National Cooperative Highway Research Program

Research Results Digest

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Effects of Weldments on Fatigue Strength of Steel Beams

An NCHRP staff digest of the essential findings from the final report on NCHRP Project 12-7, "Effects of Weldments on Fatigue Strength of Steel Beams," by John W. Fisher, Karl H. Frank, Manfred A. Hirt, Fritz Engineering Laboratory, Lehigh University; and Bernard M. McNamee, Drexel University.

Superseded by NCHRP REPT 102

THE PROBLEM AND ITS SOLUTION

The fatigue fractures observed in the cover-plated steel beam bridges of the AASHO Road Test and in other similar structures emphasize the important effect of welding and welded details on the life expectancy of highway beam or girder bridges. Also of great significance in these bridges are such factors as the loading history to which the structures are subjected, the types of materials used, the design details, and the quality of fabrication. Among the more important design details are such factors as cover plates, stiffeners, attachments, and splices. In the past, only approximate general mathematical design relationships have been possible on the basis of the limited existing experimental data. However, with the conduct of additional research and an analysis and evaluation of the many interrelated fatigue parameters, suitable basic relationships can be developed to properly design welded bridges for a desired life expectancy.

Lehigh University and its subcontractor, Drexel University, undertook the development of mathematical design relationships that could define with statistical confidence the fatigue strength of rolled and welded beams, rolled and welded beams with cover plates, and welded beams with flange splices. 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 relating the fatigue behavior of beams to design details, applied stresses, and types of steels. During the research program, 374 beam specimens were tested under constant-amplitude cyclic loading.

In addition to the beam specimens, pilot fatigue studies on variable-amplitude cyclic loading were also conducted, using notched bar specimens.

The principal design variables included three types of steel (A36, A441, and A514), various stress considerations (minimum stress, maximum stress, and stress range), and various details (square-ended cover plates, with or without a transverse end weld, variation in the thickness of the cover plate, variation in the width of the cover plate, and the use of multiple cover plates on each flange). Also studied were transitions in the flange width, including a 2-ft. radius transition and a uniform slope transition.

The study determined fatigue behavior between 50,000 and 10,000,000 cycles of loading.

The fact that 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.

Because the findings result from a meticulously designed and executed experimental program, plus 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.

FINDINGS

This study can provide the basis for several revisions to the fatigue provisions of the *AASHO Specifications for Highway Bridges*. The report contains a table that presents suggested modifications and additions to the *1969 AASHO Specifications* for the categories corresponding to the studies undertaken in this investigation. Details and other types and location of material not included in this study have been omitted from the table pending the completion of further work. Obviously, the provisions for base metal adjacent to fillet welds are lower boundary values. A more detailed breakdown of the various types of fillet welded details (i.e., stiffeners and other attachments) should be possible when further work is completed.

Values of stress range for specification provisions should be based on confidence limits. This provides a rational means of accounting for the sample size on which the value is based, the desired percentage of survival, the confidence level, and the degree of variation in the test observations.

This study has shown that the currently used fatigue provisions are not satisfactory for A36 and A441 steel rolled beams. Neither are they satisfactory for plain welded beams, cover-plated beams, and flange splices of A36, A441, and A514 steel. Stress range alone accounts for the variation in cycle life. Specification provisions should reflect this fact.

The importance of stress range was confirmed by a minimum notch-producing detail, the plain rolled beam, which provided an upper boundary for fatigue strength. Similar behavior was observed for severe notch-producing detail—the cover-plated beam--which provided a lower boundary for the fatigue strength. The report presents the stress-cycle relationship for the upper and lower boundary behavior. The mean fatigue strengths and 95% confidence limits for survival are shown for the cover-plated, welded, and rolled beams. All test data are plotted.

For purposes of design, this study has shown that the fatigue strength of a given welded detail is independent of the strength of the steel. Tests of rolled A36 and A441 steel beams yielded about the same fatigue strength. The *AASHO Specifications* should reflect this finding for rolled beams, welded beams, flange splices, and cover-plated beams.

Current *AASHO Specifications (1969)* indicate that web-to-flange fillet welds can be neglected and the beam considered as base metal for purposes of fatigue design. This assumption is incorrect and grossly overstates the fatigue strength of plain welded beams. The error is on the unsafe side and increases with an increase in steel strength, because allowable design stresses also increase for the base metal.

