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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, 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 do so through contact with the Cooperative Research Programs Staff, Transportation Research Board, 2101 Constitution Ave., N.W., Washington, D.C. 20418.

Areas of Interest: IIC Bridges, Other Structures, Hydraulics and Hydrology

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Fatigue Behavior of Welded and Mechanical Splices in Reinforcing Steel

This NCHRP Digest contains suggested revisions to the AASHTO Standard Specifications for Highway Bridges based on findings from the final report under NCHRP Project 10-35, "Fatigue Behavior of Welded and Mechanical Splices in Reinforcing Steel," conducted by Wiss, Janney, Elstner Associates, Inc. The Digest was prepared by David G. Manning, Consultant.

INTRODUCTION

In reinforced concrete design, the structural engineer is faced with the task of determining where and how reinforcing bars must be spliced in a structure.* The lap splice, when conditions permit and when it will satisfy all requirements, is generally the most common method for splicing reinforcing bars. However, when lap splices are impractical or uneconomic, mechanical or welded connections may be used to provide a direct connection between reinforcing bars.

There are a wide variety of proprietary mechanical connectors and welded joints that can be considered, depending on the circumstances. In new construction, for example, splices can be used to join large bars (codes do not permit lapped splices with No. 14 and No. 18 bars), where spacing is insufficient to permit lap splicing, where lap lengths are excessive, in "tension tie members," and at construction joints where it is undesirable to have long lengths of bar protruding from the joint. In rehabilitation projects, direct connections may be used advantageously in circumstances such as bridge widening projects, where lesser amounts of sound concrete may have to be removed, or in staged construction, where working space between the stages may be limited. In some situations, in bridge rehabilitation work for example, practical considerations may dictate that splices be placed in regions of repeated stress cycles; therefore, it is important that the fatigue behavior of splices be known.

The factors affecting the fatigue of unspliced reinforcing bars are considered to be well known and provisions are included in design specifications. Conversely, information related to the fatigue behavior of mechanical connectors and welded joints is very limited, especially considering the variety of connectors and weld details available. A consequence of the limited research is that major U.S. codes and design specifications do not include comprehensive fatigue design criteria for any type of reinforcing bar splices, whether conventionally lapped bars, welded splices, or mechanical connections.

*The term splice is used to refer to the joining of two reinforcing bars by welding or with a mechanical connector.

THE PROBLEM AND ITS SOLUTION

NCHRP Project 10-35, "Fatigue Behavior of Welded and Mechanical Splices in Reinforcing

Steel," was initiated to evaluate the fatigue behavior of welded and mechanical splices for reinforcing bars in bridges, and to develop practical design provisions.

The research was accomplished by carrying out the following four tasks: a review and summary of published and unpublished literature; the design and conduct of a laboratory program of fatigue tests; a statistical analysis of the experimental results and those obtained from the literature; and the formulation of design guidelines. The guidelines were prepared, complete with commentary, in a format suitable for incorporation in the AASHTO *Standard Specifications for Highway Bridges.*

Background

A representative curve, which relates the cyclic stress or stress range, *S*, and the number of loading cycles to failure, *N* (usually called an *S-N* curve), for unspliced reinforcing bars is shown in Figure 1. This curve shows the effect of a constant-amplitude stress range on fatigue life for a constant minimum stress, S_{min}. The *S-N* curve can be considered to consist of three distinct regions: the low-cycle region; the finite-life region; and the long-life region, which defines the fatigue limit.

For practical fatigue design purposes, the low-cycle region is unimportant (outside of seismic design), and the *S-N* curve is simplified to two straight lines representing the finite-life and long-life regions, as illustrated in Figure 2. The horizontal line represents the fatigue limit stress range, below which the bar may be expected to sustain an unlimited number of cycles, from the practical point of view, without failure. At stress ranges greater than the fatigue limit, the fatigue life decreases with increasing stress range, as represented by the sloping line for the mean fatigue life, which is derived from test data. The sloping line represents the relationship between stress range and number of cycles for which there is a 50 percent probability of failing in fatigue. Because a 50 percent probability of failure is unacceptable for design purposes, the design fatigue limit must be established and is usually taken to be the lower 95 percent tolerance limit. This results in a line defining the design fatigue life, which represents a near 100 percent probability of a bar not failing in fatigue for combinations of stress range and loading cycles falling below the line. The lines representing the design fatigue life and the mean fatigue life are parallel—the distance between them being a measure of the scatter of the data.

