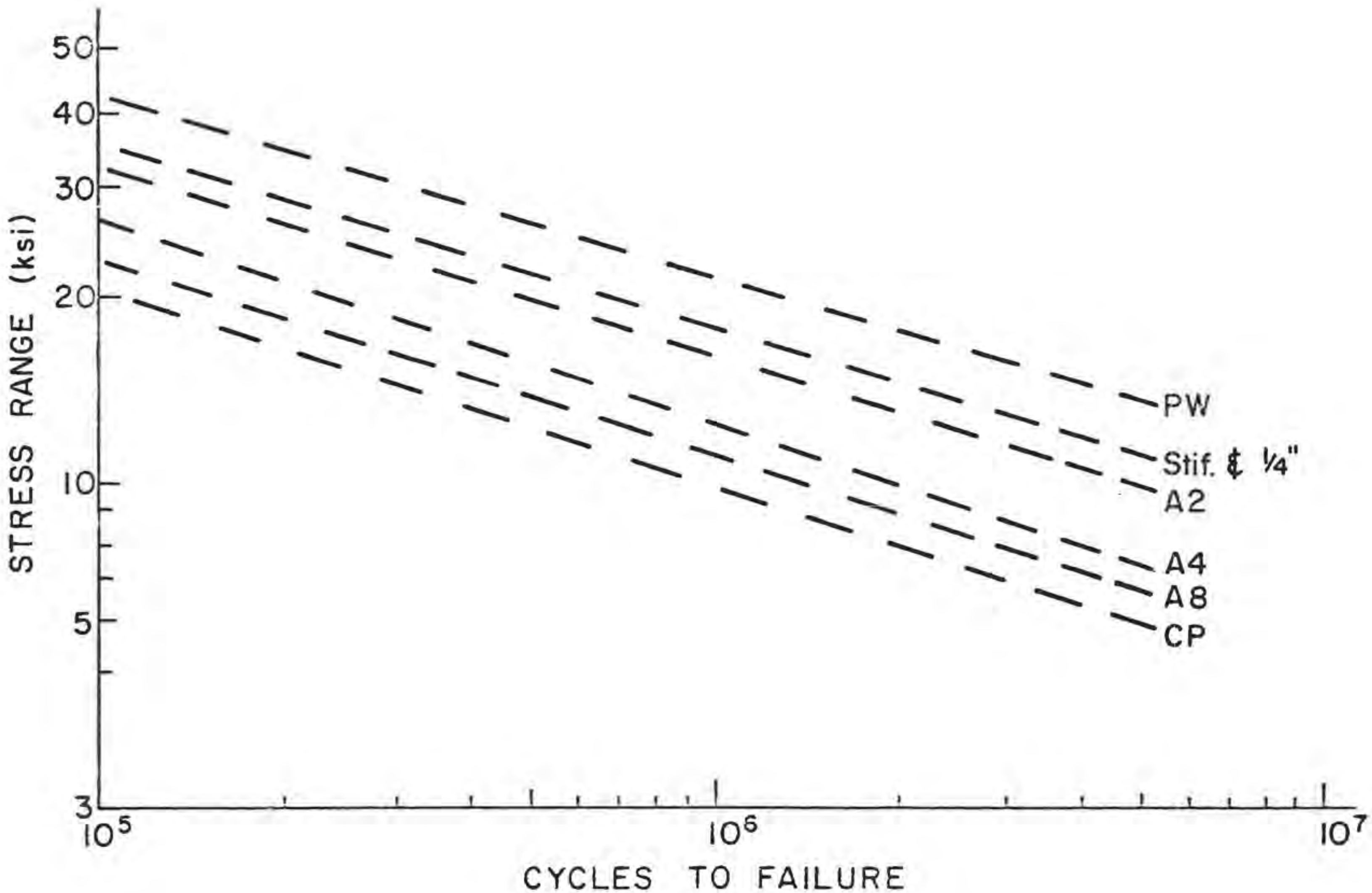

Figure 4. 95% Confidence limits for 95% survival.

TABLE 2

DESCRIPTION OF DESIGN CONDITIONS

Joint Classifications

General Condition	Situation	Kind of Stress(1)	Stress Category (See Table 1)
Plain Material	Base metal with rolled or cleaned surfaces. Flame cut edges with ASA smoothness of 1000 or less	T or Rev.	A
Built-up Members	Base metal and weld metal in members without attachments, built-up of plates or shapes connected by continuous full penetration groove welds or by continuous fillet welds parallel to the direction of applied stress	T or Rev.	B
	Calculated flexural stress at toe of transverse stiffener welds on girder webs or flanges	T or Rev.	C
	Base metal at end of partial length welded cover plates having square or tapered ends, with or without welds across the ends	T or Rev.	E
Groove Welds	Base metal and weld metal at full penetration groove welded splices of rolled and welded sections having similar profiles when welds are ground flush and weld soundness established by non-destructive inspection	T or Rev.	B
	Base metal and weld metal in or adjacent to full penetration groove welded splices at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2 1/2, with grinding in the direction of ap-	T or Rev.	B

TABLE 2 (Continued)

General Condition	Situation	Kind of Stress(1)	Stress Category (See Table 1)
Groove Welds	plied stress, and weld soundness established by non-destructive inspection		
	Base metal and weld metal in or adjacent to full penetration groove welded splices, with or without transitions having slopes no greater than 1 to 2 1/2 when reinforcement is not removed and weld soundness is established by non-destructive inspection	T or Rev.	C
	Base metal at details attached by groove welds subject to transverse and/or longitudinal loading when the detail length, L, parallel to the line of stress is between 2 in. and 12 times the plate thickness, but less than 4 in.	T or Rev.	D
	Base metal at details attached by groove welds subject to transverse and/or longitudinal loading when the detail length L is greater than 12 times the plate thickness or greater than 4 in. long	T or Rev.	E
Fillet Welded Connections	Base metal at intermittent fillet welds	T or Rev.	E
	Base metal adjacent to fillet welded attachments with length L in direction of stress less than 2 in. and stud-type shear connectors	T or Rev.	C
	Base metal at details attached by fillet welds with detail length L in direction of stress be-	T or Rev.	D

TABLE 2 (Continued)

General Condition	Situation	Kind of Stress(1)	Stress Category (See Table 1)
Fillet Welded Connections	tween 2 in. and 12 times the plate thickness but less than 4 in.		
	Base metal at attachment-details with detail length L in direction of stress (length of fillet weld) greater than 12 times the plate thickness or greater than 4 in.	T or Rev.	E

(1) "T" signifies range in tensile stress only; "Rev." signifies a range of stress involving both tension and compression during the stress cycle.