Variations in cover-plate geometry (such as cover-plate thickness, cover-plate width, multiple cover plates, and termination detail) have a negligible effect on end-welded cover plates.

Cover plates wider than the flange to which they are attached will result in a decrease in fatigue strength unless transverse end welds are used. Hence. transverse end welds should be required on wide cover plates. The end weld may be returned around the beam flange or stop short of the flange toes. Cover plates narrower than the flange to which they are attached do not require a transverse end weld.

At a transition in flange width, groove welds with the reinforcement removed provided for almost all cases the same fatigue strength as plain welded beams. The straight transition with a 1-to-2½ taper yielded the same fatigue strength as the 2-ft. radius transition. The straight taper provides adequate fatigue strength and is more economical.

Groove welds in A514 steel require close examination and inspection, as they appear more susceptible to flaws that grow and eventually lead to fatigue failure. It appears reasonable to require the 2-ft. radius transition for groove welds in A514 steel until the results of future work become available.

There were no observable differences in fatigue life that could be attributed to uncontrolled variables (such as rest periods, interruptions of the tests for up to one year, rate of loading, and environmental effects).

The results of the studies on plain rolled beams did not correlate with existing studies on plate specimens. This appears to be caused by the greater probability of a defect occurring in the beam and the stress concentration at the web-flange junction.

The pilot study on variable-amplitude loading indicated that most of the fatigue damage was caused by the higher amplitudes of loading. The RMS (root mean square) stress range of the larger stress blocks correlated with the constant-cycle tests of similar specimens. In actual bridges the assumed design stress can be considered an estimate of the RMS stress range. Hence, the use of constant-amplitude fatigue data for design provisions is reasonable.

The log transformation of cycle life resulted in a normal distribution of the test data at nearly every level of stress range.

The empirical exponential model relating stress range to cycle life was observed to provide the best fit to the test data for all beam series.

Suitable mathematical design relationships can be developed for any desired life from the mathematical models relating stress range to cycle life.

The behavior of all details with respect to stress range was the same. In each, the life was observed to be inversely proportional to the applied nominal stress range. A good fit to the data of each end detail was provided by expressing the life as inversely proportional to the third powers of the stress range.

The theoretical stress analysis based on the fracture mechanics of crack growth substantiated the empirical model that provided the best fit to the test data. In addition, it provided a means of assessing the significance of the results obtained from the experimental work; i.e., the earlier failure of wide cover plates was rationally explained.

The number of cycles for a visible crack to form was usually 75% or more of the life of the specimen for all details tested. Consideration should be given to this fact in estimating the remaining life of structures that have fatigue cracks.

Cracks formed in the compression flange of many cover-plated, plain welded, and flange-spliced beams in regions of residual tension stress. These cracks usually grew more slowly after they had grown out of the residual tension area. If the compression flange was subjected to tension (i.e., during partial reversal), a number of beams failed by fracture of the compression flange.

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. Suggested revisions to the *AASHO Specifications for Highway Bridges* are included and 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 are the result of a meticulously designed and executed experimental effort verified by analyses of crack propagation and fracture mechanics and appear to warrant very serious consideration for immediate inclusion in design specifications.

The results of this study involve primarily a solid body of basic fatigue behavior for welded steel beams and realistically describe upper and lower boundary conditions. There is, of course, still a great deal of work required to define the fatigue behavior in the intermediate region between the upper and lower boundaries. This still relatively unexplored region is of great importance to the design engineer because it includes such things as stiffeners, attachments, and lateral bracing effects on the fatigue behavior of welded steel members. Additional work will be conducted by Lehigh University under a continuation of Project 12-7 in these relatively unexplored areas, and a report is expected to be issued about 1973.

The work reported in this *Digest* and the continuation work under Project 12-7 consider only constant-amplitude cyclic loading fatigue; variable-amplitude cyclic loading fatigue remains to be explored. Therefore, NCHRP has scheduled Project 12-12, "Welded Steel Bridge Members Under Variable-Cycle Fatigue Loadings," to be under contract in the fall of 1970, and anticipates a final report some time in 1975.

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