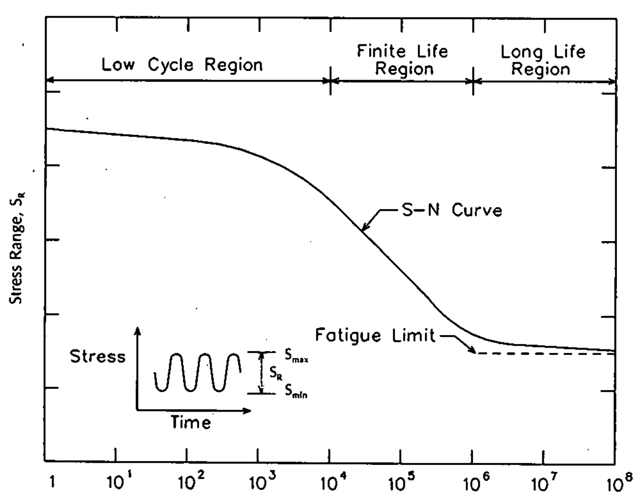

Figure 1. Representative S-N curve for steel reinforcing bars.

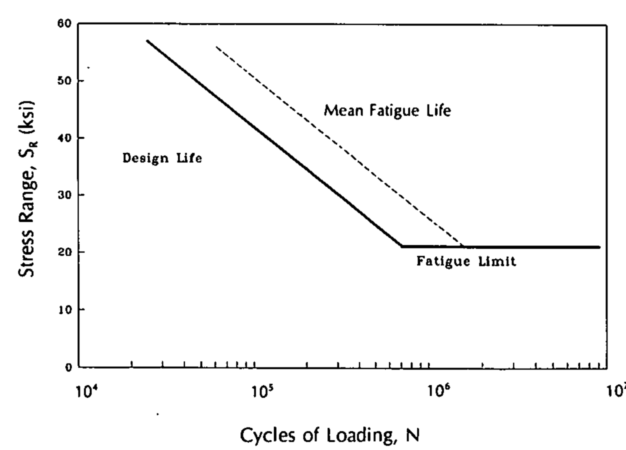

Figure 2. Fatigue lines for unspliced, Grade 60 North American reinforcing bars (equations representing the three lines are given in NCHRP Report 164).

FINDINGS

Literature Review

Numerous references to the fatigue behavior of unspliced reinforcing bars were found in the literature. The most significant factors affecting fatigue behavior have been shown to be the stress range during the loading cycle and the deformation geometry, where deformation geometry is characterized by the ratio of the radius at the base of the rolled-on deformation to its height. Minimum stress has a lesser effect. The grade of steel and bar size have a minor effect, but not sufficient to affect design procedures.

Research on the fatigue behavior of reinforcing bar splices, whether by welded joints, by mechanical connectors, or by conventional lapped bars, was found to be limited. Much of the work pertaining to proprietary mechanical connectors was unpublished. Although limited, the previous research showed that the fatigue strength of spliced bars can be substantially less than that of unspliced bars and that the type of splice has a significant effect. Where the locations of fractures were reported, nearly every fatigue failure of a mechanical splice occurred through the bar at the end of the connector, not in the mechanical connector itself. This was also true for welded splices.

Two methods of testing have been used in previous experimental investigations: specimens tested in axial tension in air, and specimens tested in flexural reinforcement in concrete beams. In some studies where splices were tested in air, parameters such as misalignment, or the deformation geometry, were not reported, making it difficult to compare data from different investigations. The difficulty was overcome, in this investigation, by calculating a representative fatigue strength for the unspliced bar used in each test, and comparing the fatigue performance of splices with the calculated values. Limited data indicate that testing in air may be a more severe condition than testing in beams, probably because secondary stresses due to misalignment are minimized, if not eliminated, by the encasing concrete.

Published data for fatigue tests on conventional lapped-bar splices were even fewer than for tests on welded or mechanical splices, but the results were consistent. In concrete beams subjected to flexural loadings, the fatigue performance of conventional lapped splices with straight bars was not reduced relative to that of the unspliced bar. Lapped splices with cranked bars (where the end of one bar is offset by bends in the region of the splice) exhibited significantly reduced fatigue performance.

Experimental Investigation

Fatigue tests on 231 spliced and unspliced bars were carried out in this portion of the research. The experimental program included tests in the finite-life region, and tests in the long-life region of the *S-N* curve. Seven proprietary mechanical connectors and two welded joint configurations, considered to be representative of splices in common use, were selected for testing. These are illustrated in Figures 3, 4, and 5, and the test program is summarized in Table 1.

The investigation was designed to permit a statistical evaluation of the constant-amplitude stress range below which each type of splice tested could sustain 5 million cycles of loading, which was regarded as the fatigue limit (long-life tests). Tests were also carried out on two types of splices at stress ranges intended to cause failure above the fatigue limit (finite-life tests). Two sizes of bar were used—No. 5 and No. 8. Unspliced bars of both sizes were tested in fatigue. Two mechanical connectors were also tested with epoxy-coated bars. A constant minimum stress of 3 ksi was used in all the testing.

The majority of the tests were conducted in axial tension in air, designated X in Table 1. For two splice configurations in which the longitudinal axes of the bars were offset at the splice location, the spliced bars were embedded in a rectangular concrete beam for testing. These splices are designated Y in Table 1. A test was also conducted on a double-lap weld splice, which was modified and tested in air, to try to establish a correlation with the single-lap weld tested in concrete.

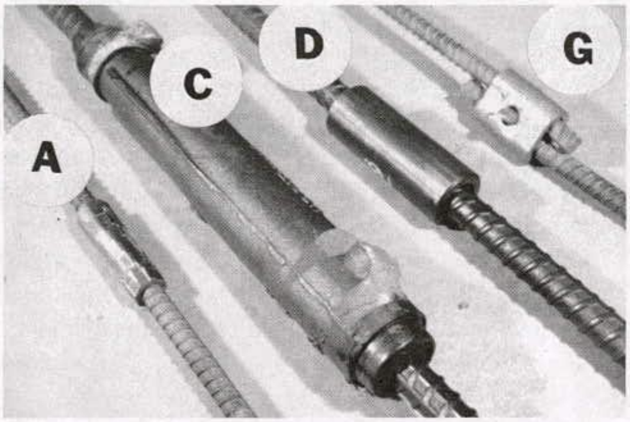

Figure 3. Nonthreaded mechanical connectors included in the experimental investigation (code letters A, C, D, and G).

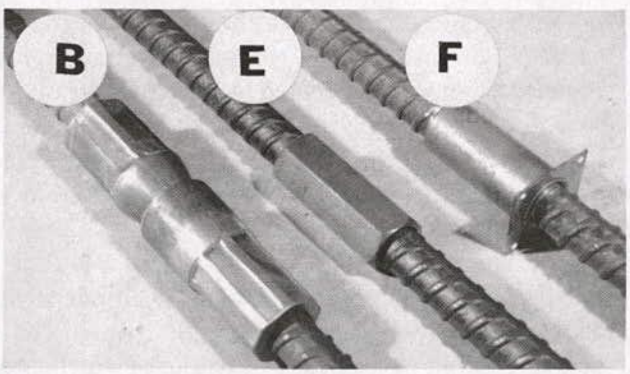

Figure 4. Threaded mechanical connectors included in the experimental investigation (code letters B, E, and F).

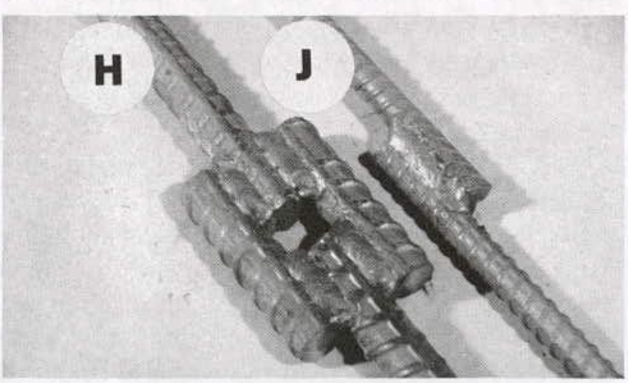

Figure 5. Welded splices included in the experimental investigation (code letters H and J).

Test Results and Statistical Analysis

For any one type of splice, the range of stress cycle was the predominant factor determining the fatigue life of the splice.

Data from the long-life and the finite-life fatigue tests were analyzed to obtain estimates of the mean fatigue strength and the standard deviation for each type of splice included in the test program. Published and unpublished test data from the literature were also included in the analysis, where appropriate. Measurements of the alignment of the specimens tested in air were also analyzed to account for the effects of misalignment or eccentricity of the spliced bars, but the effect was found to be small. Two-sided tolerance limits were calculated from the estimates of mean fatigue life and standard deviation, such that it is 95 percent probable that 95 percent of the data fall between the limits. As already noted, the historical practice has been to derive fatigue limits for reinforcing bars by taking the lower limit of the two-sided tolerance limit. The mean fatigue limit and the design fatigue limit for each splice tested are given in Table 2.

The design fatigue limit for different splices varied widely, ranging from a high of about 21 ksi for grout-filled coupling sleeves to a low of about 4 ksi for welded splices and for two-piece cold-swaged sleeves with threaded ends. In this investigation, the effect of the splice was always a reduction in fatigue performance relative to that of the unspliced bar. However, it was noted that this may not always be the case, especially if the bar has a relatively poor fatigue performance, in which case there may be no difference between the fatigue performance of the spliced and unspliced bar. Grade 60 reinforcing bar commonly has a design fatigue limit greater than 20 ksi and the bar used in this investigation had a design fatigue limit in excess of 24 ksi. In general, those splices for which a low design fatigue limit was calculated also exhibited a low mean fatigue limit. Nevertheless, for some splices the data were scattered, resulting in a large standard deviation and a large interval between the mean and design fatigue limits. Splicing epoxy-coated bars with a cold-swaged sleeve or a grout-filled sleeve was not detrimental to the fatigue performance of the splice.

Except for one bar in which the fracture

TABLE 1. ARRANGEMENT OF TEST PROGRAM

Code	Splice Type	No. 5 Bars		No. 8 Bars	
		Long-Life Tests	Finite-Life Tests	Long-Life Tests	Finite-Life Tests
U	Unspliced Bars	X	X	X	X
	Mechanical Connectors				
A	Cold-swaged steel coupling sleeve	X			
A*	Cold-swaged steel coupling sleeve	X			
B	Two-piece cold-swaged steel coupling sleeve with threaded ends	X		X	
C	Grout-filled coupling sleeve			X	
C*	Grout-filled coupling sleeve			X	
D	Steel-filled coupling sleeve			X	
E	Taper-threaded steel coupler			X	X
F	Straight-threaded coupler (bar not upset at threads)			X	
G	Steel coupling sleeve with wedge	Y			
	Welded Joints				
H	Double-lap			X	X
I	Modified double-lap	X			
J	Single-lap	Y			

X denotes a group tested in air
Y denotes a group tested in beams
* denotes epoxy-coated bar

TABLE 2. FATIGUE LIMITS AT 5 MILLION CYCLES AND PROPOSED FATIGUE CATEGORIES FOR SPLICES

Code	Bar Size (No.)	Mean Fatigue Limit (ksi)	Design Fatigue Limit (ksi)	Proposed Category (ksi)
U	5	31.3	27.3	
U	8	29.9	24.4	
A	5	17.0	12.6	12
A*	5	20.0	14.2	12
B	5	6.3	3.7	4
B	8	9.0	4.4	4
C	8	24.0	20.8	18
C*	8	25.4	19.1	18
D	8	13.4	9.8	4
E	8	20.0	14.2	12
F	8	13.0	8.6	4
G	5	22.9	16.4	12
H	8	10.3	7.7	4
I	5	7.2	3.9	4
J	5	21.2	15.6	4

NOTE: See Table 1 for codes
* denotes epoxy-coated bar

initiated at a bar mark, the fatigue fractures of all the unspliced bars initiated at the base of a transverse lug. The locations of the fatigue fractures in the spliced bars varied with the type of splice. Splices made with nonthreaded mechanical connectors typically fractured through the reinforcing bar at or near the end of the connector. This was the only mode of fracture observed for cold-swaged steel coupling sleeves and steel coupling sleeves with wedge, and it was the predominant mode for steel-filled coupling sleeves. Some steel-filled sleeves fractured transversely through the middle of the sleeve, initiating at multiple sites in the steel filler metal or the sleeve metal. Except for one specimen, which fractured through the reinforcing bar, all grout-filled coupling sleeves fractured transversely through the sleeve.

Splices made with threaded mechanical connectors fractured most often through the threaded segment of the reinforcing bar, initiating at the root of the first engaged thread immediately at the end of the coupler. All taper-threaded steel couplers fractured in this mode, as did most straight-threaded couplers. A few straight-threaded couplers fractured transversely through the coupler, initiating at the root of an internal cut thread. All two-piece, cold-swaged steel coupling sleeves with threaded ends fractured in the male-threaded half of the swaged coupling sleeve, initiating at the root of the first fully engaged male thread.

Welded splices always fractured transversely through the bar, initiating at a weld termination.

Implementation

The only commonly specified requirement for a welded splice or mechanical connector is that it must develop a tensile strength of at least 125 percent of the yield strength of the bar being connected. This requirement has little bearing on the fatigue strength of splices and, therefore, design requirements dealing specifically with fatigue are needed. Unfortunately, requirements for the fatigue strength of splices must be based largely on physical testing because fatigue strength cannot be computed analytically. Although the experimental investigation contributed substantially to the information on the fatigue splices, it studied only seven mechanical connectors out of the 20 to 30 available. An additional complication is that, because mechanical connectors are proprietary products, they are subject to modification at any time. Although factors such as the effect of the bar size (the details and dimensions of some connectors vary with bar size) and coated reinforcement were found to be small in this investigation, the number of combinations actually tested in fatigue represent only a small fraction of the available splice systems with potentially different fatigue performance. After careful consideration of all these issues, it was decided that sufficient information was available, and the most satisfactory method of implementation was to prepare a prescriptive design requirement suitable for incorporation in the AASHTO *Standard Specifications for Highway Bridges*.

Suggested Revisions to AASHTO Specifications

In the AASHTO specifications, fatigue is considered to be a serviceability requirement. An equation is presented that is applicable to straight, unspliced reinforcing bars, and which almost invariably results in a fatigue limit stress range above 20 ksi. Actual ranges of stress under service loads seldom exceed 20 to 25 ksi; the higher allowable stress range that could be justified in the finite-life region is, therefore, unimportant.

As the research in Project 10-35 showed, there appears to be a similar limiting stress range for spliced reinforcing bar, which is usually lower than for unspliced bar, but below which a spliced bar may be expected to sustain an unlimited number of cycles—from the practical point of view—without failure. As already noted, this design fatigue limit may be determined as the lower 95-percent tolerance limit to the 5 million cycle mean fatigue limit. Because the design fatigue limit for some splices is as low as 4 ksi, some adjustment for allowable stress may be needed for splices in the finite-life region. While there are limited data available, it appears that the slope of the finite-life region of the *S-N* curve for spliced bars can be represented approximately by the slope of the finite-life region for unspliced bars.

Suggested revisions to the AASHTO specifications were written, complete with commentary clauses, to incorporate requirements for the fatigue strength of splices. Splices were divided into three categories according to minimum design stress ranges of 18, 12, and 4 ksi. Category assignments were made for each of the splices tested in the investigation on the basis of the design fatigue limits and consideration of the way in which forces are transferred through the splice. The assignments are shown in Table 2.

Nonthreaded mechanical connectors (code letters A, C, D, and G) were, with one exception, assigned to the 12 ksi or 18 ksi categories. These assignments were made, in part, because the transfer of forces between the reinforcing bar and some of the connectors occurs gradually over the length of the bar within the connector, resulting in better fatigue performance. The grout-filled sleeve was the only connector for which the performance justified the 18 ksi category. Fatigue performance of the cold-swaged coupling sleeve and the steel coupling sleeve with wedge was appropriate to the 12 ksi category. Transfer of force in the steel-filled coupling sleeve resulted in concentration of stress at the junction between the bar and the sleeve, and the connector was assigned to the 4 ksi category.

Threaded connectors (code letters B, E, and F) were assigned to the two lower categories. Stress concentrations occur at the root of threads, reducing fatigue performance. However, the tapering of the threads appears to reduce the peak stress and the results from the taper-threaded coupler placed it in the 12 ksi category. The straight-threaded coupler and the two-piece, cold-swaged steel coupling sleeve were assigned to the 4 ksi category because local reductions in cross section increase stress concentrations and reduce fatigue performance.

All three weld configurations (code letters H, I, and J) were assigned to the 4 ksi category. The test results for the single-lap weld would justify the 12 ksi category but, because of unexplained differences between single-lap welded bars tested in beams and the double-lap welded bars tested in air, a conservative approach was taken. Couplers that were not tested in the investigation (and for which test data in the literature are insufficient to justify a higher category) were assigned to the 4 ksi category, with the provision that higher values may be used if justified to the satisfaction of the engineer.

A method for calculating the increase in stress range for less than one million cycles of loading was also proposed. The suggested revisions have been submitted for consideration by the AASHTO Subcommittee on Bridges and Structures.

CONCLUSIONS

The research showed that there is a large variability in the fatigue strength of mechanical connectors and welded splices, which are representative of those in common use. For any one type of splice, the range of the stress cycle is the predominant factor determining the fatigue life of the splice. Although data are limited, factors such as bar size, epoxy coating, and minimum stress appear to be insignificant for design purposes.

Under constant-amplitude stress cycles, splices exhibit a fatigue limit stress range below which they will sustain a virtually unlimited number of cycles from the practical point of view. Above the fatigue limit, the fatigue life decreases from about one million cycles with increasing stress range. The fatigue limit for any type of splice can be derived from a test program. This was done for the seven types of mechanical connector and the two types of welded splices included in the investigation. Based on these results, splices were divided into one of three categories and suggested revisions to the AASHTO specifications were prepared. By implementing the results of the research in this manner, it is anticipated that the findings will benefit bridge designers and stimulate improvements in proprietary connectors.

REPORT AVAILABILITY

The overall objective, research approach, findings, conclusions, and recommendations are presented in the main body of the agency final report, Project 10-35 titled, "Fatigue Behavior of Welded and Mechanical Splices in Reinforcing Steel." Detailed descriptions of the literature review, experimental investigation, and statistical analyses are presented in the Appendices. Appendix A presents a review of the literature and unpublished test reports; Appendix B the design and results of the experimental investigation; and Appendix C the statistical analyses of the test results.

The agency final report will not be published in the regular NCHRP report series. However, the agency report is available on microfiche by contacting Transportation Research Board, National Cooperative Highway Research Programs, 2101 Constitution Avenue, N.W., Washington D.C. 20418.

ACKNOWLEDGMENTS

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Conrad Paulson, of Wiss, Janney, Elstner Associates (WJE), and John M. Hanson, formerly of WJE and now with North Carolina State University, were the principal investigator and co-principal investigator, respectively. The work at Materials Research Laboratories was conducted under the supervision of John E. O'Donnell. Dr. E.J. Ripling of Materials Research Laboratories provided assistance as project advisor.

Splices used in the test program were provided by the following: Barsplice Products, Inc., Dayton, Ohio; Dayton Superior Corp., Miamisburg, Ohio; Erico Products, Inc., Cleveland, Ohio; and Splice Sleeve North America, Inc., Sacramento, California. The assistance of these companies is appreciated.

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